Field Evaluation of Dynamic Load Factors
for Historic Through-Truss Bridges

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by
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Abstract

The late 1800s was a time of great innovation in bridge engineering. The advent of steel and the demand for longer spans under higher loads spurred the development of many new bridge designs. Out of this innovative time period, a wide range of historic through-truss bridges were developed and constructed. These bridges, now well beyond their design life, are being evaluated for capacity; many being demolished, few being rehabilitated. The evaluation process requires the estimation of the complex dynamic behavior of historic through-truss bridges under traffic loading. Structural engineers often evaluate complex dynamic behavior with the use of a dynamic load factor (DLF), which enables the enveloping of dynamic response values by scaling the maximum static response. For this reason the current study set out to evaluate the magnitude, distribution, and influential variables of DLF for historic through-truss bridges. This was accomplished through a combination of field evaluation of existing specimens, and digital signal processing; to produce both the maximum static and dynamic response values for instrumented bridge members for each traffic event. Scatter plots, correlation coefficients, and histograms were then created to evaluate DLF magnitude, distribution, and correlations with test variables. Correlations and trends were investigated between DLF magnitude and: maximum static strain, vehicle speed, vehicle static weight, and bridge span. The most influential variable on the magnitude of DLF was determined to be member peak static strain. The present study concludes that historic through-truss bridges exhibit DLFs with magnitudes higher than contemporary slab on girder bridge types.
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1. Introduction

1.1 Background

The late 1800s was a time of great innovation in bridge engineering. The advent of steel and the demand for longer spans under higher loads spurred the development of many new designs. Out of this process, through-truss bridges of many designs became common structural forms used in the Northeastern United States. These now historic bridges have unique and aesthetically pleasing geometric forms that are rarely exhibited in modern construction practices. Only a select few remind us of these once prominent structural forms, now in danger of being lost as many bridges have been replaced or swept away in floods in the past century.

Adding to this danger, recent bridge collapses like the I-35W disaster in Minnesota have focused national attention on the state of America’s deteriorating infrastructure. This renewed focus in conjunction with the age and deterioration of historic truss bridges, puts these structures in serious risk of being found structurally deficient, demolished, and replaced. This would result in a great loss of American structural and architectural history.

A primary factor in determining the fate of a historic bridge lies in the treatment and understanding of their complex dynamic behavior when exposed to contemporary traffic loading. The most commonly used technique in practice for dealing with complex dynamic loading is the application of a dynamic load factor (DLF). This technique allows bridge dynamics to be evaluated in a static analysis by scaling the magnitude of a static traffic load to envelope the relatively higher dynamic loads of moving traffic. The bridge components can then be designed or evaluated for this effective static load, removing the need for detailed dynamic analysis. Standard values for dynamic load factors exist in all major bridge construction codes, but have been developed primarily for use with modern bridge forms such as girder bridges. Historic truss
bridges having a lower mass to stiffness ratio are likely to exhibit different dynamic behavior than modern girder bridges resulting in different dynamic load factors.

1.2 Problem Statement

Previous research on the dynamic behavior of historic bridges has not provided a widely accepted DLF. This leaves only approximations of the dynamic behavior resulting from the extension of research performed on modern bridge types. Due to the complex geometries and lower mass to stiffness ratios, it is likely that these truss bridges do not behave in the same manner as previously researched bridge types and, therefore, require different DLFs when evaluating service conditions. The use of modern codified DLFs could be unnecessarily condemning many historic bridges. This concern warrants research specifically dealing with the dynamics of historic truss bridges.

1.3 Focus of Research

The broad subject of dynamic bridge response will be narrowed to the determination of dynamic load factors used to scale response to service loads for the evaluation of historic truss bridges. The domain of historic truss bridge types encompasses a wide variety of different bridge geometries; Pratt and lenticular arch trusses will be the focus of this research. Pratt trusses were selected due to the relative abundance of existing examples open to traffic and their generic geometry that will provide a baseline for historic bridge behavior. Lenticular arch trusses were selected to help the preservation effort of their limited numbers and the relative lack of knowledge of their behavior. For lenticular arch trusses, the contribution of the longitudinal girder in carrying the applied loads to the supports will also be investigated in addition to the DLF. This secondary investigation is a result of the unknown design intent of the longitudinal girder, which complicates the calculation of accurate member forces.
1.4 Scope of Research

The research presented will determine, through field testing, specific dynamic load factors for lenticular and Pratt truss bridges. The results of the study will provide a more accurate DLF for the evaluation of service conditions of historic bridges. This analysis can then be used to aid in the structural certification of the remaining examples of these structural forms. Through better understanding of the dynamic behavior, retrofit and restoration efforts are expected to be more effective in maintaining the structural health for future generations.

Specifically, bridges will be selected for field-testing to meet the following criteria:

- Pratt or lenticular truss geometry
- Date of original construction: 1880 - 1900
- Through-truss design
- Open to highway traffic
- Ideally, unaltered original construction

Three bridges were selected for field-testing using one or more of the following three loading conditions: controlled truck traffic, normal traffic, and controlled dynamic shaker testing. Under controlled truck loading various vehicle velocities will be used to test the influence of velocity on DLF. Instrumentation will be placed on up to sixteen members of interest in order to record the bridge response under the previously described loading. These response quantities will then be used to calculate DLFs for each test case.

To evaluate the influence of many competing variables several test parameters will be controlled or documented for each test case in order to perform a comparative study of the resulting DLF. The test parameters considered to influence DLF and corresponding values to be controlled or documented for the present study are:
• Span length (natural frequencies): 100, 150, 200+ft

• Vehicle speed
  • Truck testing: crawl, 1/3, 2/3, full speed limit
  • Normal traffic testing: approximated using known distance and time

• Vehicle static weight
  • Truck testing: constant, documented value
  • Normal traffic testing: obtained through digital filtering of field data

1.5 Objectives

The primary objective of this research was to experimentally calculate Pratt and lenticular truss specific DLFs. The secondary objective of this research was the determination of relationships between parameters of interest and the DLF. The final and furthest reaching objective is to provide this information in an effort to reduce the cost of accurate analysis of historic truss bridges and therefore aid in preservation efforts.
2. Literature Review

2.1 Introduction

This literature review consists of four key areas concerning the conducted research. The first of these areas is the investigation of code mandated dynamic load factors (DLF) from leading national bridge design and construction codes. Comparison of these DLF specifications will further demonstrate the need for research in this area as it relates to truss bridges. Also, these DLF specifications can be used as a baseline comparison for field determined DLFs measured later in the research study. The second key area of interest is investigation of published results of bridge dynamic field testing and monitoring techniques. This is crucial to the completion of the proposed research because field data used in the determination of DLF will be obtained using the best available of these methods. The third key area of interest consists of published research results of bridge DLF, including dynamic load allowance, dynamic amplification factor, and impact factors. Specifically, from these published studies the type of bridge tested, parameters of interest, and DLFs reported will be presented to assist with the development of an experimental program for the proposed research. The fourth key area of interest consists of an overview of relevant sensor, data acquisition, and filtering technology as it pertains to the proposed research. This will establish the specifics of the experimental and analytical programs used to conduct the research. Many sources were consulted to gather the information required to provide the review of the state of the field presented here; a full list of reviewed references is available at the end of the document.

2.2 DLF Definition

Before further discussion can take place, definition and explanation of the DLF and the variables that affect the dynamic response of a bridge are required. The most common definition
of dynamic impact, \( I \), is presented in equation (2.1) (Billing, 1984; Paulvre, 1991; Laman, 1999; Ashebo, 2006; Moghimi, 2007)

\[
I = \frac{R_{\text{dyn}} - R_{\text{static}}}{R_{\text{static}}}
\]  

(2.1)

where: \( R_{\text{dyn}} \) is the maximum dynamic response; \( R_{\text{static}} \) is the maximum static response

Figure 2-1: Bridge Response to Dynamic Loading

Close examination of Figure 2-1 expands the concept of dynamic amplification and DLF. From the graph the relative differences between the low frequency static response and high frequency dynamic response of a structure can be observed. The dynamic impact, \( I \), is essentially the percent difference between the maximum dynamic and static responses. Adding 1.00 or 100% to \( I \) then obtains the DLF, accounting for the original static response. Figure 2-1 presents the static response scaled by the calculated DLF for this particular case. This DLF scaled
response envelopes the dynamic response in the shape of the static response. This obtains the maximum dynamic response from the easily calculated static response. The goal of the DLF is to simplify analytical calculations while ensuring the maximum stress in the structure is identified. This simplification, shown in Figure 2-1, is obtained by scaling the static response by the DLF.

2.3 Code Prescribed Dynamic Load Factors

The following list of existing factors prescribed by design codes and historical references provides an overview of the state of DLF in bridge engineering. In the study of DLFs, or dynamic amplification, the bridge engineering community has yet to come to a consensus; each code having its own way of dealing with the dynamic loads and, therefore, unique DLF. Although each specification is unique, three categories of DLF exist with respect to the variables of interest: constant, length varying, and frequency varying.

2.3.1 Constant DLF

The simplest way to prescribe a DLF is to specify a constant factor that can be applied to all bridges of all makes, lengths, and materials. AASHTO LRFD Bridge Design Specification, article 3.6.2.1 simply sets three constant impact factors \((IM)\) seen in table 2-1. These \(IM\) factors can then be used in equation 2.2 to determine the DLF. The general case of 33% is most relevant to the research in question and would result in a DLF of 1.33. AASHTO LRFD (1994) states that these values were obtained through field testing. This testing resulted in a maximum \(IM\) factor of 25% for most highway bridges. The 25% is a result of the design truck loading only and is not applied to the design lane loading, for this reason the reported value is then multiplied by \(4/3\) resulting in the 33% value that is reported in table 2-1.

\[
DLF = 1 + IM
\]  

(2.2)
2.3.2 Length Varying DLF

The commonly included variable in the DLF formula is span length. These specifications take into account the tendency for longer span bridges to exhibit lower dynamic amplification. Most specifications have a similar format, where $IM$ is determined by some constant divided by the sum of another constant and the length of loading in the span that produces the highest static load. The length of loading can be taken as the bridge length in simply supported cases. Table 2-2 presents the code specifications that vary with length and their respective DLF equations.

Table 2-2: Length varying DLF  (Equations 2.3-2.5 from Moghimi 2007)

<table>
<thead>
<tr>
<th>Specification</th>
<th>DLF</th>
<th>Equation #</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO 1992</td>
<td>$DLF = 1 + \frac{50}{L(ft) + 125} \leq 1.3$</td>
<td>(2.3)</td>
</tr>
<tr>
<td>Japan</td>
<td>$DLF = 1 + \frac{6.1}{L(ft) + 15.2}$</td>
<td>(2.4)</td>
</tr>
<tr>
<td>French</td>
<td>$DLF = 1 + \frac{1.2}{L(ft) + 1.5}$</td>
<td>(2.5)</td>
</tr>
<tr>
<td>Merriman and Jacoby</td>
<td>$DLF = 1 + \frac{300}{L(ft) + 300}$</td>
<td>(2.6)</td>
</tr>
</tbody>
</table>
Figure 2-2 presents a graphical comparison of the previously presented DLF equations. It is clear from the figure that Merriman and Jacoby prescribes a higher DLF than contemporary codes for bridges of all lengths. This difference can be explained by the change in the dynamic loading applied and subsequently the dynamic behavior exhibited in bridges over the last century. In the time of Merriman and Jacoby predominant impact loads were caused by locomotive traffic, which have can have a large mass relative to the causing higher dynamic response. In contrast, modern bridge live loading has a lower load to bridge mass ratio resulting in a lower percentage of total stress from dynamic loading. The higher values could also reflect the quality of dynamic measurements that were available in the early 1900s. Regardless of the time period, the values reported demonstrate a clear disagreement over the appropriate DLF for bridges of different lengths.

Figure 2-2: DLF of AASHTO, French, Japanese, and Merriman and Jacoby (Moghimi 2007)
2.3.3 Frequency Varying DLF

The third category of DLF specification is natural frequency varying load factors. This method for determining the DLF was first introduced in the 1983 Ontario Highway Bridge Design Code (OHBDC) and was later adopted by the Australian specification, AUSTROADS, in 1992. OHBDC states that this method was developed based on test results. The values obtained from these respective codes are presented graphically in Figure 2-3. It can be observed that this method will produce vastly different DLFs than either of the previously discussed methods. The rationale of this specification is that bridges with natural frequencies close to the frequency of normal traffic loading will experience higher dynamic amplification, therefore the increase in DLF for bridges with natural frequencies in the 2-5Hz range.

![Graph showing DLF vs. First Flexural Frequency for OHBDC (1983) and AASHTO (1994)]

Figure 2-3: Canadian, Australian, and American DLF vs. First Flexural Frequency
2.4 Dynamic Field Testing of Bridges

Accurate observation of complex dynamic behavior of bridges is most readily accomplished through field testing. Field testing from published research falls into two major categories: controlled traffic testing and normal traffic testing. Controlled traffic testing involves passing a test vehicle of known weight over the bridge at a specific speed and horizontal location relative to the bridge deck while recording the dynamic response. In this case the relationship between applied load and observed response is well known. However, this test does not guarantee that the test vehicle will excite the bridge in the same dynamic manner as normal traffic conditions due to the limited sample size of the test and the wide variety of truck types on the market. In contrast normal traffic testing allows the bridge specimen to remain open to all traffic during testing, producing a realistic sample of different truck types. As a result of the live traffic, no parameters of the load are known making the relationship between applied load and observed response less clear. However, this ensures that the bridge is being excited in a realistic dynamic manner. Normal traffic testing relies on the ability to determine the loading parameters from the collected data, for this reason the data collection system is often calibrated with an initial controlled traffic testing period.

2.5 Other DLF Studies

Experimentally or numerically determining dynamic amplification has been a popular research topic over the past 40 years in many countries. Studies were conducted by a number of independent researchers employing a wide array of techniques to observe and determine values for dynamic amplification. However all studies concerned with determining DLFs fall into two broad categories: analytical and field.
2.5.1 Field Determined DLF

The direct measurement of bridge dynamic behavior is a straightforward approach to determining dynamic response and ultimately DLF of real world bridges. With careful implementation, field testing of bridges can determine accurate DLFs considering all variables affecting the dynamic behavior of a bridge. This leads to a more realistic picture of the total response and therefore DLFs that closely mimic actual dynamic amplification. For this reason most recent research concerning bridge dynamics involves observation of actual bridges. This method focuses on a limited sample of bridges and therefore the results are bridge specific, making it difficult to reliably expand the results to other bridges.

2.5.1.1 Field Testing Procedure

The calculation of DLF requires the dynamic and static response of a structure to be known. When field testing is implemented, the dynamic response can be measured directly from sensor readings. The static response cannot be directly measured because of the dynamic nature of all real loading conditions. From published research, field static response is obtained using two different methods depending on the type of testing being performed. When controlled traffic testing is utilized, the static response can be estimated by performing a crawl speed test. During a crawl speed test the load travels along the length of the bridge at a low speed such that the static conditions are approximated and this response can be directly recorded. This however requires the dynamic and static response to be recorded from two different passes of the test vehicle. To produce accurate results the truck must make all test passes in the same position relative to the bridge deck. This may be possible at low speeds but becomes impractical at highway speeds. When normal traffic conditions are employed, crawl speed testing becomes impossible. For these reasons many researchers obtain the static response from the measured dynamic response with
the aid of a low pass digital filter. The static and dynamic responses are then determined from the same test pass, which removes the need to ensure identical positioning of the test load. The digital filter employed in this method removes the higher frequency dynamic response from the low frequency static response, and has been used with success by (Billing 1984), (Paulitre 1991), (Kim and Nowak 1997) (Laman 1999), (Ashebo 2006).

In their review of bridge dynamics, Paulitre, Chaallal, and Proulx (1991), establish specific concerns that need to be addressed when designing a digital filter to ensure accuracy when obtaining the static response. These special considerations include: a passband at $v/L$ Hz (velocity/span length) or the static loading frequency, and a stopband at a frequency below the first fundamental frequency of the bridge. In the case that the passband and the stopband become close in frequency, the filtering method introduces error as the static and dynamic responses cannot be separated accurately. According to Paulitre, this becomes a problem for relatively short span bridges; less than 15m in length. Billing (1984) also suggests using a long leader of data prior to the test vehicle crossing the bridge in order to remove the transient and impulse effects of the digital filter.

2.5.1.2 Field Testing Results

Many field studies have been performed recently to determine DLFs and the variables that affect its value. Presenting a summary of these studies provides insight into the methods and results of other independent researchers. Overviews of several published field studies are presented here including: method of measurement, bridge type, variables of interest, correlations observed, and values obtained.
2.5.1.2.1 Girder Bridge Studies

O’Connor and Pritchard (1985) conducted two separate impact studies on a small composite girder bridge near Brisbane, Australia. Dynamic measurements were obtained from midspan strain measurements under live traffic loading. The static loads were determined with the aid of a mandatory weigh station near the bridge, which provided the static weight for each of the test vehicles. The static loading effects were then analytically calculated using this static load. Calibration tests were conducted with a known weight vehicle to accurately determine girder distribution factors to use in the calculation of static effects. The variables of interest for this study are: vehicle gross weight, vehicle suspension type, and surface roughness. The authors observed some correlation between lower static effect and higher impact value, but conclude that vehicle gross weight has a small negligible influence on dynamic impact. Through analytical studies the authors state that no correlation exists between impact and surface roughness, although they go on to say field testing will be required to validate this claim. The most significant result from the study is the observation of a strong correlation between vehicle suspension type and impact factors. The authors specifically state that dual axle suspensions with load sharing capability have a strong influence in producing large impact values. The authors reported a mean impact value of 0.38 and a maximum value of 1.32. This would translate to DLFs of 1.38 and 2.32 respectively. With respect to other research the authors recommend the use of live traffic testing as a result of the strong correlation between suspension type and impact factor and the need to test many suspension types.

Kim and Nowak (1997) conducted load distribution and impact factor testing on two I-girder bridges in Michigan. Dynamic strain measurements were recorded under normal traffic conditions using strain transducers attached to the lower surface of the bottom flange at midspan
of each girder. One calibration vehicle of known weight provided a reference level for strain data collection for each bridge. Static strain levels were obtained through digital filtering of the dynamic data. The variable of interest for this study was peak static strain. The authors found that impact factor decreases as the peak static strain increases. During the testing girders far from the source of loading exhibited much higher impact values than those near the load. The reported values for dynamic impact have a mean of 0.12, a standard deviation of 0.15, and a maximum value of 0.6 obtained far from loading corresponding to low static strain. These values result in DLFs of 1.12, 1.15, and 1.6 respectively. The authors note that the average values fall well below the AASHTO specifications for all members with large static strains.

2.5.1.2.2 Box Girder Bridge Studies

Demeke B. Ashebo, Tommy H.T. Chan, and Ling Yu published results from DLF studies conducted on a skewed continuous box girder bridge in 2006. The bridge was instrumented with strain gages, accelerometers, and axle sensors used to determine bridge response, natural frequencies, and vehicle speed and position respectively. Data was collected for five consecutive days under normal traffic conditions and under controlled pseudo static conditions for one test vehicle. Dynamic response of the bridge came directly from recorded data and the static response was obtained using a low pass digital filter. The variables of interest for this study were speed, weight, number of axles, axle spacing, and vehicle position all of which were recorded during field tests. Through comparison of 309 good cases the authors established a negative correlation between vehicle weight and DLF. A weak positive correlation was observed for vehicle speed. Vehicles moving in a group or side by side consistently resulted in lower DLF than a single vehicle; this corresponds with vehicle weight correlation as multiple vehicles increase the static strain. The authors found no correlation with respect to number of
axles. The maximum DLF recorded was 1.91 occurring in a negative bending region of the bridge. After statistical analysis of the computed DLFs the author recommends a DLF for box girder bridges of 1.24 which corresponds to a 90% confidence interval.

2.5.1.2.3 Truss Bridge Studies

Laman, Pechar, and Boothby (1999) conducted dynamic impact testing for three, 1930s era, through-truss bridges. Strain gages were used to measure the dynamic response of critical members, and the static response was obtained through digital filtering of the dynamic data. Testing was initially conducted under controlled loading with known axle weights and vehicle speed to produce calibration data. Normal traffic testing was also conducted to obtain results from a wide range of loading scenarios. Variables of interest for this study include: component type, component peak static stress, vehicle type, and vehicle speed. The authors found a strong negative correlation between dynamic impact values and peak static strain for floor beams and stringers. From the reported data stringers away from the load (low static stress) exhibit dynamic impact up to 2 times that of stringers near the loading. The authors observed no correlation between vehicle speed and impact. Overall the study found that dynamic amplification was dependent on truck location, component location, component type, and component peak static stress. The author recommends an IM of 10% to be used for through truss bridge stringers and floor beams with good surfaces.

2.5.1.2.4 Comprehensive Bridge Types

J.R. Billing (1984) conducted one of the largest studies investigating bridge DLF for the Ontario Highway Bridge Design Code (OHBDC). For this study 27 bridges of various construction material, span length, and configuration were dynamically tested. Selected bridges were instrumented with both accelerometers and displacement transducers to determine the
natural frequencies and dynamic response respectively. Pressure switches located a known distance apart were used to measure the vehicle velocity. Testing was conducted with known weight control test vehicles and under normal traffic conditions. The static response for each test was obtained with a low pass digital filter, a Fourier analyzer was used to determine the location of the cut off frequency. During the controlled tests the author observed vehicle suspension dependent DLF; the mean for similar test vehicles with air bag suspension was 60% lower than those with leaf suspension. A general decrease in DLF as vehicle weight increased was observed for longer span bridges. The maximum mean value of DLF reported in the study was 1.36, which was observed in the bridge during free vibration (after a truck had passed). The study also observed a much higher mean coefficient of variation of 0.82 than the initial assumption of 0.45.

2.5.2 Analytically Determined DLF

Analytical determination of dynamic amplification has become more prevalent in recent research. This method uses finite element modeling of vehicle bridge interaction to determine the dynamic behavior of the bridge. To accomplish this task the bridge structure, test vehicle, and often the roadway surface need to be modeled using idealized representations of in-field conditions. A summary of published studies will be presented here including: bridge type, model type, idealizations required, variables of interest, correlations observed, and values obtained.

2.5.2.1 Three Dimensional Analysis

Lu Deng and C.S.Cai (2009) completed an analytical study of the variables effecting impact factors for multi-girder concrete bridges. For this study a 3D vehicle-bridge coupled model was used. The test vehicle for this case was taken as an AASHTO HS20-44 truck, modeled as a rigid body connected to two linear elastic springs in series to represent the suspension and tires. The road surface was also modeled using a zero-mean stationary Gaussian
random process based on a power spectral density function. From the model, coupled equations of motion for the test vehicle and bridge were created and solved in the time domain using Runge-Kutta method. Comparing to the results of a previous study in which dynamic behavior of multi-girder concrete bridges was directly measured then validated the results of the solved equations of motion. A parametric study was then conducted using the validated model; variables of interest are: span length, vehicle speed, and surface roughness. For each permutation the model was run 20 times with 20 sets of randomly generated road surfaces, providing 20 average values of DLF. These results produced a strong negative correlation between road surface condition and DLF. The author concluded that the other variables of interest did not show clear correlations with DLF. It was observed that the shortest bridge tested produced the highest load factors. The reported values of DLF ranging from 1.1 to 2.5, values higher than 1.3 corresponding to poor or very poor road surface condition. The author recommends using a modified version of the AASHTO 1994 specification for impact factor for bridges. This modification produces higher impact values for “Poor” and “Very Poor” surface conditions shown below in equation 2.7:

\[
IM = RSI \times \begin{cases} 
0.33 + 0.01 \times (16.76 - L) & L < 16.76m \\
0.33 & L \geq 16.76m
\end{cases}
\]  

(2.7)

where: RSI corresponds to the road surface index taking values of 0.7, 1, 1.5, 3, and 6 corresponding to very good, good, average, poor, and very poor road surface conditions.

Hassan Moghimi (2007) conducted an analytical study using non-linear dynamic simulation to determine the dynamic impact for composite steel bridges. The modeling involved in this study consisted of: several vehicles as rigid bodies attached to nonlinear suspensions; a composite steel girder bridge as an assembly of 3D thin walled beam elements, shear connectors, and concrete slab; and pavement profiles as a realization of exponential functions combined with
inverse Fourier transformations. These models provided simultaneous equations of motion, which were solved using direct integration method and a time step of 0.001 to 0.0001 seconds. Results from solving these equations were then compared to a previous field study of the bridge in question to validate the modeling and analysis techniques. Adjustments were made to better replicate field measurements so the model could be used to conduct a parametric study. Variables of interest in the parametric study include: vehicle velocity, span length (aspect ratio), stiffness of elastomeric bearings, mass of vehicle, vehicle eccentricity with respect to deck centerline, and initial vehicle bounce due to surface roughness. From the parametric study it was determined that DLF is: positively correlated with velocity and initial bounce; negatively correlated with length, bearing stiffness, and lane eccentricity; and not correlated with vehicle mass. The author reports values ranging from 1.05 to 1.30 for DLF and concludes that current code specified values are adequate for normal design situations.

2.5.2.2 Two Dimensional Analysis

Hwang and Nowak (1991) investigated dynamic loads and amplification in prestressed concrete and composite steel girder bridges. Bridges for this study were modeled as prismatic beams by assuming that the static and dynamic load distribution factors were the same. The truck for this study was modeled as a distributed mass in rigid body motion connected to leaf springs treated as nonlinear springs connected to tires treated as linear elastic springs. The road surface was also modeled using a power spectral density function to create 20 random surface profiles for each test conducted. These three models were combined to generate coupled equations of motion, which were then solved using Newark’s direct integration method. Normal traffic conditions were conducted using the Monte Carlo simulation method with the assumption that truck traffic would be 20% 3-axle and 80% 5-axle trucks. A parametric study was conducted; the
variables of interest were: span length, vehicle static weight, vehicle speed, axle spacing, and 
surface roughness. The author’s results show that DLF has: 1) a negative correlation with vehicle 
static weight, 2) a weak positive correlation with axle spacing and vehicle speed and, 3) no clear 
correlation with other testing parameters. Huang and Nowak also investigated trucks travelling 
side-by-side which resulted in lower DLFs than single vehicle tests. The published results report 
mean DLF values of 1.09 to 1.21 extrapolated for a 75-year lifespan of the bridge.

2.6 Low Pass Digital Filters

Many published DLF research studies obtain static bridge response from measured 
dynamic data using a digital low pass filter; therefore a working knowledge of the digital filter 
process is helpful in understanding the published results. Owen Bishop (1996) explains the 
details of analogue and digital filters; a summary of his low pass digital filter discussion will be 
presented here. Figure 2-4 presents a graphical representation of the digital filter circuitry. A 
digital filter has 3 main components: registers, multipliers, and summing blocks, which are all 
connected by data busses. In Figure 2-4 the square blocks represent data registers, the middle 
row of circles represent the multipliers and the bottom row of circles represent the summing 
blocks. The unfiltered signal \(X_n\) is cycled through the data registers, once in the register the 
signal value is multiplied by the corresponding coefficient \(a\) for that register, then the product 
of all of the registers are added together to produce the filtered signal \(Y_n\). In this fashion when 
each \(X_n\) value is in the first data register the filter produces the corresponding \(Y_n\) value. The 
whole process can be thought of as a moving average that reduces the impact of short-term or 
high frequency signals on the filtered response. The characteristics of a particular digital filter 
are a function of the number of data registers and the multiplying coefficients, by varying these 
values any type of filter can be created. Data registers take initial values of 0 and, therefore, 
accurately filtered data only occurs when all data registers have been filled with unfiltered data.
2.7 Summary

The published analytical and experimental field studies presented provide a general outline for the study of DLF but do not provide uniform conclusions concerning the value of DLF or the variables influencing this value. This lack of consensus suggests the need for further research to gain a better understanding of the complex relationship that exists between bridge loading and dynamic response. The need for accurate DLF is especially important when evaluating the adequacy of historic structures nearing the end of their design life, for which little research has been conducted. The proposed research will provide insight into the possible differences in dynamic behavior of historic through-truss bridges as compared to modern bridge designs.
3. Study Design

3.1 Introduction

The study of historic bridge DLF relies on the development of a logical process to obtain the dynamic and static response of a historic bridge during a traffic event. For the present study these quantities were obtained using the process outlined in Figure 3-1. Starting at the top of the figure, the study required historic bridges that could be exposed to traffic events. This dramatically decreased the number of bridges available because most historic bridges from the late 1800s have been closed to traffic. The bridges selected for the present study and the traffic events studied will be detailed in Chapter 4. Once a historic bridge that was open to traffic was located, an experimental program needed to be implemented to gather field data. The specifics of this experimental program will be detailed in Chapter 4. The data of interest observed in the next level of the flowchart is: vehicle presence, vehicle speed, member strains due to traffic events, and member strains due to a generated forcing function. The data of interest was determined by examining the required inputs for the analytical program. These needs were established by examining the process in which the static and dynamic responses would be produced. For the present study a digital filter was used to produce the dynamic response, and the static response. The use of a digital filter then requires the determination of cutoff frequencies between the static, dynamic, and noise portions of the raw. This necessitated the forcing function and corresponding member strains for identification of natural dynamic frequencies, and the vehicle presence and vehicle speed for identification of the static frequency. The analytical program will be further detailed in Chapter 5.
Figure 3-1: Flowchart of Study Processes
4. Experimental Program

4.1 Introduction

In development of the experimental program, three key areas will be discussed: traffic testing procedure, modal testing procedure, and bridge selection. As previously discussed, the data required to conduct the analytical program included: vehicle presence, vehicle speed, member strains during traffic events, and member strains due to a generated forcing function. The key areas discussed in this chapter will detail the process and methods used to collect this raw field data.

4.2 Traffic Testing Procedure

The first and most important data required to complete the present study are member strain time histories caused by traffic events. To collect this type of data, sensors were installed on selected bridge members and data was recorded during traffic events. Using a standardized testing procedure at each bridge and for each test run allowed data to be compared from different test runs and different bridges. Detailing the traffic testing procedure requires four sections: Data Acquisition System, Member Selection, Data of Interest, and Test Vehicle Selection. Controlling these factors validates a comparative study based on the data collected.

4.2.1 Data of Interest

Understanding the dynamic amplification of loads for the selected bridges under normal traffic conditions is the overall goal of this research. In order to accomplish this goal the dynamic response of the bridge under normal traffic conditions was observed. For the present study it was decided to observe the bridge member strains as a direct indicator of the bridge dynamic response. To collect this data, a data acquisition system was left in place to measure bridge response to normal traffic for a period of time required to obtain an optimum of 50 traffic events.
The data of interest from each of these traffic events was the peak static and dynamic strain for each selected bridge component. A digital filter was proposed to obtain the static response during data analysis and therefore only the dynamic response was recorded during testing. To help the implementation of the digital filter, the vehicle speed and presence were also recorded by placing rubber coated tape switches on the bridge approaches.

4.2.2 Data Acquisition System

A reliable data acquisition system was developed to record the dynamic response of the selected bridges. The factors considered in the development of this system were: availability, portability, ease of installation, and accuracy of measurements. The data acquisition system used for the present study is illustrated in Figure 4-1. The data acquisition process consists of 3 key processes: analog sensing, analog to digital signal conversion, and data storage. To conduct the analog sensing two types of sensors were selected: linear variable displacement transducers (LVDTs) were selected as the measurement device to capture the bridge member strains, and rubber coated tape switches were selected to provide information about the vehicle presence and speed. To convert and process the analog signals an IOtech, Daqbook 216 data acquisition system was selected and connected to a notebook computer used to trigger the system. A 32-gigabyte flash drive was used to store the digital data produced after each traffic event. The system was supplied with electric power in the field by a portable generator.
Figure 4-1: Data Acquisition System

4.2.2.1 LVDT Sensors

When supplied with a constant reference voltage, LVDTs return an electronic analog signal with a voltage relative to the mechanical displacement of the sensor body and plunger. For this reason, LVDT sensors are normally used in structural engineering to capture displacements directly. For the purposes of the present study, bridge displacements were of little concern compared with dynamic strain values; therefore, the LVDTs were attached and oriented in such a way as to measure strains. Figure 4-2 is a picture of an LVDT attached to a bridge member in
this fashion. As can be seen in the picture the LVDT is clamped to the bottom flange, oriented with the longitudinal axis of the member. In this manner the change in length of the flange due to bending moment or axial force will produce an equal movement of the LVDT plunger. Dividing this change in length by the gage length, or the distance between the tip of the LVDT plunger and the LVDT clamp then produces the measurement of strain in units of inch/inch. This process is summarized in equation 4.1. The resin block, seen in Figure 4-2, was used to isolate the LVDT from direct contact with the metal clamp and bridge member preventing interference with the LVDT internal coil.

![Diagram of LVDT Strain Gage Application](image)

**Figure 4-2: LVDT Strain Gage Application**

\[
\frac{signal \times gage \ factor}{gage \ length} = \frac{volts \times \text{inch/volt}}{inch} = \text{strain} \quad (4.1)
\]
The key factor in determining dynamic strains from the LVDT analog voltages using Equation 4.1 is the calibration or gage factor, which converts the voltage into a distance. The gage factor is unique to each LVDT and can change over long periods of time. To accurately determine this constant for each LVDT, each sensor used in the testing process was independently calibrated using a commercially available calibration apparatus. To complete this calibration the LVDT is placed in the apparatus and a micrometer is used to displace the LVDT core a known distance creating a change in voltage. These values are then divided to produce sensor specific gage factors. This procedure was then carried out ten times to produce a mean value of the gage factor.

Two types of LVDTs were available for the present study, having sensitivities and ranges of: 200 and 80 volts per inch and ranges of 0.050 and 0.125 inches respectively. The range of accurate measurements able to be captured using the selected data acquisition system are limited by two factors, the range of the LVDT sensors, and the signal to noise ratio. The range sets a physical maximum value and the signal (sensitivity) to noise ratio sets a practical minimum value. The range of the selected sensors was well known but the signal to noise ratio changed with each testing location. The noise level in the signal changed because the electromagnetic coil and the long lead wires connecting the sensors to the Daqbook were affected by different electromagnetic signals at each test area such as: electric or communication lines, cell phone signals, or any other electronic equipment. When selecting the members and locations to install sensors these practical limitations had to be considered to ensure the data collected was useable. To mitigate these problems members were selected that had the highest dynamic strains, this increased the signal amplitude with respect to the level of noise and therefore produced more
useable information. In the event that a particular test run resulted in a signal to noise ratio such that the signal was indistinguishable from the noise the data was removed from the present study.

4.2.2.2 Rubber Coated Tape Switches

Determining vehicle presence and speed played an integral part in the data collection effort. For this task four rubber coated tape switches were used; placed in the configuration shown in Figure 4-3. These switches were adhered to the roadway surface in the wheel path such that passing car tires would depress the switch completing an electrical circuit in the data acquisition system. A constant voltage was supplied to this circuit via a 9V battery producing a voltage time history for each switch with various spikes as each car tire passed over it. By placing switches at each end of the bridge it can easily be established when a vehicle enters and exits the bridge span. This information is useful in determining what potions of the recorded signal correspond to vehicle presence. By placing two sensors at each end of the bridge, spaced at a known distance (8 feet) the vehicle’s approach and exit speeds can be determined, and by knowing the length of the bridge and the time that the first wheel of the vehicle took to cross the bridge an average speed over the bridge can be calculated. This information is vital in determining the static loading frequency needed for the analytical program.
Analog to Digital Conversion

An analog to digital converter (A/D converter) is needed to transform the continuous analog signal provided by the sensors into a digital signal that is easily stored for future analysis. To accomplish this task an IOtech Daqbook 216 portable data acquisition system, capable of converting signals at 100 kilohertz and 16-bit resolution, was selected. Due to the high voltage requirements of the LVDTs, an external power supply was used to provide the required reference voltage of 15V to the LVDTs. Three IOtech DBK8 input modules were daisy chained to the Daqbook to increase the number of available channels. The Daqbook is then connected to a notebook PC in order to store and view the data using the IOtech data acquisition software, Daqview.

Whenever using an A/D converter, determining the correct sampling rate is a key decision in the testing procedure. A sampling rate must be chosen that is sufficient to capture the maximum values at the highest frequency observed in the data. The lower bound of sufficient
sampling rates is often referred to as the Nyquist rate, defined as twice the highest frequency of interest in the signal. For the purpose of the present study the frequencies of interest were the natural frequencies of the bridge and bridge components. The values of natural frequency for major contributing modes of highway bridges that fall in the length of the present study generally range from 1-10Hz. Preliminary calculations of member local natural frequencies for stiffer members such as bridge stringers and floor beams fell in the range of 20-40 Hz. Considering this range of interest and the need to ensure accurate maximum values in the data, a sampling rate of 200Hz was selected which corresponds to more than two times the Nyquist rate.

When determining the minimum time required for the test, or sampling duration, the lowest frequency of interest, or the static frequency, is the primary concern. The lowest static frequency is determined by the speed of the test vehicle divided by the length of the bridge. This value will change for each test but an extreme value can be calculated for each bridge by assuming a test speed of 5mph (8 kph) and dividing by the bridge length. These calculations provide a lower bound for the static period or the time for the vehicle to cross the bridge. Additional time to account for the free vibration period of the bridge should also be added. For the purposes of the present study an additional 10 seconds was added to the sampling duration to account for the free vibration period. From these calculations and assuming a maximum 300-foot bridge span a conservative sampling duration can be set at 30 seconds.

4.2.3 Member Selection

For each bridge tested, individual members to be instrumented were selected prior to the testing date. When selecting these members and sensor locations, several factors were considered: accessibility, signal to noise ratio, and the goal of the study.
The first of the member selection factors, accessibility, presents a practical constraint to member selection with regards to safety while instrumentation is being installed. Traffic will remain open during sensor installation removing the possibility of using a ladder or man-lift from the bridge deck to reach members. An additional obstacle to member accessibility is the depth of water below the bridge. Deep and or strong currents make the installation of sensors impractical and unsafe. Selecting members that can be accessed from the bank or shallow water below the bridge with a ladder, climbing equipment, or from the bridge deck without a ladder will satisfy these practical constrains and maintain safety during the testing procedure.

The second selection factor, signal to noise ratio, has already been discussed briefly and relates more to sensor placement on the member. To produce the most favorable signal to noise ratio, members and sensor placement were selected to locate sensors in the areas of the highest dynamic strains. In all bridges the members closest to the loading are likely to have higher dynamic strains. For the bridges selected, this meant instrumenting the stringers and floor beams. The sensors for these members were also located at approximately midspan corresponding with the maximum bending moment (assuming simple supports). For other selected members that act primarily under axial forces sensors were placed for accessibility while making sure to avoid stress concentrations at connection areas.

4.2.4 Controlled Test Vehicle Selection

Each bridge included in the study was tested under normal traffic loading as well as under controlled traffic loading. Controlled testing consisted of passing a known weight vehicle at known speed over the bridge. Controlled testing was required because many historic bridges are located on low volume traffic routes, making testing a bridge under normal truck traffic especially difficult and time consuming. The weight, type, and speeds of the control vehicle had
to be selected for each bridge test. The weight of the test vehicle was the primary concern in selection; for each bridge a vehicle at or near the weight limit of the bridge was selected. With this goal in mind a different type of truck had to be located for each test. For light bridges, a standard single axel dump truck was the vehicle of choice, but for heavier bridge weight limits, water tankers became a viable option. Specifics of the control vehicles used will be presented within the bridge test descriptions later in this chapter. Control vehicle speeds were selected starting with 5mph (8kph), increasing by 5mph (8kph) until either the speed limit was reached or the maximum practical truck speed was reached.

4.3 Modal Testing Procedure

To aid in the identification of natural frequencies, experimental modal analysis was carried out for each bridge. This consisted of exciting the bridge with a known forcing frequency while taking strain measurements. In doing this the system input and output are known and therefore a frequency response function (FRF) can be created to relate the input to the output. Viewing this FRF in the frequency domain makes the identification of natural frequencies possible. To describe this testing procedure requires a discussion of the data of interest and the data acquisition system used.

4.3.1 Data of Interest

Every modal testing program can be explained as the experimental development of a function to relate a known (supplied) input and a corresponding output. The focus of this modal testing will be supplying the selected bridge with a known and validated force time history and observing corresponding member strains. An electromagnetic shaker supplied with a varying analog voltage was used to produce the force time history that acted as the input. For each bridge tested, the response type (member strains) and the individual members to be instrumented had
been selected according to the traffic testing procedure. The output for the modal testing was obtained using these same members and sensor locations.

4.3.2 Data Acquisition System

The data acquisition system is comprised of two major sub-systems: input and output. Figure 4-4 presents the overall data acquisition system used for this portion of the field testing. Comparison of the output portion in Figure 4-4 and the output portion in Figure 4-1, shows that they are identical. This is intentional and beneficial because the natural frequencies identified in the modal testing will be used in the filtering the traffic data. By making the systems identical, unnecessary testing variables were removed. The input portion of the acquisition system consists of three key processes: analog signal creation, mechanical force generation, and force validation. These three input processes will be the focus of this portion of the thesis, for details on the output processes refer to the traffic testing procedure.
4.3.2.1 Analog Signal Creation

The selection of an electromagnetic shaker to generate the force time history necessitated the ability to produce and control a time varying analog output voltage. The amplitude and frequency of this output voltage determines the forcing function supplied to the bridge. To complete this task a National Instruments USB-6221 hardware package was used in conjunction with Labview as a controlling software package. A second piece of data acquisition hardware was required because the previously selected Daqbook 216 was incapable of producing an
analog output voltage. This combination of hardware and software was able to create an analog voltage range ±10V at any frequency required. A software package was developed to be able to produce sine waves at constant frequency and at linear varying frequencies also known as a chirp signal. This analog signal was then supplied to an amplifier to increase the signal voltage to the ±120V required to operate the electromagnetic shaker. The amplifier also served as a safety, “off”, switch for the shaker system as the voltage could quickly be reduced to zero in an emergency situation or if the shaker needed to be moved out of the way for traffic.

4.3.2.2 Mechanical Force Generation

To produce an accurate force time history an APS 400 Electro-Seis long stroke shaker was selected. This system is capable of producing controlled frequency force time histories in the range of 90 to 100 pounds (45kg). The shaker was placed on the bridge deck at midspan and used to excite the bridge in a vertical motion, emulating the primary direction of traffic loading. As can be seen in Figure 4-5 the shaker uses a set of rubber bands to suspend a large mass, then an electromagnetic coil forces the suspended mass up and down to create the time varying force. When using an electromagnetic shaker, care must be taken to tune the voltage range of the signal such that the mass will not strike the top or bottom of the apparatus. If this is not done damage to the shaker can occur as well as a change from harmonic excitation to impact excitation. Impact excitation can also be useful in determining a FRF but requires a much higher sampling frequency and therefore was not desirable for the present study. During testing the voltage level was controlled manually at the amplifier and adjusted such that impact would not occur ensuring harmonic excitation.
4.3.2.3 Force Validation

To connect the input and output halves of the data acquisition system an accelerometer was placed on the shaker and connected to the Daqbook 216. This allowed the forcing function to be validated and time stamped with the member strain output. A PCB Piezotronics 352A78 shear accelerometer was chosen because of the ease of connection with the existing Daqbook 216 and the ease of mounting on the shaker. This accelerometer was monitored using the same sampling frequency and duration as the LVDTs. Employing this system also eliminated any unforeseen variables in the analog signal or force generation such as friction or noise. During testing, the shaker was started on the bridge and tuned for the maximum amplitude before the suspended mass impacted the shaker frame. Then the system was given time to come to the steady state condition at such time the member strains and the forcing function were recorded.
using the Daqbook system. This process was repeated for a range of constant frequency signals (1-15 Hz) and several times with the chirp signal.

**4.4 Bridge Selection**

The focus of this research is the evaluation of DLF for historic Pratt and lenticular arch bridges. This goal drove the primary selection criteria as well as accessibility for instrumentation. Because of a lack of remaining bridge specimens, data comparison, and to limit the scope of the project, it was decided to select 1 lenticular arch bridge and 2 Pratt truss bridges. The term “historic bridge”, for the purposes of the present study, refers to a bridge with an original construction date between 1880 and 1910. This age range reflects a time period of great innovation in the structural field, the introduction of steel, and the age of bridges at the highest risk of replacement.

Investigation of the effects of length and natural frequency on DLF required bridges of various spans. Observation of remaining historic bridges suggests representative span lengths of approximately 100, 150, and 250 feet. The tested bridges will remain open throughout instrumentation and testing, therefore requiring relatively low average daily traffic (ADT) to ensure safety and to limit traffic interference with modal testing. Table 4-1 summarizes the bridges included in the present study.

Table 4-1: Selected Bridges for Testing

<table>
<thead>
<tr>
<th>#</th>
<th>Bridge</th>
<th>Location</th>
<th>Truss Type</th>
<th>Span</th>
<th>Year Built</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Pine Creek Bridge</td>
<td>Jersey Shore, PA</td>
<td>Lenticular Arch</td>
<td>287’-0” (88.4 m)</td>
<td>1889</td>
</tr>
<tr>
<td>2</td>
<td>Old Forge Bridge</td>
<td>Camp Hill, PA</td>
<td>Pratt Truss</td>
<td>113’-11” (34.7 m)</td>
<td>1887</td>
</tr>
<tr>
<td>3</td>
<td>Neff Bridge</td>
<td>Alexandria, PA</td>
<td>Pratt Truss</td>
<td>153’-0” (46.6 m)</td>
<td>1889</td>
</tr>
</tbody>
</table>
Bridge 1, Pine Creek Bridge, is a lenticular arch, through-truss bridge built by the Berlin Iron Bridge Company in 1889. This bridge carries River Road over Pine Creek approximately 1 mile Southwest of Jersey Shore, PA. At 287'-0" (88.4 m), this bridge was the longest tested and one of the longest known existing lenticular trusses built by the Berlin Iron Bridge Company. The bridge did not have a posted weight limit at the time of testing but the owner verbally communicated a weight limit of 20 tons. The Warren truss configuration connecting the top and bottoms chords also makes this bridge rare. In 1964 this bridge was rehabilitated; in 2008 it was disassembled and refurbished. Reconstruction was completed in 2011. For the current configuration, some connection details and member sizes have been altered to ensure occupant safety and constructability. Figure 4-6 is a picture of Pine Creek Bridge in late November 2011, post reconstruction.
The geometry and members selected for sensor installation at Bridge 1 are shown in Figure 4-7. Sensors were placed approximately at midspan for stringers and floor beams. In all other members the sensors were located for accessibility while ensuring placement far enough from stress concentrations to minimize local effects. Field testing for this bridge took place over a two-day period, one day for sensor installation and one day for data collection.

The control test vehicle for this bridge was a 3-axle, 2,500 gallon water tanker, weighing approximately 21 tons (209kN), supplied by the local fire department. Controlled testing consisted of 5 centered passes of the water truck over the bridge. The testing commenced with a 5 mph (8kph) test and each consecutive pass the speed was increased by 5mph (8kph) until at 25 mph (40.2 kph) the maximum practical speed was obtained. During testing it was observed that the largest deflections in the bridge occurred during the 20 mph (32.2 kph) test run. Forty-five other vehicles consisting of both trucks and passenger vehicles were observed during normal traffic testing.
Bridge 2, Old Forge Bridge, is a 113′-11″ (34.7 m) pin-connected, Pratt, through-truss bridge, located near Camp Hill Pennsylvania. This bridge was fabricated in 1887 by the Phoenix Bridge Company then erected by Dean & Westbrook. Compression members for this truss were constructed using Phoenix Columns, several wrought iron pieces riveted into a hollow round cross-section, which is a common signature of the Phoenix Bridge Company. This bridge was selected because of the representative length, age, and Pratt truss geometry. In 1975, the floor beams, stringers, and bridge deck were replaced with an updated design. Bridge 2 carries Sheepford Road over Yellow Breeches Creek, South of Camp Hill Pennsylvania. At the time of testing the bridge had a posted weight limit of 5 tons. Figure 4-8 is a picture of Old Forge Bridge taken in March 2012.
The geometry and members selected for sensor installation at Bridge 2 are shown in Figure 4-9. Sensors were placed at the approximate midspan for stringers and floor beams. In all other members the sensors were located for accessibility while ensuring placement far enough from stress concentrations to minimize local effects. Field testing for this bridge was accomplished in one day.

The control test vehicle for this bridge was a 2 axle dump truck, weighing approximately 5 tons (50 kN). Controlled testing consisted of 10 centered passes of the dump truck over the bridge. The testing commenced with a 5 mph (8 kph) test and every two passes the speed was increased by 5 mph (8 kph) until at 25 mph (40.2 kph) the maximum practical speed was obtained. During testing it was observed that the largest deflections in the bridge occurred during the 25 mph (40.2 kph) test run. 40 other pieces of traffic were observed during normal traffic testing.
4.4.3 Bridge 3

Bridge 3, Neff Bridge, is a 153'-0" (46.6 m) long, pin-connected, Pratt, through-truss bridge located near Alexandria, Pennsylvania. This bridge was originally fabricated in 1889 by the Pittsburgh Bridge Company. This bridge was recently (2011) rehabilitated, replacing the deck stringers and floor beams and increasing the top chord cross section. The new deck system consists of continuous stringers acting in composite action with a concrete deck. Currently, the bridge carries Alexandria Pike over the Frankstown Branch of the Juniata River. This bridge was selected because of the representative length, age, and Pratt truss geometry. At the time of testing the bridge had a posted weight limit of 20 tons. Figure 4-10 is a picture of Neff Bridge taken in October 2012.
Figure 4-11: Neff Bridge Sensor Map

The geometry and members selected for sensor installation at Bridge 3 are shown in Figure 4-11. Sensors were placed at the approximate midspan for stringers and floor beams. In all other members the sensors were located for accessibility while ensuring placement far enough from stress concentrations to minimize local effects. Field testing for this bridge was completed in one day.

The control test vehicle for this bridge was a 3 axle fire engine, weighing approximately 23 tons (229kN), supplied by the local fire department. Controlled testing consisted of 9 centered passes of the fire engine over the bridge. The testing commenced with a 5mph (8kph) test and every two passes the speed was increased by 5mph (8kph). The maximum speed obtained was approximately 30mph (48kph). During testing it was observed that the largest deflections in the bridge occurred during the 25mph (40.2kph) test run. 33 other pieces of traffic were observed during normal traffic testing.
4.5 Summary

The experimental process described in this chapter was used for each bridge specimen included in the present study. To complete the experimental program for this research three historic through-truss bridges were selected for testing. Sensors were installed on members of interest at each site and the bridges were excited under normal traffic, controlled traffic, and harmonic excitation. Strain time histories, vehicle speeds, and bridge geometry were collected as the primary data to be passed on to the analytical program.
5. Analytical Program

5.1 Introduction

After field-testing of the selected bridges was complete, the dynamic response data was processed to extract the DLFs for each strain time history. These calculated DLFs provide the basis for the comparative study to determine relationships between DLF and span length, vehicle gross weight, vehicle speed, and component peak static stress. The key factors in determining the DLF are the peak static response and the peak dynamic response. For the present study, DLFs were determined using low-pass digital filtering. The overall process used to produce the DLF is shown in Figure 5-1. The output obtained using this selected method relies on three key variables: static cutoff frequency, dynamic cutoff frequency, and filter type and order. This chapter will focus on the methods used to determine or select these values.

![Figure 5-1: Flowchart of Analytical Process](image-url)
5.2 Static Cutoff Frequency

The static response is obtained through digital filtering of the collected dynamic response. In this process a low pass digital filter is used to separate the low frequency static response from the higher frequency dynamic response. The primary concern in this process is the proper selection of the cutoff frequency, which sets the boundary between filtered and passed signals. The determination of this frequency will be accomplished through several numerical steps including: theoretical static frequency, static influence lines, and a power spectral density (PSD) of collected data.

5.2.1 Theoretical Static Frequency

The static frequency of a test response corresponds to the time the test vehicle was influencing a particular bridge member, which can be calculated by dividing the vehicle velocity by the influence length. For simple members such as a simply supported stringer the theoretical static frequency is just the vehicle velocity divided by the stringer span length. The data collected by the rubber coated tape switches placed at each end the bridge was used to determine the vehicle velocity (known time and distance) and field measurements of member geometry was used to determine span lengths. This sets a minimum value for the cutoff frequency, as filtering the signal below this frequency would begin to remove amplitude from the static response. Due to the time required for load transfer to occur and the small changes in vehicle velocity over the span length, the static response will not occur at a single frequency but will be clustered around the calculated static frequency. This concept is also hard to apply when global members which are not in direct contact with the loading are considered. For this reason the theoretical static frequency alone is insufficient to determine the cutoff frequency.
5.2.2 Static Influence Lines

Using static influence lines to determine the static cutoff frequency is very similar to the theoretical static frequency but removes the problems associated with members not in direct contact with the loading. The first step in this process was to produce the influence line for the member of interest with respect to a point load moving along the length of the bridge. Due to the nature of pin-connected trusses, influence lines consist of linear lines at various slopes. The static frequency of loading was then determined by dividing the vehicle speed by twice the length between local maximums and minimums. By selecting the shortest length between the local maximum and minimums in the influence line the highest static frequency was determined. This method still ignores the time required for load transfer and changes in vehicle speed and length and, therefore, is insufficient by itself to determine the static cutoff frequency.

5.2.3 Power Spectral Density

To experimentally determine the optimum location of the static cutoff frequency a power spectral density (PSD) was used to complete a frequency analysis of the field data. This method allows the time required for load transfer and any other unforeseen field variables such as boundary conditions of individual members to be included in the determination of the static cutoff frequency. A PSD provides the amplitudes of signals present at different frequencies. This relies on Fourier’s theory that any arbitrary function can be replicated with several sine wave functions at various frequencies and amplitudes. To compute the PSD a Fast Fourier Transform (FFT) can be used. The FFT is a computer algorithm that speeds the calculation of the PSD for large sets of data. For this research Matlab was used to conduct the FFT and produce the PSD for each set of data. The prominent frequencies present in a data set can be identified as large spikes on the PSD plot of the data. On a PSD plot the static portion of a signal is commonly referred to
as the DC spike and is located in the low frequency range. The highest frequency in the DC spike can be used to establish a minimum for the static cutoff frequency. This value can easily be determined graphically but due to the large amounts of data was determined numerically by finding the local minima in in the low frequency range.

Taking the maximum of the three minimum cutoff frequencies resulting from the PSD and the calculated theoretical static frequencies then approximated the optimum cutoff frequency. This selected frequency is then guaranteed to include all portions of the static response but remain below the dynamic and the majority of the noise frequencies.

5.3 Dynamic Cutoff Frequency

The dynamic response is obtained through digital filtering of the collected raw data. This is necessary to remove noise in the signal. In this process a low pass digital filter is used to separate the lower frequency dynamic response from the higher frequency noise. The primary concern in this process is the proper selection of the cutoff frequency, which sets the boundary between filtered and passed signals. To determine this cutoff frequency three numerical methods were employed: frequency response function of modal testing, theoretical modal frequencies, and power spectral density of traffic testing.

5.3.1 Frequency Response Function (FRF)

Experimentally determining modal frequencies is most commonly done with the aid of a frequency response function (FRF). This is a function that is determined using known input and output that displays the amplitude of the structure’s response over a range of frequencies. Local maxima in this function are considered natural or modal frequencies. For the present study the modal testing procedure outlined in the previous chapter was applied to each bridge resulting in a known input and output in the time domain for each bridge. For each set of input and output an
individual FRF was created. This process was completed by first using the FFT to transition to the complex frequency domain (frequency and phase). Then the complex conjugate was applied to create a PSD of each signal transitioning from the complex system to a real system of just frequencies. Finally the PSD of the output signals is normalized by divided by the PSD of the input signal resulting in a FRF. Then the local maximums of the FRF were numerically determined to identify the natural dynamic frequencies. This process is helpful in determining the dynamic cutoff frequency but insufficient because the type of excitation between the modal and traffic testing is different as well as the influence of noise on the system.

5.3.2 Theoretical Modal Frequencies

Basic dynamics and mechanics can be applied to derive and equation for vibration frequencies of individual components. Equation 5.1 shows the theoretical relationship of modal frequencies of a simple beam, with distributed mass $m$, as a function of geometric properties, material properties, and boundary conditions. Only one of these factors, geometric properties, was easy to determine in the field, for this reason a few assumptions had to be made. Material properties were assumed to be those of ASTM- A992 steel, likely, stiffer than the historic material in place. For boundary conditions two scenarios were investigated, pin-pin and fixed-fixed. Because of these uncertain assumptions and the inherent theoretical nature these calculations alone are not sufficient to establish the dynamic cutoff frequency. Table 5-1 shows a summary of the calculated theoretical natural frequencies for key components of stringers and floor beams. These local modes were focused on because flexural modes of the deck were those observed in the field during testing.
\[ \omega_1 = \pi^2 \sqrt{\frac{E*I}{m*L^4}} \]  

(5.1)

Table 5-1: Theoretical First Modal Frequencies (Hz)

<table>
<thead>
<tr>
<th>Bridge #</th>
<th>Member</th>
<th>Pin-Pin</th>
<th>Fixed-Fixed</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Stringer</td>
<td>11.6</td>
<td>26.2</td>
</tr>
<tr>
<td></td>
<td>Floor Beam</td>
<td>22.5</td>
<td>51.1</td>
</tr>
<tr>
<td>2</td>
<td>Stringer</td>
<td>32.2</td>
<td>73.1</td>
</tr>
<tr>
<td></td>
<td>Floor Beam</td>
<td>29.6</td>
<td>67.2</td>
</tr>
<tr>
<td>3</td>
<td>Stringer</td>
<td>9.6</td>
<td>21.9</td>
</tr>
<tr>
<td></td>
<td>Floor Beam</td>
<td>18.0</td>
<td>40.8</td>
</tr>
</tbody>
</table>

5.3.3 Power Spectral Density of Traffic Data

To experimentally approximate the optimum location of the dynamic cutoff frequency a PSD was used to complete a frequency analysis of the field data. This method includes every experimental variable and only relies on the quality of the data collected. To compute the PSD the same method outlined in the static cutoff frequency section was used. The prominent frequencies present in a data set can be identified as large spikes on the PSD plot of the data. With this step complete, the frequencies determined from the modal testing FRF and those from the theoretical modal frequencies could be compared to those observed in the PSD to identify the portions of the signal that were predominately noise and needed to be filtered out.
The optimum cutoff frequency was then approximated by finding the next highest local minimum in the PSD above the maximum of the previously determined cutoff frequencies resulting from the FRF and the calculated theoretical modal frequencies. This selected frequency is then guaranteed to include nearly all portions of the dynamic response but remain below the majority of the noise frequencies.

5.4 Digital Filtering

The performance of a digital filter is controlled by three factors: the type, order, and the direction of filtering. Many types of digital filter were considered during the selection process but ultimately a high order Butterworth filter was selected for the present study because of the particular requirements of the filter. These requirements include a smooth near ideal passband that limits distortion of the remaining signal, and a narrow transition band allowing very accurate cutoff frequencies to be selected.

The order of a digital filter controls the slope of the transfer function around the cutoff frequency. A higher order is preferable as it corresponds to a sharper transition between unfiltered and filtered frequencies. Figure 5-2 presents the transfer function of two different low-pass digital filters. These filters have the same cutoff frequency of 250 Hz but different orders. The plot illustrates that as order increases the filter behaves more like the ideal or theoretical value. In the present study the static frequency and the first natural frequency were close to the same frequency. For this reason a 10th order filter was selected to ensure the transfer function is sufficiently steep to separate these close frequencies without removing a portion of the remaining signal.
Figure 5-2: Transfer Functions of Low-Pass Digital Filters

The direction of filtering refers to the order that the digital signal is fed into the registers of the digital filter. A signal can be filtered forward, backward, or both. The direction of the filtering can distort the phase of the remaining signal. Forward filtering creates a phase lag and backward filtering creates a phase advance. For the present study the filtering was carried out in both directions to minimize this effect. Filtering in both directions also has the added benefit of doubling the effective order of the filtering function. In this manor a much quicker fifth order filtering function can be used to the same effect as a more arduous tenth order filtering function. Figure 5-3 shows a representative static and dynamic response of a floor beam from bridge 2 after the digital filter has been applied.
5.5 DLF Calculation

DLFs will be calculated for each corresponding set of static and dynamic strain time histories obtained through the digital filtering process, using equation 5.2. The purpose of the DLF is to scale the maximum static response to obtain the maximum dynamic response irrespective of time. Therefore maximum response values will be selected without regard to occurrence time, this concept is clearly shown in Figure 5-3 as the maximum static response occurs approximately 0.2 seconds before the maximum dynamic response.

\[
DLF = 1 + \frac{Max_{\text{dyn}} - Max_{\text{static}}}{Max_{\text{static}}}
\]  

(5.2)
5.6 Summary

The analytical process described in this chapter was used to produce DLFs for each strain time history collected during field testing. This consisted of using a digital filter to obtain the static and dynamic response from the raw data; requiring carefully selected cutoff frequencies. DLFs were then calculated based on the maximum static and dynamic responses observed in each strain time history. This process was implemented using an algorithm developed in Matlab and included in Appendix A. The DLFs obtained in this process will be used in a comparative study to determine key factors affecting the magnitude of DLFs.
6. Results and Comparative Study

6.1 Introduction

A comparative study of the DLFs produced during the analytical program was required to examine both the magnitude of the observed DLF and relationships between the magnitude and experimental variables. The magnitude of the observed DLF was evaluated by comparison with values determined using existing codified recommendations. Plots and histograms of the data as a function of the variable of interest were used to measure correlations indicative of relationship strengths between DLF and variables. The variables of interest in determining possible relationships are: member peak static strain, vehicle speed, vehicle static weight, and bridge length.

6.2 DLF Magnitude

As was observed in the results published by other studies, and the results of the current study the magnitude of DLF is highly variable. The present study collected a sample size of 419 individual DLFs with a maximum observed magnitude of 1.95, 50th percentile of 1.19, and 95th percentile of 1.49. Figure 6-1 presents the distribution of the magnitude of the observed DLF from the combined experimental results of all three bridges. Interval size for DLF range was chosen to limit the number of ranges with less than 3 data points. It can be observed that this distribution resembles a positively skewed distribution. Comparison of experimentally derived data to the value specified by the AASHTO LRFD Bridge Design Specifications 6th edition indicates that 82% of the observed sample falls below the specified DLF of 1.33 (AASHTO 2012). Although some experimentally derived DLFs are large, the values observed are of a similar order of magnitude to that currently specified by AASHTO.
In Figure 6-2 the DLF magnitude distribution is presented for each bridge independently. It is important to examine the data in this form to observe differences in the distribution between bridge types that indicate a difference in behavior. With the samples separated it can be observed that the positively skewed distribution is evident for Bridge 2 and Bridge 3 DLF distributions but is not as well defined for the Bridge 1 DLF distribution. This difference in distribution could be reflecting a difference in behavior between lenticular and Pratt trusses or a change in loading. This is a very difficult assertion to defend due to the limited sample size from Bridge 1. This limited sample size was caused by a combination of higher than expected noise levels and stiffer than expected members; effectively corrupting every normal traffic signal with high levels of noise. Therefore to definitively determine that these truss types behave differently further testing would be required.
Figure 6-2: DLF Magnitude Distribution

a) Bridge 1 (24 Samples)

b) Bridge 2 (342 Samples)

c) Bridge 3 (53 Samples)
Table 6-1 presents the sample size, the observed 95\textsuperscript{th} percentile, the 2012 AASHTO codified value, and the percentile of the observed distributions that corresponds with the 2012 AASHTO code value. These values clearly show that the 95\textsuperscript{th} percentile for each bridge tested was well above the code value; also as sample size increases the percentile corresponding to the AASHTO code increases. This second observation implies that at some very large sample size the 95\textsuperscript{th} percentile of the observed data could correspond with the AASHTO code value.

Extending this observation shows that the overall larger DLFs produced could be an artificial product of a small sample size. Although without further testing to increase sample size it has to be concluded that overall the historic through-truss bridges tested under normal traffic conditions responded with DLFs considerably higher than those suggested by the current AASHTO specifications.

### Table 6-1: Comparison of DLF Magnitudes to AASHTO Values

<table>
<thead>
<tr>
<th>Bridge #</th>
<th>Sample Size</th>
<th>Observed 95\textsuperscript{th}</th>
<th>AASHTO 2012</th>
<th>AASHTO Percentile</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>24</td>
<td>1.43</td>
<td>1.33</td>
<td>79\textsuperscript{th}</td>
</tr>
<tr>
<td>2</td>
<td>342</td>
<td>1.49</td>
<td>1.33</td>
<td>82\textsuperscript{nd}</td>
</tr>
<tr>
<td>3</td>
<td>53</td>
<td>1.47</td>
<td>1.33</td>
<td>81\textsuperscript{st}</td>
</tr>
</tbody>
</table>
In Figure 6-3 the DLF magnitude distribution is presented for global members such as truss chords, portal members, and truss diagonals at service level loading (controlled traffic testing). It is important to examine the data in this form to observe differences in the distribution when low level loading from light passenger vehicles has been removed from the samples. The exclusion of light vehicle loading follows the logic of the AASHTO specification, were design and evaluation loads correspond to heavy truck traffic and extreme events. Figure 6-3 a, presents the DLF distribution for the top chord, bottom chord, portal and diagonal members of Bridge 2 and Bridge 3 obtained from the controlled testing portion of the experimental program. High levels of noise combined with low strain levels necessitated the exclusion of data from Bridge 1 global members. The two bridges included were loaded at or above their posted weight limits during controlled testing and therefore produced DLFs correspond to service level loading. To further eliminate low level loading from the samples minimum static strain levels of 50, and 100 microstrain were applied to the samples. The distributions corresponding to these criteria are presented in Figure 6-3 b and c. With the samples separated in this manner it can be seen that the positively skewed distribution is evident in all of the samples and is accentuated in the samples with higher static strain levels. This clearly shows that high DLF values correspond to low levels of loading which are of little concern during the bridge design or evaluation process. For this reason the lower values for DLF determined by the distributions corresponding to high level loading are more appropriate for the evaluation of historic through truss bridges.
a) Maximum Static Strain Greater than 0 Microstrain (65 samples)

b) Maximum Static Strain Greater than 50 Microstrain (40 samples)

c) Maximum Static Strain Greater than 100 Microstrain (21 samples)

Figure 6-3: DLF Magnitude Distribution for Global Members at Service Level Loading
Table 6-2 presents the sample size, the observed 95th percentile, the 2012 AASHTO codified value, and the percentile of the observed distributions that corresponds with the 2012 AASHTO code values from the DLF distributions corresponding to service level loading. These values clearly show that the 95th percentile of the DLF magnitude distribution was still above the code specified value but as minimum static strain increases the observed 95th percentile decreases and the percentile corresponding to the AASHTO code increases. These observations imply that at higher levels of service loading such as a load corresponding to a factored load combination the 95th percentile of the observed data would correspond with the AASHTO code value. Although without further testing to increase sample size and static strain levels it has to be concluded that the historic through-truss bridges loaded at their current posted weight limits responded with DLFs slightly higher than those suggested by the current AASHTO specifications.

Table 6-2: Comparison Global Member DLF at Service Loading to AASHTO

<table>
<thead>
<tr>
<th>Microstrain Limit</th>
<th>Sample Size</th>
<th>Observed 95th</th>
<th>AASHTO 2012</th>
<th>AASHTO Percentile</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>65</td>
<td>1.49</td>
<td>1.33</td>
<td>74th</td>
</tr>
<tr>
<td>50</td>
<td>40</td>
<td>1.40</td>
<td>1.33</td>
<td>90th</td>
</tr>
<tr>
<td>100</td>
<td>21</td>
<td>1.36</td>
<td>1.33</td>
<td>91st</td>
</tr>
</tbody>
</table>
6.3 DLF and Maximum Static Strain

A negative correlation between maximum static strain and DLF is the strongest correlation observed by several previous studies, and therefore will be presented first in the present study. To investigate this correlation, observed values of DLF were plotted as a function of the maximum static strain obtained through digital filtering in the analytical program. The scatter plot in Figure 6-4, including all bridges at all load levels, presents a general decrease in the magnitude of DLF as the maximum static strain increases supported by a Pearson’s correlation coefficient of -0.367. The Pearson’s correlation coefficient is a statistical value that ranges from +1 to -1 that is used to quantify the level of linear dependence between two variables. To further investigate this negative correlation, Figure 6-5 was created to condense and clarify the information presented in the scatter plot. The average DLF magnitudes are shown for each range of maximum static stress and the standard deviation is depicted using the error bars. This histogram establishes a clear negative relationship between the maximum static strain and DLF magnitude. The sudden increase in DLF and variability observed in the final two ranges presented can be attributed to a low sample size; including only DLFs from one vehicle crossing each. Figure 6-6 investigates the observed trend in each of the three bridges individually and yields supporting evidence for the same relationship. The large sample size of Bridge 2 is the best representation of this relationship, with the smaller sample sizes of Bridge 1 and Bridge 3 in general agreement. Explaining this trend involves examining the definition of the DLF, the calculation of DLF involves dividing a value by the maximum static response therefore high values can be produced if this response tends to zero. Putting these results in perspective, the higher levels of static strain scaled by the lower DLF produce higher dynamic strain than the
lower static strains scaled by the higher DLF. This correlation suggests that at higher levels of loading it is reasonable to use a lower DLF than would be observed from low level loading.

To further investigate DLF caused by high level loading, DLFs obtained from global members during controlled traffic testing of Bridge 2 and Bridge 3 were plotted as a function of maximum static strain and are shown in Figures 6-7 and 6-8. These values more closely correspond to service level loading but it can be seen that low maximum static strain values are still present in the data. The overall trend observed in the complete sample is further supported by these figures as higher levels of static strain produce much lower DLF. The data presented in Figure 6-7 produces a correlation coefficient of -0.496 demonstrating a stronger relationship than when low level loading is included.
Figure 6-4: DLF versus Maximum Static Strain (All Samples)

Figure 6-5: Histogram of DLF versus Max Static Strain (All Samples)
Figure 6-6: Histogram of DLF versus Max Static Strain

a) Bridge 1

b) Bridge 2

c) Bridge 3
Figure 6-7: DLF for Global Members at Service Loading versus Maximum Static Strain

Figure 6-8: Histogram of DLF for Global Members at Service Load versus Max Static Strain
6.4 DLF and Vehicle Speed

By evaluating the static frequency of loading and first modal frequency it can be observed that, at a particular speed, the two frequencies coincide and theoretically result in resonance or a DLF of infinity. The amount of energy introduced into the system by a vehicle moving at a higher speed also suggests that increased speed could increase DLF. For these reasons, the correlation between vehicle speed and DLF magnitude was investigated using the procedure established in the previous section. A scatter plot of DLF against vehicle speed is shown in Figure 6-6. From the scatter plot it is difficult to visually establish a trend between the vehicle speed and DLF magnitude. This is supported by a correlation coefficient of 0.004. However when the sample is resolved into the histogram format, as can be seen in Figure 6-7 a relationship can be observed. The magnitude of DLF first increases with speed until 20mph (32kph) then decreases with further increase of speed. The histograms for individual bridges in Figure 6-8 also support this observation. During field testing this pattern was qualitatively observed through visual observations of bridge deflections. This observed trend is most likely caused by similar static loading frequency and a low modal frequency coinciding when the vehicle speed is approximately 20mph (32kph). When these two frequencies coincide the magnification of the response is limited because only one period of the static loading frequency is observed for each vehicle crossing, which explains the weakness of this relationship. The optimization of vehicle suspensions for higher highway speeds provides a secondary hypothesis. This would mean at very low speeds the pseudo static condition will be approximated and at relatively high speeds the vehicle suspension will dampen the dynamic component, but between these ranges the DLF would be larger. More testing, including a method for determining vehicle suspension characteristics, would be required to investigate this second hypothesis.
Figure 6-9: DLF versus Vehicle Speed (All Samples)

Figure 6-10: Histogram of DLF versus Vehicle Speed (All Samples)
Figure 6-11: DLF versus Vehicle Speed
6.5 DLF and Vehicle Static Weight

Investigation of the correlation between the vehicle static weight and DLF magnitude was limited by the ability to determine the static weight of vehicles passing over the bridge. For this reason the comparison was limited to four known vehicle static weights, three from the control test vehicles weighing 10, 42 and 46 kips (44, 187, and 205kN), and one from a personal vehicle weighing 5kips (22kN). From the scatter plot in Figure 6-9 the only conclusion that can be drawn is that the lowest vehicle static weight produced the highest DLFs. The correlation coefficient for this data is -0.013 indicating nearly zero correlation. Figure 6-10, was used to further investigate the possibility of a relationship but the histogram does not visually support a clear relationship between DLF and vehicle static weight. It does however suggest a possible relationship with the type of vehicle as it can be seen that the light passenger vehicle produced both the largest and the most variable DLFs while the heavy commercial vehicles produced lower and less variable DLFs. This suggests that the type of vehicle suspension is likely a factor that DLF magnitude is dependent on. This is supported by the fact that both the 5kip and 10kip vehicles were passed over the same bridge and resulted in very different magnitudes of DLF, but a further increase in vehicle weight at other bridges seemed to have no effect. Investigation of this hypothesis would require further field research because vehicle suspension type was not recorded during the present testing.
Figure 6-12: DLF versus Vehicle Static Weight

Figure 6-13: Histogram of DLF versus Vehicle Static Weight
6.6 DLF and Bridge Span

The most general variable in the behavior and design of a bridge is span length. Also many specifications that include equations for DLF are based on the span length of the bridge. For these reasons the correlation between DLF magnitude and span length was investigated. The scatter plot in Figure 6-9 shows the DLF magnitude plotted as a function of span length. This plot shows that the shortest bridge produced both the largest and the most variable DLFs while both the variability and magnitude decreased with increasing bridge length. Although this trend can be observed, the calculated correlation coefficient for this data was 0.011 indicated zero correlation. In the histogram format presented in Figure 6-10, a relationship is observed as it can clearly be seen that the DLF magnitude gradually decreases as the bridge length increases. This pattern agrees with the negative correlation expected at the outset of the present study Assuming a fixed span to depth ratio, the material volume required to construct a bridge increases exponentially as it gets longer and therefore the dead weight of the bridge grows dramatically. This affects the dynamic response of the bridge because the initial vertical acceleration that a vehicle carries onto the bridge must be distributed between the mass of the vehicle and the mass of bridge; therefore as the mass of the bridge becomes much larger than the mass of the vehicle the dynamic portion of the response is reduced. Applying this reasoning to the tested specimens can help to explain the observed relationship and the difference in the rate of change along the plot in Figure 6-9. The shortest bridge in the study was by far the lightest and the other two bridges were of comparable weight due to the medium length bridge having a concrete deck and the long span having an open grate deck. Therefore the differences in deck type counteracted the length effects and produced DLFs of similar magnitude and distribution for the second two bridges.
Figure 6-14: DLF versus Span Length

Figure 6-15: Histogram of DLF versus Span Length
Table 6-2 presents a comparison of the 95th percentile of the sample of DLF from each bridge and the corresponding values obtained for that bridge using the length varying equations previously presented in chapter 2; the results are presented in order of increasing span. The observed 95th percentile of DLF is greater than every codified value except that of the historic reference. This is likely a function of the bridge types and live loading assumed for each of these references. For the contemporary references the assumed bridge type is a slab on girder system with truck loading being the design live load. In the historic reference the assumed bridge type is a through truss and the live load was heavy railcars. The historic assumptions more closely resemble the live to dead load ratio of the bridge specimens selected because the dead load of the bridge is much closer to the live load magnitude. In contrast in contemporary structures the dead load is generally a higher portion of the total load therefore exhibiting lower DLF magnitudes.

Table 6-3: Comparison of DLF Magnitudes to Length Varying Codified Values

<table>
<thead>
<tr>
<th>Bridge #</th>
<th>Observed 95th</th>
<th>AASHTO 1992</th>
<th>Japan</th>
<th>France</th>
<th>Merriman and Jacoby</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1.49</td>
<td>1.21</td>
<td>1.24</td>
<td>1.10</td>
<td>1.72</td>
</tr>
<tr>
<td>3</td>
<td>1.47</td>
<td>1.18</td>
<td>1.21</td>
<td>1.08</td>
<td>1.66</td>
</tr>
<tr>
<td>1</td>
<td>1.43</td>
<td>1.12</td>
<td>1.15</td>
<td>1.04</td>
<td>1.51</td>
</tr>
</tbody>
</table>
6.7 Summary

In this chapter the DLF distribution and test variable dependence was investigated through the implementation of scatter plots, correlation coefficients, and histograms. Overall the distribution was observed to be a positively skewed distribution centered on 1.18 and having a 95th percentile of 1.49. The calculated correlation coefficients and the observed trends are listed in Table 6-3. The test variable that seemed to have the biggest influence on DLF was the maximum static strain, followed by bridge span length. For other test variables it was hard to determine a definite correlation or observed trend. Also it was observed that vehicle suspension type could play a larger role than expected in the behavior of the bridge vehicle system. In general larger sample sizes would be required to statistically validate the correlation coefficients and trends presented and ultimately to determine if historic trusses exhibit higher DLFs (1.49) than currently specified (1.33).

Table 6-4: Test Variables and Observed Correlations

<table>
<thead>
<tr>
<th>Variable</th>
<th>Correlation Coefficient</th>
<th>Observed Trend</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Static Strain</td>
<td>-0.367</td>
<td>Negative</td>
</tr>
<tr>
<td>Vehicle Speed</td>
<td>0.004</td>
<td>Weak Parabolic</td>
</tr>
<tr>
<td>Vehicle Static Weight</td>
<td>-0.013</td>
<td>Weak Negative</td>
</tr>
<tr>
<td>Span Length</td>
<td>0.011</td>
<td>Negative</td>
</tr>
</tbody>
</table>
7. Conclusions

7.1 Introduction

Historic through-truss bridges are an important and beautiful chapter in the story of American structural history that must be preserved for future generations to enjoy and appreciate. Many of these bridges are reaching the age of rehabilitation or demolition and, therefore, are being evaluated using contemporary design philosophies, namely dynamic amplification of static loads (DLF), that may not capture their complex dynamic behavior. The specifications used for design and evaluation have been developed or targeted to apply to modern girder construction practices, which, because of inherent mass and stiffness differences, behave much differently under dynamic loading than historic structures. For this reason the present study investigated the magnitude and possible influencing variables for DLF in historic through truss bridges. This was accomplished through field testing of existing historic bridges still open to traffic. Dynamic member strains were collected at each bridge specimen during normal and controlled traffic conditions to capture that natural variability of traffic loading. Digital filtering was then used to produce DLFs for each member and vehicle crossing. Scatter plots and histograms were then produced to investigate the distribution of DLF magnitude, possible correlations, and observable trends with respect to test variables. The present study considered the following test variables to have a possible influence on the magnitude of DLF: maximum static strain, vehicle speed, vehicle static weight, and bridge span length. Conclusions and future research recommendations that are supported by the collected data are presented in the following sections.
7.2 Summary

The results of the present study support several conclusions:

1) The 95th percentiles of the recorded DLF magnitude samples at each bridge under normal traffic loading are higher than the AASHTO specified 1.33. This suggests that historic through-truss bridges behave differently under dynamic loading than slab on girder structures and it may be advisable to use an increased DLF in the order of 1.49 for the evaluation and rehabilitation of these structures under normal traffic conditions.

2) The 95th percentiles of the recorded DLF magnitude samples under service level loading are only slightly higher than the AASHTO specified 1.33. This suggests that historic through-truss bridges under high level loading behave more like slab on girder structures but it still may be advisable to use an increased DLF in the order of 1.36 for the evaluation and rehabilitation of these structures under service level loading.

3) Normal traffic and service level loading conditions produce a positively skewed DLF distribution. Comparison of individual bridge DLF distributions indicates that as sample size increases, the 95th percentile decreases. Comparison of global member DLF distributions indicates that as static strain level increases, the 95th percentile decreases.

4) DLF magnitude is most strongly correlated with member maximum static stress with a Pearson’s correlation coefficient of -0.367 for normal traffic conditions and -0.496 for service level loading. This trend was further observed in the histograms for the entire sample, individual bridges, and service level loading.

5) For individual bridges, DLF magnitude is inversely related to vehicle static weight, but when expanded to all bridges, this correlation is not supported resulting in a correlation coefficient of -0.013. The trend observed for individual bridges is similar to the
dependence of DLF magnitude on maximum static strain because a heavier vehicle will usually produce higher static strains when placed on the same bridge as a lighter vehicle. The low correlation coefficient is explained when comparing Bridge 2 with the much stiffer Bridge 3 controlled vehicle tests, which resulted in similar static strains and DLF values with a 5-ton (25kN) truck and a 23-ton (229kN) truck respectively.

6) High values of DLF correspond with low level loading while low values of DLF correspond to high level loading. This observation aligns well with the trend observed in maximum static strain but can also be observed in the vehicle static weight trends. At Bridge 2, a 2.5-ton (25kN) passenger vehicle and a 5-ton (50kN) commercial vehicle were both passed over the 5-ton (50kN) weight limit bridge; the lighter vehicle produced DLFs with greater magnitude and variability than the heavy vehicle. Considering this fact it may be appropriate to use a lower percentile than the 95th when evaluating structures under high loading situations. This is suggested because the values presented in the current study include many low level loading situations.

7) DLF magnitude has a negligible correlation with span length, resulting in a correlation coefficient of 0.011. Although a negative trend is supported by the histogram clearly showing a decreasing value of DLF magnitude as span length increases. From the data collected in the present study, it is impossible to determine whether this trend is caused by a fundamental behavior different between bridges of different length such as, increased bridge mass or lower 1st natural frequency, or if it could be caused by a low number of samples.

8) DLF magnitude does not appear to be dependent on truss type. Two truss types were selected for bridge specimens, and although different truss types of the same span were
not tested with the same control vehicle, this conclusion is supported because the two bridges of similar geometry produced widely different DLF distributions. The collected data suggests that the overall bridge mass and stiffness, which may be related to truss type, are more responsible for changes in DLF magnitude.

9) Although DLF magnitude was observed to have a parabolic relationship to vehicle speed, the correlation coefficient of 0.004 does not support its inclusion as a variable of interest in design or evaluation of DLF. Besides this fact, vehicle speed is a difficult variable to control on existing structures and is often unknown during the design of new structures; therefore DLF should be applicable to all possible vehicle speeds to ensure safety.

10) It can be concluded that the most influential variable in determining DLF magnitude is maximum static strain because it had the strongest correlation coefficient and observed trend.

11) At current posted weight limit loading maximum static strain was less than 200 microstrain for all sensored members in the three bridges tested.

12) Digital filtering is an effective method for producing the static response of a component from a collected dynamic signal and therefore effective at producing accurate DLFs.
7.3 Recommendations for Future Research

In future efforts to study DLF, it would be highly beneficial to increase the sample size of traffic and bridge specimens in order to better explain this complex behavior. Actions that could have been taken to increase the value of the data collected for the present study include: using strain gages or other sensor types less susceptible to noise, investigating the structures pre and post rehabilitation, using a variety of control vehicles with known suspension characteristics as well as vehicle weight, and implementing a weigh-in-motion system to better characterize the normal traffic conditions present during testing. Furthermore, to investigate the differences in the behavior of historic through-truss bridges and slab on girder bridges, data will need to be collected from a variety of modern structures, using the same testing procedure as was developed for historic structures.
References


Appendix A: Selected Strain Time Histories

Figure A-1: Strain time history Old Forge Bridge floor beam

Figure A-2: Strain time history Old Forge Bridge stringer
Figure A-3: Strain time history Old Forge Bridge stringer

Figure A-4: Strain time history Old Forge Bridge stringer
Figure A-5: Strain time history Old Forge Bridge stringer

Figure A-6: Strain time history Old Forge Bridge bottom chord
Figure A-7: Strain time history Old Forge Bridge floor beam
Appendix B: Selected Power Spectral Densities

Figure B-1: Power Spectral Density Old Forge Bridge floor beam

Figure B-2: Power Spectral Density Old Forge Bridge stringer
Figure B-3: Power Spectral Density Old Forge Bridge stringer

Figure B-4: Power Spectral Density Old Forge Bridge stringer
Figure B-5: Power Spectral Density Old Forge Bridge stringer

Figure B-6: Power Spectral Density Old Forge Bridge bottom chord
Figure B-7: Power Spectral Density Old Forge Bridge floor beam
Appendix C
Matlab Code from Analytical Program

clc;
clear all;
format long;

%Sampling Constants
Fs=200;
Ps=1/Fs;
Bridge_Length=70;
Strip_Distance=8;

%Import Data to Structured Array from TXT.
numfiles=50;
umchan=7;
Length=3000;
trafficdata=cell(1,numfiles);
for i=1:numfiles
    myfilename=sprintf('Traffic00%d.TXT',i);
    trafficdata{i} = importdata(myfilename,'\t',2);
end

%Create Matrix of Raw Data
raw_data_1= zeros([numfiles,16,Length]);
for i= 1:numfiles
    for j=2:16
        if j==2||j==3||j==4||j==5||j==6||j==7||j==8
            raw_data_1(i,j-1,:)=trafficdata{i}.data(:,j);
        elseif j>13
            raw_data_1(i,j-1,:)=trafficdata{i}.data(:,j);
        end
    end
end

%Create Time Vector
time=zeros([1 Length]);
for i=1:Length-1
    time(i+1)=time(i)+Ps;
end

%Extract Data While Traffic on Bridge
Speed_Strip_1=abs(raw_data_1(:,13,:));
Speed_Strip_2=abs(raw_data_1(:,14,:));
Speed_Strip_3=abs(raw_data_1(:,15,:));
Start_Data=zeros(numfiles,1);
End_Data=zeros(numfiles,1);
Avg_Speed=zeros(numfiles,1); %fps
for i=1:numfiles
    [pks1,locsl]=findpeaks(Speed_Strip_1(i,:),'minpeakheight',8);
    [pks2,locs2]=findpeaks(Speed_Strip_2(i,:),'minpeakheight',8);
    [pks3,locs3]=findpeaks(Speed_Strip_3(i,:),'minpeakheight',8);
if isempty(locs1)+isempty(locs2)+isempty(locs3)>2
    Start_Data(i)=1;
    End_Data(i)=Length;
    Avg_Speed(i)=7.333;
elseif isempty(locs1)+isempty(locs2)>1
    Start_Data(i)=min(locs3);
    End_Data(i)=Length;
    Avg_Speed(i)=25;
elseif isempty(locs2)>0
    Start_Data(i)=min(locs3);
    End_Data(i)=Length;
    Avg_Speed(i)=7.333;
elseif isempty(locs3)>0
    Start_Data(i)=min(min(locs1),min(locs3));
    End_Data(i)=max(max(locs1),max(locs3));
    Avg_Speed(i)=(Bridge_Length+Strip_Distance)./abs((time(min(locs1))-
    time(min(locs3))));
elseif isempty(locs1)>0
    Start_Data(i)=min(min(locs2),min(locs3));
    End_Data(i)=max(max(locs2),max(locs3));
    Avg_Speed(i)=Strip_Distance./abs(time(min(locs1))-
    time(min(locs2)));
else
    Start_Data(i)=min(min(locs1),min(locs2),min(locs3));
    End_Data(i)=max(max(locs1),max(locs2),max(locs3));
    Avg_Speed(i)=Bridge_Length./abs(time(min(locs2))-
    time(min(locs3)));
end
end
raw_data=zeros(numfiles,numchan,Length);
for i=1:numfiles
    k=1+End_Data(i)-Start_Data(i);
    raw_data(i,:,1:k)=raw_data_1(i,1:numchan,Start_Data(i):End_Data(i));
%     figure(i)
%     plot(time(1:k),squeeze(raw_data(i,6,1:k)))
end
% Scale Raw Voltages to create Strains
for i=1:numfiles
    for j=1:numchan
        if j==1
            raw_data(i,j,:)=raw_data(i,j,:)/208.765/3.7;
        elseif j==2
            raw_data(i,j,:)=raw_data(i,j,:)/208.126/3.54;
        elseif j==3
            raw_data(i,j,:)=raw_data(i,j,:)/208.425/3.43;
        elseif j==4
            raw_data(i,j,:)=raw_data(i,j,:)/205.726/3.5;
        elseif j==5
            raw_data(i,j,:)=raw_data(i,j,:)/206.928/3.5;
        elseif j==6
            raw_data(i,j,:)=raw_data(i,j,:)/206.802/3.31;
        elseif j==7
            raw_data(i,j,:)=raw_data(i,j,:)/207.988/3.46;
        elseif j==8
            raw_data(i,j,:)=raw_data(i,j,:)/82.713/3.74;
        elseif j==9
            raw_data(i,j,:)=raw_data(i,j,:)/83.105/3.54;
        elseif j==10
            raw_data(i,j,:)=raw_data(i,j,:)/83.105/3.54;
        end
    end
end
raw_data(i,j,:)=raw_data(i,j,:)./82.981./3.50;
elseif j==11
raw_data(i,j,:)=raw_data(i,j,:)./82.883./3.54;
end
end

end

%Determine Global and Local Static Frequencies and Natural Frequencies

Glob_Static_F=zeros(numfiles,1); %Hz
Local_Static_F=zeros(numfiles,numchan); %Hz
Local_Dynamic_F=zeros(numfiles,numchan); %Hz
for i=1:numfiles
    Glob_Static_F(i)=Avg_Speed(i)./Bridge_Length;
    for j=1:numchan
        if (j==2||j==3||j==4||j==5)
            member_length=10;
            Local_Dynamic_F(i,j)=33;
        elseif (j==1||j==7)
            member_length=10;
            Local_Dynamic_F(i,j)=30;
        else
            member_length=10;%Bridge_Length;
            Local_Dynamic_F(i,j)=25;
        end
        if Avg_Speed(i)/member_length<2
            Local_Static_F(i,j)=2;
        elseif Avg_Speed(i)/member_length>4.95
            Local_Static_F(i,j)=4.95;
        else
            Local_Static_F(i,j)=Avg_Speed(i)./member_length;
        end
    end
end

%Zero Data (Remove Offset Voltages)
zero_data= zeros([numfiles,numchan,Length]);
for i=1:numfiles
    for j=1:numchan
        avg=mean(raw_data(i,j,1:200));
        k=End_Data(i)\-Start_Data(i)+1;
        zero_data(i,j,1:k)=raw_data(i,j,1:k)-avg;
    end
end
%     hold on
%       figure(i)
%       hold on
%       plot(ti
me,squeeze(zero_data(i,4,:)),'-b')
end

%Frequency Analysis using FFT and PSD
fft1=zeros([numfiles,numchan,Length]);
psd1=zeros([numfiles,numchan,Length]);
Fft=zeros([numfiles,numchan,Length]);
power=zeros([numfiles,numchan,Length]);
w=100/(Fs/Length);
f1=(Fs/Length)*(0:Length/2);
f=f1(1,1:w);
for i=1:numfiles
    window=hann(End_Data(i)-Start_Data(i)+1);
    for j=1:numchan
        a=squeeze(zero_data(i,j,1:size(window))).*window;
        fft1(i,j,:)=fft(a,Length);
        psd1(i,j,:)=fft1(i,j,:).*conj(fft1(i,j,:));
        Fft(i,j,1:w)=fft1(i,j,1:w);
        power(i,j,1:w)=psd1(i,j,1:w);
        clear fft1;
        clear psd1;
    end
end
for i=1:numfiles
    for j=1:numchan
        [minimas,locs_minimas]=findpeaks(squeeze(-1.*power(i,j,1:w)),'NP'EAKS',50,'MINPEAKDISTANCE',5);
        k=1;
        while k<1000
            if Local_Static_F(i,j)<f(locs_minimas(k))
                Local_Static_F(i,j)=f(locs_minimas(k));
            elseif Local_Static_F(i,j)==f(locs_minimas(k))
                k=1001;
            else
                k=k+1;
            end
        end
    end
end
% Plot PSD vs. Freq
% for i=1:numfiles
%     figure(i)
%     for j=1:numchan
%         subplot(4,3,j)
%         hold on;
%         plot(f.',squeeze(power(i,j,1:w)),-'g');
%         title(sprintf('Power Spectral Density Channel %d',j))
%         xlabel('Frequency (Hz)');
%         ylabel('Power Spectral Density');
%         axis([0,100,0,.1])
%         hold off;
%     end
% end

% Static and Dynamic Filtering Data
Local_Static_data=zeros([numfiles,numchan,Length]);
Global_Static_data=zeros([numfiles,numchan,Length]);
Local_Dynamic_data=zeros([numfiles,numchan,Length]);
Max_Static=zeros(numfiles,numchan);
Max_Dynamic=zeros(numfiles,numchan);
DLF=zeros(numfiles,numchan);
for i=2:numfiles
    for j=1:numchan
Fc_L=Local_Static_F(i,j);
[B,A]=butter(10,Fc_L/(Fs/2));
Local_Static_data(i,j,:)=filtfilt(B,A,zero_data(i,j,:));
clear B;
clear A;
Fc_G=Glob_Static_F(i);
[B,A]=butter(10,Fc_G/(Fs/2));
Global_Static_data(i,j,:)=filtfilt(B,A,zero_data(i,j,:));
clear B;
clear A;
Fc_LD=Local_Dynamic_F(i,j);
[B,A]=butter(10,Fc_LD/(Fs/2));
Local_Dynamic_data(i,j,:)=filtfilt(B,A,zero_data(i,j,:));
clear B;
clear A;

D_Length=size(Local_Dynamic_data(i,j,:),3);
S_Length=size(Local_Static_data(i,j,:),3);

Max_Static(i,j)=max(abs(Local_Static_data(i,j,.1*S_Length:.9*S_Length)));
Max_DYNAMIC(i,j)=max(abs(Local_Dynamic_data(i,j,.1*D_Length:.9*D_Length)));
DLF(i,j)=(Max_DYNAMIC(i,j)-Max_Static(i,j))/Max_Static(i,j);

% ploting Data
% hold on
% figure(i)
% hold on
% subplot(3,4,j)
% plot(time,squeeze(zero_data(i,j,:)),'-r',time,squeeze(Local_Dynamic_data(i,j,:)),'-b',time,squeeze(Local_Static_data(i,j,:)),'-g');
% axis([0,time(End_Data(i)-Start_Data(i)+1),.0002,.0002])
end
end
for i=2:numfiles
    for j=1:numchan
        d_vs_MS((i-1).*numchan+j,:)=[Max_Static(i,j)',DLF(i,j)');
        d_vs_S((i-1).*numchan+j,:)=[Avg_Speed(i)',DLF(i,j)'];
        d_vs_L((i-1).*numchan+j,:)=[Bridge_Length,DLF(i,j)'];
        if i>34 && i<45
            d_vs_VW((i-1).*numchan+j,:)=[10,DLF(i,j)'];
        end
        if i==33|| i==31
            d_vs_VW((i-1).*numchan+j,:)=[5,DLF(i,j)'];
        end
    end
    end
    xlswrite('old_forge_dlf_VS_MaxStatic.xls',d_vs_MS,'A1');
    xlswrite('old_forge_dlf_VS_Speed.xls',d_vs_S,'A1');
    xlswrite('old_forge_dlf_VS_Length.xls',d_vs_L,'A1');
    xlswrite('old_forge_dlf_VS_Weight.xls',d_vs_VW,'A1');