Alaska Department of Transportation & Public Facilities
Alaska University Transportation Center

Life Cycle Costs for Alaska Bridges

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A study was implemented to assist the Alaska Department of Transportation and Public Facilities (ADOT&PF) with life cycle costs for the Alaska Highway Bridge Inventory. The study consisted of two parts. Part 1 involved working with regional offices of ADOT&PF to assemble bridge costs (initial, construction, maintenance and repair) for a sample of bridge types. Results were limited by available data and format; therefore, it is recommended that ADOT&PF develop an available online simple Bridge Management archiving system.

Part 2 focused on identifying how a bridge scheduled for replacement deteriorated over time. Load tests were conducted to help assess the bridge response of an aged structure. Noticeable and measurable differences in strain at the end of the bridge life for the structure studied were found.

A service life cycle costing approach has advantages over a traditional life cycle cost approach. For example, a bridge has essentially three lives; structural, functional and service. Structural life can be extended almost indefinitely with the right repairs. The service life approach does not assume a life. Rather it used to estimate a life that provides the lowest life cycle cost. A service life approach allows comparisons of alternatives for an infinite planning horizon.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
	NOTE: volumes greater than 1000 L shall be shown in m^3			
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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EXECUTIVE SUMMARY

This study was implemented to assist the Alaska Department of Transportation and Public Facilities (ADOT&PF) with its life cycle costs for the Alaska Highway Bridge Inventory. The study consisted of two parts. Part 1 involved working with regional offices of ADOT&PF to assemble initial costs, construction costs, and maintenance and repair for a sample of the different bridge types. As part of that effort, ADOT&PF provided the research team with Pontis (the AASHTO Bridge Management software) that was being used by the department. The software has since been updated and is now called AASHTOWare (2014). The results of this effort were limited by the available data. Presently, it is not feasible to finalize life cycle costing for the Alaska Highway Bridge Inventory because each region files its data in a different format and archived construction costs are extremely difficult to find. It is recommended that ADOT&PF develop a simple Bridge Management archiving system that is available online. This archiving system can then be used to maintain bridge records for initial construction, maintenance, and rehabilitation costs, and their relationship to the bridge inventory records.

Part 2 involved an attempt to identify how a bridge that was scheduled for replacement deteriorated over time. The initial plan was to take samples from the steel members of the Noyes Slough Bridge and identify their stress-strain response to load. The proposed research plan was to compare the behavior of steel subjected to load over its lifetime with that of new steel having nearly the same material properties. However, because of environmental constraints, this portion of the work was not funded. Therefore, the research findings presented are limited to the available resources of field testing and computer simulation to evaluate structural response and compare it with that of the initial bridge, before it had aged. The research results showed noticeable and measurable differences in the strain behavior of the Noyes Slough Bridge at the end of its life cycle when compared with its theoretical beginnings. Strain gauges were used to calculate strength loss. Using measured changes in strain in the bridge girders, a comparison was made between the bridge's condition at the time of evaluation and its condition at the time of original construction. The results illustrated that the structure had yielded at several points within the girders. These strain values indicate that the structure may have been overloaded at least once during its lifetime.

The application of a service life cycle costing approach has a number of advantages over the traditional life cycle cost approach. A bridge has essentially three lives; structural, functional

and service. All of these lives are highly variable. For example the structural life of a bridge can be extended almost indefinitely with the right repairs. The service life approach does not assume a life. Rather it estimates the life that provides the lowest life cycle cost. Doing so allows comparisons of alternatives assuming an infinite planning horizon.

1 INTRODUCTION

1.1 General

The life of a bridge can be defined in multiple ways; structural, functional and service life. The most common of these is the structural life, which can be defined as the point at which the cost of repairing the bridge due to a structural element becomes economically unattractive. The structural life of a bridge is critical, because at some point the bridge requires repair, load reduction, or replacement. At this point, the associated costs should be compared to select the most cost-effective alternative.

The functional life of a bridge is reached when a bridge no longer serves the need of the public because of increased traffic, vehicle loading, or other required functions. As with the structural life, there are multiple alternatives that correct functional deficiencies, including replacement and upgrading, or rerouting traffic. Again, each alternative can be analyzed to ascertain which is the most cost-effective.

The service life of a bridge is defined as that which minimizes its life cycle costs. The advantage of this approach is the ability to merge functional and structural costs into a single analysis. The service life of a bridge can be determined by computing either present worth or equivalent annual costs, including the initial cost, for each year until a minimum life cycle cost is found. The service life of the bridge does not mean the bridge has reached either its structural or functional life. Rather the service life is simply the point at which life cycle costs begin to increase after being at a minimum. Ideally, actual cost data adjusted for inflation would be used to establish service life for each structural type. Unfortunately, those data do not exist, which leaves two options: develop anticipated costs based on anecdotal data or develop a process that can be applied as data become available. After a consultation with ADOT&PF, it was decided that the second option would be chosen. That process is presented in Chapter 3.

The Noyes Slough Bridge (#0283), in service for over 60 years, provides an excellent example of the life cycle of a bridge. This three-span composite steel girder, 172-foot-long bridge had a roadway width of 30 feet (Figure 1.1, and 1.2). A 5-foot-wide sidewalk was built on either side of the bridge in 1951. The structure was designed for AASHTO H20-44 highway loads. The steel girders met the ASTM A242-4b. The 7¼-inch-thick reinforced concrete deck was designed for 3000 psi 28-day concrete compressive strength. In 1982, about 850 square feet

of deck surface (the deck was 3843 sq ft) was delaminated and overlaid with 1 inch of polymer concrete.

Figure 1.1 Side view of the Noyes Slough Bridge

Figure 1.2 Westbound view of Noyes Slough Bridge

Recently, the Noyes Slough Bridge, located in Fairbanks, Alaska, near the intersection of Illinois Street and College Road, was replaced. Six decades of traffic, as well as extreme thermal expansion and contraction, had taken its toll on the bridge's structural integrity. This research

provides an estimate of strength loss in the structural steel of the bridge, as well as the bridge's most economical life cycle.

1.2 Overview and Scope of Work

When the Noyes Slough Bridge was considered for demolition, a project to evaluate the structure's remaining life was envisioned. The steel would be reclaimed, and strength tests would be conducted. The bridge's strength loss over its service time and its structural life cycle would be learned. Due to unforeseen situations and higher-priority projects, the bridge was not demolished during the study period; thus, an alternate course of action was planned.

A SAP2000 model of the bridge was created for use in determining the deflections of the bridge as though it were newly constructed. After the bridge was instrumented, the structure was tested by driving and stopping on the bridge at predetermined locations. The test vehicle was a fully loaded ADOT&PF International 7600i dump truck. The dump truck was parked at locations to establish the highest shear forces and moments in the support girders. A CR5000 Campbell Scientific data logger was used to record data from strategically placed thermistors, accelerometers, and strain gauges. Deflections recorded for maximum shear and moment tests were used to assess strength loss. Using this information, a theoretical life cycle was calculated.

It was proposed that construction and maintenance data for bridges in Alaska would be gathered from ADOT&PF's master archive list and used for cost comparison with other similar standing highway bridges in the state.

In other words, the study was performed in three stages: (a) Evaluate the remaining life of the Noyes Slough Bridge by experimentally determining its condition; (b) create a SAP2000 three-dimensional finite element model of the bridge and analyze the structure for loaded dump trucks; and (c) based on ADOT&PF records, perform an economic analysis of highway bridges throughout Alaska.

1.3 CR5000 Data Monitoring System

The programmed Campbell Scientific CR5000 data logger (see Figure 1.4) is a rugged, high-performance data acquisition system with a built-in keyboard, graphics display, and PCMCIA card slot. It combines 16-bit resolution with a maximum throughput of 5000 measurements per second, has multiple input channels, and can measure a large number of sensors.

The integrated keyboard and display screen on the CR 5000 data logger allows the user to program, manually initiate data transfers, and view data on-site during testing. The integrated PCMCIA slot accepts memory cards up to 2 GB for stand-alone data collecting. The CR5000 includes a current excitation channel that allows a direct connection of PRTs or other sensors that use current excitation.

A battery-backed SRAM and clock ensure that data, programs, and accurate time are maintained while the data logger is disconnected from the main power source. The CR5000 can be used to collect and store experimental data; it can control peripherals and has proven valuable for cold weather applications.

Figure 1.4 Campbell Scientific CR5000 data acquisition system

1.4 Instrumentation

Installation of the sensors required a staging platform, so a rolling platform was designed to roll underneath the girders using the flanges as a guide (see Figure 1.5). Each gauge was programmed into the data logger and installed on the bridge. A 12V deep cycle car battery was used as a power source, and data was recorded and stored every time a vehicle triggered the data logger.

Using as-built bridge construction drawings provided from ADOT& PF, a three-dimensional model was prepared in SAP2000. Using this computer model, the bridge response

was simulated and compared with experimental data collected from the bridge site where a loaded dump truck was positioned at various locations on the bridge. These tests were used to correlate bridge behavior in relation to expected behavior when the bridge was new.

Finally, research was conducted to evaluate maintenance and construction data that was provided by ADOT&PF.

Figure 1.5 Rolling platform and data collection staging area

2 LITERATURE REVIEW

Of the approximately 600,000 bridges in the United States, 24% are considered structurally deficient or functionally obsolete. This pattern continues when focusing on Alaska. Of the state's 1156 bridges, about 22% are considered in the same state of disrepair as bridges in other states (Ahmad, 2011). Many of these bridges have to be repaired or replaced. The question is, Which bridge type is the most economical? And another, How much money can be saved if bridges are replaced with the longest lasting, lowest construction and maintenance cost bridge?

Low-cost sensors and data collection systems allow rapid collection of data that may indicate bridge health including loss of strength. Such information can be the first step in determining many previously unknown facets of bridges, namely an accurate life cycle, an economical life cost, and the type of bridge that is most cost-effective (Kerley and Lwin, 2011; Kendall, Keoleian, and Helfand, 2008; Hawk, 2003). Other associated life cost studies were reported by (Zhang, et al, 2003, 2005 and 2008).

Similar economics studies have been done on Bascule bridges in Chicago, Illinois, where the bridge life exceeded the customary 75-year life cycle (Krizek et al., 2003). Their study found that

> the useful life is defined as the period from the initial construction to the point where the reconstruction cost is greater than the initial cost. Most of the Chicago bridges studied have never been reconstructed and have had a useful life greater than 75 years, which is commonly assumed to be bridge design life. The first Chicago-type Bascule, the Cortland Street Bridge, is currently more than 102 years old, page 2.

Krizek et al. (2003) also determined that

> the results of their brief study showed that (a) the achievable useful life of a bascule bridge can be more than 100 years, (b) the total life cycle cost of a 100-year-old bascule bridge can be less than five times its initial cost, and (c) timely [maintenance, repair, and rehabilitation] actions can lower the total life cycle cost of the bridge, page 6.

Therefore, clearly there is sparse data or supporting evidence that shows these bridges are capable of withstanding 75 years of service, even though many are being used long after their expiration date. America's infrastructure is aging, and sufficient data to help us understand exactly what is happening to aging bridges is unavailable. Even though Alaska is younger due to

statehood being more recent in comparison with the rest of the United States, the situation of having insufficient data is no different.

Consider that throughout the useful life of bridge structures, each bridge is subjected to routine and periodic maintenance, occasional rehabilitation, and some replacement work. Some parts of the bridge deteriorate faster than others, and in some cases, a structure can experience overloads or crash loads caused by different types of traffic accidents. Therefore, bridges require a number of expenditures for various activities during their useful life (Hawk, 2003).

For bridge structures, maintenance and rehabilitation may be a significant part of the Bridge Life-Cycle Cost Analysis (BLCCA), which can be expressed by

$$LCC = DC + CC + MC + RC + UC + SV \tag{1}$$

where
LCC = life cycle cost;
DC = design cost;
CC = construction cost;
MC = maintenance cost;
RC = rehabilitation cost;
UC = user cost, and
SV = salvage cost.

Typically, user costs for bridge structures are a small part of the costs. The majority of these costs occur during traffic congestion, bridge maintenance, or bridge rehabilitation. Mohammadi et al. (1995) illustrated that a single parameter may be used to quantify the bridge decision-making process. In this study, three elements were combined in the BLCCA: (a) bridge condition rating, (b) costs resulting from work on the bridge, and (c) bridge life expectancy. Mohammadi et al. (1995) illustrated this as follows:

$$VI = F(r, c, t) \tag{2}$$

where
VI = bridge value index,
F = objective function,
r = condition rating,
c = costs, and
t = time or bridge service life expectancy

Life cycle costs have been used in the transportation field for some time; however, disagreement remains as to which cost items should be included with a given analysis. Delay

costs, fuel costs, vehicle operating costs, etc., are at the heart of the controversy. Other non-agency costs such as environmental factors, expected life, changes in traffic patterns, the influence of overloaded vehicles, deterioration rates, are difficult to estimate. In summary, bridge life expectancy is not well defined; bridge behavior will affect a bridge's expected life and can change with type of materials, climatic exposure, traffic loads, overloads, and vehicle crashes with the bridge structure (Hawk, 2003).

This study examines expected costs for different bridge types in the state of Alaska and the aged response of the Noyes Slough Bridge. Initially, an evaluation of the condition of the structural steel bridge members was proposed, sampling and testing their response in relationship to the response of a new steel member. This idea was abandoned because of the risks of handling and the expense associated with containment of the lead paint on the existing steel members. Therefore, the second part of this study was limited to evaluating the bridge girder response to a heavy truck load in an effort to determine the overall condition and remaining life of the structural elements.

3 NOYES SLOUGH BRIDGE, PART 1

3.1 General Information

The initial portion of the analysis of the Noyes Slough Bridge is based on experimental results that were found by measuring strain, accelerations, and member temperature. Measurements were taken using weldable strain gauges, accelerometers, and thermistors. The intention with this approach was to provide a better understanding of how a small multispan bridge in Alaska reacts to daily traffic loading. Data from the measurements would be useful in determining aspects of the bridge's characteristics, such as the life cycle of the major structural carrying members or strength loss over time.

3.2 Construction/Planning

To fully examine how the Noyes Slough Bridge reacts to everyday traffic loading, one end span and the mid span were instrumented. Due to budgetary limitations and accessibility of the undercarriage, focus was on only two of the three spans. This bridge is theoretically symmetrical, so the two end spans should act the same (see Figures 3.1, 3.2, and 3.3). The concrete deck was constructed to act compositely with the steel girders, but each steel girder is simply supported between bents. In 1951, the concrete deck was built with a 28-day concrete compressive strength of 3000 psi. By 1982, approximately 22% of the deck was delaminated. That same year, deck repair consisted of replacing reinforcing bars, installing polymer concrete patches, and placing a 1-inch polymer modified concrete overlay on the bridge deck.

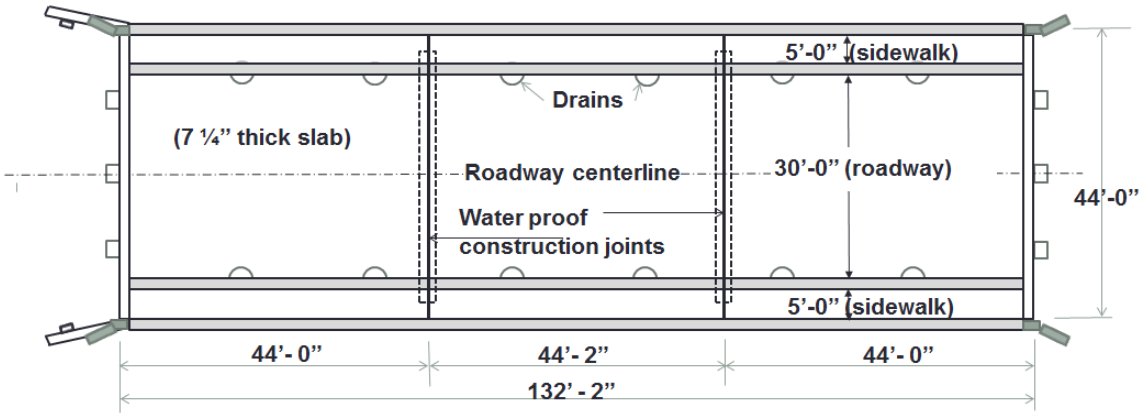

Figure 3.1 Plan view of Bridge #0283, Noyes Slough Bridge

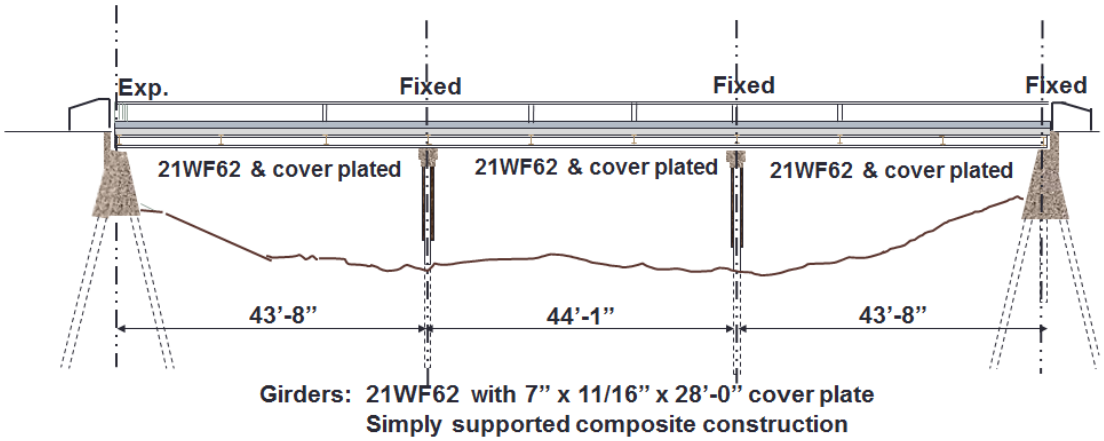

Figure 3.2 Elevation view of Bridge #0283, Noyes Slough Bridge

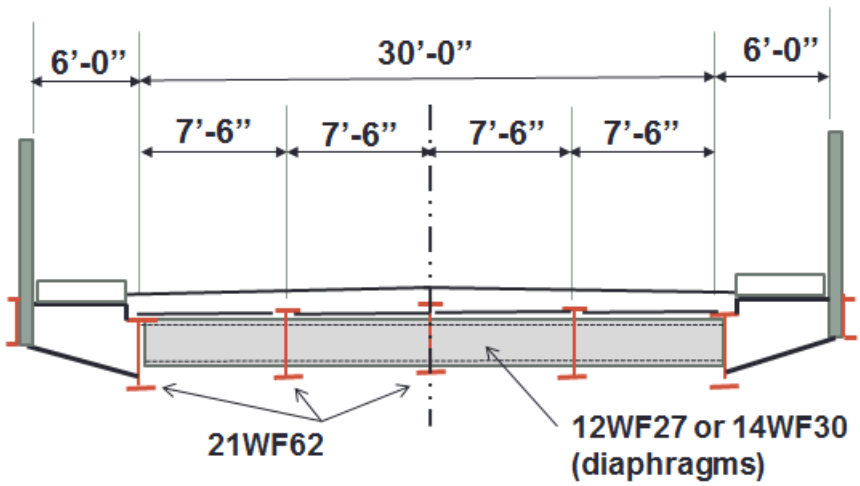

Figure 3.3 Bridge #0283, cross section of the steel girder superstructure

This approach—instrumenting one end span and the mid span—minimized the instruments used and helped minimize managing the wiring harness at the undercarriage of the bridge. Further, an attempt was made to minimize the wiring clusters because of their vulnerability to those curious and those wanting to sell the wiring for cash. Prior to installing the instrumentation and the data monitoring system, a safe method for doing so without the expense of hiring a company for traffic control needed to be established. Thus, the research team designed and built a rolling cart that could be suspended from the lower flanges of the main west and east (W-E) girders (see Figure 3.4).

Figure 3.4 Hanging rolling platform

The movable instrumentation platform was designed to be lifted, installed, and moved by only one person. When in use, the platform is approximately 15 to 20 feet above the ground. This distance helped protect the onboard equipment from potential damage by the public. The platform was designed and built so that it would roll along the bridge girders above the slough, and so that one person could prepare and install the instrumentation at mid span. The platform was built using two-by-fours with an OSB base. Four steel angles were attached at each corner. Welded steel wheels and bearings were used to enable the platform to roll along the steel girder flanges. To install the rolling platform, a chain link pulley system was designed and implemented using modified S-hooks to anchor the chain at any place along its length, and steel C-clamps for the chains to use as pulleys. Once the platform was completed and installed on the bridge, tools, wires, and sensors could be applied to the bridge structure.

3.3 Instruments (Types/Configurations)

Three types of instruments were used to monitor the response of the Noyes Slough Bridge: strain gauges, thermistors, and accelerometers. The sensors were monitored using a Campbell Scientific Model CR000 data acquisition system. Continuous monitoring was conducted by using a marine battery for power. Initially, the research team intended to communicate with the equipment from the University of Alaska Fairbanks, but because of the location of the bridge and the surrounding buildings and associated costs, that idea was abandoned. Thus, the data were downloaded periodically and examined.

3.3.1 Instrumentation Details

All data was recorded by a Campbell Scientific CR5000 data logger with a AM16/32B multiplexer. Twelve thermistors, four accelerometers, and sixteen strain gauges were installed on the structure:

Twelve (12) Thermistors

- Each was a YSI 55033, 2252 ohm resistor supplied by ThermX Southwest
- Two (2) were used to measure ambient air temperature
- Ten (10) were installed on the girder webs to measure temperature distribution through the depth of the girder
    - Five (5) gauges were installed on two different girder webs to measure temperature variation over depth. The gauges were placed evenly from top to bottom of the web. The gauges were insulated to reflect the girder temperature as accurately as possible.

Four (4) Accelerometers

- One 20g uniaxial accelerometer was placed on top of the lower flange at mid span on two different girders
- One 20g triaxial accelerometer was placed on top of the lower flange at mid span on two different girders

Sixteen (16) Strain Gauges

- Each sensor was a five (5) volt 350 ohm full bridge weldable strain gauge supplied by HITEC Products, Inc. (HPI)
- Four gauges were placed at mid span on the top and bottom flanges of two different girders to measure the girder flexural response to load
- Two gauges were placed at the web centerline at the ends of two different girders to measure girder shear

Summary

- Sensors were only placed on one end span (span #1) and at mid span of span #2
- All thermistors were placed on span #2
- Two accelerometers were placed at mid span for span #1 and two at mid span for span #2
- Six strain gauges were placed at mid span of the first end span (span #1)

- Six strain gauges were placed at mid span of span #2
- Four strain gauges were placed at the web centerline located at the beginning of the first span of the bridge (span #1)

Each sensor was assigned a unique five-digit label to help separate and describe them. The first position (X - - -) is used to describe the sensor location (i.e., which of the two spans). For example, “E” is end span and “M” is mid span. The second position (- X - -) is used to determine which of the five girders the instrument is located on; that is, the girders are labeled from 1 to 5 or South to North. Please note, only girders 2, 3, and 5 were instrumented. The third position (- - X -) is used to denote the type of instrument; whether it is a strain gauge (S), a single strain gauge of a pair used for shear (R), a thermistor (T), or an accelerometer (A). The fourth position (- - - X) is used to designate the vertical position of the instrument; the highest position is 1. Each location may have multiple instruments above or below it, designated by a 1 through 6, for instance, showing that there are 6 separate instruments near each other, one being the highest relative position vertically to the others. A list of instruments and their positions is shown in Table 3.1.

Table 3.1 Sensor installation documentation

Along bridge	Girder	Sensor	Vertical position (1 to 6)						Label
End span	2nd	Strain	x	x					E2S1-2
End span	3rd	Strain	x	x					E3S1-2
End span	5th	Strain	x	x					E5S1-2
Mid span	2nd	Strain	x	x					M2S1-2
Mid span	3rd	Strain	x	x					M3S1-2
Mid span	5th	Strain	x	x					M5S1-2
End span	3rd	Shear strain	x	x					E3R1-2
End span	3rd	Accelerometer							E3A1
End span	5th	Accelerometer							E5A1
Mid span	5th	Accelerometer							M5A1
End span	3rd	Thermistors	x	x	x	x	x	x	E3T1-6
End span	5th	Thermistors	x	x	x	x	x	x	E5T1-6

Examples: End span, 2nd girder strain gauges #1-2 (E2S1-2)
Mid span, 5th girder strain gauges #1-2 (M5S1-2)
End span, 5th girder strain gauges in shear #1-2 (E5R1-2)
End span, 3rd girder accelerometer (E3A1)
End span, 3rd girder thermistors #1-6 (E3T1-6)

3.3.2 Thermistors

The thermistors used in this study were a simple two-wire epoxy encapsulated YSI 44033. Created by ThermX, the thermistors are a standard 2252 ohms with an interchangeability tolerance of ±0.1°C and a full operating temperature range of -80°C / +75°C (see Figures 3.5 and 3.6).

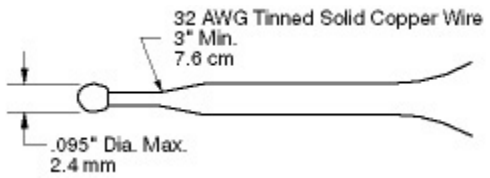

Figure 3.5 Wiring diagram for a YSI 44033

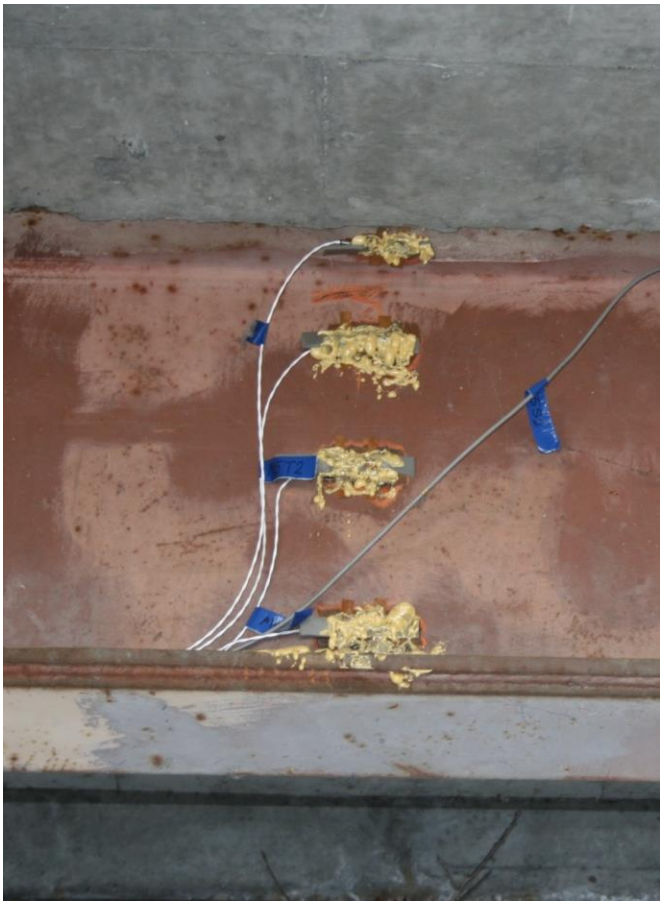

Figure 3.6 Thermistors before the final insulation application

A simple thermal resistor with metal sheathing for protection, the thermistors operate within a 5V excitation, and record and convert resistance to temperature.

The thermistors were placed in a column of five along the web of two girders, just off center of the end span of both the 3rd and 5th girders, fastened using weather stripping to insulate them from the outside air. The weather stripping helped hold the instruments in place, while two strips of shim stock steel were tack welded to permanently fasten the system in place (see Figure 3.6). Two thermistors were installed to hang from the girder so that an accurate ambient air temperature could be obtained. Altogether, twelve thermistors were installed on the bridge structure. Installation proceeded by first coating the steel and then insulating the thermistors with a two-part expanding foam. The foam insulated the sensors from the outside air. This type of application gives a more accurate representation of the actual temperature of the steel.

The placement of the thermistors provides information as to how the steel girders react to the change in temperature as the day warms and cools. Throughout the day, every hour on the hour, the thermistors were sent an excitation voltage so that the temperature could be recorded.

3.3.3 Strain Gauges

Girder strain was measured with full bridge amplified Wheatstone bridge weldable strain gauges. These gauges were built and supplied by Hitec Products, Inc. (HPI). Classified as HBWF-35-125-6-75UP-SS (x2) and HBWF-35-125-6-50UP-SS (x14), the only difference between these two types of strain gauges is the length of the insulated cable; the former has 75 feet of UP cable, which is a four-conductor polyurethane (#22 AWG)-shielded jacket cable, and the latter has 50 feet of cabling. It was determined that no discernable signal loss would occur over the extra 25 feet of cabling.

The HPI strain gauges use Wheatstone bridge electrical circuits to ascertain the strain in the girders on which they are attached (see Figure 3.7). The change in strain varies linearly with the change in resistance. Resistance changes with the length change of the legs of the bridge circuit. This change in resistance alters the outgoing measurable voltage.

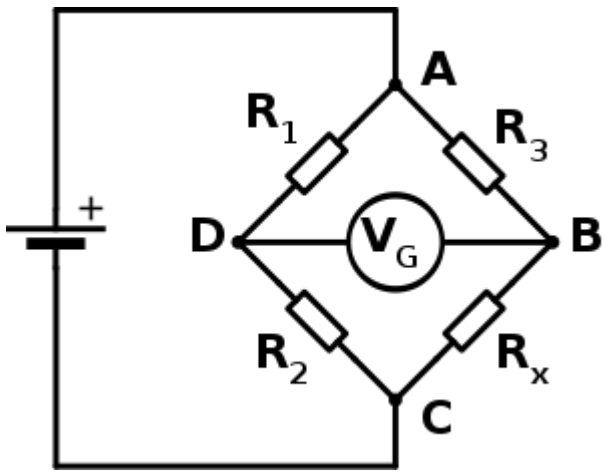

Figure 3.7 Electrical diagram of a Wheatstone bridge

3.3.4 Accelerometers

Four accelerometers were used to instrument the Noyes Slough Bridge; three were uniaxial in the *z*-axis (the direction of gravity), and one was triaxial. Accelerometers and strain gauges behave similarly; only accelerometers use a piezoelectric crystal that alters the voltage as it moves around, rather than a resistor stretched in a strain gauge (see Figure 3.8).

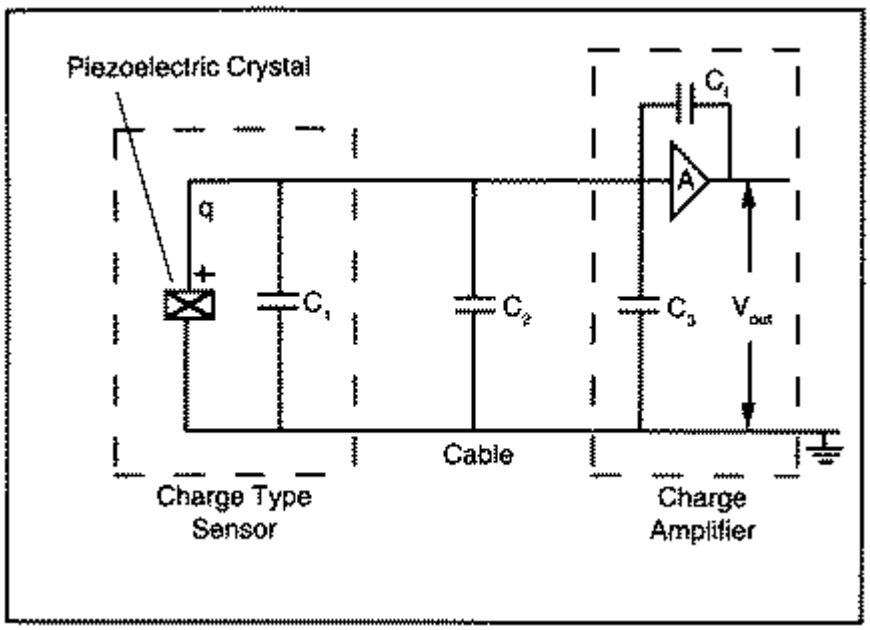

Figure 3.8 Wiring diagram of a single axial accelerometer

This voltage output is directly related to the force that is applied to the accelerometer. The sensor voltages were recorded at a rate of 50 times per second (50 hertz). This measurement provides an accurate view of how the bridge reacts to each vehicle that travels across it. The

acceleration data were used to signal the data logger to begin its collection cycle, rather than the data logger constantly gathering data while the bridge is at rest. More about the collection cycle, as well as how the accelerometers were used as triggers, is discussed in Section 3.5.

3.4 Programming (Campbell Scientific/Methods/Code)

In conjunction with the instruments described in the previous section, the Campbell CR5000 and an AM415 were used to compile the data in real time as the bridge responded to traffic. This system was programmed to record ambient air temperature and temperature distribution throughout the girders every hour on the hour, and programmed to record strain and acceleration as each vehicle crossed and activated the system. The data monitoring system was written with the CRBasic Editor. The system was programmed to record data for the next 50 cycles once activated by a large enough G-force. The system resets the cycle count if a large enough force is recorded before the initial 50 cycles is completed.

The system was designed to continuously monitor the accelerometers without recording until activated. Once activated, the CR5000 records at a rate of 50 hertz, and during that time, the system records acceleration data and strain data. Because of a lack of channels to place the temperature gauges, the AM415 multiplexer was used to act as a hub where a different channel is selected each time the excitation voltage is sent. The AM415 was programmed to activate every hour on the hour and overwrite the program even if the acceleration and strain gauges were occupied, momentarily taking precedence over the other gauges for only a few moments before it resumed recording normally.

The rate of data gathered, even with the idle system in place, was greater than was first predicted, and the final amount was in the hundreds of gigabytes. The data collected over the year were trimmed to be more manageable; outliers and small G-forces were removed, and only the largest forces were examined.

An unfortunate set of circumstances as well as ADOT&PF's priority rating set the deconstruction of the Noyes Slough Bridge further and further down the list of importance. The delays in bridge destruction combined with the expense of handling members with lead paint resulted in an inability to utilize the structural steel in the bridge testing applications. This circumstance caused the daily data from the loggers to be largely unanalyzed, because without the current strength data of the structural steel, the strength lost over its age is impossible to

establish with just the bridge's daily reaction data. Therefore, an alternate form of analysis was implemented and used to find the bridge's strength loss.

4 NOYES SLOUGH BRIDGE, PART 2

This section of the Noyes Slough Bridge analysis provides the reader with a measure of strength loss over the life of the bridge. The measure is based on evaluation without testing the steel in a laboratory setting. Due to the constraints on ADOT&PF's project workload, the bridge remained in use a year longer than was expected, and structural health traffic monitoring was conducted during the life of the structure. The strength loss was evaluated by calculating the strain and deflection of the steel girders caused by test loads that were measured in preselected positions on the structure. Calculated values were compared with the experimental data.

4.1 Testing (Dump Truck)

An International 7600i 6x4 day cab tipper was used to load the bridge structure with known loads at known positions. These known truckloads were also used to test the monitoring equipment (see Figure 4.1). Subsequently, the loaded truck was stopped at different locations to measure static response for maximum moments and maximum shears. The dynamic response of the bridge was measured by requesting the driver to drive across the bridge at a maximum safe speed. Tests were conducted by having the driver drive in both directions across the bridge.

Figure 4.1 International 7600i day cab tipper

Thirty-six static load tests were used to determine the highest possible shear stress and deflection on the two spans that were instrumented—the mid span and the west end span—as well as six dynamic loads where the dump truck reached speeds upwards of 30 mph. These tests were organized in a fashion that minimized the amount of time the truck was parked on the

bridge while the flaggers stopped traffic, and so that only a minimum number of truck turnabouts were needed.

The first deflection test was conducted by having the truck driver position the front axle at the middle of the west end span. For the next test, the driver positioned the truck so that the next axle was positioned over the same midpoint. The following test was conducted by positioning the truck with the rear axle at the same place on the bridge. The first test was at the farthest outside edge of the eastbound lane. The driver moved the truck forward and repeated the process on the midpoint of the mid span. The next set of deflection tests was a repeat of the process, but westbound and on the outside edge of the westbound lane. The following set of tests repeated the last but differed in lane placement; this time both tests were repeated but the dump truck was in the inside edge of the lane. Finally, to finish the deflection tests, the dump truck was directed eastbound in the very center of the bridge. The truck was positioned the same as for the westbound tests.

Six static shear tests were conducted next. For these tests, the driver turned the dump truck around and placed the front axle on a paint mark that was placed on the bridge deck prior to testing. This paint mark was located so that the center of the load was one girder depth distance away from the pier. The painted placement for the wheel was located to produce the largest shear stresses. This test was at the west end span of the bridge. Then the truck was turned around and the process was repeated for the eastbound outside edge of the lane. Once again, the procedure used in the previous tests was repeated, with the truck driven down the inside edge of the lane. Finally, the truck was positioned for an additional set of tests to evaluate the maximum girder shear. This set of tests corresponded to the truck stopping on painted marks with the truck located in the centerline of the bridge.

The last set of tests was the dynamic test, where the driver accelerated the dump truck as much as possible. Due to the bridge's proximity to the nearby intersection, reaching speeds over 25 to 30 mph proved difficult. The truck once again followed the previous test pattern by accelerating westbound while staying on the outside edge of the lane and then turning around to repeat the process eastbound. Then, the dump truck driver repeated the previous two tests while being as far to the inside of the lane as possible. In the last two dynamic tests, the dump truck was driven along the bridge centerline eastbound and then westbound. The truck axles, wheel spacing, and weight were measured prior to testing on the bridge.

4.2 SAP2000 Model

An analysis of the Noyes Slough Bridge was done using a structural finite element analysis program called SAP2000 (2014). The finite element model of the bridge was employed to analyze the way the bridge reacted not only to daily traffic, but also to static and dynamic field tests that were conducted with a pre-weighed and pre-measured ADOT&PF dump truck. The model showed the reaction of the bridge when new to different positions and speeds of the loaded dump truck. The virtual three-dimensional computer model was created from the specifications and as-built drawings for this bridge.

For the model, the bridge's main frame was mapped out and placed using basic frame/cable lines, which later were changed to W21x62 beams for the main E-W girders. The sub N-S girders had C12x25 channels at both ends. The sub N-S girders directly over the two piers were W12x26 beams, and the sub N-S girders between the piers were W14x30 beams. Once the main structure was created, the driving surface was placed and fixed to the girders. The driving surface was treated as a shell, or thick plate section property, and sectioned by using the intersection of the girders as a reference point. Each section of the driving surface was divided into 8 equal pieces about 45 inches square. Then, due to the concrete welded onto the girders, each thick plate intersection had a node that corresponded to a node on the girder 14 inches below it, that was connected by a 2-joint link. This link essentially fastened the driving surface to the girder below it, so all forces and moments were fully transferred between those nodes.

Once the model was completed and preliminary testing of the system showed that the model was reacting to simple loading properly, a loading schedule was set up to the specifications and locations of the dump truck tests (see Figure 4.2).

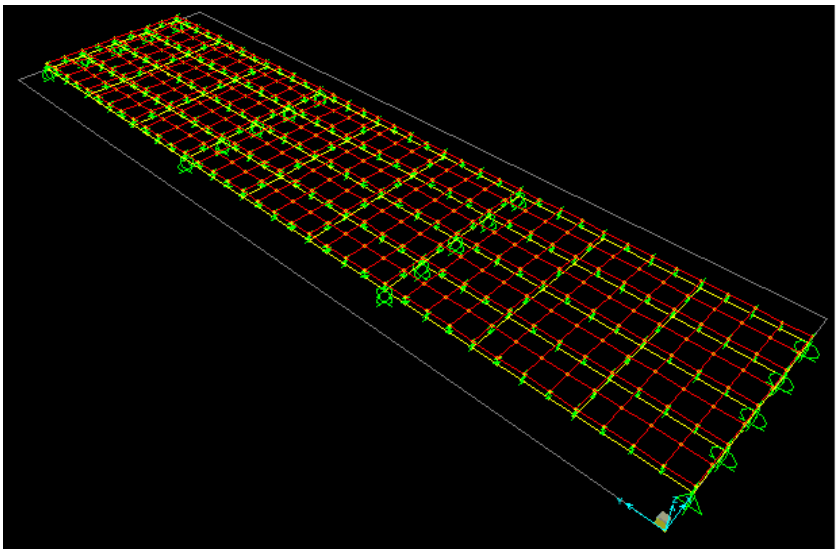

Figure 4.2 SAP2000 model of the Noyes Slough Bridge

After the computer model was checked to make sure that the forces were moving through the model correctly, the final forces in each location corresponding with the previously described dump truck test were placed, and the model was analyzed. This analysis gave the stresses throughout the model and showed how each member of the bridge reacted to the tests. Using the finite element model, stresses were calculated from strains and evaluated for strength comparison. This approach was used as the basis for determination of strength loss over the Noyes Slough Bridge's life cycle.

5 ADOT&PF BRIDGE COST DATA

5.1 Original Maintenance Data

The Pontis database provided by ADOT&PF was used to evaluate all maintenance data. The data were analyzed for fiscal years 2005 to 2010. The data were categorized in an effort to better understand the maintenance trends of Alaska's bridges throughout their life cycle. Additionally, the data were compared with ADOT&PF's construction records to determine the most economical types of bridges currently built. One aspect not covered in depth here is the drastic changes in temperature in the northern region versus the more temperate climates of the southeast region. However, discrepancies appear even in this aspect, since Valdez is under the northern region's jurisdiction. With these problems aside, the data have been collected and analyzed to maximize the information available.

5.2 Results of Statistical Analysis

Some parameters are more influential than other parameters, such as spending per fiscal year, current age of the bridge, and type of bridge. Maintenance cost data were sorted by spending per fiscal year, as shown in Figure 5.1.

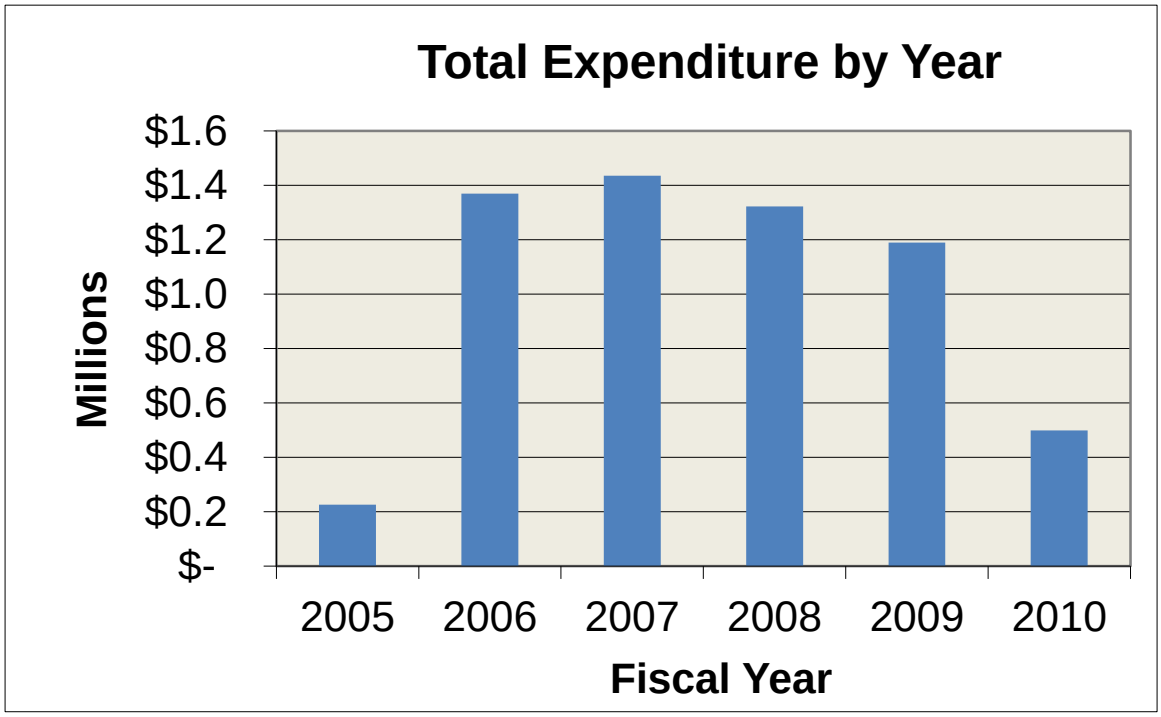

Figure 5.1 Total expenditure by fiscal year

Bridges were sorted into the following 13 unique bridge types stored in the Pontis database:

- Arch Deck
- Box Beam Multi
- Box Beam Single
- Culvert
- Girder and Floor beam
- Orthotropic
- Slab
- Stayed Girder
- Stringer
- Tee Beam
- Truss-Deck
- Truss-Thru
- Misc.

The bridge types were counted and organized to show relative expenditure for each type. Data were then averaged over the total number of bridges for each type. The total for each type is in parentheses after the name type and summarized in Figure 5.2.

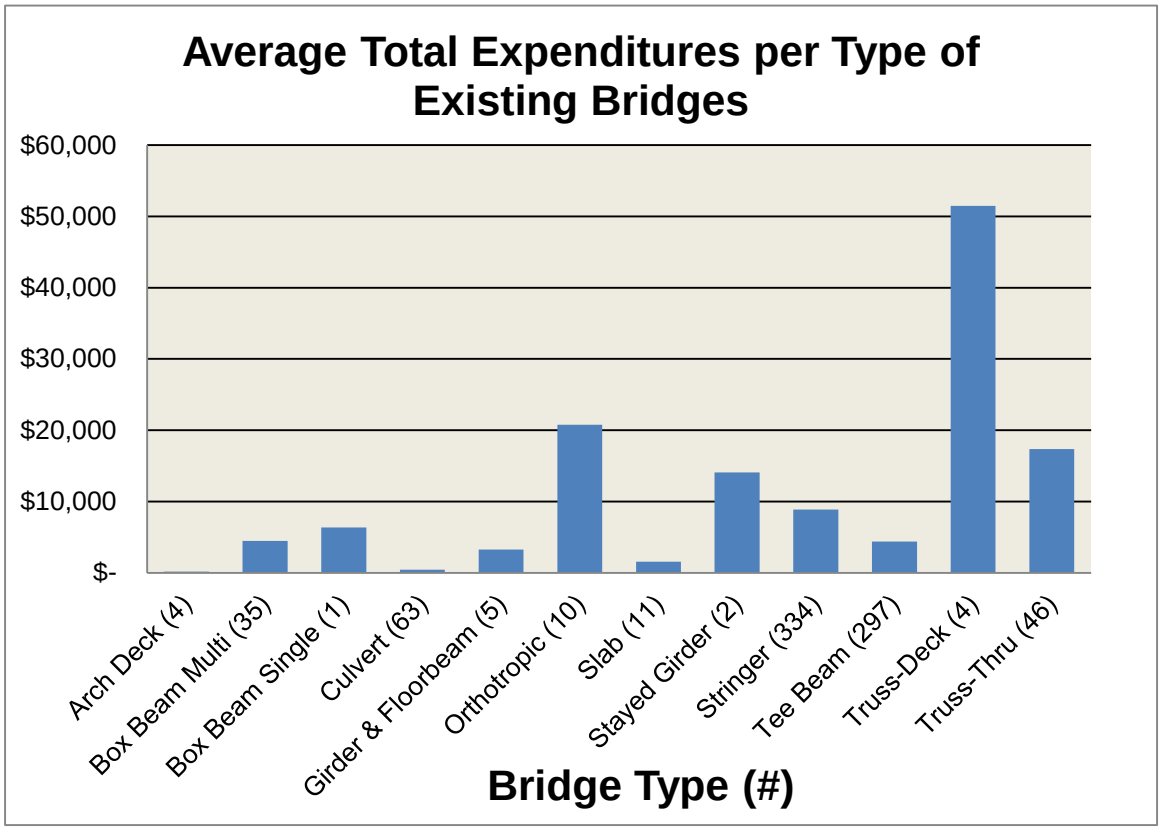

Figure 5.2 Average expenditure per type of bridge

However, this number is not indicative of the actual average amount spent on each bridge type over the course of the five years. To derive that data, the number of bridges that were actually maintained were counted and compared with the total number of bridges in each category, as provided in Figure 5.3.

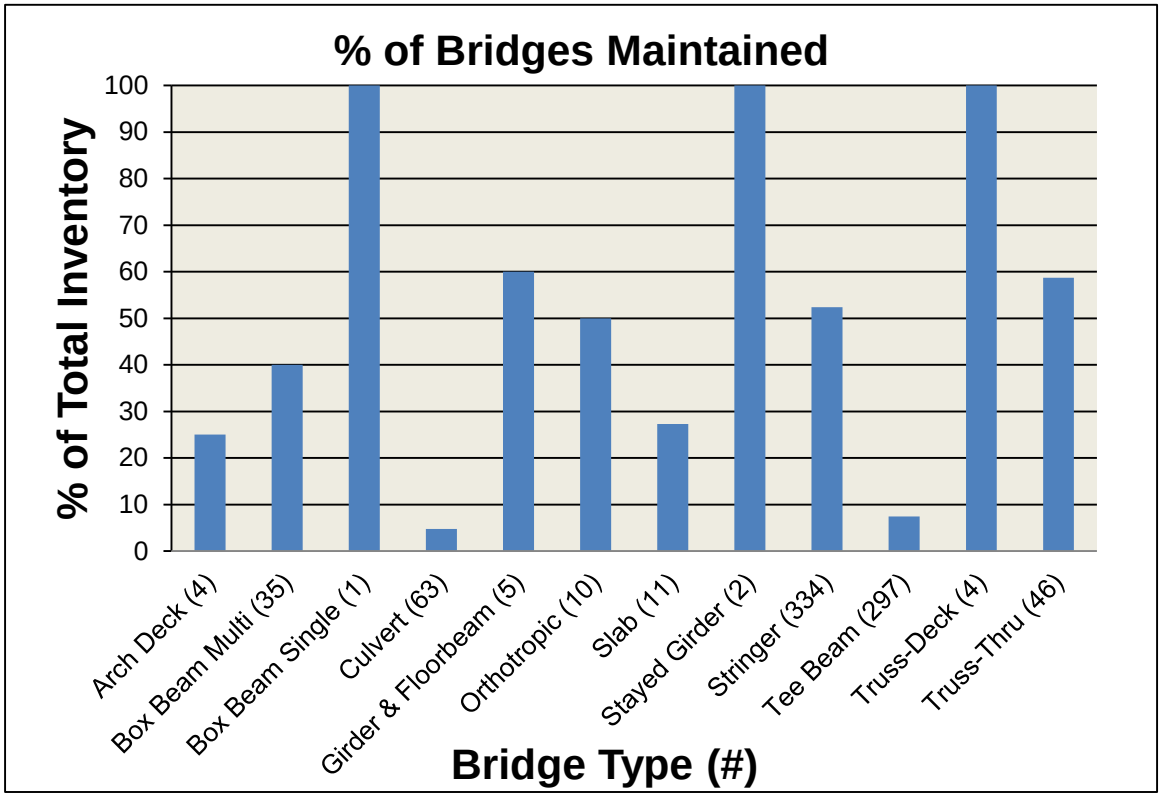

Figure 5.3 Percent of bridges maintained

Once the percent of each type of bridge that was worked on was found, the next step was to find the average cost for each type of bridge based on number of bridges maintained. This figure would show how much is spent, on average, to maintain one of each type of these bridges. In Figure 5.4, the number of bridges maintained is in parentheses after the name type

Data were separated into bridge type and how the bridges compare with one another over the five-year period. Further, the data were examined to evaluate how each bridge type compared in each fiscal year and to learn which type cost more to maintain (see Figure 5.5).

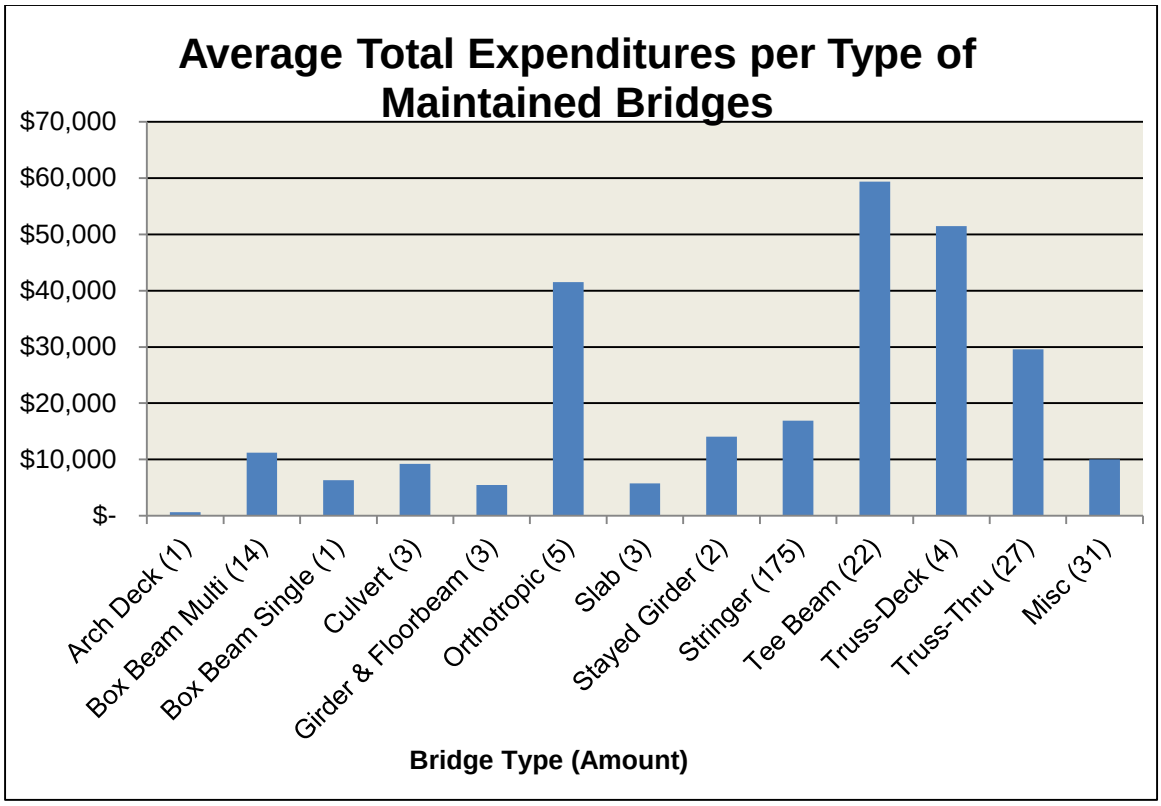

Figure 5.4 Average total expenditures per type of maintained bridges

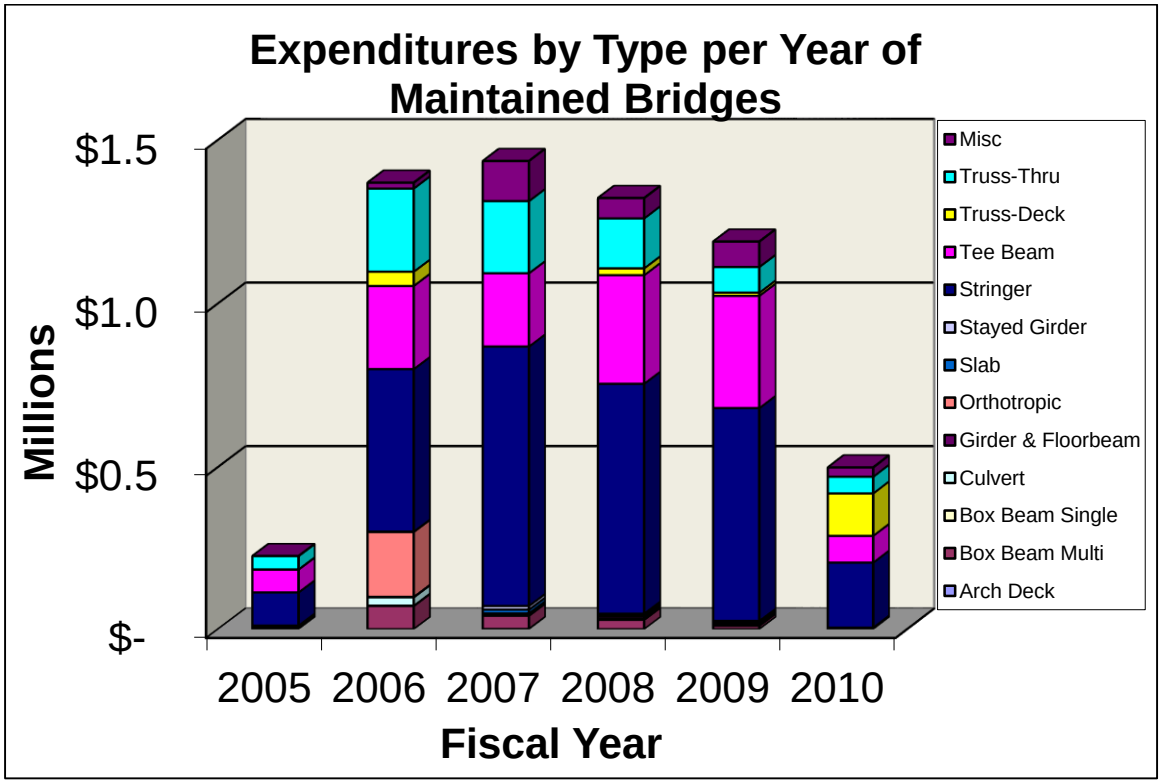

Figure 5.5 Expenditures by type per year

Annual expenditures for maintenance were compared by bridge type. The comparison was made by evaluating the percentage of total maintenance costs for bridges in Alaska. Figure 5.6 shows that most of each year's expenditures were made on Tee Beams and Stringers, which is due to there being more bridges in those two categories.

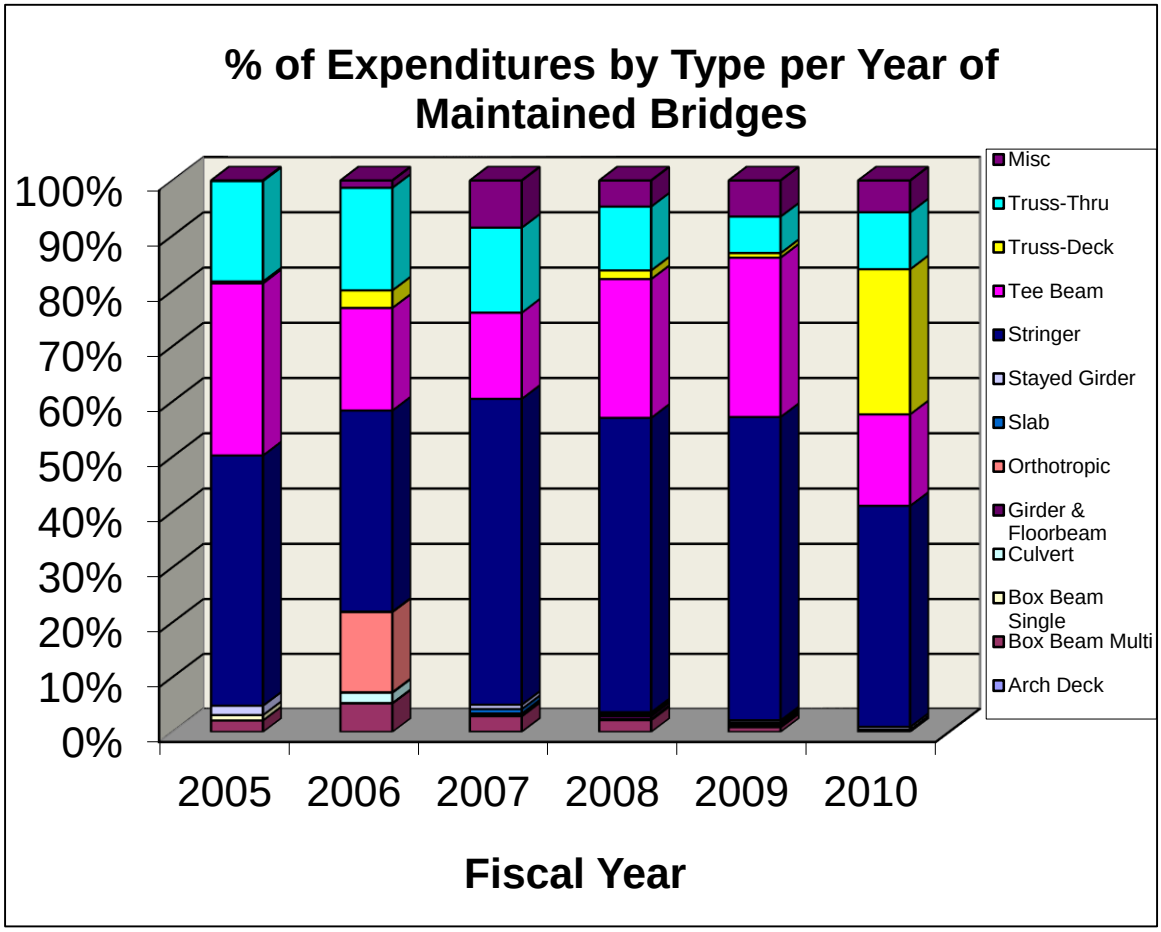

Figure 5.6 Percent of expenditures by type per year of maintained bridges

To provide a better understanding of how much it costs to maintain a given type of bridge, it is helpful to examine the average cost by bridge type as compared with the entire bridge population. The average expenditure per bridge when compared with its total population is shown in Figure 5.7.

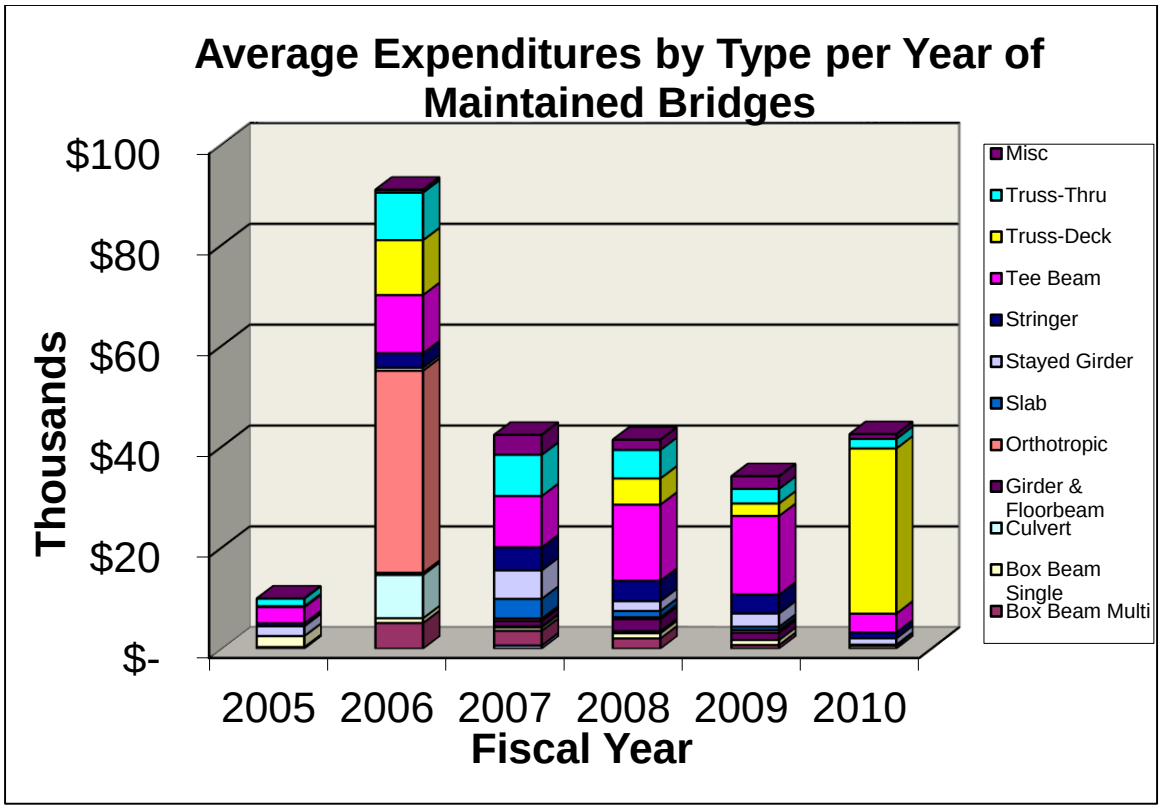

Figure 5.7 Average expenditure by type per year of maintained bridges

Figure 5.7 shows that the cost of Tee Beam bridges is nearly half of the costs per year, not because of that population, but rather because the few that are maintained each year cost quite a lot in comparison with other bridges like the steel Stringer.

The data from Figure 5.7 were divided into percentages for a clearer indication of the average expenditure on each bridge type. This information is shown in Figure 5.8, which presents how the expenditures are being divided per year and per bridge type.

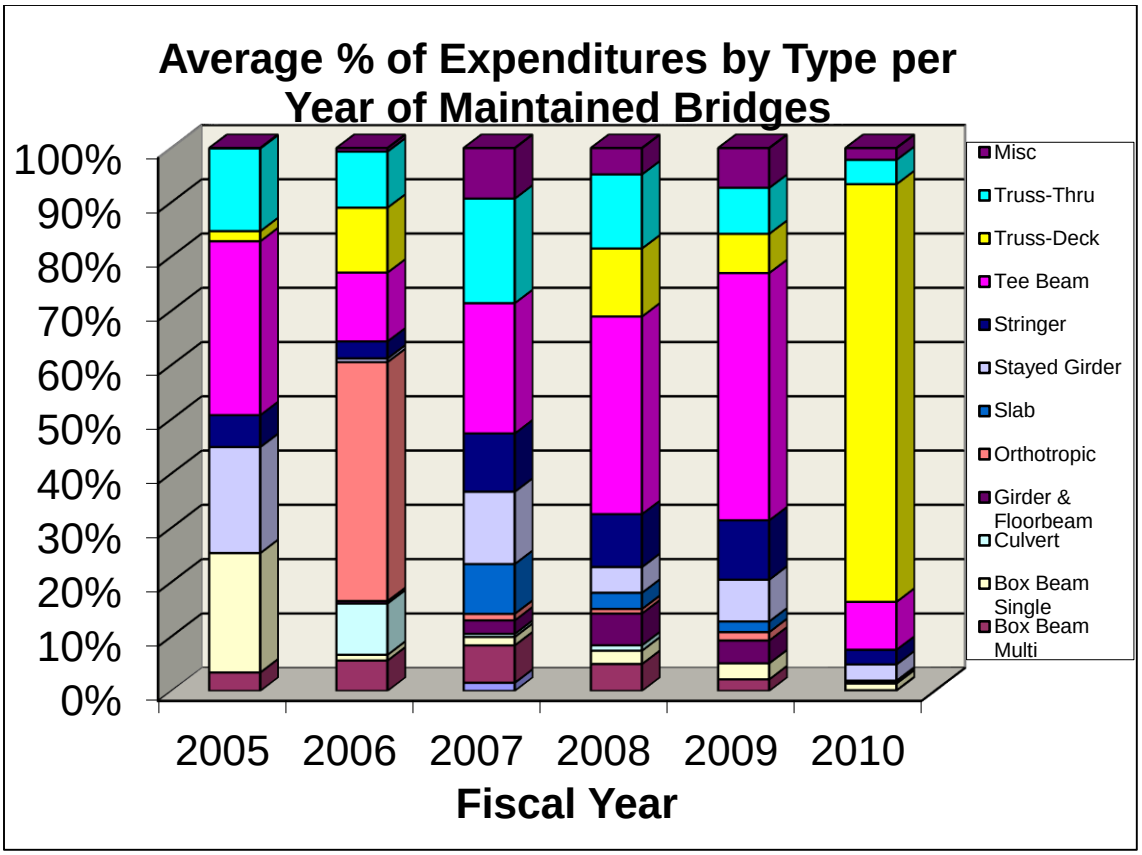

Figure 5.8 Average percent of expenditures by type per year of maintained bridges

Another explored aspect of these bridge types is how the age of each bridge affects the cost of its maintenance. Using Pontis database "yearbuilt" data, age was found and the data then examined to show the differences in the amount spent on maintaining each type of bridge as it aged. Figures 5.9, 5.10, and 5.11 show three examples of bridge types that had the highest maintenance expenditures: Stringer, Tee Beam, and Truss-Thru.

In fact, a general trend shows that as a bridge ages it requires more and more maintenance. This need is extremely clear in Figure 5.11, as many of the older bridges are costing more to maintain. Note the large difference in age ranges of the Stringer bridge, 3 to 76 years, and the Tee Beam bridge, 4 to 49 years.

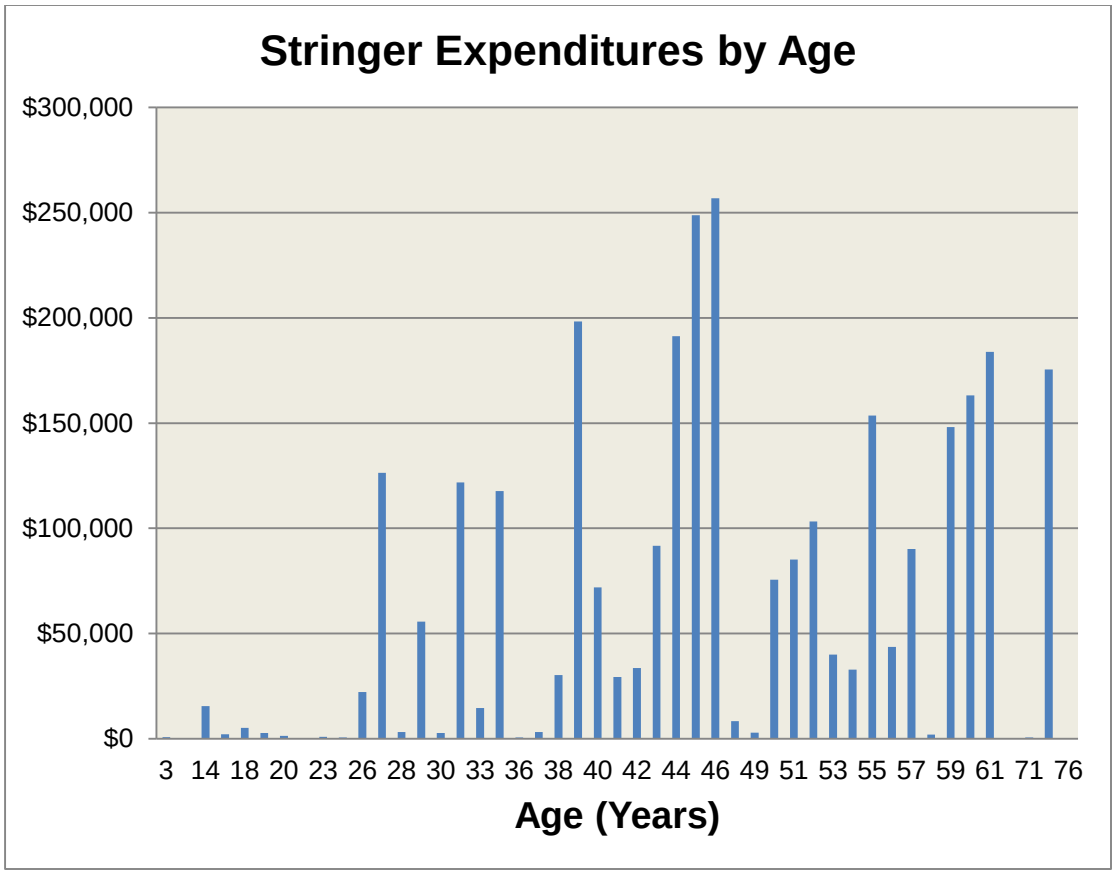

Figure 5.9 Stringer expenditures by age

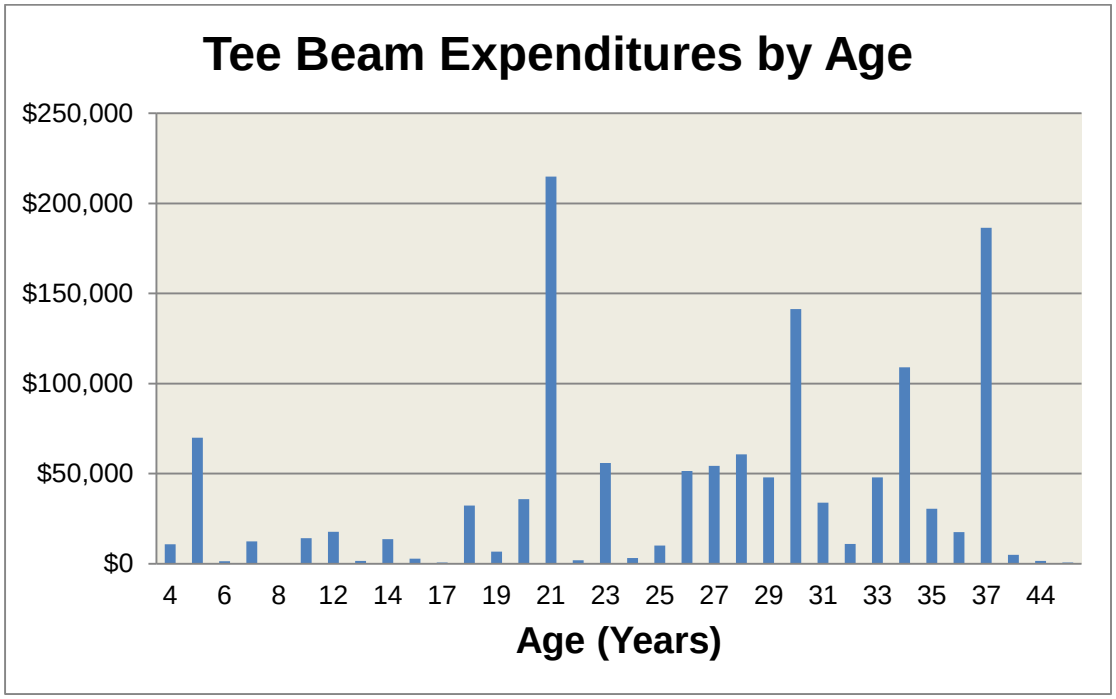

Figure 5.10 Tee Beam expenditures by age

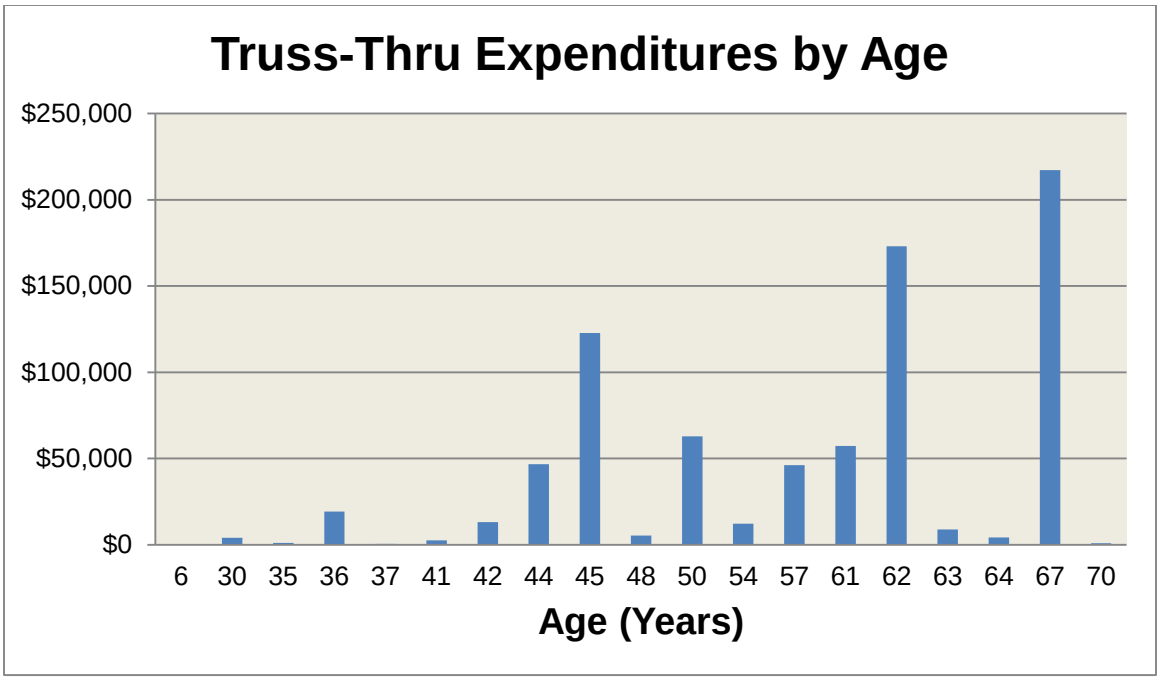

Figure 5.11 Truss-Thru expenditures by age

5.3 Construction Cost Situation

After maintenance costs were determined and analyzed, the initial construction costs were needed to find an accurate economic model. These records are held at each of the corresponding regional office archives. Therefore, individual trips were required to gather the appropriate information from each of the sites.

The Northern Region office of the ADOT&PF is organized differently than the other two regional offices. The Northern Region office records are in hardcopy form and are filed in three-ring binders in the master archive library, which makes finding initial construction costs quite simple and intuitive. One simply checks the master archive list for the specific bridge or job. The region's master archive has a listing of whether the library contains information on anything more than as-builts. If it does, then finding the binder is straightforward. If construction data are there, the binder usually contains a final report along with a final accumulative change order.

The Central Region and Southeast Region ADOT&PF offices both use a different system of data management. Due to the sheer quantity of records, a hardcopy of each record would simply require too much space, so these two regions have opted to use microfiche for storing records. This storage method is a great space saver, but it adds two more steps to finding construction data: first finding as-builts and then finding the microfiche roll in the master list.

Once these steps are taken, the data must be sifted through using projection readers—a rather slow process and difficult to do for long periods.

It was not feasible to find the final construction costs for all of Alaska's bridges due to the time and effort needed for this task. A few small changes could be made by ADOT&PF that would dramatically increase the feasibility of future economic analyses. Some of these changes include but are not limited to the following:

- organize databases according to final LPOs and final costs
- cross-reference maintenance databases
- organize final reports for each project

Currently, the information is simply too spread out for a full economic analysis to be completed. Implementation of Bridge Life-Cycle Cost Analysis will not be practical until final construction costs are organized in such a fashion as to facilitate that effort.

6 ESTIMATING THE LIFE CYCLE COST

There are multiple methods of determining the life cycle costs of bridges. The most common method is to determine the present worth or equivalent annual cost of all expenditures on the bridge throughout its life. This method is commonly found in engineering economy textbooks. The difficulty is that the life of the structure must be assumed at the beginning. In most life cycle cost determinations, the design life of the structure is used, knowing that, generally, the expected life is considerably longer. As a result, the life cycle cost of the bridge may be misleading.

A second method of estimating life cycle cost is using what is commonly called the service life. In principle, the approach is quite straightforward. The present worth or equivalent cost of the structure is computed for each life span beginning in year zero through the life, which provides the lowest life cycle cost. Service life could be shorter or longer than the design life. The service life does not directly assess the structural life, but simply estimates the life, which minimizes the life cycle costs.

Consider the following example: The initial cost of a bridge is $2 million with a design life of 75 years. The annual cost of maintenance is $3000 in Year 1 with an anticipated increase in cost of 5% per year. The deck will require repaving every 15 years at a cost of $150,000. An interest rate of 4% is assumed. Inflation will not be considered in this case. In determining service life using the equivalent annual cost method, it is assumed that the reader has an understanding of engineering economy.

The life cycle cost for Year 1 would be the (principle)(Capital Recovery Factor) plus the maintenance costs in Year 1 or ($2 million)(1.04) + 1000 = $2,081,000.

Repeat this method for Year 2.

($2 million)(A/P,4,2) + [1000(1.0025)(P/F,4,2) + 1000)(A/P,4,2)]

= ($2 million)(.5302) + [1025(.9246) + 1000](.5302) = $1,061,432

Since the equivalent annual cost for Year 2 is less than for Year 1, compute the equivalent annual cost for Year 3. Repeat these steps until a minimum is found.

This method lends itself very well to a spreadsheet, and Excel has financial functions built in that easily allow these computations. The graph in Figure 6.1 shows a minimum life cycle cost of about $109,873. For this example, those costs represent spending at the end of Year 69. Again, this value represents the economic service life of the structure and does not

necessarily relate to the structural or functional life. However, if costs during the life of the structure can be anticipated, they can easily be incorporated into the analysis. For example, if it is anticipated that an additional lane will be added in Year 50, the cost of the addition of that lane can be incorporated into the analysis.

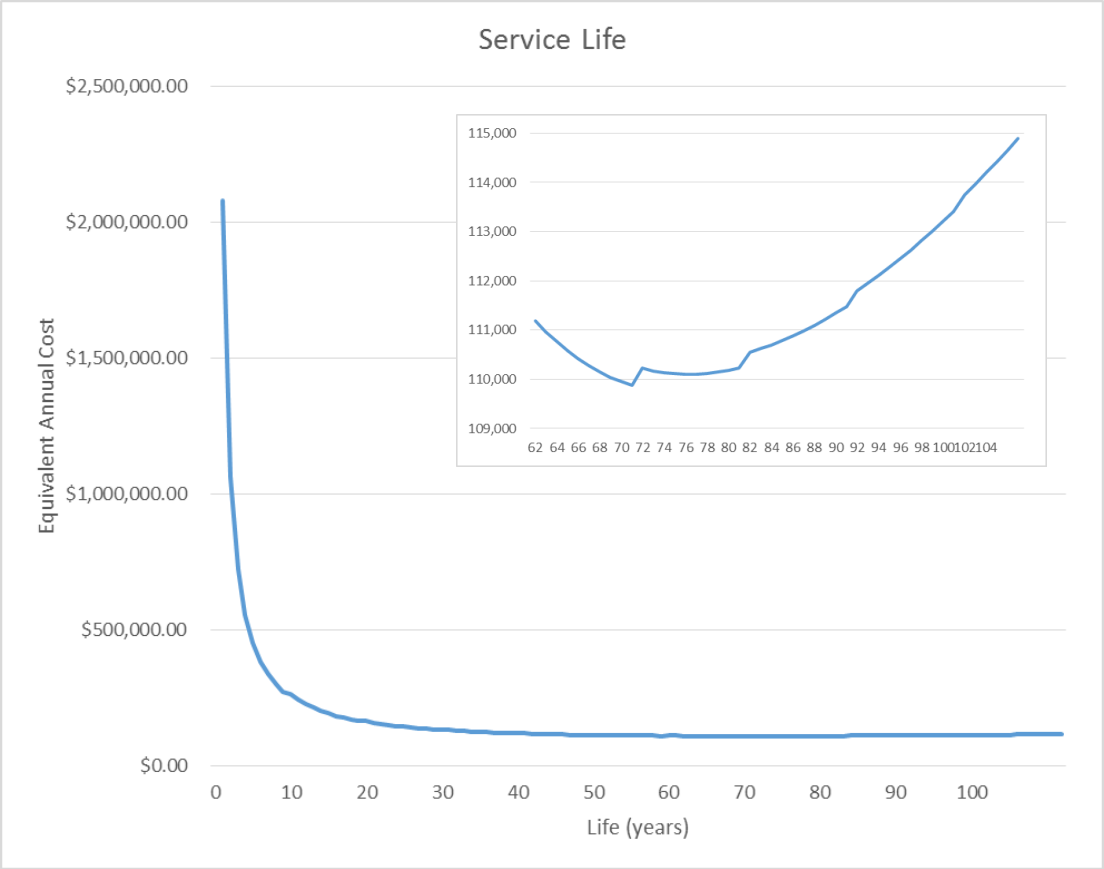

Figure 6.1 Minimum service life for a bridge structure

The advantage of the service life approach is that the analysis is not tied to a fixed life. Rather, the economic life of each structure is estimated, and the associated cost of that structure is estimated. Design alternatives are then compared based on this life cycle cost. As data become available, the analysis can be tuned based on those data.

The challenge becomes the replacement strategy. Since bridge construction is a long-term investment with no clear replacement requirement, it is assumed that the planning horizon is infinite. Further, the service life does not necessarily represent optimal replacement timing. As the structure ages, both structural and functional changes may occur. Consequently, the assumptions made at the design stage may become invalid. A change to the structure or

replacement strategy may emerge, which then requires an analysis that determines how best to address the new need.

The existing bridge, called the defender, will be compared with all alternatives, called the challengers. Since the costs incurred to date are sunk costs, they cannot be considered in any of the decision strategies. The steps listed below should be followed in selecting a replacement strategy:

1. Compute the service life of both the defender and the challenger.
2. Compare the service lives. If the cost of the defender is higher than the cost of the challenger, choose the challenger. If the cost of the challenger is higher than that of the defender, choose the defender.
3. If the defender should not be replaced now, estimate when it should be replaced by the challenger.

To demonstrate this process, two scenarios will be considered. The first scenario is whether the bridge should be replaced in-kind in Year 71. The second scenario is whether to add a lane to the existing bridge in Year 50 to accommodate traffic or to replace the bridge.

If in Year 71, the question is whether to replace the bridge or to replace the bridge in-kind, the process is to look at the cost of maintaining the bridge for one more year and compare that cost with the economic life of the new bridge. Since the bridge is being replaced in-kind, the cost of maintaining the existing bridge one additional year must be less than the economic service life of the new bridge, which is 69 years, with an equivalent annual cost of $109,873. Looking at Figure 6.1 the estimated cost of maintaining the bridge in Year 70 is $86,993 plus a capital cost of $150,000 for deck replacement. However, a deck inspection indicates the deck does not need repairs for four years. Thus, the cost of routine repairs is less than $109,873. This calculation suggests that the bridge be left in place for one more year. The process described is continued until the costs exceed the economic life costs of the challenger. If only routine maintenance is performed, the bridge would be replaced in Year 75 if the cost projected is accurate. In truth, costs should be evaluated with current data rather than projected design data.

The second scenario essentially compares two alternatives. The first alternative is to retrofit the bridge with an additional lane to accommodate traffic. The second alternative is to build a new bridge to accommodate additional traffic. Both alternatives will occur in Year 50.

In this case, the service lives of the two alternatives are compared. Alternative 1 assumes a retrofit cost of $1,000,000, which includes the cost of resurfacing the existing deck and some modifications to reduce routine maintenance costs. Because the bridge has aged, the maintenance costs will be higher than a new bridge at $15,000 per year at a growth rate of 5% per year. The estimated cost of resurfacing the deck increases to $175,000 every 10 years. Using the same procedure as before, the service life of the bridge after rehabilitation and widening is an additional 47 years at an equivalent annual cost of $146,737.

Assume a new bridge can be constructed for $4 million with an initial maintenance cost of $2000 per year with an annual growth rate of 3% due to new design technologies. It is anticipated that the deck will be re-decked every 15 years at a cost of $225,000. In this case, the service life is 81 years at an equivalent annual cost of $104,409. Replacing the bridge becomes the most economically attractive decision.

As always, intrinsic values, including available funding, environmental impacts, and community input, must be considered. In summary, the economic life approach offers a number of advantages over traditional life cycle cost analysis for infrastructure that has a very long life. The most attractive advantage is that the procedure does not require assigning an analysis life. Rather, the procedure determines the life at which the life cycle costs are at a minimum. This method allows ready comparison of multiple alternatives with long lives and an analysis of modification of potential changes in strategy at any point in the life of the structure. While the procedure is somewhat more complex, spreadsheets allow for rapid analysis. Once the spreadsheet is set up, it can be rapidly altered to accommodate changes in assumptions or modified for other alternatives.

7 COMPARATIVE RESPONSE (STRUCTURAL DETERIORATION)

7.1 Comparison

The research findings from this study show that there were noticeable and measurable differences in the strain behavior of the Noyes Slough Bridge at the end of its life cycle when compared with its theoretical beginnings. Strain gauges were used to calculate strength loss. Using measured changes in strain in the bridge's girders, a comparison was made between the bridge's condition at the time of evaluation and its condition at the time of original construction. With this method, changes in stress could be quite easy to find.

Not every strain gauge responded as predicted. Many of the strain gauges showed signs of change during the test, yet after analysis, these changes proved to be negligible. Therefore, only strain gauges that showed changes in strain that paralleled the SAP2000 model were used for the final analysis. Also, for the final analysis, only girders that experienced the greatest amount of strain from the location of the dump truck were used for the most accurate picture of strength loss.

Careful consideration of the data analysis indicated a clear trend of increasing strain, or increasing stress, over the life of the structure. Figures 7.1, 7.2, and 7.3 show what the strain in the beams should have been when the bridge was first built, as compared with the strain in the beams during the test.

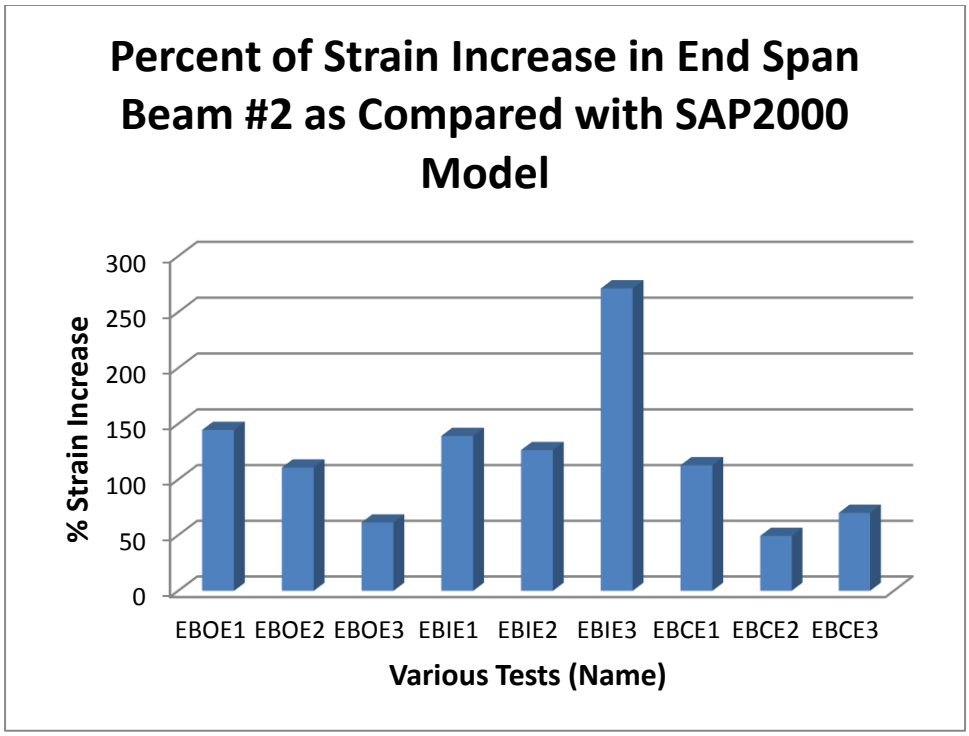

Figure 7.1 Strain test results comparison in End Span Beam #2

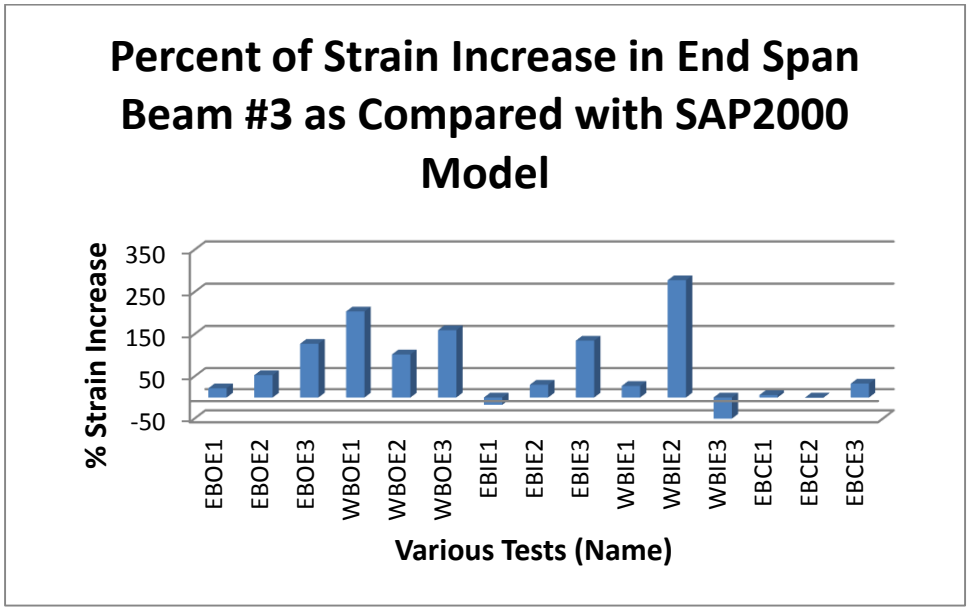

Figure 7.2 Strain test results comparison in End Span Beam #3

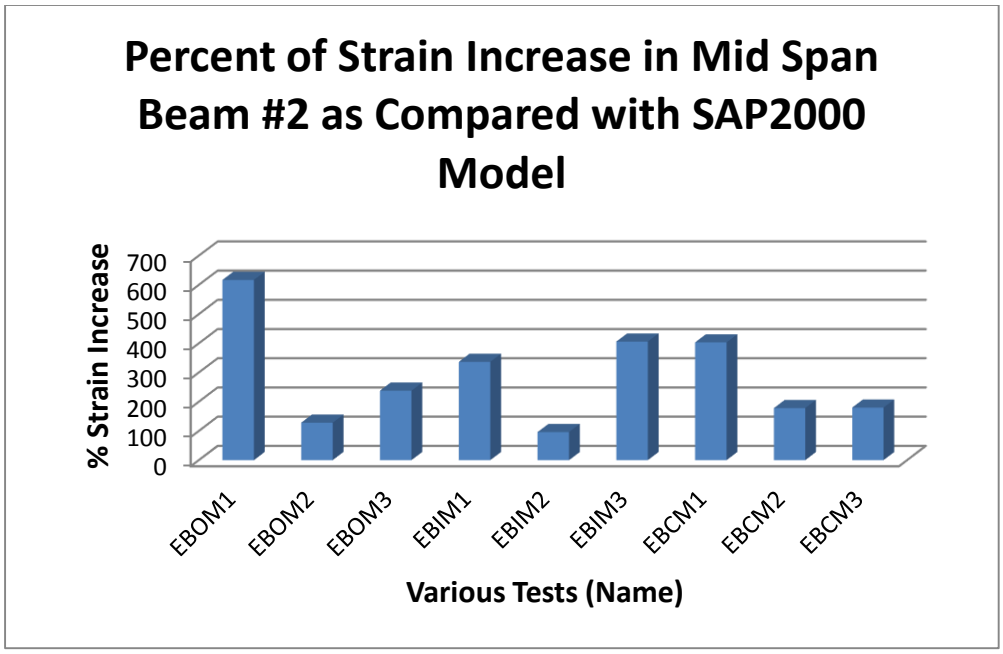

Figure 7.3 Strain test results comparison in Mid Span Beam #2

In each of these cases, it can be seen conclusively that beam strains increased and, in turn, their stress increased. In many cases, the increase was well over 100%. The stress values associated with these increases are shown in Figures 7.4, 7.5, and 7.6.

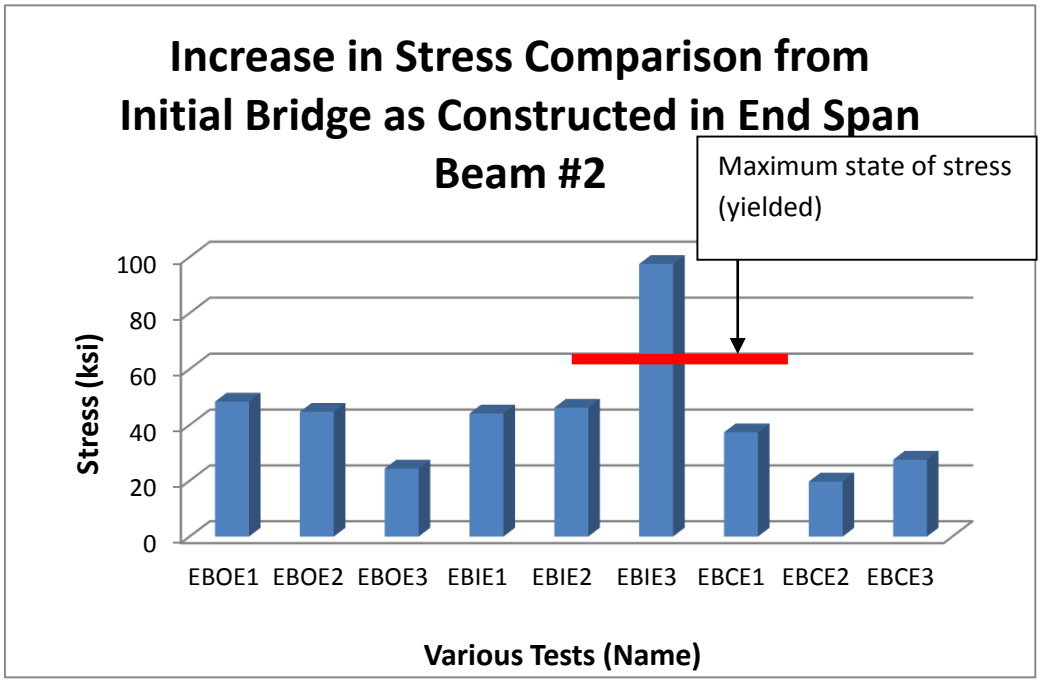

Figure 7.4 Changes in stress comparison in End Span Beam #2

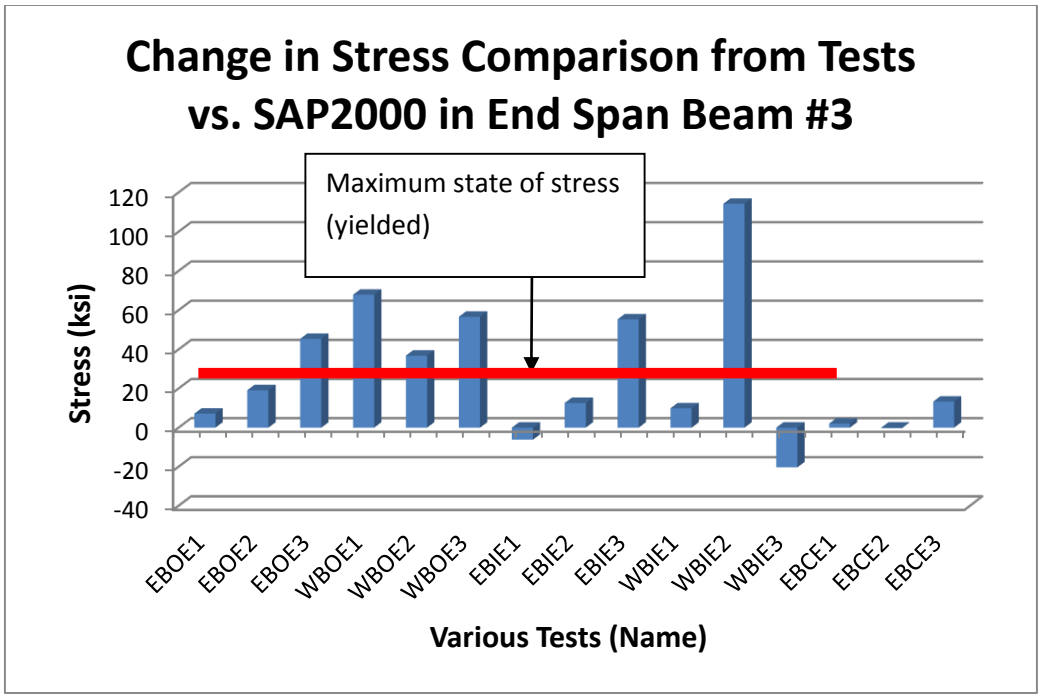

Figure 7.5 Changes in stress comparison in End Span Beam #3

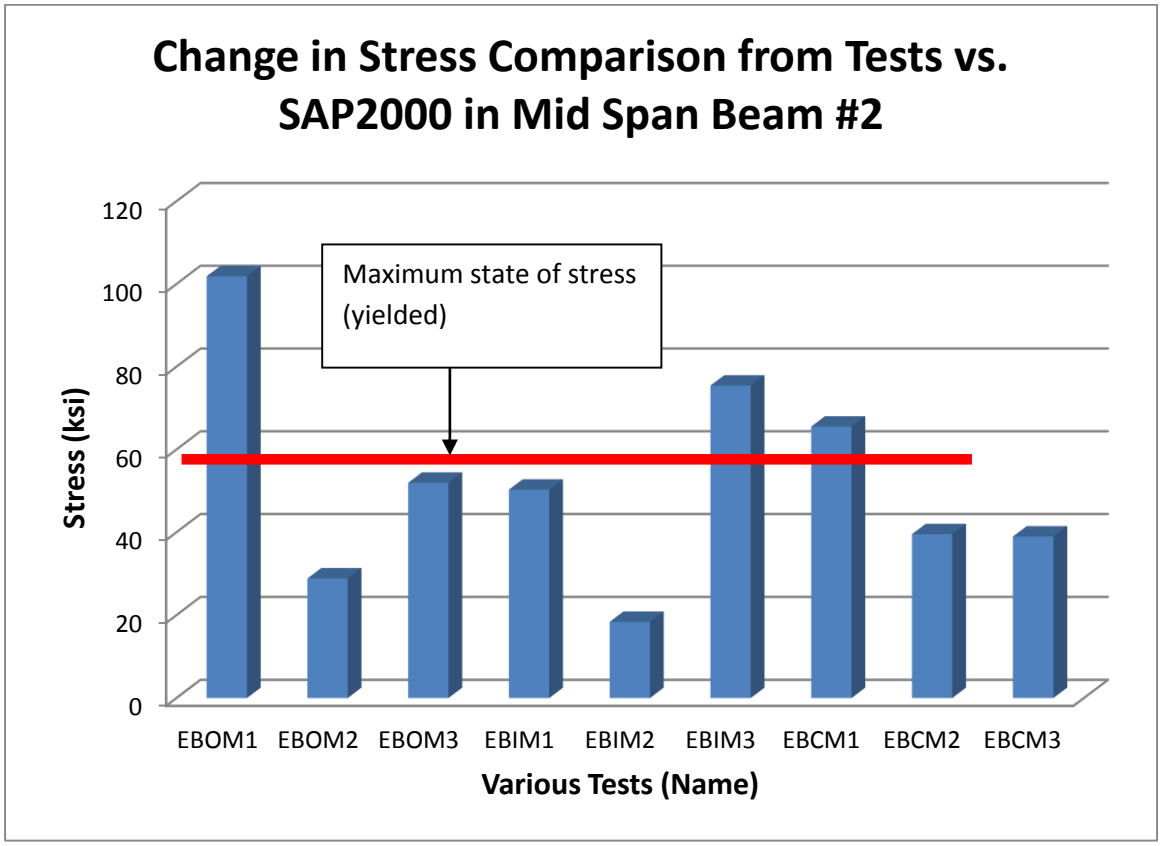

Figure 7.6 Changes in stress comparison in Mid Span Beam #2

Nearly every test showed a marked increase in strain and stress in the beams when compared with its SAP2000 theoretically calculated values based on the original (new) construction condition. With an average stress increase of 43.35 ksi in End Span Beam #2, 27.60 ksi in End Span Beam #3, and 52.32 ksi in Mid Span Beam #2, it is easy to see a direct correlation between the use of 62 years and the bridge's theoretical beginnings. These stress increases are based on the concept that stress is proportional to strain (linear behavior). Clearly, in this case, parts of the structure may have experienced yield, and the member carrying capacity may have been redistributed so that areas less stressed would contribute to the load-carrying capacity of the structure. These stresses were calculated from the strains measured in the gauges; therefore, assuming that the stress increases of these beams show the beams have undergone yielding deformation is reasonable. The minimum yield strength of these beams was F_y = 36 ksi, and the effective yield stress was F_{ye} = 54 ksi. The effects of traffic and weather caused the Noyes Slough Bridge beams to experience significant states of stress, and these effects led to the redistribution of stress throughout the rest of the beam.

7.2 Maintenance Data Compared with Noyes Slough Bridge

The Noyes Slough Bridge is classified as a steel Stringer type with a reinforced concrete deck, placing it in the largest category for bridge types in the state of Alaska. In 2013, Alaska had 334 bridges with the classification of Stringer. This means that a lot of money was spent maintaining these bridges; however, as noted in Chapter 5, Stringer bridges are some of the cheapest to maintain per year, per bridge, making them a very efficient bridge type—little maintenance cost and a long life span.

When Alaska achieved statehood in 1959, the Noyes Slough Bridge was already about ten years old. The ADOT&PF was decentralized, and most of its records were transferred. The original construction cost of the bridge has not been found; therefore, an accurate assessment of its full life cycle is not possible. Prior to this study, the bridge was decommissioned after visual inspections. During this study, the bridge was replaced. Construction of the new Noyes Slough Bridge, located near the intersection of Illinois Street and College Road in Fairbanks, was part of a street improvement project for Illinois Street.

7.3 Importance of Construction Cost

The initial construction costs of many bridges in Alaska are not available. Only 28 initial construction costs from 5 different bridge types were found. Of those bridges, nearly all of them were 40 years old, or younger, with the average being 35 years old. Only the most current records on bridges are readily available from ADOT&PF. Table 7.1 shows the 5 types of bridges that rank the most cost-effective by initial cost plus yearly maintenance over their life span:

Table 7.1 Summary of costs for Alaska bridge types

Bridge Type	Initial Cost Plus Annual Maintenance Cost
Arch Deck	$1,195,088
Box Beam	$1,475,702
Tee Beam	$4,443,132
Stringer	$7,199,380
Orthotropic	$36,812,406

The list provided in Table 7.1 is only a partial list of the bridge types in Alaska. In the case of both the Arch Deck bridge and the Orthotropic bridge, only one entry was found. The Noyes Slough Bridge falls under the category of Stringer bridges and, therefore, according to the data, is one of the most expensive bridge types on this list. Without proper data and accurate records, it is nearly impossible to analyze the work of so many engineers and contractors who labored to create before us. Therefore, it is vital that a system, easily accessible and instantly maintained by the ADOT&PF, be implemented. Pontis is the current software system in place at ADOT&PF, and it has a long way to go before it can be considered a central system of data access and storage throughout the department.

8 CONCLUSIONS

Throughout the course of this study, many insights were gained,into the way a bridge reacts under live load, to the cost of maintaining bridges throughout the state of Alaska and how the life cycle cost of bridges might be determined.

A clear loss of strength was found in the Noyes Slough Bridge; the structure exceeded twice the strain it was designed for when new. All bridges throughout Alaska were cataloged and then sorted by bridge type. The cost to maintain each bridge was determined, and the maintenance costs were statistically analyzed. Initial construction costs were missing for many of these bridges, including the Noyes Slough Bridge, so though its bridge type is one of the cheapest to maintain, an accurate cost benefit analysis of the bridge was impossible.

The Noyes Slough Bridge was demolished in the summer of 2013, ending any further data that could have been recorded from traffic usage over the girders. While replacing the bridge was the ultimate plan, demolition occurred later than planned due to higher ADOT&PF priorities. Essentially, with this study, much more is now known about the Noyes Slough Bridge and its relationship to the rest of the bridges throughout Alaska.

The application of the service life cycle costing approach has a number of advantages over the traditional life cycle cost approach. A bridge has essentially three lives; structural, functional and service. All of these lives are highly variable. For example the structural life of a bridge can be extended almost indefinitely with the right repairs. The service life approach does not assume a life. Rather it estimates the life that provides the lowest life cycle cost. Doing so allows comparisons of alternatives assuming an infinite planning horizon.

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