STRUCTURAL HEALTH MONITORING OF A CABLE STAYED PEDESTRIAN BRIDGE

WITH INTERFEROMETRIC RADAR

by

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Thesis directed by Professors Fredrick R. Rutz and Kevin L. Rens

ABSTRACT

Due to an aging infrastructure inside the United States and advances in technology, innovative structural health monitoring methods are emerging. Both short and long term health monitoring of structures can yield valuable data which can be used to determine the condition and capacity of the structure. Much research has been performed in the area of long term health monitoring (defined as monitoring where the instruments are left for days, months or years) but short term monitoring is an emerging field. This document focuses on short term monitoring utilizing an instrument that is new to North America as of 2009.

The subject instrument is the “IBIS-S” system which uses local interferometric radar to monitor structural movement wirelessly and in a non-contact manner. As this system is lightweight and wireless it is easy to quickly deploy, without interrupting the use of the structure, and allows the user to begin collecting data under live loads within hours. Outside of Europe, little research and verification of interferometric radar technology has been conducted on structures. This thesis presents interferometric radar theory, development and application as it relates to cable stayed bridges, particularly towards monitoring the health of the cables and overall natural frequencies of the bridge.

It will be shown that interferometric radar can successfully be used to monitor the tension force and health of the cables as well as the global frequencies of the bridge. A protocol for monitoring the cables and the overall natural frequency for the City and County of Denver’s cable stayed pedestrian bridge where 16th street crosses the Platte River is presented. Through the use of interferometric radar, baseline data was
established for the subject bridge and it was determined that the fundamental frequency of the bridge is below the 3 Hz recommendation set forth in AASHTO standard.

The form and content of this abstract are approved. I recommend its publication.

Approved: Fredrick R. Rutz
DEDICATION

This thesis is dedicated to my incredibly supportive wife, Melissa, and my two daughters, Hannah and Amelia, without whom this thesis and my graduate work never could have happened – thank you for your unwavering support. Additionally, I dedicate this work to my father, Alan, who sacrificed so much in life to teach me about work ethic and a belief that I can accomplish anything in life. I also would like to dedicate this thesis to my Lord and Savior Jesus Christ who is my ‘all in all’ and who has given me the drive and discipline to succeed.
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LIST OF SYMBOLS

SYMBOL

A  Cross sectional area of cable

d  The object’s relative change in position

E  Modulus of elasticity of cable material

\( f \)  Fundamental frequency in the vertical direction

\( f_c \)  Fundamental bending frequency

\( f_n \)  Fundamental frequency of member or structure

g  Acceleration due to gravity

H  Cable force in chord direction

I  Moment of inertia of cable cross section

\( l \)  Cable chord length

L  Length of the main span

\( L_e \)  Effective cable length

m  Mass per unit length of cable

n  Mode number

T  Axial Tension Force on Cable

W  Weight of supported structure, including only dead load

\( \Delta \varphi \)  The change in radar phase

\( \lambda \)  The radar wavelength

\( \lambda^2 \)  A non-dimensional characteristic parameter that reflects the influence of the sag-extensibility on the cable natural frequencies

\( \pi \)  Mathematical constant

\( \rho \)  Cable mass per unit length
\xi \quad \text{A non-dimensional parameter which represents the effect of cable bending stiffness on the natural frequencies of cable vibration}

\omega_1 \quad \text{Calculated fundamental frequency based on the taut string theory}

\omega_{1s} \quad \text{Fundamental frequency based on Ren et. al’s theoretical work which considers cable sag}
CHAPTER I
INTRODUCTION

Introduction

Radar has long been used to track the movement of objects or masses across large distances. This includes aircraft, sea vessels, precipitation, and vehicles. Regarding static objects, such as structures, radar has been successfully used to locate hidden objects. An example of this is the process of locating reinforcing steel in concrete or underground utilities utilizing ground penetrating radar (GPR).

More recently radar has been used by satellites to monitor movement of soil/land masses to an accuracy of several meters (IBIS-S Controller User Manual July 2010). With time and technology, the accuracy of radar has greatly increased to the point where it is now feasible to monitor very small movements. In the 1990’s, an Italian corporation known as “Ingegneria Dei Sistemi”, or IDS – translated “Systems Engineering” – partnered with the University of Florence to research the possibility of utilizing radar to monitor earth movement to a higher degree of precision (i.e. millimeters (mm)) (25.4mm=1.0 inches) (Farina et al, 2011). IDS and the University of Florence were successful in applying radar technology in monitoring land subsidence and slope stability, particularly in mining applications (Farina et al, 2011). In this application, a semi-permanent radar station is set up to focus on a particular slope (Farina et al, 2011). IDS’ technology for monitoring slope stability has successfully been used in Europe for over a decade (Farina et al, 2011).

In the late 1990’s, Farrar et. al presented that radar could be used to monitor movement in structures (Farrar et. al 1999). IDS and the University of Florence have also researched radar applications on structures, namely monitoring small movements in structures or structural members. Through all this research, IDS has developed a
system entitled Image By Interferometric Survey (IBIS) which utilizes radar to monitor
deflections in structural members with a precision of up to 1/100 mm (0.000394 inches).
The IBIS system is a non-contact, non-destructive, rapidly deployable system that has
positive implications for the structural health monitoring community. Both the benefits
and limitations of the IBIS instrument are discussed in this thesis. As it relates to
structures, the IBIS system has been used in Europe for approximately 10 years.

Inside of North America few IBIS systems exist and the few systems that do exist
are privately owned and operated by the mining industry to monitor slope stability.
Olson Instruments Inc. (Olson) of Wheatridge, Colorado currently has the only privately
held, for-hire, IBIS system in North America. Olson has graciously agreed to help
sponsor this thesis in the spirit of increasing the body of knowledge of the structural
health monitoring community.

Scope and Objective of Thesis

Because of its superior accuracy there are many potential IBIS applications in
structures. However, many such applications are still being researched. The primary
objective of this thesis is to explore, via laboratory and field experiments, an application
of IBIS technology to cable stayed bridges. Of particular interest is a pedestrian bridge
owned by the City and County of Denver, Colorado. A method to monitor the health of
the cables and overall bridge health with IBIS technology was developed and
implemented. The scope of this thesis is limited to one bridge and to vibration
monitoring on that bridge. Recommendations for future bridge health monitoring are
provided herein.

Outline of Thesis

This thesis presents the results of theoretical and experimental testing in which
interferometric radar technology is used to monitor the structural health of a cable stayed
pedestrian bridge. Chapter Two presents a review of available literature regarding interferometric radar and its use on structures, including cable stayed bridges. Chapter Three presents the results of a Finite Element Analysis (FEA) theoretical determination of a particular cable stayed bridge’s fundamental frequency. Chapter Four presents the results of laboratory experiments in which interferometric radar was used to determine the tension force in a cable based on the fundamental frequency of vibration and an attempt was made to determine the tension force in a steel channel through the use of interferometric radar. Chapter Five presents the results of field work wherein interferometric radar, an alternate instrument, and an accelerometer were used to monitor vibrations in a cable stayed pedestrian bridge and the results of all three instruments were compared. Chapter Six contains discussion and conclusions reached as a result of the work in the previous four chapters as well as a testing protocol for the subject pedestrian bridge. Chapter Seven presents final conclusions and recommendations.
CHAPTER II

LITERATURE REVIEW

Introduction

The concept of a cable suspended bridge (a bridge in which smaller vertical suspender cables that support the deck, hang from a larger catenary shaped cable which anchors to the earth) has been around for centuries as historians note remote foot bridges constructed with vines and ropes. However, the concept of a cable stayed bridge – a bridge in which the cables are diagonal, putting the bridge deck into compression, and the cables attach to a structurally critical tower or mast is a more recent (past 400 years) development.

The oldest known cable stayed bridge design concept dates back to a design completed by the Venetian engineer Faustus Verantius in 1607 (Podolny 1999). The oldest known constructed cable stayed bridge dates back to a 32 Meter (M) (105 Feet) long span completed by Loscher in 1784 (Podolny 1999). The oldest known cable stayed bridge in the United States is a still intact steel bridge located in Texas and was designed by E.E. Runyon (Historic American Engineering Record 1968).

The concept of cable stayed bridges did not become common in place in the United States until approximately 40 years ago. As can be seen in Figure 2.1, the use of cable stayed bridges in new construction in the United States continued to increase until the mid 1990's. The increased use of cable stayed bridges is due both to their aesthetic appeal and their cost effectiveness for moderate bridges.
With an increase in the number of cable stayed bridges in the United States in the past 40 years there becomes a need for the development of structural health monitoring techniques for cable stayed bridges. In contrast to a suspension bridge wherein the failure of one of hundreds of suspender cables is likely not catastrophic, the failure of a cable in a cable stayed bridge has a higher likelihood of being catastrophic.

For instance, consider the Martin Olav Sabo cable stayed pedestrian bridge in Minneapolis, Minnesota (Figure 2.2) - constructed over a light rail and highway in 2007 with a main clear span of 67 m (220 feet) - which had a serious failure on February 19, 2012 that led to the closure of the bridge. The failure, involved the failure of two cables which caused a portion of the bridge deck to deflect which in turn caused an increase in loading on adjacent connections causing concern or a progressive collapse. Reportedly, the cables failed due to a fatigue failure, induced by wind born vibrations, in a diaphragm plate that connected to the bridge tower (WJE 2012). Many engineers have suspected
that the failure was caused by fatigue stresses that were induced by excessive vibrations in the bridge's cables as members of the community had expressed concern over excessive vibrations. It is unknown to this engineer what type of structural health monitoring program was in place for this bridge but perhaps closer monitoring of cable vibrations and global vibrations would have predicted fatigue failure.

Much research exists regarding the structural health monitoring of cable stayed bridges and a fundamental part of any such health monitoring plan involves vibration monitoring. However, most, if not all, current vibration monitoring instruments require persons to physically access the cables in order to obtain measurements which usually results in temporary bridge closure. The purpose of this thesis is to explore the use of radar technology for monitoring structural health in cable stayed bridges – this concept allows data to be gathered without closure of the bridge and in a non-contact manner.
History and Concept of Radar

The word ‘radar’, a noun in the English language, stems from an acronym developed in the U.S. Navy – RAdio Detection And Ranging (RADAR). As the name implies, radar technology relies upon the use of radio waves to detect the location (range) of mass. On the electromagnetic spectrum, radar is a subarea of the microwave region having a range of frequency between 0.3 and 300 GHz (Figure 2.3). Radar technology has been around for over 100 years but it wasn’t until World War II when its use became widespread and wide known as it was found very useful to track both enemy and ally ship and aircraft locations and movements.
Figure 2.3 Electromagnetic spectrum.
Radar is an invisible wave which, like any waveform, when emitted will reflect—or bounce back— from objects in its way. Metallic objects are a particularly good reflector. The quality of the reflection is dependent upon both the material and the angles inherent to the object. An ideal radar reflector is a metallic object in the shape of an open pyramid where the radar is directed at the ‘inside’ of the pyramid - this shape forces all incoming waves to be reflected back towards their source (Figure 2.4).

![Figure 2.4 Ideal radar reflector.](image)

In a real world scenario, the rather sharp angle created at the point where an aircraft’s tailfin joins the fuselage is a good reflector of radar. With this basic knowledge of radar one can view Lockheed Martin’s stealthy, nearly radar invisible, F117 aircraft and come to understand the purpose of the oblique angles and out of plumb tail fins.

In order to locate or track the movement of an object, a radar emitter sends out radar waves with a known frequency, amplitude, and wave length. An antenna is engineered and setup to receive the radar waves. Due to the Doppler Effect, the reflected waves
have a different wavelength, frequency, phase and amplitude from the initial signal (IDS 2008). Utilizing the change in the waveform it is then possible to determine how far away the object is and by this information one can track the relative movement (displacement) of an object.

How IBIS Works

IDS’ IBIS system utilizes radar technology to detect small movements in structures or structural members. The subject IBIS instrument, one that is licensed by the Federal Communications Commission for use in the United States, emits microwaves at a Continuous Wave Step Frequency (CWSF) of 17.1-17.3 MHz with a wavelength of approximately 18mm (0.71 inches)(IDS 2010). The frequency and wavelength have importance as they determine the size of object that can be detected, the maximum measurable deflection, and the minimum distance two objects must be separated by – a concept known as ‘range bins’ for which further discussion is presented below.

IBIS utilizes a form of radar known as ‘interferometric’ radar. In most radar applications the amplitude change in the reflected wave form is used to determine the location of the desired object. However, interferometric radar utilizes the change in phase of the reflected wave to determine position. By monitoring the phase change, a more accurate measurement of movement is possible (Figure 2.5).
An object's change in position is given by the equation 2.1 shown below.

\[ d = -\frac{\lambda}{4\pi} \cdot \Delta \varphi \]

**Equation 2.1 Relative Displacement Equation (Gentile 2008).**

Where:
- \( d \) = Object’s relative change in position
- \( \lambda \) = Radar wavelength
- \( \Delta \varphi \) = Change of Phase as shown in Figure 2.5
- \( \pi \) = Mathematical constant

Inside of the interferometric radar technology are two concepts; Synthetic Aperture Radar (SAR) and Real Aperture Radar (RAR). IDS manufactures a system for long term monitoring of slopes and structures (IBIS-L) and a system for short term health monitoring (IBIS-S). The instrument used in this research is the IBIS-S system. In a static setup the IBIS-S relies upon RAR technology which, in contrast to a typical two dimensional (2D) SAR image, results in a one dimensional (1D) view (Dei et al, 2009).
Due to the 1D limitation it is critical to understand that the displacement measured is the object's relative movement in the line of radar. As the radar head is rarely, if ever, pointed directly along the expected plane of movement (vertical in the instance of a beam deflecting downward), the measured displacement is not actual displacement (Figure 2.6). However, due to the superior accuracy of IBIS, vertical displacement will produce a change in the measured distance and utilizing basic trigometric principles one can calculate the actual displacement.

![Figure 2.6 IBIS' line of sight measurement (adapted from Gentile 2008).](image)

From a hardware standpoint, the IBIS system is fairly simple. It consists of a 'radar head' or sensor that has two antennas. One antenna sends a signal and the other
receives. The radar head sits atop a traditional tripod that is leveled prior to use. Two cables connect to the back of the sensor. One cable is used to power the system and the other is used to send data to a personal computer (PC). Any PC system with the proper software and a Universal Serial Bus port (USB) can be used to receive data.

The IBIS system has six available sets of antennas. Each antenna forms a different shaped radar cone which results in a different field of view. For example, one antenna produces a narrow focused cone and another produces a wide shallow cone. While the IDS’ provided manuals give data on each of the antenna’s there are no known publications which educate the user on which antennas are best for certain applications. In the “Field Work” section of this paper more is presented on how an individual chooses an antenna for a given application.

Once the IBIS system is assembled and powered, the user can view the radar feedback on the PC. The radar feedback is presented on a plot with distance on the X axis (in meters) and a signal to noise ratio (SNR) plotted on the Y axis (see Figure 2.7 for an example).

![Figure 2.7 Radar read out on PC.](image)
The stronger the SNR the more accurate the instrument’s output is. Thus, strong peaks on the plot (high SNR) are necessary for the structural member(s) of interest. On the low end, SNR ratios of 20 give an accuracy of 0.1mm (0.00394 inches) (IDS 2010). On the high end, SNR ratios of 60 and higher give an accuracy of 0.005mm (0.00020 inches) (see Figure 2.8). Even with the low SNR ratios, a high degree of accuracy is available.

![Figure 2.8 Accuracy vs. SNR.](Copyright 2010 by IDS, IBIS-S Controller V 02.02.000 User Manual Rev. 1.1, July 2010, Image used by kind permission from IDS)

Certain types of structural members are good natural reflectors of radar. These include steel members with geometry changes such as ‘L’, ‘C’ and ‘W’ shaped members. Steel cables are typically good reflectors. Typical plate steel and smooth rods are not good reflectors of radar. Concrete and wood are not typically good reflectors of radar. For applications where the user cannot achieve good reflections the user may be required to install a reflector as shown in Figure 2.4.

In order for the user to have confidence as to what peaks correlate to which structural members, a laser distance meter is a necessary tool. By placing the distance...
meter on the IBIS and aiming at desired structural elements, the user can quickly
determine which peaks relate to which member(s). For the purpose of determining the
distance to smaller diameter cables, a distance meter with a live video feed and
crosshairs is necessary to ensure the laser is reflecting from the correct cable. For the
purposes of this thesis, a Leica DISTO D5 instrument (a laser distance meter which has
a range of up to 200 M (650 Feet) and an accuracy of 1mm (0.0394 Inches)) was used.
Thus, as long as the user has confidence in the objects each radar peak corresponds to
the user can collect displacement data on multiple structural elements at the same time.
For instance, one technique involves viewing the underside of a bridge, looking parallel
to the structure, allowing the user to collect displacement data for all cross
frames/members at the same time with one setup and one instrument.

In a typical static/tripod type of setup the IBIS system produces a 1D image. Due
to the static setup and CWSF, “range bins” are created in which the radar cannot
distinguish the difference between objects that are within a certain distance of each
other along the path of the wave propagation (see Figure 2.9). The range bin distance
for the IBIS-S system available in North America is 0.75 M (2.46 Feet); that is to say that
the instrument cannot differentiate between objects closer than 0.75 M (2.46 Feet) to
each other on a given radial distance from the instrument. Dei et al. (2009) have
successfully explored monitoring techniques, monitoring torsional motion, wherein the
instrument is mounted and moved slowly along a rail to produce a 2D, SAR image that is
not limited by range bins.
IBIS can sample at a rate of 200Hz (100Hz Nyquist Frequency) permitting the user to obtain accurate data when monitoring vibrations. The 200Hz sampling rate also permits the user to see higher order natural frequencies and mode shapes in most structures and cables.

### Limitations of IBIS-S System

The IBIS system is limited to objects that are further than 0.75 M (2.46 Feet) apart from each other. In addition to this limitation, IBIS’ 18mm (0.71 Inches) wavelength requires that the object be larger than 18mm (0.71 Inches) in order that the waves do not pass over the object. The implication of these limitations is that IBIS cannot practically be used to monitor the tensile force in cables that have a diameter of less than 18mm (0.71 Inches) (although a limitation of 25.4mm (1.00 inches) cable diameter is typically recommended).
IBIS’ 18mm (0.71 Inches) wavelength also presents a dilemma for larger deflections. If the deflections in the member exceed 18mm (0.71 Inches), then certain portions of the radar waves will pass over the object resulting in incorrect data. For cables with low tensile forces that are constantly excited by ambient conditions (wind, traffic) this proves to be a problem. However, this problem can be overcome by mounting onto the cable, or beam, a reflector that is much larger than the 18mm (0.71 Inches) wavelength – obviously this would require access to the cable (which may not always be possible).

Benefits of IBIS are that its results are reportedly unaffected by weather conditions and it typically requires no contact with the structural members so that monitoring can occur while the structure is in use.

**IBIS Software**

IDS’ IBIS system is supplied with an outdoor rated laptop computer which features hardware drivers to connect to and collect data from the radar head. However, any personal computer can be used; the user does not have to use the supplied laptop. Also provided with the IBIS system is MatLab-based post processing software. After the field data—referred to as a mission—is complete, the user can launch the post processing software and begin processing the gathered data (displacement data as a function of time) for each point of interest. The post processing software is surprisingly simple to use and is quite powerful. In particular, the software performs a Fourier Transform function (which takes displacement data that was acquired in the time domain and plots it in the frequency domain) after which the displacement, velocity and acceleration can all be viewed in both the time and frequency domains.

IBIS’ MatLab based software also has the capability to determine mode shapes and to form a movie that depicts the structure’s movement over time – this is done in a click of a button. These capabilities allow the user to process and view their data in the field.
and within minutes form an opinion on the accuracy of the data as well as the behavior of the structure.

**Historic Use of Radar on Infrastructure**

Radar has long been used to inspect roadways and structures through the use of Ground Penetrating Radar (GPR). In contrast to IBIS’ use of interferometric radar technology, GPR utilizes conventional radar technology to ascertain information. GPR use in the United States began in the 1970’s with its use on roadways, including bridge decks, and its use has expanded to investigation of: buried utilities, voids below roadways, pavement thicknesses, depth to bedrock and determination of deteriorated areas of bridge decks, all utilizing high frequency microwaves of 0.5 to 1 GHz (Morey, 1998; Maser, 1994; Alongi et al, 1992, Scullion et al, 1992).

Today, GPR is still used for all of the above and its use has further expanded to buildings and bridges. GPR is used to locate steel rebar and strands in mildly reinforced concrete and pre/post stressed concrete, respectively. Tallini et al, successfully showed that GPR can be used to determine foundation type and to investigate the quality (depth and bulbs) of micropiles (Tallini et al, 2004)

As it pertains to interferometric radar, research indicates that the technology has been successfully used to monitor the structural health of wind turbines (Pieraccini et al, 2008), buildings (Luzi et al, 2008), towers and culverts (Beben, 2011). Other research involves multiple IBIS systems working together to measure torsion in structures (Dei et al, 2009). Much research has also been dedicated to utilizing IBIS technology in the monitoring of slope stability (see Pieraccini et al., 2001).
Historic Use of Radar on Cable Stayed Bridges

As previously mentioned, Farrar et. al explored the application of radar to monitor movement in structures – this included bridges (Farrar, et. al, 1999). Since that time many groups - including Pieraccini et al, Gentile, Dei et al - have successfully undertaken the monitoring of bridges with IBIS technology (Gentile 2008, Pieraccini et al., 2001). These individuals have been successful in monitoring both deflections/vibrations in bridge members as well as determining global natural frequencies and mode shapes of the bridges. These bridges included cable stayed bridges.

Specifically regarding the structural health of cables in the cable stayed bridges there is very little published literature. In 2010 Gentile theorized that IBIS technology could be used to monitor the health of cables in cable stayed bridges via monitoring changes, over time, in the natural frequency of the member (Gentile 2010). Further, Gentile relied upon previous research which determines the tension force in a cable based on the cable properties and measured natural frequency. Over time, changes in the cable’s frequency would indicate there is a change in the tension force which could indicate deterioration in the cable or overall changes in the load paths in the bridge as a whole.

Predicting Tensile Force in Cables Based on Fundamental Frequency

The earlier work that Gentile did relied upon work done by Mehrabi in 2006 (Mehrabi 2006). Mehrabi indicates that the tension force in a cable member can be determined by relying upon a taut string model which is represented by equation 2.2 (Mehrabi 2006):
Where: $T$ = Axial tension force on cable  
$f_{n} = $ Fundamental frequency of member or structure  
$n = $ Mode number  
$L_{e} = $ Effective cable length  
$\rho = $ Cable mass per unit length

As the name implies, applying the taut string theory to cables assumes the cable is taut. However, in many instances cables in cable stayed bridges are not taut. Such is the case for the bridge that is the subject of this thesis. Both the degree of sag and the bending stiffness of cables effect their fundamental frequency and changes the accuracy of Equation 2.2 above. Ren et al sought to quantify the effect of bending stiffness and sag on the taut string theory equation and their work indicates that indeed unacceptable errors in predicted tensile force can occur due to cable sag and stiffness (Ren et. al, 2007). Ren et. al. performed a theoretical analysis on the effect of cable sag and in that analysis they introduce a non-dimensional term $\left( \frac{f_{n}}{n} \right)^{2}$, shown in Equation 2.3 below, which “is an important characteristic parameter that reflects the influence of the sag-extensibility on the cable natural frequencies” (Ren et. al, 2007).
\[ \lambda^2 = \left( \frac{mg \ell}{H} \right)^2 \frac{EA \ell}{HL_e} \]

Equation 2.3 A non-dimensional characteristic parameter that reflects the influence of the sag-extensibility on the cable natural frequencies.

Where: 
- \( E \) = Modulus of elasticity of cable material 
- \( A \) = Cross sectional area of cable 
- \( L_e \) = Effective cable length 
- \( H \) = Cable force in chord direction 
- \( \ell \) = Cable chord length 
- \( m \) = Mass per unit length of cable 
- \( \lambda^2 \) = A non-dimensional characteristic parameter that reflects the influence of the sag-extensibility on the cable natural frequencies 
- \( G \) = Acceleration due to gravity

When utilizing the first mode, Ren et al conclude that for \( \lambda^2 \) values greater than 1.0, large amounts of error exist (20% when \( \lambda^2 \) equates to 5.48, for example) (Ren et. al, 2007). This phenomena is shown graphically in Figure 2.10 below wherein \( \omega_1 \) is the calculated fundamental frequency based on the taut string theory and \( \omega_{1s} \) is the fundamental frequency based on Ren et. al’s theoretical work which considers cable sag (Ren et. al, 2007).
Ren et al’s work went on to analyze the effect of cable sag on higher modes of vibration and the results are very useful in this thesis’ work. Specifically, Ren et al performed theoretical analysis for higher vibration modes and showed that the effects of cable sag become negligible for higher vibration modes. A plot showing the variation of $\lambda^2$ for different vibrations modes is shown in Figure 2.11 below. Note that in Figure 2.11, the ‘y’ label on the y axis is a variable that represents different curve functions, each of which contain the natural frequencies of the symmetric in-plane modes of a sagged cable. On the same note, the $\beta l / 2$ label on the x axis is the algebraic roots
(1\textsuperscript{st}, 2\textsuperscript{nd}, 3\textsuperscript{rd}, etc.) to the equation represented by variable ‘y’ which represent corresponding (1\textsuperscript{st}, 2\textsuperscript{nd}, 3\textsuperscript{rd}, etc.) mode shapes.

![Figure 2.11 Effects of cable sag on higher vibration modes.](Copyright Emerald Group Publishing Limited 0264-4401, Engineering Computations: International Journal for Computer-Aided Engineering and Software, Volume 25, No.2, 2008, Determination of Cable Tensions Based on Frequency Differences, Ren, Wei-Xin; Liu, Hao-Liang; Chen, Gang; Page 177, Figure 3, used with kind permission from Inderscience Enterprises Limited.)

Ren et. al. conclude that when $\lambda^2=300$, the discrepancy between the first three symmetric natural frequencies of the sagged cable obtained from their theoretical equation and those of the taut-string cannot be neglected, but the sagged cable frequency and taut-string frequency become very close after the fourth symmetric vibration mode (3.5pi on Figure 2.11). Ren et al continue with lab and field work which prove their theories. Ren et al’s work has significance for those in the structural health monitoring community that work on cable stayed bridges since, prior to this work, cable sag on many bridges introduced unacceptable errors in the calculated cable tension.
forces. In order to remove the error associated with cable sag, one must determine the fourth, or higher, cable vibration mode. The only drawback to this method is that associated with determining the higher modes of vibration. The process of determining the higher modes of vibration relies upon field work and review of power density plots and sometimes during that process certain modes can be missed.

Ren et al’s work also looked at the effect of cable bending stiffness on the results of the taut string theory. Ren et al’s work analyzes the cable as a simply supported beam with an axial tension force and they introduce a non-dimensional parameter \( \xi \) which represents the effect of cable bending stiffness on the natural frequencies of cable vibration. \( \xi \) is defined in equation 2.4 below (Ren et. al, 2008).

\[
\xi = \ell \sqrt{\frac{T}{EI}}
\]

Equation 2.4 A non-dimensional parameter which represents the effect of cable bending stiffness on the natural frequencies of cable vibration.

Where:  
\( E \) = Modulus of elasticity of cable material  
\( I \) = Moment of inertia of cable cross section  
\( T \) = Axial Tension Force on Cable  
\( \ell \) = Cable chord length  
\( \xi \) = A non-dimensional parameter which represents the effect of cable bending stiffness on the natural frequencies of cable vibration

Ren et al prepared a plot of \( \xi \) versus natural frequency which is shown in Figure 2.12 below where \( \omega_1 \) is the calculated fundamental frequency based on the taut string theory and \( \omega_{1s} \) is the fundamental frequency based on Ren et. al’s theoretical work which considers cable stiffness (Ren et. al, 2007). Essentially, Ren et. al. concluded that for lower values of \( \xi \) (less than 50), the cable bending stiffness cannot be neglected but for higher values (more than 50) cable bending stiffness can be neglected (Ren et. al,
Also, the lower values of $\xi$ effect the results of the taut string theory for higher order frequencies as shown in Figure 2.12.

Figure 2.12 Effects of cable bending stiffness on fundamental frequency.
(Copyright Emerald Group Publishing Limited 0264-4401, Engineering Computations: International Journal for Computer-Aided Engineering and Software, Volume 25, No.2, 2008, Determination of Cable Tensions Based on Frequency Differences, Ren, Wei-Xin; Liu, Hao-Liang; Chen, Gang; Page 180, Figure 4, used with kind permission from Inderscience Enterprises Limited.)
In summary, Ren et. al’s work shows that when predicting cable tension based on fundamental frequency utilizing the taut string theory, that cable sag effects the results of the taut string theory for lower vibration modes but for higher vibration modes the error is negligible (Ren, et. al, 2007). In contrast Ren et. al showed that the effect of cable bending stiffness on the results of the taut string theory effect the higher frequency modes more so than the lower modes (Ren, et. al, 2007).

**Effect of Vibrations on Pedestrian Bridges**

In their book, “Vibration Problems in Structures”, Bachmann et. al. indicate that 95% of pedestrians walk at rates between 1.65 and 2.35Hz (Bachmann, et.al, 1995). Most structural engineers are aware that as the forcing frequency approaches the fundamental frequency of the structure resonance can occur. It is for these reasons that
modern pedestrian bridge codes limit the natural frequency of pedestrian bridges. For the subject bridge, the applicable bridge code – 1997 AASHTO Guide Specifications for Design of Pedestrian Bridges – indicates that for pedestrian bridges the fundamental frequency in the vertical direction must be above 3Hz, and the fundamental frequency in the lateral direction must be above 1.3Hz (AASHTO 1997). Steel footbridges, such as the subject one, are more susceptible to vibration problems due to pedestrians (Bachman et. al, 1995). Work done by Wiss et al. indicates that the most severe response in pedestrian bridges occurred when the fundamental frequency of the bridge is closest to 2Hz (Wiss et al 1974).

Bachmann et al. present a basic empirical equation relating the fundamental frequency of cable stayed bridges to their span length (Bachmann et.al, 1995). This equation is presented in Equation 2.5 below (Bachmann et.al, 1995).

\[ f_e = \frac{110}{L} \]

**Equation 2.5 Fundamental bending frequency of cable stayed bridge (Bachmann et al, 1995).**

Where: \( L \) = Length of the main span  
\( f_e \) = Fundamental bending frequency

AASHTO 1997 indicates that if an analysis of the bridge's fundamental frequency in the vertical direction is not evaluated then the bridge may be proportioned to satisfy the following criteria in equations 2.6 and 2.7:
\[ f \geq 2.86 \ln\left(\frac{180}{W}\right) \]

Equation 2.6 Minimum fundamental frequency of bridge in the vertical direction (AASHTO 1997)

or

\[ W \geq 180e^{-0.35f} \]

Equation 2.7 Prescriptive weight of supported structure, including only dead load (AASHTO 1997).

Where:  
- \( f \) = Fundamental frequency in the vertical direction
- \( W \) = Weight of supported structure, including only dead load

**Summary**

In summary, there are numerous cable stayed bridges inside of the United States and those bridges require structural health monitoring. It is critical to design those bridges to prevent fatigue failures which can occur due to excessive vibrations. A proper health monitoring program can identify a loss of stiffness or excessive vibrations via monitoring the global natural frequency of the bridge and the natural frequencies of the cables. Equations exist which can correlate the tension in cables to the measured natural frequency.

Radar technology has been around for decades but it has only been in the past 10 years that this technology has been used for structural health monitoring purposes. By using interferometric radar technology, IDS has developed an instrument that can monitor vibrations in cable stayed bridges in a non-contact manner.
CHAPTER III
THEORETICAL ANALYSIS

Overview

In order to have baseline data to compare field testing results to, a theoretical
Finite Element Analysis (FEA) was performed for the bridge of interest- the City and
County of Denver’s cable stayed pedestrian, 16th Street over the Platte River. Utilizing
construction documents for the subject bridge, a three dimensional Computer Aided
Drafting (CAD) model was created. Next, the CAD model (a .dxf file) was imported into
SAP 2000 and the theoretical global fundamental frequency of the bridge was
determined.

Three Dimensional CAD Model

The subject bridge has a fairly complex geometry as is typical for cable stayed
bridges. In particular, the bridge is ‘crowned’ vertically with the highest point being the
middle support; the towers are swept in three dimensions; and, the geometry is not
symmetrical about the center support. Utilizing elevations and dimensions from the
bridge’s original construction drawings, shown in Appendix A, the three dimensional
CAD model was created using AutoCAD software. Figures 3.1 and 3.2 below show
three dimensional CAD views.

Figure 3.1 Three dimensional view of bridge (purlins not shown for clarity sake).
Figure 3.2 Three dimensional view of bridge.

SAP 2000 Model

General Bridge Construction

Construction drawings (See Appendix A) for the subject bridge were obtained and through inspection of the bridge it was concluded that the bridge was built in substantial conformance with the construction documents regarding dimensions and materials used.

Structurally, the bridge is fairly simple. The bridge features a decay resistant wood decking (spanning perpendicular to the length of the bridge) that transmits gravity loads to wolmanized wood stringers that span approximately 3.96 M (13 feet) (parallel to the length of the bridge) and bear upon steel floor beams that span approximately 6.10 M (20 feet) (perpendicular to the length of the bridge). The steel joists attach at either end to two main 40.64 Centimeter (CM) (16 inch) diameter steel tubes, or girders. The steel tubes constitute the main gravity carrying element. The steel tubes rest upon
bearing pads near both ends of the bridge and at their midspan the tubes bear upon a perpendicular built up steel member which attaches to the towers and the towers transmit that load to a cast in place mildly reinforced concrete foundation. A photograph of the bridge is shown below in Figure 3.3. The main tubes span approximately 31.70 M (104 feet) between supports and gain significant support from cables. The cables are 25.4 mm (1 inch) diameter Class A structural steel strands (manufactured in accordance with ASTM A586). The cable properties, as provided by the manufacturer (WireCo) are as follows:

- **Tensile Capacity** = 55,338 Kilograms (KG) (61 tons)
- **Weight per Unit Length** = 30.65 Newtons/M (2.1 lbs/ft)
- **Modulus of Elasticity** = $1.66 \times 10^{11}$ Pascals (Pa) (24,000 Kips per Square Inch (ksi))
- **Cross Section Area** = 3.87 CM² (0.60 in²)

The cables connect to the tube via a welded plate connection and the cables connect to the plate via a pin ended connection. At their tops, the cables connect to 40.64 CM (16 inch) diameter steel tube masts via a similar connection.
Support Conditions

The subject bridge features cantilevers, approximately 3.05 M (10 feet) long on the west end and 20 feet long on the east end, at both ends. Through inspections, it was noted that the supports nearest the ends of the bridge are both roller bearing supports as evidenced by the loose and slotted vertical bolts and the elastomeric bearing pads shown in Figure 3.4 below.
The bearing at the middle of the bridge was determined to be consistent with a pin ended condition. This is evidenced by the main supporting tubes which bear upon the aforementioned perpendicular built up steel member, as shown in Figures 3.5 and 3.6 below, via a welded steel plate that has limited moment capacity.
The built up perpendicular steel section transmits the gravity loads from the main tubes to the towers, or masts, via a bolted connection as shown in Figure 3.6. The
towers, or masts, are supported via a moment resistant connection at their base as shown in Figure 3.6.

**Bridge Fundamental Frequency**

Through the FEA analysis it was concluded the global fundamental frequency of the bridge is 3.39 Hz. In accordance with AASHTO recommendations, all the frequency determinations in SAP 2000 were determined utilizing un-factored self-weights. The SAP model was edited multiple times in order to determine its sensitivity to cable force and individual element boundary conditions and no change in the fundamental frequency or first mode shape occurred. This is due to a 7.16 M (23.5 foot) long steel framed cantilever that exists on the east end of the bridge. The cantilever dominates the first mode shape and that cantilever is unaffected by cable tension force as no cables connect to that portion of the bridge. The first mode shape from the SAP model can be seen in Figure 3.7 below.

![Figure 3.7 First mode shape from sap model.](image)

**Conclusions**

In summary, the fundamental frequency of the bridge was determined to be 3.39 Hz in SAP 2000 software and the first mode shape was found to be dominated by the east end cantilever section. Changes in the boundary conditions of individual elements and the cable tension forces resulted in no change in the fundamental first mode shape due to the dominance of the east end cantilever. The east end cantilever dominance is...
consistent with field observations which are discussed in the Field Work portion of this thesis.
CHAPTER IV
LABORATORY WORK

First Laboratory Experiment

Setup of First Experiment (Cable Test)

Bearing in the mind that one of the primary purposes of this thesis is to determine if IBIS technology can be accurately used to measure the natural frequency of cables, a laboratory experiment was designed and implemented. The first phase of the laboratory experiment involved purchasing a 1.25 inch diameter cable (wire rope) with looped ends and fixing the cable in UC Denver’s (UCD) Material Testing System (MTS), stretching the cable to a known tensile load, striking the cable with a rubber mallet to induce free vibration and measuring the vibration motion with three instruments (see Figure 4.1 for an image of the lab setup). The three instruments used were the subject IBIS-S system (sampling at a rate of 100Hz Nyquist Frequency), a programmable accelerometer (GP1 Programmable Accelerometer manufactured by Sensr, sampling at a rate of 50 Hz Nyquist Frequency and set at a ‘g’ range of +/- 2.5g) and a non-contact instrument manufactured by DynaTension (Model P1000) which measures the frequency of vibration in cables. Loading was applied in 8.90 Kilonewton (KN) (2 Kips) (1Kip=1000 Pounds) increments starting at 0 KN (0 Kips) and going through to 115.65 KN (26 Kips). Ten readings were taken with the P1000 and the results were averaged for a final value. A minimum of four cable strikes were conducted with the accelerometer and the data from those strikes were included in the Fourier analysis. A minimum of four cable strikes were carried out with the IBIS instrument and that data was used in IBIS’ post processing MatLab based fourier analysis.
Figure 4.1 Cable in MTS equipped with accelerometer.

The P1000 instrument has certain limitations based on cable tension, diameter and length. Due to these limitations the P1000 could only be used to measure tension forces above 44.48 KN (10 kips) in the laboratory setting. Due to the ‘g’ range limitations the accelerometer cut off certain portions of data early in the vibration after the cable was struck. In the laboratory setting no limitations on the IBIS system were encountered; this is certainly one advantage of IBIS technology, along with the fact that the user does not need to have access to the cables as is required with traditional instruments.

Once the fundamental frequency for the cable was found for each load case with each of the three instruments, the fundamental frequency was utilized in the taut string
equation (Equation 2.2) to calculate the cable force. Since the cable had no sag during the test Equation 2.2 was relied directly upon with no correction for sag extensibility.

**Results of First Experiment (Cable)**

From the 44.48-115.65 KN (10-26 Kip) range the IBIS system results and the P1000 results proved to be very close - this finding is encouraging. The data can be seen in tabular form in Table 4.1 and graphically in Figure 4.2. The accelerometer provides the user with acceleration as a frequency of time. Utilizing the Fourier Transform function in Microsoft Xcel, the time domain data was plotted in the frequency domain in order to view the peaks (natural frequencies) in the data. In Table 4.1 below the findings from each of the three instruments are presented and it can be seen that all three instruments are providing similar values reinforcing the confidence in IBIS’ results. Based on these findings it can be concluded that IBIS technology can successfully be used to measure frequency of vibration in cable stayed members with a diameter of 25.4MM (1 inch) or larger.

After the lab work was completed the data was incorporated into Mehrabi’s tension equation. Mehrabi’s equation is a function of cable length and being that the length term in the equation is squared, the equation is highly sensitive to cable length. Due to the laboratory setup with looped cable ends, the exact effective length of the cable is not exactly known. Further, during initial loading 0-44.48 KN (0 to 10 Kips) the cable stretched substantially (over 25.4MM -1 inch); thus, the cable length substantially changed during this region. Given these observations it is no surprise that the subject equation does not fit the data well from 0-44.48 KN (0-10 Kips). The important finding is that all three instruments provide close data; there has been plenty of previous research on the application of Mehrabi’s tension equation and that is not the purpose of this research. After 44.48 KN (10 Kips) –once cable slack/loss was mostly removed-
Mehrabi’s tension equation proved to be fairly accurate (+/- 5%) as can be seen in review of the data in Table 4.1.

Frequency domain plots from Microsoft Excel of each cable force increment measured are shown in Appendix B and on those plots the reader can view the peak which corresponds to the measured natural frequency. The figures shown in Appendix B include frequency domain plots from IBIS’ MatLab based post processing system in which the reader can view the peak which corresponds to the measured natural frequency.
Table 4.1 Results of Lab Testing for Tensile Force Determination in Cable.

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<th>Actual Force (Kips)</th>
<th>fn Measured with IBIS</th>
<th>fn Measured with P1000</th>
<th>fn Measured with accelerometer</th>
<th>Tension Force Based on IBIS (Kips)</th>
<th>Tension Force Based on P1000 (Kips)</th>
<th>Tension Force Based on Accelerometer (Kips)</th>
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Figure 4.2 Graph of calculated vs. actual cable force.
Conclusions from First Experiment (Cable)

Based on the laboratory results from the cable test it was concluded that the IBIS-S system can successfully be utilized to measure the fundamental frequency of tension members such as wire rope and structural steel strands. It was also concluded that for higher tension loads, once a majority of the slack has been removed, Mehrabi’s tension equation can successfully be applied. Thus, it is shown that by using interferometric radar, the fundamental frequency and tension force in cables can be determined in a non-contact remote manner.

Second Laboratory Experiment (Steel Channel)

Setup of Second Experiment (Steel Channel)

After experiencing success with IBIS in measuring the natural frequency in cables the idea came to try the same experiment with a rigid steel member. In particular, tension members in steel truss bridges are of interest. A change in the natural frequency of a steel truss member could indicate decay in the member or its associated welds is occurring. In order to physically fit a steel member into the MTS a rather large steel member (C10x15.3 A36 steel, as shown in Figure 4.3 was chosen. The steel member was put under tension loads ranging from 2.22 to 177.93 KN (0.5 to 40 Kips) (Figure 4.4). The steel member was struck with a rubber mallet and the vibration was measured with IBIS and an accelerometer that was mounted to the channel. Again, the accelerometer data was imported into Microsoft Excel and the time domain data was plotted in the frequency domain in order to view the peaks (natural frequencies) in the data. Due to the limitations of the accelerometer data was only collected under tension loads up to and including 44.48 KN (10 Kips) with the accelerometer.
Figure 4.3 Drawing of channel.

Figure 4.4 Channel testing in MTS.
Results of Second Experiment (Steel Channel)

Initially the accelerometer data displayed errors and it was realized the accelerometer was not well adhered to the channel and was moving independently of the channel as evidenced by shifts in the acceleration data in the frequency domain. Thus, the experiment was repeated. The results of this experiment were counter intuitive in that the accelerometer data indicate a decrease in natural frequency as the tension load increased. Further, the IBIS data and the accelerometer data did not correlate well. Through reviewing the data it became apparent that over a large increase in axial load (88.96KN (20 Kips) for instance) the natural frequency did not change drastically and as the axial load got higher, increments in axial load resulted in smaller changes in the natural frequency indicating a non-linear relationship exists between the two. It was also noted that the channel sustained plastic deformation around the holes that secured the channel to the MTS. It is likely that due to the relatively large stiffness of the channel, when compared to typical truss bridge tension members, the natural frequency is not a good indicator of the tension force.

Frequency domain plots from Microsoft Excel of each channel force increment measured are shown in Appendix C and on those plots the reader can view the peak which corresponds to the measured natural frequency. Appendix C includes frequency domain plots from IBIS’ MatLab based post processing system in which the reader can view the peak which corresponds to the measured natural frequency.
Conclusions from Second Experiment (Channel)

In summary, the results of this experiment were counter intuitive (the accelerometer data indicated a decrease in natural frequency as the tension load increased) and the IBIS data and the accelerometer data did not correlate well. Based on the results from the second experiment it was concluded that interferometric radar cannot be utilized to measure the fundamental frequency in steel channels with a ‘C’ shape until further research is conducted. It is possible that the material yielding and stress concentration at the connection points negatively affected the experiment.
CHAPTER V
FIELD WORK

Setup

Setting up IBIS in the field is very simple and intuitive. Setup simply involves setting up and leveling a tripod followed by attaching the radar head and connecting to the PC via a USB cord. In order to gather data on the global bridge movement setup is fairly simple; particularly for steel framed bridges with cross members or cross frames.

For the subject bridge (Figure 5.1) steel ‘W’ cross beams are spaced 3.96 M (13 feet) on center. The 90 degree bend at the web/flange connection on ‘W’ sections is a great reflector of radar. In this instance the user sets up the IBIS as close to one abutment or the other as reasonably possible and aims the radar head along the length of bridge as shown below in Figure 5.2. As long as the cross beams are at least 0.75 M (2.46 feet) apart (for range bin reasons) this setup allows the user to collect displacement data on all visible cross members at once. For the purposes of monitoring vibrations, the subject of this thesis, it is not critical to input the geometry of the bridge into the IBIS software. Inputting the bridge geometry and radar head angle/position is necessary if the user desires actual displacement data (refer to Figure 2.6 for explanation on how IBIS measures line of sight movement, not vertical displacement).
Figure 5.1 Subject cable stayed bridge.
Once setup and turned on, the user must determine which peaks on the radar display correspond to which structural elements. This is easily done with a laser distance meter which is positioned on the radar head. By documenting the position of the peaks, the user can accurately identify the structural elements.
each member and its corresponding radar peak the user can easily view the data at a later time and correctly understand/apply the data.

For the purposes of monitoring vibration in cables the positioning of the IBIS system is not as straightforward. Firstly, the user desires a location in which multiple cables can be viewed at once. For the subject bridge the maximum number of cables that can be viewed at once are three. However, the user must take care to insure that the cables are at least 0.75 M (2.46 feet) apart along the line of sight. Depending on the position of the IBIS it is quite possible that along the line of sight the cables could fall within the same range bin. An example of this is a setup where the IBIS is aimed directly perpendicular to the length of the bridge/cables – in this instance all the cables are in the same vertical plane. One might think that an ideal setup is at the end of the bridge looking down the length of the cables such that the cables will be more than 0.75 M (2.46 feet) apart along the line of sight. However, depending on the bridge geometry and construction, this setup would likely result in viewing vertical masts on the bridge and could possibly result in the first cable blocking the view of the additional cables. An ideal setup for a cable stayed bridge, when monitoring the cables, is one in which the instrument is located near the end of the bridge, looking down the length of the bridge, and the instrument is aimed such that no other bridge elements are in the field of view. In some instances the user can setup near the middle of the bridge and look upwards (Gentile, 2010). However, it is recognized that once a bridge is in service it is not very realistic to setup near the middle of the bridge due to the waterway or roadway.

Setup Comparisons

Setup and data were obtained both by setting up the instrument on the bridge deck (near the masts) and by setting up the instrument on the ground, isolated from the bridge vibrations. The data gathered from the setup on the bridge was noted to
sometimes exclude lower vibration modes which is possibly due to the instrument vibrating in-sync with the bridge. For this reason all subsequent setups were conducted from the ground so as to isolate the instrument from the vibrations. It should be noted that the vibrations on the subject bridge are very notable with large deflections (on the order of one CM (2.54 inches). Thus, for bridges that are larger and more stiff (automotive bridges) it may be possible to successfully monitor vibrations from the bridge deck itself; future research is needed on this topic.

**Testing and Results**

**Cable Vibration Measurement**

The subject bridge contains a total of 12 cables. According to the engineer of record (EOR) the top four cables were originally tensioned to 17.79 KN (4 Kips) each and the remainder of the cables were tensioned to sufficiently bring the bridge deck into the desired shape and alignment; thus, the original tension is only known in 4 of the 12 cables. A DynaTension P1000 instrument, an accelerometer and the IBIS instrument were used to measure the natural frequency in each of the 12 cables. Six samples per cable were taken with the P1000 and the results were averaged. With the accelerometer, a minimum of 2000 data points were collected and utilized in the Fourier analysis and this process was repeated and the results averaged. With the IBIS, measurements were taken for each cable with a minimum of 2000 data points and this process was repeated and the results averaged.

The results from the DynaTension meter are counterintuitive due to the relatively high frequencies reported by the instrument. It was theorized that the DynaTension meter was reporting higher mode shapes but review of the accelerometer data does not show good correlation for higher mode shapes and as such it was concluded the DynaTension meter data was erroneous. An attempt to repeat the measurements with
the DynaTension meter was made but after several attempts the exercise was abandoned due to the DynaTension meter not working correctly. Further, even if the DynaTension meter was successfully reporting higher mode shapes, the instrument has no way of letting the user know which mode shape he or she is collecting data on. Since Merhabi’s cable force determination equation is a function of both the natural frequency and mode shape, the DynaTension meter is not helpful for determining the force in the cable since the user has no way of knowing which mode he or she is measuring. The DynaTension meter results are shown in Table 5.1 below.

It took several attempts to collect valuable data with the accelerometer (see Figure 5.3 for a photograph of the accelerometer in use). After processing the accelerometer’s field data from the initial attempts it was realized errors must have occurred as the data had no clear peaks in the frequency domain. Based on permanent shifts in the accelerometer data when viewed in the time domain it was realized the instrument was vibrating independently of the cables and it was physically slipping downward on the cable due to poor attachment. Thus, the experiment was repeated by better securing the instrument to the cables. During the initial attempts to collect cable vibration data with the accelerometer it was thought the cables needed to be struck into free vibration with a rubber mallet as was done in the laboratory cable experiment. However, after securely affixing the instrument the data still did not appear correct in the time domain – in fact the only data that appeared correct was the initial data collected prior to striking the cable for the first time. Thus, it was concluded the bridge has enough vibration on its own (movement in the bridge and its cables is visible under no pedestrian loads which is a result of ambient environmental conditions such as minor wind loads) and there is no need to strike the cables. Once the experiment was repeated without striking the cables the data appeared very correct in both the time and
frequency domains and the data proved to be repeatable as the experiment was conducted twice. The final accelerometer data was determined to be accurate due to its appearance, intuitiveness, alignment with IBIS’ data, and the finding that cables in similar (mirror) positions on opposite sides of the bridge was very close to each other. The frequency data for each cable found from the accelerometer is shown in Table 5.1 and the frequency domain plots of the data for each cable are shown in Appendix D.

![Figure 5.3 Accelerometer installed on cable on subject bridge.](image)

It also took several attempts to gather usable cable data with the IBIS instrument. This was largely due to the learning curve encountered in trying to find the optimal setup position and in determining which radar peaks corresponded to which cables. Once the optimum setup position was found for each set of cables, data was gathered quickly. With a couple exceptions, the IBIS and accelerometer data correlated well (within 1/100th of a Hz). In the instances where the two don’t correlate well (error of 0.60 Hz) no real logical conclusion was achieved. In these instances the experiment should be repeated.
with both instruments in order to validate data, however this was not possible due to IBIS availability constraints. Nonetheless, there were multiple instances where the IBIS instrument’s data correlated very well with the accelerometer data and as such it was concluded the IBIS instrument can successfully be used to monitor the fundamental frequency in cables. The results of the IBIS measurements are shown in Table 5.1.

Both the accelerometer and IBIS frequency domain plots for the cables show smaller peaks at 1.67 and 1.98 Hz in multiple instances. The smaller peaks are consistent with global mode shapes. After review of the plots it was concluded that for the bottom (shorter cables with greater tension force) no smaller peaks existed and due to the large tension force in the cables and the cable’s close proximately to the center (fixed) support, the global mode shape was not visible. For the middle cables, a consistent small peak between 1.95 and 2.00 Hz was visible which is consistent with the measured global fundamental frequency of the bridge as discussed below. For the upper (longest) most cables a consistent small peak at approximately 1.66 Hz was observed. As no global mode shape at 1.66 Hz was observed, this peak was not expected and is not readily explainable. In the instances where the 1.66 peak was observed, no peak was observed at 1.98 with the accelerometer. However, in the same instances where a peak was observed at 1.66 with the IBIS instrument, a peak was observed at 1.98 which is an indication the first global mode shape is observable with IBIS but not with the accelerometer in some instances.
Table 5.1 Measured Fundamental Natural Frequency of Bridge Cable by Cable Position (Hz).

<table>
<thead>
<tr>
<th></th>
<th>NE Top</th>
<th>NE Middle</th>
<th>NE Bottom</th>
<th>NW Bottom</th>
<th>NW Middle</th>
<th>NW Top</th>
<th>SW Top</th>
<th>SW Middle</th>
<th>SW Bottom</th>
<th>SE Bottom</th>
<th>SE Middle</th>
<th>SE Top</th>
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</thead>
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<tr>
<td>IBIS</td>
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<td>3.80</td>
<td>5.21</td>
<td>6.03</td>
<td>3.46</td>
<td>2.44</td>
<td>2.14</td>
<td>3.77</td>
<td>6.62</td>
<td>6.12</td>
<td>4.05</td>
<td>2.72</td>
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<tr>
<td>Accelerometer</td>
<td>2.24</td>
<td>3.42</td>
<td>5.42</td>
<td>6.00</td>
<td>3.52</td>
<td>2.44</td>
<td>2.24</td>
<td>3.17</td>
<td>7.08</td>
<td>5.85</td>
<td>4.00</td>
<td>2.73</td>
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<tr>
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<td>16.74</td>
<td>23.1</td>
<td>24.47</td>
<td>23.47</td>
<td>22.52</td>
<td>17.15</td>
<td>17.16</td>
<td>25.2</td>
<td>21.58</td>
<td>23.45</td>
<td>22.85</td>
<td>15.75</td>
</tr>
</tbody>
</table>
Global Bridge Vibration Measurement

In order to determine the bridge’s global fundamental frequency the IBIS instrument was setup under the bridge and vibration data on multiple cross beams was gathered. This experiment was repeated numerous times and from both sides of the bridge as the center foundation prohibited viewing of all the cross members at once. The data gathered indicate that the cross member’s lowest vibration mode occurs at a peak of 1.98Hz consistently and as such it was concluded the fundamental frequency of the bridge is 1.98Hz.

Summary and Conclusions

In summary, field testing indicates that interferometric radar can successfully be used to monitor the global bridge natural frequency, mode shape and the individual bridge element’s (cables, masts, railing) fundamental frequencies.
CHAPTER VI

DISCUSSION

Introduction

Within this section of the thesis, the theoretical, laboratory and field testing results are combined to determine the success of utilizing interferometric technology to monitor cable stayed bridges. Further, the cable tension forces, and global bridge frequencies are determined.

Cable Calculations

As significant sag (more than 1 foot in some instances) was observed in the cables on the subject bridge it was considered if the taut string theory could be utilized to determine the tension force in the cables. As the cables were noted to be very flexible it was determined unlikely that the cables’ bending stiffness would affect the results of the taut string theory. Nonetheless, Ren et al’s research was utilized to determine if the cable bending stiffness or sag were going to significantly affect the taut string theory results.

The force in each cable was determined utilizing Merahbi’s equation and the measured frequency and that tension data is shown in Table 6.1. Also, the non-dimensional parameters set forth in Ren et. al’s work were calculated for each cable in order to determine the accuracy of the taut string theory for each of the cables. As discussed previously, Ren et al concluded that when $\lambda^2$ exceeds 1.0 for the first mode, the cable sag can be neglected and when $\zeta$ is greater than 50, the cable stiffness can be neglected. In Table 6.1 it can be seen that $\zeta$ is greater than 50 in every instance, thus the cable stiffness was neglected. The $\lambda^2$ term was calculated less than 1.0 in all but three cases and in one of the three cases the term was 1.32 which corresponds to a
relatively small error. For the two cases where the $\lambda^2$ term greatly exceeds 1.0, error in
the calculated tension force exists (likely on the order of 10%). In order to better
calculate the tension force in those two cases, a higher mode shape should be used.
For those two instances, the 5th mode was used, the tension force was corrected and
that corrected data is shown in Table 6.2. In comparing the corrected data in Table 6.2 it
is noted that the calculated tension force based on the 5th mode is quite different than
that calculated from the first mode. Given that the corrected tension force should only
be approximately 10% different between the two methods it was concluded that the 5th
mode was incorrectly determined and repeated attempts to re-analyze the data did not
yield better results. Thus, the data from Figure 6.1 was relied upon recognizing that in
some instances an error of approximately 10% likely exists.

It can be seen that the cable forces are relatively consistent across the bridge for
similar cables in similar positions. The only exception is one lower cable that has a
substantially higher (40% higher) tension force. As the IBIS and accelerometer data
compare well for that cable and multiple samples were taken it was determined the data
was accurate. One possible explanation is simply that the subject cable had to have a
higher tension force during construction in order to obtain the desired shape of the
bridge.
Table 6.1 Cable Force Determination.

<table>
<thead>
<tr>
<th>Effective Length (Feet)</th>
<th>Cable</th>
<th>Actual Length (Feet)</th>
<th>( f_n ) Measured with IBIS</th>
<th>( f_n ) Measured with Accelerometer</th>
<th>Tension Force Based on IBIS (Kips)</th>
<th>Tension Force Based on Accelerometer (Kips)</th>
<th>( \xi ) term from Ren et al.</th>
<th>( \chi^2 ) term from Ren et al.</th>
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<td>106.6</td>
<td>NE Top</td>
<td>108.8</td>
<td>2.22</td>
<td>2.24</td>
<td>6.59</td>
<td>6.71</td>
<td>127.92</td>
<td>2.67</td>
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<td>71.6</td>
<td>3.80</td>
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<td>15.65</td>
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<td>5.21</td>
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<td>39.30</td>
<td>101.61</td>
<td>0.00</td>
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<td>99.4</td>
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<td>101.4</td>
<td>2.44</td>
<td>2.44</td>
<td>7.97</td>
<td>7.97</td>
<td>131.06</td>
<td>1.32</td>
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<td>63.3</td>
<td>NW</td>
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<td>4.05</td>
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<td>31.8</td>
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<td>144.08</td>
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<td>36.4</td>
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Table 6.2 Corrected Cable Force Determination, Based on 5th Mode

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<th>Cable</th>
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<td>NE Top</td>
<td>127.9</td>
<td>7.08</td>
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<td>SW Top</td>
<td>144.2</td>
<td>7.56</td>
<td>3.06</td>
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</table>
Global Bridge Vibration Discussion

As seen in the cable data in Appendix D, global peaks in the cable data were also observed at 1.98Hz which confirms these conclusions. While gathering data on the cables it was possible to also gather data on the bridge masts and railing and both those items were also found to have a first global mode at 1.98Hz again confirming the conclusion that the bridge’s first global mode is 1.98Hz. The results of the floor beam, mast and tower data are shown in Figures 6.1 through 6.6. One particular floor beam displayed a smaller peak at 1.67Hz which is consistent with that observed on certain cables.

Figure 6.1 Radar plot of underside of bridge (peaks correspond to floor beams).
Figure 6.2  Frequency domain plot of underside of bridge, data for seven floor beams overlaid (Note: all members display peak at 1.986 Hz which was determined to be the first global natural frequency of the bridge).
Figure 6.3 Frequency domain plot of floor beam vibration on underside of bridge (Note: this member displays peaks at 1.67 (which correlates to cable vibration data) and 1.98 Hz which corresponds to the measured global bridge natural frequency of 1.98Hz).
Figure 6.4 Frequency domain plot of south tower vibration (Note: This Tower displays a peak at 1.94 Hz which is very close the measured global bridge natural frequency of 1.98 Hz).
Figure 6.5 Frequency domain plot of north tower vibration (Note: This Tower displays a peak at 1.98 Hz which matches the global bridge natural frequency of 1.98 Hz; Smaller peaks after peak at 1.98 Hz correspond to other global bridge mode shapes; Peak at 9.86 Hz which corresponds to the first local natural frequency of north tower).
Figure 6.6 Frequency domain plot of north railing (Note: This Tower displays a peak at 1.98 Hz which corresponds to the measured global bridge natural frequency of 1.98Hz).

Other peaks observed in the global survey was a large peak (the largest) at 2.64Hz and the next largest peak was at 3.3Hz which corresponds very closely with the SAP model output which indicated the fundamental frequency was 3.39Hz. The discrepancy between the SAP model and the field data is not readily explainable by this engineer and would require more research.

The MatLab based post processing used by IBIS has the ability to take all of the cross member time domain deflection data and overlay it simultaneously to create an animation of the bridge’s motion and to determine the mode shape. This process was completed and the deflected shape corresponds with that gathered from SAP in that the
two cantilevered ends, especially the east end, dominate the first mode shape (see Figure 6.7 below).

When inspecting the bridge it was noted that the vibrations were very noticeable and during time spent on the bridge pedestrians would routinely stop and ask if the bridge was supposed to vibrate in the manner which it is. These findings are consistent with Wiss et al’s work which indicates that the most severe and noticeable vibration in pedestrian bridges occurs at a frequency of 2Hz (the subject bridge vibrates at 1.98Hz).

**Pedestrian Bridge Application of Work**

As can been seen in the above work, the fundamental frequency of the subject bridge violates the AASHTO criteria which the bridge was designed with in regards to the 3.0Hz limitation for the fundamental frequency. It is unknown to this engineer as to whether or not the engineer of record performed calculations/models to check the bridge for vibrations both vertically and laterally as should have been done. The scope of this thesis is vertical vibrations; no investigation of lateral vibrations was conducted.
A quick check of Bachmann, et.al’s empirical equation for fundamental frequency of cable stayed bridges shows that their equation either does not apply well to pedestrian bridges or to bridges with multiple spans (Bachmann, et.al, 1995). Applying their equation to the subject bridge indicates a fundamental frequency of 3.47Hz. It is interesting that Bachmann et al’s equation (which is intended to be used for cable stayed bridges) is rather close to the SAP model output of 3.39Hz and that one of the modes measured in the field was at 3.3Hz. However, as the lowest mode measured in the field was 1.98Hz it was determined this is coincidental.

**Recommendations for City and County of Denver’s Cable Stayed Pedestrian Bridge at 16th Street Over The Platte River**

It is recommended that the subject bridge be monitored for signs of fatigue. Others have undertaken a bridge health monitoring protocol, index and baseline data for the bridge. Through time spent on the bridge it was noted that the fasteners that secure the decking to the structure often are backing out which result in not only loose deck planks but protruding screws which are a trip hazard. It is likely that the loose screws are a result of fatigue failure. Now that baseline data is available, the fundamental frequency of the bridge, cables and masts can be checked annually, or bi-annually, and any changes would warrant further investigation into potential decay or loss of stiffness. The costs associated with trying to stiffen the bridge would be large and may not be justified. The most economic method of solving the vibration problem would involve a tuned mass damper system which is quite costly. Further, this thesis excludes an evaluation of the bridge’s lateral fundamental frequency but such an analysis should be performed and its quite possible that problems exist in the lateral direction as well.
Summary and Conclusions

In summary, the cable vibration data can be used to determine the tension force in each of the cables. The global fundamental frequency of the subject bridge was determined to be approximately 1.98Hz. The force in each of the cables is shown in Table 6.1. The discrepancy between the SAP model and the field data is not readily explainable by this engineer and would require more research which is outside the scope of this thesis. It was concluded the AAHSTO empirical equations regarding fundamental frequency are not accurate for cable stayed bridges. The subject bridge violates AASHTO limitations for the fundamental frequency in the vertical direction. The consequence of the low fundamental frequency will likely be premature fatigue failure of one or more bridge elements. Future monitoring of the bridge should include vibration monitoring.
CHAPTER VII
CONCLUSIONS AND RECOMMENDATIONS

Conclusions

In summary, this thesis shows that interferometric radar can successfully be used to determine the fundamental frequency of bridge’s and their individual elements. The use of such radar has clear advantages in regards to ease of use, not needing direct access to the bridge, and large amounts of data in short amounts of time. For these reasons it is determined that interferometric radar is a good tool that can be used to monitor the structural health of bridges and other structures.

Vibration data from cables on cable stayed bridges can be used to determine the tension force in the cables. In many instances the taut string theory can be used and no correction for cable stiffness and sag is needed. However, in the case where correction is needed, higher vibration mode data can be used to determine the tension force more accurately.

It was concluded the AAHSTO empirical equations regarding fundamental frequency are not accurate for cable stayed bridges.

Recommendations

Regarding the City and County of Denver Bridge, 16th Street over the Platte, the fundamental frequency of the bridge is below 3.0Hz which is an AASHTO violation and this is of concern. Potential premature fatigue failure exists as a result of the low fundamental frequency (1.98Hz). The noticeable vibrations in the bridge and the 1.98Hz finding are consistent with Wiss et al’s work which indicates that the most severe and noticeable vibration in pedestrian bridges occurs at a frequency of 2Hz. Now that baseline data is available, the fundamental frequency of the bridge, cables and masts
can be checked annually, or bi-annually, and any changes would warrant further investigation into potential decay or loss of stiffness.

**Future Work**

Future work should be completed in order to better determine if interferometric radar can be used to determine the tension force in axially loaded steel members.

This thesis excludes an evaluation of the subject bridge’s lateral fundamental frequency but such an analysis should be performed and it’s quite possible that problems exist in the lateral direction as well.

Future research into the AASHTO empirical equations for limitations on pedestrian bridge weight and fundamental frequency should be conducted.

Further research into the accuracy of the SAP model, and the apparent discrepancy between field data, for the subject bridge should be considered.
REFERENCES


Friedrich Gauss, 11 Parc Mediterrani de la Tecnologia E-08860, Castelldefels, Barcelona.


APPENDIX A

Construction Drawings
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<td>Irrigation Details</td>
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1. Flood flow and elevation data for the South Platte River at 16th Street is as follows:

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<th>Return Frequency</th>
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<td>5179.9</td>
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<td>50-year</td>
<td>17,900</td>
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<td>10-year</td>
<td>8,800</td>
<td>5173.8</td>
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</table>

2. Flood elevations are based on 1988 NAVD Datum.

A. CRITERIA:

1. Design Loads
   a. Floor Live Loads
      Floor Live Loads are set according to AASHTO Guide Specification for the Design of Pedestrian Bridges, as noted.
   b. Concentrated Load: 10,000 lb
      live load reduction: design live loads are reduced for structure as described in the design criteria. truck load is not applied concurrent with pedestrian uniform loading.
   c. Snow Loads
      snow loads are in accordance with the city of Denver building department requirements.
   d. Flat roof snow load
      flat roof snow load: PY = 25 psf
   e. Uniform Load
      uniform load including suspended pipes: 10 psf
      uniform load including suspended pipes: 10 psf
   f. Foundation Loads: Class B2 concrete
      foundation walls: Class B2 concrete
      foundation walls will be designed for the maximum active or at-rest pressures, subject to confirmation by the geotechnical engineer.
   g. Active Pressure: 45 psi
   h. Friction Coefficient: 0.40

2. AASHTO SEISMIC CRITERIA:
   a. Importance Classification: I
   b. Soil Profile Type: II
   c. Site Coefficient: 0.5
   d. Maximum of 0.80
   e. (The listed soil profile type is assumed)

3. FOUNDATIONS AND SUBS
   Unless noted otherwise, all concrete foundations shown in structural drawings were previously constructed at an earlier date with exception of the center pier. The information below is provided in reference to the existing construction and/or applies to new construction.
   a. Geotechnical
      (available at the office of the city and county of Denver department of public works)
      project No: 981507, February 5, 1995
      Masin technologies, inc.
      2600 18th street
      denver, co 80211
      drilled piles:
      40,000 lb allowable end bearing
      4,000 lb allowable skin friction
      6" minimum into weathered bedrock
      16" minimum drilled length
   b. Ancillary and Retaining Structures:
      spread footings, class b concrete
      minimum depth required from grade to bottom of footing = 2'-6"
      minimum dimensions of column footing = 24" square
      minimum width of continuous wall footing = 18"
      bearing capacity = 1,500 psf, 6 psf minimum
   c. Foundation Walls, Class B2 concrete
      foundation walls will be designed for the maximum active or at-rest pressures, subject to confirmation by the geotechnical engineer.

3. STRUCTURAL STEEL:
   a. The fabricator shall be pre-qualified and certified through ASC Quality Certification Program, under either Category C or C (Complex Steel Building Structures).
   b. Poles, angles, and channels shall conform to ASTM A36.
   c. WF sections shall conform to A-572 Grade 50, U.N.O. or ASTM 492.
   d. T-1 sections shall conform to ASTM A-500, Grade B, Ty=68 ksi.
   e. Pipe shall conform to ASTM A-53 Type E, Grade B, or ASTM A-333.
   f. All bolts and studs shall conform to ASTM A-307 U.N.O.
   g. Anchor bolts shall conform to ASTM A-307.
   h. Weld lengths shown on the plans are not effective lengths required. fillet weld sizes are the width of the leg. the minimum fillet weld size shall be 1/8", where the length of the weld is not shown, it shall be the full length of the joint.

3. The contractor shall notify the engineer of any conflicts or inconsistencies in the drawings or specifications. The contractor and engineer shall resolve any conflict in writing before proceeding with the work.

4. All construction materials shall be distributed on the structure to stay within the design loads. the contractor is responsible for any temporary shoring, bracing, and safety. some items will require the use of a Colorado registered Professional engineer (see 61.4).

5. The contractor is responsible for the stability of the structure during construction.
PLAN NOTES:

1. PROVIDE (3#10 THHN & 1#10 GND.) IN 3/4" PVC.

2. PROVIDE THIS WEATHER PROOF J-BOX ON THE END OF THE EXISTING PVC CONDUIT TO THE EXISTING PARK SHADE RECEPTACLE AND LIGHTING.

3. PROVIDE (3#10 THHN & 1#10 GND.) IN EXISTING 3/4" PVC CONDUIT TO THE EXISTING RECEPTACLE.

4. PROVIDE (2#10 THHN & 1#10 GND.) IN EXISTING 1/2" PVC CONDUIT TO EXISTING PARK SHADE LIGHTS.

5. CONNECT THE (9) EXISTING STEP LUMINAIRES TO THE EXISTING LIGHTING CONTROL AND PANEL "L1." INSTALL THE TWO TOP STEP LIGHTS INCLUDING LAMPS AND BALLASTS THAT EXIST IN THE BOTTOM OF THE PANEL "L1." ENCLOSURE. PROVIDE AND INSTALL ALL CONDUIT AND CONDUCTORS AS REQUIRED.

LIGHTING LEGEND

- TYPE "SL1" PEDESTRIAN STEP LIGHT LUMINARIE
- TYPE "SM1" UNDER DECK LUMINARIE
- EXISTING STEP LUMINARIE

SEE SHEET #E1.2 FOR CONTINUATION OF THIS CIRCUIT TO THE TYPE "SM1" UNDER BRIDGE LUMINAIRES.

GENERAL ELECTRICAL NOTES:

1. REUSE PANEL "L1," LIGHTING CONTROL - CONTACTOR, TIME SWITCH, PHOTOCELL AND ALL ASSOCIATED PARTS INCLUDING NEMA 4X STAINLESS STEEL ENCLOSURE THAT EXISTS NORTH-WEST OF THIS BRIDGE LOCATION AND NEAR THE UTILITY TRANSFORMER; COMPLETE THE ASSEMBLY FOR ALL COMPONENTS WITHIN THIS NEMA ENCLOSURE.

2. PROVIDE AND INSTALL CONCRETE ANCHORS ON THE EXISTING NEMA 4X ENCLOSURE NOTED ABOVE, (MINIMUM OF 8 ANCHORS REQUIRED).

3. REUSE THE EMPTY CONDUIT HOMERUNS BETWEEN PANEL "L1" AND TO THE EXISTING BRIDGE SUPPORT STRUCTURE.
DETAIL NOTES:
1. TYPE "SL1" COMPACT FLUORESCENT LUMINAIRE MOUNTED IN HANDRAIL. REFER TO DETAIL ON SHEET E1.3.
2. HORIZONTAL CONDUCTS BETWEEN THE TYPE "SL1" LUMINAIRSES SHALL BE 3" TRADE SIZE CONDUIT. REFER LUMINAIRE TYPE "SL1" MOUNTING DETAIL ON SHEET E1.3 FOR MOUNTING REQUIREMENTS.

GENERAL NOTES:
1. ALL CONDUCTORS TO THE BRIDGE LIGHTING LUMINAIRSES SHALL BE A MINIMUM OF 3/8" THICK.
2. ALL HOMERUN CONDUITS SHALL BE A MINIMUM 3/4" TRADE SIZE CONDUIT, UNLESS OTHERWISE NOTED.
3. CONDUITS ON THIS DRAWING ARE SHOWN DIAGRAMMATICALLY TO INDICATE SEPARATION OF SWITCHED CIRCUITS. ROUTE CONDUITS IN COMMON CONDUIT WHEREVER POSSIBLE.
DETAIL NOTES:

1. TYPE "SL1" COMPACT FLUORESCENT LUMINAIRE MOUNTED IN HANDRAIL. REFER TO DETAIL ON SHEET E1.3.

2. HORIZONTAL CONDUITS BETWEEN THE TYPE "SL1" LUMINAIRE SHALL BE 3/4" TRADE SIZE CONDUIT. REFER LUMINAIRE TYPE "SL1" MOUNTING DETAIL ON SHEET E1.3 FOR MOUNTING REQUIREMENTS.

GENERAL NOTES:

1. ALL CONDUCTORS TO THE BRIDGE LIGHTING LUMINAIRE SHALL BE A MINIMUM OF #10 THHN

2. ALL HOMERUN CONDUITS SHALL BE A MINIMUM 3/4" TRADE SIZE CONDUIT, UNLESS OTHERWISE NOTED.

3. CONDUITS ON THIS DRAWING ARE SHOWN DIAGRAMMATICALLY TO INDICATE SEPARATION OF SWITCHED CIRCUITS. ROUTE CONDUCTORS IN COMMON CONDUIT WHEREVER POSSIBLE.

CITY AND COUNTY OF DENVER
DEPARTMENT OF PUBLIC WORKS

As Constructed

BRIDGE LIGHTING PLAN
EAST

Project No./Code: 17141

112
SECTION

TYPE "SL1" MOUNTING DETAIL

SCALE: 1'-0" = 20" 1" = 1'-0" @ 1:117

NOTE: CONTRACTOR MUST PROVIDE THE RAIL FABRICATOR WITH SHOP DRAWINGS FOR THE TYPE "SL1" LUMINARE

SUMMARY OF QUANTITIES
(FOR INFORMATION ONLY)

<table>
<thead>
<tr>
<th>PAY ITEM</th>
<th>UNITS</th>
<th>ESTIMATED QUANTITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>LUMINARE TYPE &quot;SL1&quot;</td>
<td>LINEAR FT.</td>
<td>36</td>
</tr>
<tr>
<td>LUMINARE TYPE &quot;SL2&quot;</td>
<td>each</td>
<td>2</td>
</tr>
<tr>
<td>STEP LIGHTS INSTALLATION</td>
<td>LUMP SUM</td>
<td>1</td>
</tr>
<tr>
<td>1/2&quot; GRC ELECTRICAL CONDUIT</td>
<td>LINEAR FT.</td>
<td>80</td>
</tr>
<tr>
<td>1/2&quot; ELECTRICAL CONDUIT</td>
<td>LINEAR FT.</td>
<td>480</td>
</tr>
<tr>
<td>3/4&quot; ELECTRICAL CONDUIT</td>
<td>LINEAR FT.</td>
<td>320</td>
</tr>
<tr>
<td>WIRING</td>
<td>LUMP SUM</td>
<td>1</td>
</tr>
<tr>
<td>CONTROL CABINET MODIFICATIONS</td>
<td>EACH</td>
<td>1</td>
</tr>
<tr>
<td>ELECTRICAL CONNECTION TO EXISTING SHELTER</td>
<td>EACH</td>
<td>1</td>
</tr>
<tr>
<td>WHEEL-PROOF PULL BOXES</td>
<td>EACH</td>
<td>3</td>
</tr>
</tbody>
</table>

LUMINARE SCHEDULE

<table>
<thead>
<tr>
<th>TYPE</th>
<th>MANUFACTURER/MODEL</th>
<th>LAMP</th>
<th>FINISH</th>
<th>VOLTS</th>
<th>DESCRIPTION / NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL1</td>
<td>GARDCO #43S-SURFACE-128 LT-LV-optional color</td>
<td>(1) CF26 6/7/9/13/15/20</td>
<td>Optional Color to match galvanized steel</td>
<td>120</td>
<td>Surface mounted fluorescent step light attached to railing as detailed, complete with waterproof enclosure and mounting brackets/hardware. Step light shall be mounted in the vertical position with horizontal louvers as shown in the type &quot;SL1&quot; mounting detail on this sheet. Step light shall have vandal resistance construction with vandal resistant hardware. Special entry tools shall also be supplied if required. Fluorescent lamp shall have end of life protection and electronic ballast shall be capable of starting at -20°F. Entire housing including louvers shall be die cast aluminum with fade and abrasion resistant, electrostatically applied powdercoat finish, ETL or UL listed for wet locations.</td>
</tr>
<tr>
<td>SM1</td>
<td>KIM AFT SERIES AL12/70MH/120/20- P/HOS/LS or pre-approved equal</td>
<td>MP70C/U/MED</td>
<td>Dark Bronze</td>
<td>120</td>
<td>Stem mounted adjustable metal halide floodlight mounted underneath bridge deck to light pedestrian path under bridge; heavy duty swivel, polycarbonate lens shield; UL listed for wet location.</td>
</tr>
</tbody>
</table>
LAYOUT NOTES

1. All layout dimensions are from plans for calculating action areas, other than those noted on this drawing. All construction areas are shown for planning purposes only. Therefore, any visibility or interference with existing structures or developments shall be verified prior to construction.

2. All layout dimensions are to be used as guidelines only. Final adjustments shall be made on site.

3. Door sizes are shown for planning purposes only. Final adjustments shall be made on site.

4. All layout dimensions are from plans for calculating action areas, other than those noted on this drawing. All construction areas are shown for planning purposes only. Therefore, any visibility or interference with existing structures or developments shall be verified prior to construction.

5. Door sizes are shown for planning purposes only. Final adjustments shall be made on site.

6. All layout dimensions are from plans for calculating action areas, other than those noted on this drawing. All construction areas are shown for planning purposes only. Therefore, any visibility or interference with existing structures or developments shall be verified prior to construction.

7. Door sizes are shown for planning purposes only. Final adjustments shall be made on site.

8. All layout dimensions are from plans for calculating action areas, other than those noted on this drawing. All construction areas are shown for planning purposes only. Therefore, any visibility or interference with existing structures or developments shall be verified prior to construction.

9. Door sizes are shown for planning purposes only. Final adjustments shall be made on site.

10. All layout dimensions are from plans for calculating action areas, other than those noted on this drawing. All construction areas are shown for planning purposes only. Therefore, any visibility or interference with existing structures or developments shall be verified prior to construction.

11. Door sizes are shown for planning purposes only. Final adjustments shall be made on site.
APPENDIX B
First Laboratory Test – Cable Test

Figure B1  Laboratory cable measured with accelerometer, 2 kips tension force.

Figure B2  Laboratory cable measured with IBIS, 2 kips tension force.
Figure B3  Laboratory cable measured with accelerometer, 4 kips tension force.

23.53 Hz

Figure B4  Laboratory cable measured with IBIS, 4 kips tension force.

23.94 Hz
Figure B5  Laboratory cable measured with accelerometer, 6 kips tension force.

Figure B6  Laboratory cable measured with IBIS, 6 kips tension force.
Figure B7  Laboratory cable measured with accelerometer, 8 kips tension force.

Figure B8  Laboratory cable measured with IBIS, 8 kips tension force.
Figure B9  Laboratory cable measured with accelerometer, 10 kips tension force.

Figure B10  Laboratory cable measured with IBIS, 10 kips tension force.
Figure B11  Laboratory cable measured with accelerometer, 12 kips tension force.

Figure B12  Laboratory cable measured with IBIS, 12 kips tension force.
Figure B13  Laboratory cable measured with accelerometer, 14 kips tension force.

Figure B14  Laboratory cable measured with IBIS, 14 kips tension force.
Figure B15  Laboratory cable measured with accelerometer, 16 kips tension force.

Figure B16  Laboratory cable measured with IBIS, 16 kips tension force.
Figure B17  Laboratory cable measured with accelerometer, 18 kips tension force.

Figure B18  Laboratory cable measured with IBIS, 18 kips tension force.
Figure B19  Laboratory cable measured with accelerometer, 20 kips tension force.

Figure B20  Laboratory cable measured with IBIS, 20 kips tension force.
Figure B21  Laboratory cable measured with accelerometer, 22 kips tension force.

Figure B22  Laboratory cable measured with IBIS, 22 kips tension force.
Figure B23 Laboratory cable measured with accelerometer, 24 kips tension force.

Figure B24 Laboratory cable measured with IBIS, 24 kips tension force.
Figure B24  Laboratory cable measured with accelerometer, 26 kips tension force.

Figure B25  Laboratory cable measured with IBIS, 26 kips tension force.
APPENDIX C
Second Laboratory Test – Channel Test

Figure C1  Laboratory cable measured with accelerometer, 0.5 kips tension force.

Figure C2  Laboratory cable measured with IBIS, 0.5 kips tension force.
Figure C3  Laboratory cable measured with accelerometer, 1 kips tension force.

Figure C4  Laboratory cable measured with IBIS, 1 kips tension force.
Figure C5  Laboratory cable measured with accelerometer, 1.5 kips tension force.

Figure C6  Laboratory cable measured with IBIS, 1.5 kips tension force.
Figure C7  Laboratory cable measured with accelerometer, 2 kips tension force.

Figure C8  Laboratory cable measured with IBIS, 2 kips tension force.
Figure C9  Laboratory cable measured with accelerometer, 2.5 kips tension force.

Figure C10  Laboratory cable measured with IBIS, 2.5 kips tension force.
Figure C11  Laboratory cable measured with accelerometer, 3 kips tension force.

Figure C12  Laboratory cable measured with IBIS, 3 kips tension force.
Figure C13 Laboratory cable measured with accelerometer, 3.5 kips tension force.

Figure C14 Laboratory cable measured with IBIS, 3.5 kips tension force.
Figure C15 Laboratory cable measured with accelerometer, 5 kips tension force.

Figure C16 Laboratory cable measured with IBIS, 5 kips tension force.
Figure C17  Laboratory cable measured with accelerometer, 10 kips tension force.

Figure C18  Laboratory cable measured with IBIS, 10 kips tension force.
Figure C19  Laboratory cable measured with IBIS, 15 kips tension force.

Figure C20  Laboratory cable measured with IBIS, 20 kips tension force.
Figure C21 Laboratory cable measured with IBIS, 25 kips tension force.

Figure C22 Laboratory cable measured with IBIS, 30 kips tension force.
Figure C23  Laboratory cable measured with IBIS, 35 kips tension force.

Figure C24  Laboratory cable measured with IBIS, 40 kips tension force.
APPENDIX D
Field Measurement of Cable Vibration on Cable Stayed Pedestrian Bridge at 16th Street over the Platte River
Figure D1  Field cable measured with accelerometer, northeast top.

Figure D2  Field cable measured with IBIS, northeast top.
Figure D3  Field cable measured with accelerometer, northeast top, fifth mode.

Figure D4. Field cable measured with accelerometer, northeast middle.
Figure D5 Field cable measured with IBIS, northeast middle.

Figure D6 Field cable measured with accelerometer, northeast middle, fifth mode.
Figure D7  Field cable measured with accelerometer, northeast bottom.
Figure D8  Field cable measured with IBIS, northeast bottom.

Figure D9  Field cable measured with accelerometer, northeast bottom, fifth mode.
Figure D10  Field cable measured with accelerometer, northwest bottom.

Figure D11  Field cable measured with IBIS, northwest bottom.
Figure D12  Field cable measured with accelerometer, northwest bottom, fifth mode.

Figure D13  Field cable measured with accelerometer, northwest middle.
Figure D14  Field cable measured with IBIS, northwest middle.
Figure D15  Field cable measured with accelerometer, northwest middle, fifth mode.

Figure D16  Field cable measured with accelerometer, northwest top.
Figure D17 Field cable measured with IBIS, northwest top.

Figure D18 Field cable measured with accelerometer, northwest top, fifth mode.
Figure D19  Field cable measured with accelerometer, southwest top.
Figure D20  Field cable measured with IBIS, southwest top.

Figure D21  Field cable measured with accelerometer, southwest top, fifth mode.
Figure D22  Field cable measured with accelerometer, southwest middle.
Figure D23  Field cable measured with IBIS, southwest middle.

Figure D24  Field cable measured with accelerometer, southwest middle, fifth mode.
Figure D25  Field cable measured with accelerometer, southwest bottom.
Figure D26  Field cable measured with IBIS, southwest bottom.

Figure D27  Field cable measured with accelerometer, southwest bottom, fifth mode.
Figure D28  Field cable measured with accelerometer, southeast bottom.

Figure D29  Field cable measured with IBIS, southeast bottom.
Figure D30  Field cable measured with accelerometer, southeast bottom, fifth mode.

Figure D31  Field cable measured with accelerometer, southeast middle.
Figure D32  Field cable measured with IBIS, southeast middle.

Figure D33  Field cable measured with accelerometer, southeast middle, fifth mode.
Figure D34  Field cable measured with accelerometer, southeast top.
Figure D35  Field cable measured with IBIS, southeast top.

Figure D36  Field cable measured with accelerometer, southeast top, fifth mode.