QUANTIFYING BRIDGE RESILIENCE

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ABSTRACT

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Deviations in weather patterns have changed the demands facing the nation's infrastructure. "Extreme" weather events (i.e. those characterized as having a low probability of occurring yet result in a high consequence) are increasing in intensity, frequency, or both. Because these changes are happening on a relatively short timescale, existing probabilistic methods used by engineers to model weather events are increasingly obsolete. The most effective approach for designing structures to resist the effects of unpredictable events is to incorporate principles of resilience into their design. Resilient structures better adapt, quickly recover, and minimize the destructive outcomes of an unpredictable destructive event. In this paper, a method is proposed for rating and quantifying the structural resilience of a bridge. The proposed method specifically focuses on bridges and the relationship between their resilience and structural inputs. To demonstrate the extent of the proposed approach, four distinct bridges are chosen for a comparative case study. Each bridge falls into a category with varying combinations of high and low sufficiency and resilience ratings. Resilience ratings are calculated for each bridge and compared with their sufficiency ratings to demonstrate that high sufficiency is not indicative of high resilience. Both the sufficiency and resilience ratings of bridges should be analyzed separately and in compliment with each other when making decisions related to their design, construction, and maintenance.

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1 Introduction

1.1 Background

Increasingly extreme variations in climate and weather patterns are placing greater strains on our nation's infrastructure and its ability to adapt. Transportation infrastructure is at the forefront of this battle against shifting weather patterns yet current design standards are not keeping pace with these new demands. Highways and interstates are the lifelines of the nation's economy. Tens of thousands of bridges service these routes and these structures are susceptible to the extreme loading demands from extreme weather events. A disruption to the network of interstates has a significantly unfavorable consequence for the trucking industry and, subsequently, the nation's economy.

Currently, the Federal Highway Administration (FHWA) and the United States Department of Transportation (USDOT) rate the nation's bridges using a "sufficiency" rating. The sufficiency rating is a result of the Structure Inventory and Appraisal (SI&A) Report, which is a component of the bridge inspection process. This rating, on a scale from 0 to 100, is a measure a bridge's structural condition, functionality, and importance to the user. However, a crucial disadvantage is this rating does not take into account how a bridge responds, adapts, and/or recovers from a severe loading event. The rate which a structure recovers from a potentially damaging incident is termed it's "resilience".

Adequately resilient infrastructure has the ability to adapt to unforeseen changes and/or quickly overcome a loss of functionality. An extreme event is characterized by having a low probability of occurrence yet a high consequence. Because the timing and magnitude of an extreme event is difficult to determine, it is problematic to design for them. Incorporating principles of resilience into design eliminates the need to design for a specific extreme event and allows the structure to adapt and overcome these unpredictable events.

1.2 Problem Statement

There is a need to quantify the level of resilience contained in a bridge. A resilience rating will allow decision makers to best allocate funds for repairs and improvements. To meet this need, a resilience rating methodology for bridges is proposed based on ranks and weights of specific structural indicators.

Although several approaches for quantifying resilience have been researched, they remain largely theoretical and, as a result, lack applicability. In response, a secondary goal of the approach proposed in this thesis is to utilize as much existing data as possible. This way, state departments of transportation can easily implement the proposed methodology using information already on hand. In addition, the connection between certain bridge components and overall resilience is illustrated.

In order to emphasize the applicability of the proposed approach, a comparative case study of four distinct bridges is performed. This case study carefully evaluates four bridges currently in the bridge inventory in the state of Arizona. The four bridges target varying combinations of high and low sufficiency and resilience ratings. The resilience rating for each bridge is calculated using information found on the SI&A report, as-built construction plans, and existing inspection documents. Finally, sufficiency and resilience ratings are compared to assess their correlation.

1.3 Objectives

The following objectives are intended to meet the preceding project needs:

- Determine bridge characteristics that contribute to resilience
- Develop a methodology for quantifying structural resilience
- Compare the calculated resilience rating to the sufficiency rating currently in use by the Federal Highway Administration
- Allow for easy adaption of the resilience rating by state departments of transportation

1.4 Thesis Organization

This thesis specifically addresses the resilience of transportation infrastructure, how resilience is affected by structural inputs, and provides a measure to quantify resilience. Chapter 2 will discuss the current state of structural resilience and provide a summary of significant research performed in the area to date. The background of resilience and current proposed resilience quantification procedures will be discussed. Also, the needs and goals of this research will be discussed. Chapter 3 will provide an overview of the methodology in calculating the resilience rating. The procedure for rating and weighting a bridge will also be shown. Chapter 4 will present the results obtained calculating the resilience rating for each bridge in detail, including detailed explanations as to how each rating and weight was reached. The implications of the results found will also be discussed. Chapter 5 will discuss the results presented in chapter 4. The comparisons between resilience and sufficiency ratings will be discussed. Also, a susceptibility analysis will be presented to display the resilience rating equation's susceptibility to change.

Chapter 6 will discuss applications, implications, and conclusions drawn from the comparative case study. This chapter will propose the adaption of the resilience rating into the bridge inspection process. In the appendices, sample calculations of ratings and weights will be provided.

2 Background

2.1 Overview

The purpose of this chapter is to present the state of the art of structural resilience within the context of this research.

2.2 Resilience

In general, structural resilience is the rate which a structure can recover from damage. Resilient infrastructure is typically presented within the context of unpredictable or extreme weather events. In this context, an extreme event is categorized as having a low probability yet high consequence of occurrence. In other words, it does not occur frequently or may have never happened before, but the consequences result in substantial risks to the public, transportation system, or both.

Bruneau [2003] defines resilience through three complementary measures:

1) Reduced time for recovery;

2) Reduced consequences to the community; and

3) Reduced probability of failure.

Bruneau's [2003] study, which focused on extreme seismic events and their effects on bridges, suggested that the proper quantification of resilience required a very clear definition.

Bruneau's [2003] definition of the resilience of a structure comes from the properties of robustness, redundancy, resourcefulness, and rapidity. Robustness is a structure's strength in enduring hardships or extreme events. Redundancy is a design feature where a single component or series of components of a structural system can fail while maintaining functionality and integrity of the whole. Resourcefulness is the adaptability of a structural system while maintaining functionality. Rapidity is the rate of recovery from an event. Redundancy and resourcefulness influence the rate which a structure recovers from a damaging event.

Bocchini et al. [2014] build on Bruneau's [2003] theoretical foundations of resilience and conceptualize it as a "resilience triangle". Resiliency is quantified by the triangular area defined by the loss in functionality and rate of recovery, as defined in Equation 2-1 and illustrated in Figure 2-1.

$$R = \int_{t_0}^{t_1} \left(100 - Q(t)\right) dt$$

Equation 2-1

Where,

R= resilience

t_0= time when damaging event is initiated

t_1= time when structure is fully recovered

Q(t)= structural functionality

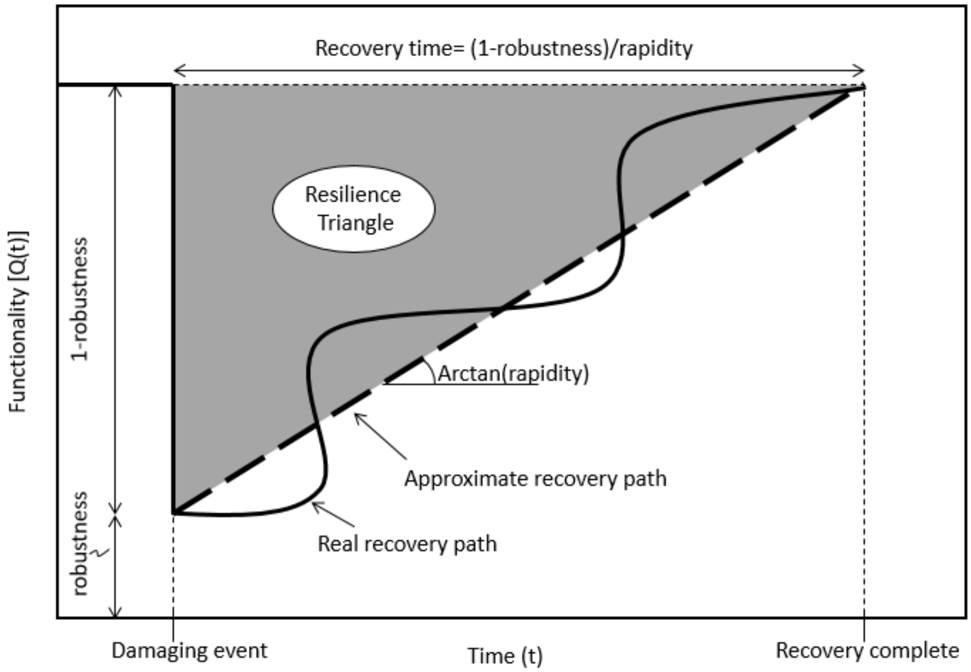

Figure 2-1: Resilience Triangle [Bocchini et al, 2014]

As seen in Figure 2-1, resilience is a function of robustness, rapidity, and time. The vertical axis measures functionality where the loss of functionality is defined as (1-robustness). The horizontal axis measures time. The horizontal leg of the resilience triangle represents the total recovery time. One of the most critical features of the resilience triangle is the hypotenuse, which is the recovery path of the structure after the introduction of a damaging event at time t_0, until the structure is completely functional at time t_1. The slope of the hypotenuse is an average approximation of the actual recovery path. This actual recovery path is represented by Bocchini et al.'s [2014] equation for resilience (Equation 2-1).

Mackie et al. [2015] define resilience as the ability of a structure to absorb external stimuli and regain functionality. They similarly implement their definition with the concepts of robustness and rapidity. Robustness is associated with a structure's ability to withstand and/or absorb an extreme event, and rapidity is the ability to recover from said event and regain functionality [Mackie et al. 2015]. Mackie et al. [2015] include carbon emissions in their resilience quantification. From their work, they conclude resilient structures emit less carbon throughout their design life because they require fewer repairs.

2.2.1 Impacts on Infrastructure

2.2.1.1 Existing Shortfalls

In the past 50 years, the average temperature of the planet has increased by roughly 0.15°C to 0.20°C per decade and has shown no signs of slowing down [Hansen, 2010]. This phenomenon is having marked effects on the intensity of droughts, hurricanes, flood events, and glacial melting [Melillo et al, 2014]. This rise in weather intensity is detrimental to our nation's infrastructure, including roadways, bridges, channels, and stormwater systems.

In the United States alone, more than 30% of bridges are more than 50 years old and, as a result, nearing the end of their design life [American Society of Civil Engineers (ASCE), 2017]. According to ASCE [2017], in the U.S., the cost to rehabilitate all of the bridges in need of repair is in excess of $120 billion. However, current funding only provides approximately $18 billion per year for infrastructure repair [American Society of Civil Engineers, 2017]. Because of this shortfall, infrastructure that is vulnerable to current weather demands may not be upgraded within an appropriate timeframe. Dong and Frangopol [2016] state it best: "The current infrastructure in the United States is losing the battle of probability."

2.2.1.2 The Battle of Probability

Although nearly every state in the U.S. had the ability to record weather data by the 1850s, historical data only exists for about the last 100 years [Fiebrich, 2009]. Furthermore, current climate models are increasingly obsolete due to the accelerated rate at which the intensity of climate events is changing. In short, extreme weather events occur more frequently and/or with higher intensity than suggested by existing models. For example, according to Huber and Gulledge [2011], events in the United States previously classified as a 500-year event now occur at a rate comparable to a 100- or 10-year event. In 2016, parts of Australia, Indonesia, and southeast China saw rainfall above the historical 90th percentile [World Meteorological Organization, 2016]. In contrast, parts of Russia saw rainfall below the historical 10th percentile in the same year [World Meteorological Organization, 2016]. Thus, current weather patterns are not becoming drier or wetter than normal— rather, the entire world is seeing weather events that are more extreme in each direction.

In addition to weather, the magnitude of natural disasters has been more extreme in recent years. This phenomenon is having a drastic impact on not only society and communities but the economy as well. In the 2015 Global Assessment Report on Disaster Risk Reduction [United Nations International Strategy for Disaster Reduction, 2015], it is reported that average annual losses due to natural disasters are about $314 billion. This estimate is solely based on losses to the built environment and does not take into account loss of life or income. Impacts from natural disasters are even more detrimental to smaller, developing countries. In fact, island communities are the most at risk group due to their relatively high exposure to hurricanes and earthquakes and because their access to supplies is limited [United Nations International Strategy for Disaster Reduction, 2015]. As a result, when a tragedy strikes these areas, it takes much longer for the country to recover both physically and economically.

Recent hurricane seasons have been more severe than the historical trend. For example, Hurricane Harvey hit southeastern Texas in late August 2017 and Hurricane Maria hit Puerto Rico shortly after. Hurricane Harvey caused $180 billion in damages, over 40 inches of rainfall, record flooding, and 80 deaths [van Oldenborgh et al., 2017, NOAA, 2018]. At the time of writing this thesis, residents of Puerto Rico still have limited access to food, water, and cell phone service [Gomez and Jervis, 2017].

The inertial forces from an earthquake are very large for a structure as massive as a building or a bridge. Generally, designing structures to withstand these forces without damage is not economically feasible. Since the frequency of the event is so rare, structures are designed to withstand damage but not collapse. Thus, with increasing frequency of seismic events, existing designs fall short of their intent.

Finally, with recent changes in weather patterns, some regions are experiencing more frequent and/or extreme droughts. These drier than normal conditions have caused agricultural strains, ecological damage, and devastating wildfires — the magnitudes of which are the greatest some areas have ever experienced. For example, the popular wine region of northern California experienced a fire season in the fall of 2017 that burned over 245,000 acres, destroyed around 8,900 structures, and claimed the lives of 43 people, making it one of the most destructive California wildfire in terms of structures and lives lost [CalFire, 2017].

2.2.2 Resilience vs Sustainability

2.2.2.1 Interrelationship

Sustainability and resilience, although related, are not the same construct. Resilience is the ability to recover after suffering a loss or failure of functionality while sustainability is the ability to conserve resources during construction and operations. In fact, as Bocchini et al. [2014] concluded, it is sometimes necessary to be resilient at the cost of being less sustainable. Most current design practices either end up focusing solely on resilience or solely on sustainability. The focus on one or the other has the effect that the "other" is often overlooked. Using this approach to design or manage infrastructure has proven to be very inefficient [Bocchini et al., 2014]. Rather, an optimal system is both sustainable and resilient, combining the positive aspects of both and optimizing them to work in concert with each other.

2.2.2.2 Sustainability

As a way of practice, sustainability gained popularity in the late 1980's and early 1990's. In 1987, the World Commission on Environment and Development published a special report, "Our Common Future" [Brundtland, 1987] in which the committee presents strategies for policies and regulations to reach more sustainable development and address environmental concerns. The report states that the goal of sustainable development is to "meet the needs and aspirations of the present without compromising the ability to meet those of the future" [Brundtland, 1987]. The report suggests that sustainable development need not be limiting. Policies and practices can be developed and improved to adapt to increasing changes in climate. To date, the biggest impact on sustainable development is due to changes in public policy.

The American Society of Civil Engineers defines sustainability through the "Triple Bottom Line," where societal, environmental, and economic conditions all have an equal weight. From this philosophy, the sustainability rating system "Envision" was created through the Institute of Sustainable Structures (ISI) [Minkser, 2015]. This rating system is much like other rating systems that award structures based on their implementation of sustainable concepts [e.g. Leadership in Energy and Environmental Design (LEED), Energy Star, Green Globes, Net Zero Energy Building (NZEB), Building Research Establishment Environmental Assessment Method (BREEAM) in the United Kingdom, and the Living Building Challenge (LBC)] [Kibert and Hakim, 2014 and Hossaini et al., 2015]. These rating systems operate similarly in that they provide points and certifications for buildings using innovative technologies to become more sustainable, reduce energy usage, produce less waste, or a combination. Envision is similar but provides certification specific to civil infrastructure by going a step further to determine if the infrastructure is appropriate or necessary for the current needs of the community and environment [Minsker, 2015].

2.2.2.3 Resilience

While sustainability focuses on impacts to the natural environment, resiliency focuses on consequences of failure and impacts on the community. A resilient system, according to Bocchinni et al. [2014], is "more reliable, since it has a lower probability of reaching limit states…fast recovery…[and] the rapidity of functionality restoration…[and] low socioeconomic consequences."

In the event of a natural disaster or other harmful event, damage to a non-resilient structure can cause cascading negative consequences on a community. For example, if water lines are damaged and no alternatives are available, the community may be without potable water until lines are restored. Building failures are not only harmful to a community— they can also be deadly. Depending on the building's use, members of a community may be out of a home, place to work, place to worship, or means to life necessities. Furthermore, a bridge failure can cut off a community from essential resources. Feasible detours may not exist or may cause travelers to drive considerably out of the way to reach their destination. This leads to not only driver frustration but also lost productivity, increased emissions, and/or increased collisions.

2.2.3 Current State of the Art

2.2.3.1 Research

In recent years, research has shown that resilience is becoming a more vital aspect of design. Historically practiced in the material science field, resilience has been placed in new contexts increasingly over the last 50 years [Park, 2013]. The first published research about resilience (not in a material science context) was from Crawford Stanley (C.S.) Holling [1973] related to the resilience of ecology. In his work, Holling [1973] suggests that ecosystems can be resilient if a foreign external or internal stimulus can be absorbed and without entirely changing the ecosystem. Holling [1973] also states that even though future events are unknown, planning and preparing for unknown events is a critical aspect of resilience.

In engineering, there has also been an increasing amount of research in resilience. Park et al. [2013] discuss resilience in engineering systems. Alipour et al. [2013] research the resilience of bridges during combined low frequency, high consequence events, specifically earthquake and scour. Dong and Frangopol [2016] focused on the impacts climate change and extreme weather events have on civil infrastructure. Ikpong and Bagchi [2015] developed a way to describe a bridge's resilience in quantitative terms. A common theme throughout this literature is that the construction of structures that can recover quickly from an intense and unpredictable weather event is dependent on purposeful integration of resilience principles into their design.

Design policies cannot quickly adapt to the changing environment because predicative models are based on antiquated assumptions. Thus, infrastructure needs to incorporate the adaptability associated with a resilient approach. A resilient bridge system can be "ahead of" the current standards of practice by being designed to adapt to an extreme event [Dong and Frangopol, 2016].

Currently, no national standard or requirement exists to assess the resilience of a bridge in quantitative terms. However, recent studies have proposed a theoretical framework for the measurement and quantification of a bridge's resilience [Dong and Frangopol 2016, Ikpong and Bagchi 2012, Deco 2013, Ikpong and Bagchi 2015]. A quantitative measure would give engineers and decision makers an objective and historical record of a bridge's resilience over time. In addition, decision makers would have information on the state of a bridge's resilience in order to make decisions that are more informed.

According to Park et al. [2013], current practice incorrectly merges resilience with risk when resilience is truly "an emergent property of what an engineering system does, rather than the static property the system has." Park et al. [2013] suggest the system should be analyzed as a whole instead of investigating parts individually and that infrastructure cannot be managed by risk alone. Identifying possible hazards can be difficult in the built environment because many risks are unknown. It is not viable to analyze risk if the event that could disrupt the system is unidentifiable. Resilience in engineering systems should be controlled and managed through "resilience analysis." It is impossible to separate the two concepts of risk and resilience in an engineering context due to the deliberate nature of engineering design. Only through resilience analysis are risk and resilience properly combined [Park et al., 2013].

Park et al. [2013] suggest that resilience should be perceived as an adaptive process and not a state. In other words, a resilient design should adapt and acclimate to unforeseen events. A resilient structure does not lose functionality after an event and does not sustain any permanent loss of functionality. With that said, a structure is dynamic throughout its life. Thus, to be resilient, routine inspections, maintenance, and rehabilitation should be performed throughout a structure's lifetime.

Current practices put too much faith and confidence in current safety measure technologies [Park et al. 2013]. For example, the Fukushima power plant disaster did not implement new protective measures for the facility even though earthquakes and tsunamis in the area had been accurately predicted for decades because the owners did not want to raise any suspicion of a potential disaster [Park et al. 2013]. In other words, if the public thought that the plant was unsafe, the facility would lose political and financial support. On the other hand, sometimes too much trust is placed in existing safety systems, as seen in the Deepwater Horizon oil spill [Park et al. 2013]. Risks and warning signs were seen, but trust was put into the system as designed.

Since resilient practices involve being accommodating and inventive, Park et al. [2013] suggest the engineering process includes four additional steps. These steps are "sensing, anticipation, adaptation, and learning" (SAAL). Originally presented by Hollnagel et al. [2006] and later adapted by Park et al. [2013], these steps form the foundation of resilient engineering design:

- **Sensing** involves taking in new stimuli from the system and its environment and giving particular attention to anything out of the ordinary. A stress that is foreign or a sudden increase in demand should not be overlooked. They provide opportunities to better the system.
- **Anticipation** occurs when, after recording and paying attention to stimuli, designers and operators can anticipate new events.
- **Adaption** occurs when designers and operators are able to anticipate potential disasters or events and can implement contingencies.
- **Learning** is achieved after adaptation occurs when designers plan for ways to manage new stimuli. This involves rehabilitation, alternative disaster management strategies, or designing for a structure to fail safely or partially.

From the SAAL framework, new ways of designing in the same environment are learned and implemented. This strategy involves teamwork and communication between many groups and may even involve other disciplines. Thus, through the SAAL approach, resilience becomes a qualitative approach to risk management.

Finally, resilient design should not be a mere re-design of existing practices. Being resilient in design means being innovative and creative, which may result in straying from traditional or standard design practices [Park et al., 2013]. This process may also involve a change in perspective and behavior in engineers. Instead of practicing "tried and true" methods that have proved to be insufficient in low probability/high consequence events, a new paradigm emerges requiring designers to conceive creative solutions for recovery from unforeseen events. It is improbable to know every possible low probability event that could cause damage to a structure, but it is practical to incorporate structural resiliency by providing contingencies to failure. Marias et al.'s [2004] strategy for changing mindsets resonates: "It is not necessary to predict all potential causes of a ship sinking in order to provide lifeboats and other emergency measures."

2.2.3.2 Examples of Resilient and Non-Resilient Design

Although resilience is not explicitly incorporated into current design standards, there are several ways structures can be resilient, such as providing a safe to fail mechanism or increased redundancy. Safe to fail design provides contingences for unpredictable events. Similarly, incorporating redundancy into a structure's design will increase its resilience by allowing for multiple alternative load paths. In other words, if one member of a structure fails, the remaining members can still carry loads and maintain functionality.

In 1927, intense rainfall caused the Mississippi River valley in southeastern Missouri to flood to unprecedented levels. It was called the Great Flood of 1927 and, at the time, was one of the worst in America's history [*Mississippi River flood of 1927*, 2018]. The floodwaters were so great that the entire levee system along the river failed. Over 250,000 square miles of land were submerged under water, homes were destroyed, thousands of structures were wiped out, and more than 250 people died [*Mississippi River flood of 1927*, 2018]. Based on the lesson from this disaster, The US Army Corps of Engineers (USACE) built a levee system in the New Madrid Floodway and included safe to fail mechanisms in the event of another unprecedented event.

In 2012, when floodwaters at the confluence of the Mississippi and Ohio Rivers rose to unsafe levels, the USACE intentionally breached the levees around the New Madrid Floodway. One of the breached levees allowed the floodwater to flow into the floodway, and another allowed the floodwater to slowly re-enter the river. This breaching caused 133,000 acres of farmland to be flooded [Morton, 2013]. Although several farm buildings and homes were flooded, no lives were lost. Thus, the resilient design and management of the New Madrid Floodway and surrounding levees prevented a repeat of the 1927 catastrophe.

In contrast, the 2015 collapse of the I-10 Tex Wash Bridge in California is an example of how a non-resilient design can be dangerous and costly. The Tex Wash Bridge on interstate I-10, built in the 1960's, spans the Tex Wash Dry River. This section of the I-10 connects Southern California to Phoenix and is a major roadway for the trucking industry. About 20% of the average daily traffic in this portion of the I-10 is truck traffic [FHWA, 2012]. During the monsoon season in July of 2015, the Tex Wash Dry River just outside of Palm Springs, California experienced a large amount of rainfall in a very short period of time. The area experienced approximately 6.5 in. of rainfall in less than six hours [Tabbakhha et al., 2016]. During this event, flash flooding caused the eastbound section of the Tex Wash Bridge to collapse. Because of this collapse, the Tex Wash Bridge was closed to traffic for five days, costing the trucking industry an estimated $2.5 million for each day it was closed due to detour lengths and delays [Masunaga, 2015]. Furthermore, traffic was reduced to one lane in each direction for two months while crews worked to replace the collapsed bridge. The total cost to replace the bridge was approximately $8 million dollars [Kelman, 2015].

According to Tabbakhha et al. [2016], several design flaws led to the collapse of this bridge. First, the bridge was constructed on a shallow foundation and not a pile foundation. This caused floodwaters to erode away the soil under the foundation of the bridge, triggering the abutment to fail. Second, the original construction of the bridge called for the existing delta-like dry river to be condensed into a narrow channel. This was done by adding soil to the delta plain, unnaturally reshaping the flow of the waterway, resulting in a sharp turn in the floodwaters and a bottleneck at the location of the abutment [Tabbakhha et al., 2016].

Although the failure of the Tex Wash Bridge was catastrophic, no warning signs were present during inspections. The year prior, the bridge received a 91.5 out of 100 sufficiency rating during inspection [Kelman, 2015]. There were no major repairs or structural flaws according to the inspection standards at the time. This bridge was structurally sound and safely served its purpose for normal everyday conditions. Nonetheless, clearly, when this bridge experienced an unforeseen 1000-year rain event on July 19, 2015 [Tabbakhha et al., 2016], its design proved to be non-resilient.

Another example of a non-resilient design is the 2007 collapse of the bridge carrying I-35 westbound in Minneapolis. The collapse, which happened during rush hour in August of 2007, killed 13 people and injured over 140. Several issues are attributed to the collapse: 1) thin gusset plates; 2) unusual loading; and 3) the lack of redundancy [National Transportation Safety Board, 2008]. This bridge was fracture critical, meaning there were no repeated members in the primary structure. While economical to construct, the statically determinant nature of the trusses in the bridge requires all primary load-carrying members to maintain their integrity for the bridge to remain safe. There were no secondary load paths to accommodate the failure of a single gusset plate. A non-redundant bridge can also be described as a non-robust structure [Olmati, 2013]. Robustness, the inverse of resilience, is a structure's ability to endure damage while maintaining functionality [Olmati, 2013].

Ideally, a resilience analysis would help stakeholders better identify "weak links" in their systems, like the Tex Wash and I-35W bridges, and potentially allocate resources toward improving robustness and/or recovery. Unfortunately, these designs were not resilient and the results were both deadly and costly.

2.2.3.3 Quantifying Resilience

2.2.3.3.1 Bruneau et al. [2003]

Bruneau et al. [2003] proposed an equation to quantify resilience as a function of the quality of a structure and recovery time (Equation 2-1). The quality of a structure is a measure of its ability to perform its designed task where loss of quality is given as: 100-$Q(t)$. This relates resilience to a structure's functionality but is not sensitive to specific features or design choices in a particular structure. Figure 2-2 is a graphical representation of the quality of infrastructure from the onset of an event, t_0, to when the structure regains functionality at a level equal to before the event. While theoretically sound, implementation of Bruneau et al.'s [2003] framework requires the designer to quantify the nebulous "quality of infrastructure" and "rate of recovery" after an event.

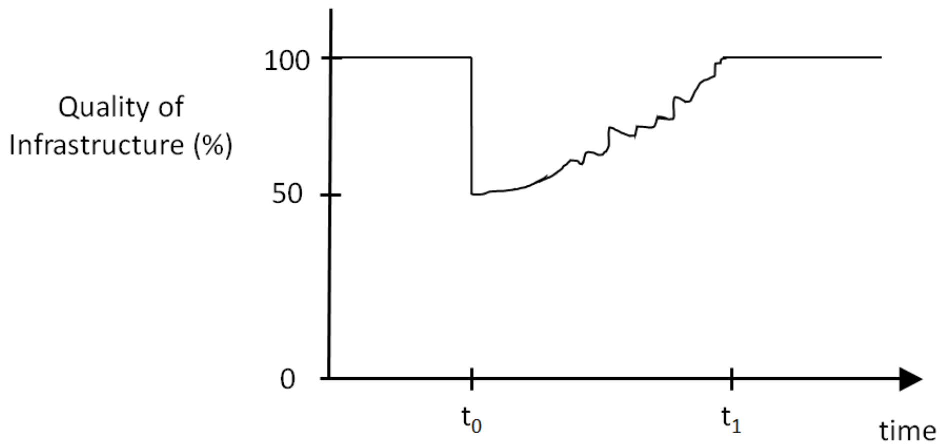

Figure 2-2: Quality Function [Bruneau et al., 2003]

2.2.3.3.2 Deco et al. [2013]

Deco et al. [2013] propose a similar probabilistic approach to bridge design using resilience and rapidity restoration techniques. Through their investigation, Deco et al. [2013] quantify specifically seismic resilience through assessment preceding a seismic event. Deco et al. [2013] propose Equation 2-2:

$$R = \frac{\int_{t_0}^{t_h} Q(t)dt}{t_h - t_0}$$

Equation 2-2

Where,

R= resileince

Q(t)= time-dependent functionality

t_0= time of the seismic occurrence

t_h is the total time the bridge is under investigation

Deco et al. [2013] provide several different techniques for restoring the functionality of the bridge after a seismic event. The results of their study found the most effective techniques to be the most expensive but also result in the least number of indirect consequences and, subsequently, the highest resilience because they reduce the recovery time. Unfortunately, since this research focuses on seismic events, the approach is not applicable to any extreme event.

2.2.3.3.3 Dong and Frangopol [2016]

Dong and Frangopol [2016] quantify the functionality of a bridge over time:

$$R_{Resi} = \frac{1}{\Delta t_r}\int_{t_0}^{t_0+\Delta t_r} Q(t)dt$$

Equation 2-3

Where:

R_{Resi}= resileince

Δt_r = the amount of time the bridge is under investigation after the extreme events

t_0= the time of investigation

Q(t)= the bridge's level of functionality over time

This is similar to the studies presented above, but their functionality losses are due to the combined event of an earthquake on a bridge that has experienced severe scour. This approach is more practical yet very specific to only certain cases. The approach has two shortcomings: it does not apply to all U.S. bridges and the analysis is retroactive, occurring only after a bridge is damaged.

2.2.3.3.4 Ikpong and Bagchi [2015]

Ikpong and Bagchi [2015] develop a way to quantify a bridge's resilience, specifically as it is impacted by climate change. The researchers propose five "bridge resilience indicators" (BRI's) to quantify a bridge's resilience: 1) abutment permafrost stability; 2) abutment washout; 3) pier scour; 4) abutment erosion; and 5) deck flooding. The BRIs are then compared to their capacity measures, which are a way to "indicate how well a bridge performs under climatic changes" [Ikpong and Bagchi, 2015]. The corresponding capacity measures are: 1) hydraulic capacity; 2) pier scour protection; 3) abutment thermal insulation; and 4) the presence of a pile foundation. The capacity measures only factor in climatic events due to climate change. For example, an increase in temperature would cause permafrost to thaw, increase snowmelt, and increase flow in channels.

Resilience indicators are weighted based on replacement cost, consequence of failure, and user cost. These weighted resilience indicators are then compared to the applicable capacity measures. In other words, abutment washout is compared to hydraulic capacity, pier scour is compared to pier scour protection, abutment erosion is compared to abutment thermal insulation or the presence of a pile foundation, deck flooding is compared to hydraulic capacity, and abutment permafrost stability is compared to abutment thermal insulation or the presence of a pile foundation.

Next, a BRI score is calculated based on the weighted indicators and comparative capacity measure. Figure 2-3 provides a flowchart of the proposed calculation of resilience:

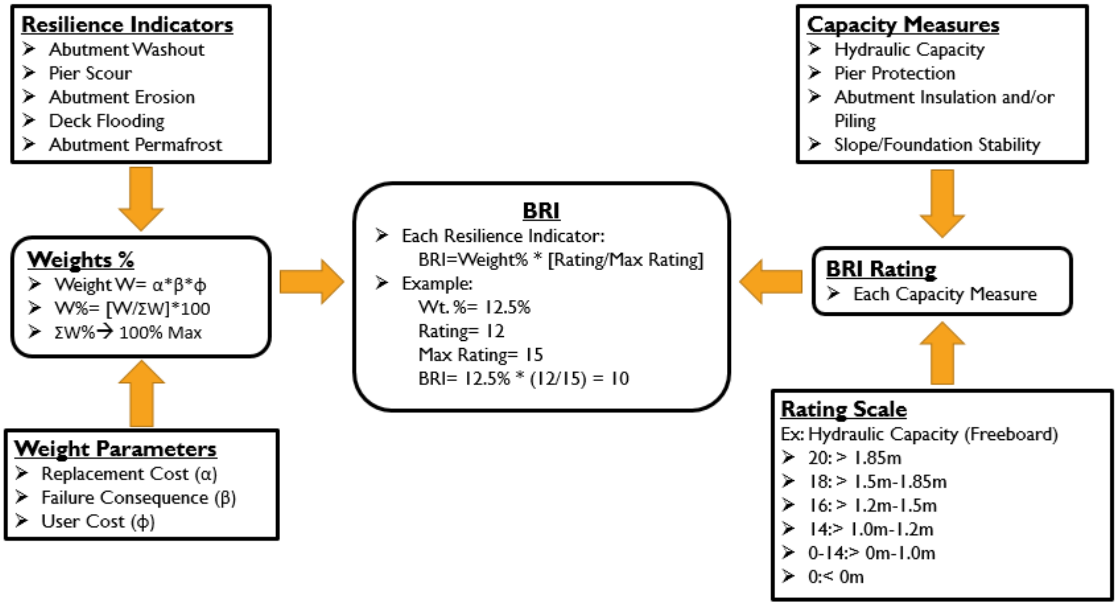

Figure 2-3: BRI Flowchart [Ikpong and Bagchi, 2015]

Calculation of a BRI happens in two steps. First, a weight factor (WF) is applied to account for the relative importance of the bridge and its effect on the cost, performance, and consequence of failure. The weight factor is the product of three parameters, as seen in Equation 2-4, and has a minimum value of 3.38 and a maximum value of 125.

$$WF = \alpha * \beta * \phi$$

Equation 2-4

Where:

α= cost of the component of the bridge that is directly affected by the climatic event

β= consequence of the climatic event on the overall performance of the bridge

ϕ= cost to the user in terms of the bridge's serviceability after a climatic event

Resilience indicators are weighted by these weight factors. Replacement cost is scored by a percentage of the replacement cost as compared to the cost of the original bridge construction. The consequence of event is scored by the severity of the event on the bridge's health, i.e. would it warrant investigation or would the bridge be out of service. User cost is scored by the impact on the road user in terms of reduced number of lanes or detour length. A sample WF calculation is presented in Table 2-1.

Table 2-1: BRI Weight Calculation [Ikpong and Bagchi, 2015]

Example Calculation: BRI Weights- Sample Bridge					
Resilience Indicator	Replacement Cost (α)	Consequence of Event (β)	User Cost (ϕ)	Weight Factor WF= α*β*ϕ	Weight (%)
	5: 20-25%	5: Out of service	5: Out of service and no detour	WF_{max}= 125	Weight= WF/ΣWF
	4: 15-20%	3: Requires immediate repair/replacement	3: Detour 300-500 km	WF_{min}=3.38	
	3: 10-15%	2: Triggers Deterioration	2.5: 150-300 km		
	2: 5-10%	1.5: Warrants testing or investigation	2.0: 0-150 km		
	1.5: 0-5%	0: N/A	1.5: Delays, number of lanes reduced		
	0: N/A		0: N/A		
Abutment Washout	4	5	2	40	44.3
Pier Scour	5	1.5	1.5	11.25	12.5
Abutment Erosion	4	2	1.5	12	13.3
Deck Flooding	5	2	1.5	15	16.6
Abutment Permafrost	4	2	1.5	12	13.3

The second step of the BRI calculation involved comparing each resilience indicator to its capacity measures. Each capacity measure rating is based on a capacity guide, thereby allowing an aspect of objectivity to the score. For example, Ikpong and Bagchi [2015] use a 2070's demand for their capacity measures, which included a projected rise in sea level due to the effects of climate change. Ikpong and Bagchi's [2015] rating scales for hydraulic capacity, pier scour resilience, and abutment insulation are shown in Table 2-2.

Table 2-2: Rating Scale for Capacity Measures

Hydraulic Capacity	**Pier Scour Resilience**	**Abutment Insulation**
20: > 1.85 m	15: Engineered apron	15: Piling and blanket
18: 1.5-1.85 m	12: Adequate riprap	13: Piling only; new
16: 1.2-1.5 m	9: Some riprap	11: Pile foundation and blanket
14: 1.0- 1.2 m	6: No riprap, slow flow	8: Pile foundation only
0-14: 0- 1.0 m	3: No riprap, fast flow	6: Wood and slow creek
0: <0 m	0: No riprap, threatened	4: Wood and fast creek
-	-: N/A	2: Crumbling wood

After the weight factors and the resilience indicators are determined, the BRI is calculated. The BRI is the sum of the product of the weight factors and the resilience indicators, as shown in Equation 2-5:

$$BRI = \sum_{i=1}^{n} W(x_i) * R(x_i)$$

Equation 2-5

Where:

W= weight factor

R= resilience indicator rating

Ikpong and Bagchi [2015] performed a sensitivity analysis to determine the effect of each resilience indicator on the calculation of the BRI. Hydraulic capacity and detour availability and length were chosen as parameters for the sensitivity analysis. The researchers demonstrated that the BRI score is sensitive to the selection of freeboard depth as the BRI decreased with an increase in freeboard. The BRI score is also sensitive to the selection of detour length, as the BRI score increased with a reduction in detour length.

An advantage of the approach proposed by Ikpong and Bagchi [2015] is it is easily incorporated into current practice because it uses information collected during routine inspections. One disadvantage is it is only applicable in regions that have permafrost. Regions in warmer climates, such as Arizona, are not subjected to melting permafrost from increased temperatures.

2.2.4 Current Practice

2.2.4.1 LRFD Design

Current design standards utilize an approach based on limit state philosophy, which is applied to serviceability, fatigue, the strength of the structure, and extreme events [Alipour, 2013]. The process of a limit-based design involves identifying all ways a structure can fail, determining a level of safety for each limit state, and identifying the important limit states for each individual case [MacGregor, 1976]. Of these criteria, the only limit state that has been calibrated using a reliability-based approach is the strength limit state [Alipour, 2013]. In order to better design for unpredictable events, performance goals should be based on the applicable limit states, not the event itself [Wen, 2001].

Reliability-based design involves taking the variability in strength, material, loading, and size of individual members in a structure and their limit state, ultimate limit state, and serviceability limit state. This has mainly come in the form of a load and resistance factor design (LRFD) approach. Currently, the American Association of State and Highway Transportation Officials (AASHTO) specify the use of LRFD for the design of bridges. In LRFD, criteria for design are based on probability of failure. Current practice dictates that structures are designed for high frequency, high probability events.

2.2.4.2 Performance-Based Design

Estes and Frangopol [2001] propose a reliability-based approach for identifying bridges in need of repair. They analyzed the bridges as a system, rather than individual parts, in order to determine serviceability and performance. This approach is very similar to modern performance-based design (PBD) approaches. Ghosn et al. [2016] found that current reliability-based design methods had large variations in desired reliability levels of evaluating the strength of structures. Ghosn et al. [2016] suggest that performance-based design may be the best next step in design standards due to its analysis of varying failure modes, performance levels, and structure uses as well as the analysis of risk. It is clear that some risk analysis performed in the design phase of a structure can aid in the structure's performance.

Performance-based design involves designing a structure to meet certain performance goals throughout the design life, from initial design to service and maintenance to demolition [Augusti and Ciampoli, 2008]. This can limit the ability of a designer to control the results of a performance-based design since many of the stages of life are out of their control [Augusti and Ciampoli, 2008]. However, this can be a very resourceful and customized process because every aspect of design is precisely chosen for each unique project. Since a PBD approach is non-prescriptive, creative solutions can be chosen for extreme events not currently standardized in design codes [Ghobarah, et al. 2001]. Currently, PBD is most commonly used in seismic design [Zameeruddin and Sangle, 2016]. In addition, PBD requires a contemporary contractual arrangement because the process involves the design team, owner, and contractors from the onset requiring a more collaborative approach.

2.2.4.3 Bridge Inspections

Federal law requires routine inspection of bridges to determine their overall condition, damage in need of repair, and/or needed maintenance. Typically, a visual inspection will suffice. Pictures are collected of the overall bridge, approaches, deck, abutments, piers, superstructures, substructure, and any identified maintenance or repair items. Very little quantitative data is collected other than measurements of the vertical clearance, average span, and overall length. Currently, inspectors perform a site visit to a bridge for a visual inspection. Inspectors gather information about the condition of bridge components through a qualitative rating system. The deck, substructure, superstructure, and any applicable channel, channel protection, and culverts are scored with a condition rating. This condition rating compares the current state of the structure to when it was first constructed [US Department of Transportation Federal Highway Administration., 1995]. Scores are based on the condition as seen visually by the inspectors. The scores also indicate whether local failures are possible and whether the bridge should be closed to traffic [US Department of Transportation Federal Highway Administration, 1995]. Most ratings are comparisons to previous inspection reports. If the inspector cannot access a particular section of a bridge— for example, a footing or pile— then that section is not assessed for damage. Results are reported to the FHWA and stored in the National Bridge Inventory (NBI) in accordance with National Bridge Inspection Standards (NBIS).

An important metric which results from a bridge inspection is the sufficiency rating (SR), which is a numerical value calculated using parameters such as functionality, serviceability, detour length, average daily traffic, and structural safety [US Department of Transportation Federal Highway Administration, 1995]. The sufficiency rating equation uses information from the Structure Inventory and Appraisal (SI&A) report and is shown below in Equation 2-6:

$$SR = S1 + S2 + S3 + S4$$

Equation 2-6

Where:

SR= Sufficiency rating

S1= 55 – (A+B), 0<S1<55

Where:

A= function of superstructure, substructure, and culvert rating

B= $(32.4 - IR)^{1.5} * 0.3254$

Where:

IR= Inventory rating

S2= 30- [J + (G + H) +I], 0< S2 <30 and 0< (G + H) <15

Where:

J= (A + B + C+ D + E + F), 0<J<13

Where:

A= function of deck condition

B= function of structural evaluation

C= function of deck geometry

D= function of underclearances

E= function of waterway adequacy

F= function of approach road alignment

G= function of bridge roadway width, approach roadway width, and structure type

H= function of ADT, bridge roadway width, and lanes

I= function of vertical clearance and Strategic Highway Network (STRAHNET) designation

S3= 15 – (A + B), 0< S3 <15

Where:

$$A=15 * \frac{ADT*Detour\ Length}{320{,}000*K},\ 0< A <15$$

Where:

$$K= \frac{S1+S2}{85}$$

B= function of STRAHNET designation

If the inspector discovers any major issues requiring repair, then a report is compiled detailing what the issue is, where it is located on the bridge, and its repair priority [US Department of Transportation Federal Highway Administration, 1995]. The sufficiency rating is also used to determine whether a bridge merits further investigation. If an issue were detected from the sufficiency rating, maintenance or repairs would be triggered. A drawback of this approach is it is reactive as opposed to proactive. Repairs and maintenance are not performed until inspectors discover an issue. There are no proactive measures taken to prevent such issues from occurring in the first place.

2.2.5 Current Needs

Resilience is an integral component of a bridge's performance and should be incorporated into its design, construction, and maintenance. There is a need to adapt resiliency concepts into the management of infrastructure. As technologies, abilities, and techniques improve, new bridges may not fit into the “normal” criteria that form the basis of current design codes. In addition, in light of the increasing frequency of intense and/or unforeseen loading events, we will need our structures to be better equipped to adapt. New design strategies that address resilience may be the next step to design the bridges of the future [Casas, 2015].

In order to include resilience in design, there is a need for a measured and objective assessment. A quantitative value for resilience will provide an easily interpretable measurement for a bridge’s ability to recover from loss of functionality. In addition, this value can further the advancement of PBD approaches for bridges.

Along with a quantitative measure, there is a need for “shovel-ready” guidelines for incorporating resilience into the structural design, analysis, and inspection of bridges. Through adoption of resilience policies, we may be able to preemptively limit or prevent the economic or life losses of future catastrophes.

2.2.6 Goals

In light of the above needs, the goal of this research aims to incorporate a resilience measure into the design of new bridges and the assessment of existing bridges. This will be accomplished by first identifying the factors that will be used to inform the resilience measure and identifying a process for using these factors. Then, a number of case study bridges will be selected. A purposeful sampling includes new and old bridges, as well as bridges with high and low sufficiency ratings. Condition assessments will be performed, and sufficiency ratings and load ratings calculated. Based on preliminary assessments of the case study bridges, a bridge resilience rating system will be derived and refined. Finally, the resilience of the bridges of this study will be rated. The results of the ratings between bridges will be compared and the implications discussed.

3 Methods

3.1 Overview

The purpose of this chapter is to explain the methodology for the assessment of an existing bridge's resilience. Initially, all bridges in Arizona are surveyed and four are selected for a comparative case study. To rate their resilience, information on these bridges, such as the as-built construction drawings and the Structure Inventory and Appraisal (SI&A) Report are obtained through the Arizona Department of Transportation (ADOT).

3.2 Collection of Existing Data

3.2.1 ADOT Bridge Database

The ADOT bridge database contains information on all of the bridges in Arizona that are owned and managed by the state of Arizona. The information contained in this database includes the structure number, route number, route milepost, bridge name, district, year built, kind of material, type of design/construction, number of spans, span length, structure length, inventory rating, operating rating, and sufficiency rating, among others. This information is used to identify and select four case study bridges, as detailed in Section 3.5.

3.2.2 As-Built Construction Documents

After selection of the four case study bridges, as-built documents were obtained from ADOT. These documents include information about the bridge's location, site characteristics, material properties, and design loading. They also provide construction details of the embankment, foundation, substructure, and superstructure. This information was used to perform resilience calculations as detailed in Sections 3.4.2 through 3.4.3.

3.2.3 Inspection Documents

3.2.3.1 Contents

Inspection documents for each bridge were obtained from ADOT. These documents include a Structure Inventory and Appraisal (SI&A) report, inspection report, inspection photographs, channel diagram, and any applicable repair reports, maintenance reports, or fracture critical member in-depth inspection reports.

3.2.3.2 Structure Inventory and Appraisal (SI&A) Report

The SI&A report contains information on functionality, integrity, and essentiality for public use. Functionality refers to a bridge's ability to accommodate current traffic conditions as a function of ADT and the bridge's geometry. Integrity refers to a bridge's structural safety and is a function of the condition rating of the substructure and superstructure, and inventory rating. Essentiality for public use is a function of detour length and ADT. The SI&A report culminates in the sufficiency rating (SR), which is a measure of how "sufficient" (i.e. functional, structurally sound, and essential) a bridge is for its current use. The SR value is used by the FHWA to justify the allocation of repair or replacement funding. For example, if a bridge has a sufficiency rating below 80, it is eligible to receive funding for repairs [US Department of Transportation Federal Highway Administration, 2012]. If a bridge has a sufficiency rating of 50 or less, it is eligible to receive funding for replacement [US Department of Transportation Federal Highway

Administration, 2012]. The SI&A report can also classify a bridge as structurally deficient, functionally obsolete, or fracture critical.

A goal of this project is to rate a structure's resilience using information contained in the SI&A report because it is readily available. A reason for this is to ensure easy adaptation by bridge management groups.

3.2.3.3 Inspection Report

The inspection report contains detailed information about the condition of the bridge. The report contains the National Bridge Inventory (NBI) condition ratings from the SI&A report and provides in-depth explanations for why the deck, superstructure, substructure, and channel receive their ratings. The deck, superstructure, and substructure are rated on a scale from 0-9 as seen in Table 3-1.

Table 3-1: Condition Ratings for Deck, Superstructure, and Substructure [US Department of Transportation Federal Highway Administration, 1995]

Code	Description
N	**Not applicable**
9	**Excellent condition**
8	**Very good condition**- no problems noted.
7	**Good condition**- some minor problems.
6	**Satisfactory condition**- structural elements show some minor deterioration.
5	**Fair condition**- all primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.
4	**Poor condition**- advanced section loss, deterioration, spalling, or scour.
3	**Serious condition**- loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	**Critical condition**- advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.
1	**“Imminent” failure condition**- major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	**Failed condition**- out of service- beyond corrective action

The channel and channel protection is similarly rated on a scale as seen in Table 3-2.

Table 3-2 Condition Rating for Channel and Channel Protection [US Department of Transportation Federal Highway Administration, 1995]

Code	Description
N	**Not applicable**. Used when bridge not over waterway
9	**There are no noticeable or noteworthy deficiencies** which affect the condition of the channel
8	**Banks are protected or well vegetated.** River control device such as spur dikes and embankment protection are not required or are in a stable condition
7	**Bank protection is in need of minor repairs**. River control devices and embankment protection have a little minor damage. Banks and/or channel have minor amounts of drift
6	**Bank is beginning to slump**. River control devices and embankment protection have widespread minor damage. There is minor stream bed movement evident. Debris is restricting the channel slightly
5	**Bank protection is being eroded**. River control devices and/or embankment have major damage. Trees and brush restrict the channel
4	**Bank and embankment protection is severely undermined**. River control devices have severe damage. Large deposits of debris are in the channel
3	**Bank protection has failed**. River control devices have been destroyed. Stream bed aggradation, degradation or lateral movement has changed the channel to now threaten the bridge and/or approach roadway
2	**Bridge is near state of collapse** because of changed in the channel
1	**Bridge closed because of channel failure**. Corrective action may put back in light service
0	**Bridge closed because of channel failure**. Replacement necessary.

The inspection report also includes the appraisal ratings from the SI&A report for the structural evaluation, deck geometry, vertical and horizontal clearances, waterway adequacy, approach roadway alignment, and scour critical designation. These items are also rated on a scale from 0 to 9, as seen in Table 3-3. Finally, a detailed explanation on the status of any maintenance or repairs is also reported in the inspection report.

Table 3-3: Condition Rating for Structural Evaluation, Deck Geometry, Vertical and Horizontal Clearances, Waterway Adequacy, and Approach Roadway Alignment [US Department of Transportation Federal Highway Administration, 1995]

Code	Description
N	Not applicable
9	Superior to present desirable criteria
8	Equal to present desirable criteria
7	Better than present minimum criteria
6	Equal to present minimum criteria
5	Somewhat better than minimum adequacy to tolerate being left in place as is
4	Meets minimum tolerable limits to be left in place as is
3	Basically intolerable requiring high priority of corrective action
2	Basically intolerable requiring high priority of replacement
1	*This value of rating code not used*
0	Bridge closed

Scour critical bridges is rated on a scale from 0 to 9, as seen in Table 3-4.

Table 3-4: Rating Scale for Scour Critical Bridges [US Department of Transportation Federal Highway Administration, 1995]

Code	Description
N	Bridge not over waterway
U	Bridge with "unknown" foundation that has not been evaluated for scour. Since risk cannot be determined, flag for monitoring during flood events and, if appropriate, closure
T	Bridge is over "tidal" waters that has not been evaluated for scour, but considered low risk. Bridge will be monitored with regular inspection cycle and with appropriate underwater inspections
9	Bridge foundations (including piles) on dry land well above flood water elevations
8	Bridge foundations determined to be stable for assessed or calculated scour conditions. Calculated scour is above top of footing
7	Countermeasures have been installed to correct a previously existing problem with scour. Bridge is no longer scour critical
6	Scour calculation/evaluation has not been made
5	Bridge foundations determined to be stable for calculated scour conditions. Scour within limits of footing or piles.
4	Bridge foundations determined to be stable for calculated scour conditions. Field review indicates action is required to protect exposed foundations from effects of additions erosion and corrosion
3	Bridge is scour critical. Bridge foundations determined to be unstable for calculated scour conditions: 1) scour within limits of footing or piles, 2) scour below spread-footing base or pile tips
2	Bridge is scour critical. Field review indicates that extensive scour has occurred at bridge foundations. Immediate action is required to provide scour countermeasures
1	Bridge is scour critical. Field review indicates that failure of piers/abutments is imminent. Bridge closed to traffic
0	Bridge is scour critical. Bridge has failed and is closed to traffic

3.2.3.4 Bridge Inspection Photographs

Inspection reports are accompanied with a "photo report", which contains photographs the inspectors collected during inspection. Typical photographs in this report include the roadway identification, bridge elevation view, deck condition, joint conditions, and any significant damage.

3.2.3.5 Channel Diagram

The channel diagram is a sketch of the elevation of the bridge and waterway. This sketch shows the transverse characteristics of the channel and substructure, such as slope and elevation, number of piers, span lengths, and overall bridge length.

3.2.3.6 Repair Report

The repair report contains information on needed repairs. Damaged items that contribute to the structural integrity of the bridge are the only items considered for repair. For example, deck deterioration, cracks in girders or abutments, and bearings in need of replacement are considered repair items. The repair report also includes detailed information about the extent and cost of the repairs, and a timeframe on when the repairs are to be completed.

3.2.3.7 Maintenance Report

The maintenance report lists needed maintenance items, which are different from repair items in that maintenance items are not concerned with the integrity of the bridge. For example, damaged guardrails, excessive debris, or cracks in the wearing surface are considered maintenance items. The maintenance report provides an estimate of the total cost of maintenance as well as priority scheduling requirements.

3.2.3.8 Fracture Critical Member In-Depth Inspection

An in-depth inspection is required for bridges that are designated as "fracture critical". A fracture critical bridge is a structure that could totally collapse upon the loss of a single component. The in-depth reports contain information about which members are fracture critical and their condition. Photographs are taken of these members along with a sketch or drawing of their location on the structure.

3.3 Characteristics Related to Resilience

3.3.1 Sufficiency Rating

A bridge's sufficiency rating is listed in the SI&A report and contains information on functionality, structural integrity, and essentiality or usefulness to the public. The sufficiency rating is scaled from 0 to 100. Bridges with higher scores are typically considered "more sufficient" bridges. A major drawback of a sufficiency rating is it is not indicative of a structure's resilience.

While not a measure of resilience, a component of a structure's sufficiency, "user cost", is also a component of its resilience. The determination of user cost, *S3*, is listed in Equation 3-1 [US Department of Transportation Federal Highway Administration, 1995]:

$$S3 = 15 - (A + B)$$

Equation 3-1

Where:

A= $15\left(\frac{ADT * DL)}{320{,}000 * k}\right)$

k= $\frac{S1+S2}{85}$

S1= a function of the condition ratings for

S2= a function of ADT and bridge geometry

ADT= average daily traffic

DL= detour length (miles)

B= 2 if Item 100 > 0

B= 0 if Item 100 =0

Item 100= STRAHNET highway designation

User cost rating is easy to implement into a resilience rating because it is part of the inspection report.

3.3.2 Scour Critical Designation

A scour critical designation is listed in the SI&A report, where a score of 4 or less is a scour critical rating. In addition, for a bridge to be classified as scour critical, it also must have the characteristic of unstable piers or abutments due to either existing scour depth or the potential for scour as determined by an in-depth scour evaluation study [US Department of Transportation Federal Highway Administration, 1995]. Scour critical bridges with existing scour damage are routinely monitored to determine if repairs are needed, based on the results and recommendations from the in-depth scour study. Scour is also a sufficiency parameter related to resilience because of its relationship to the intensity of a weather event.

3.3.3 Scour Protection

To quantify resilience, it is necessary to characterize the level of a bridge's scour protection. In the SI&A report, the type of scour protection is listed for both the abutments and the piers. The protection type is recorded on a scale, with the highest numbers designating the highest level of protection and lower numbers designating a lower level of scour protection.

3.3.4 Detour Length and Average Daily Traffic (ADT)

The combination of both the detour length and average daily traffic is a measure of how essential the route is for a community and the cost to the users. A bridge with a small detour length but large ADT may have a similar user cost to a bridge with a large detour length but small ADT. These parameters are included in the SI&A report and are used in Equation 3-1 to calculate a bridge's essentiality for public use.

3.3.5 Fracture Critical Designation

A bridge is classified as fracture critical if the loss of one member or component can cause the entire structure to fail. The fracture critical designation of a structure is related to its resilience (or lack of) because a structure that is fracture critical does not have redundancy. In other words, if a bridge can fail with the loss of one member, it is a non-redundant structure and therefore non-resilient because it does not have the ability to recover.

3.3.6 Inventory and Operational Rating

The inventory and operational rating designates the largest truck that can be driven over the bridge without exceeding the design capacity of its components. Currently, bridges are commonly designed for an AASHTO LRFD HL-93 loading, as illustrated in Figure 3-1:

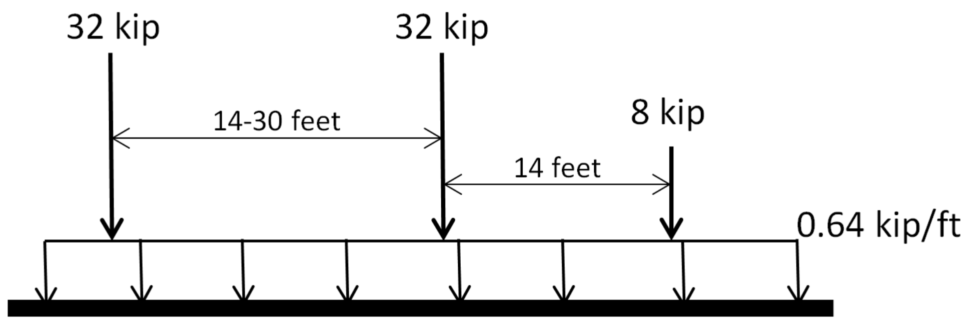

Figure 3-1: HL-93 Truck Loading

The two 32 kip point loads represent the two trailer axles of a truck and are separated by a distance of 14 to 30 feet. The 8 kip point load represents the axle for the cab of the truck and is located 14 feet away from the 32 kip load. A lane loading of 0.64 kips per foot is included to account for the dynamic effect of the moving vehicles. If a bridge cannot resist the loads produced by an HL-93 truck, Arizona state regulations require the maximum loading of the bridge be posted along the route.

3.3.7 Foundation Type

A bridge's foundation type is related to its resilience because it determines how a substructure withstands flows that could potentially wash them out. The foundation type is categorized as spread footings, piles, or drilled shaft foundations.

3.3.8 Supports

A bridge's supports are related to its resilience because they influence how a bridge responds to thermal deformations caused by temperature differentials. As such support types are categorized based on their ability to alleviate thermal deformations.

3.3.9 Capacity and Demand

Typically, engineers proportion the structural elements of a bridge to fail by a flexural mechanism. Flexural failures are more ductile and show warning signs of yielding before complete loss of equilibrium. Ductile failure modes are easier to predict, as indicated by their strength reduction factors. For example, flexure has a much higher strength reduction factor of 0.90 compared to shear at 0.75. Strength reduction factors are calibrated so a more ductile mechanism is the controlling failure mode. If the ductile and brittle failure modes are close to their respective capacities, the member has a higher probability of a brittle failure. A bridge that would fail suddenly in shear results in a low resilience due to the lack of warning signs and higher probability of failure.

3.4 Proposed Resilience Rating Method

This section outlines the proposed methodology for rating the resilience of a bridge. This approach is adapted from the method proposed by Ikpong and Bagchi [2015] (see Section 2.2.3.3.4). The resilience of a bridge can be rated as given in Equation 3-2:

$$RR = \sum_{i=1}^{n} W(x_i) * R(x_i)$$

Equation 3-2

Where:

RR= resilience rating

$W(x_i)$= weight factor for each resilience indicator

$R(x_i)$= rating of each resilience indicator against its capacity measure

n= number of resilience indicators

The interaction between weight factors, resilience indicator ratings, and capacity measures is illustrated in Figure 3-2:

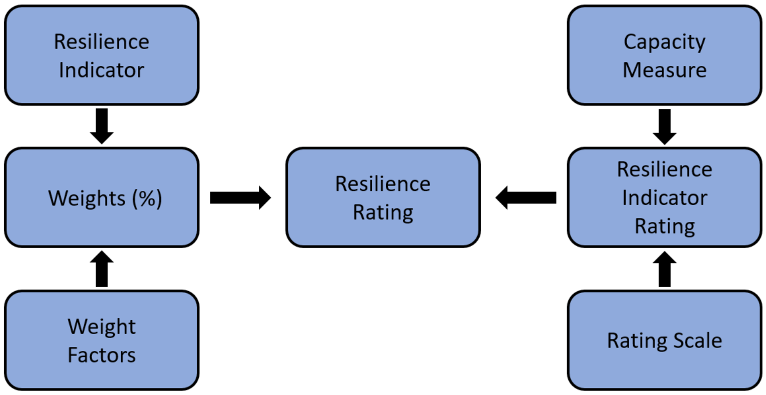

Figure 3-2: Resilience Rating Flowchart

3.4.1 Weight Factors

In addition to the capacity measure ratings, the resilience indicators are weighted based on the structural integrity, functionality and essentiality of the bridge. A weight factor is calculated to determine the relative importance of each resilience indicator relative to the other indicators. The three proposed weight factors are adapted from Ikpong and Bagchi [2015] as follows:

1) Replacement cost (α)

2) Consequence of event (β)

3) User cost (φ)

The composite weight for each resilience indicator is determined using Equation 3-3:

$$W(x) = \frac{WF}{\sum_{i=1}^{n} WF}$$

Equation 3-7

Where:

W(x)= composite weight for a given resilience indicator

WF= weight factor, $\alpha \times \beta \times \varphi$

n= number of resilience indicators

The weights provide a relative comparison between resilience indicators and are described in detail in Sections 3.4.1.1 through 3.4.1.3.

3.4.1.1 Replacement Cost, α

The replacement cost weight factor is the cost of a component of the bridge affected by the applicable resilience indicator. In other words, the replacement cost is the monetary cost to repair or replace that component for the bridge to regain functionality. The weight factor is based on the ratio of the cost to replace the single component over the cost of the construction of the entire bridge. Since total construction costs are reported for the year they were built and do not take into account inflation, this ratio can be simplified into a ratio of the size of the component over the size of the entire bridge. The weight factors are adapted from Ikpong and Bagchi [2015] through the authors' "several years of experience" with bridge inventory management (Table 3-5).

Table 3-5: Replacement Cost Weight Factor

Weight Factor	Replacement Cost, α
5	20-25%
4	15-20%
3	10-15%
2	5-10%
1.5	0-5%
0	N/A

3.4.1.2 Consequence of Event, β

The consequence of event weight factor refers to the state of the bridge's structural integrity and functionality if it were to sustain damage from the applicable resilience indicator. These ratings are also adapted from Ikpong and Bagchi [2015] as given in Table 3-6.

Table 3-6: Consequence of Event Weight Factor

Weight Factor	Consequence of Event, β
5	Out of service
3	Requires immediate repair/replacement
2	Triggers deterioration
1.5	Warrants testing or investigation
-	N/A

3.4.1.3 User Cost, φ

The user cost weight factor is determined by considering the cost of the bridge to the user if it were to sustain damage. This factor is based on detour length and average daily traffic (ADT). "Essentiality for Public Use", *S3,* is used to quantify user cost. This metric uses information given in the SI&A report, including detour length, ADT, and

STRAHNET highway designation. However, this user cost rating applies to the entire bridge and does not change with varying components. Therefore, the *S3* rating is used in conjunction with the direct user impact cost rating scale from Ikpong and Bagchi [2015]. The highest rating between *S3* for the entire bridge and the direct user impact rating is used. The weight factors are listed in Table 3-7 and are adapted from Ikpong and Bagchi [2015].

Table 3-7: User Cost Weight Factor

Weight Factor	User Cost, φ
5	$0 < S3 \leq 3$ Out of service and/or no detour
3	$3 < S3 \leq 6$ Temporary closure and detour over 98 miles
2.5	$6 < S3 \leq 9$ Temporary closure and detour 50-98 miles
2	$9 < S3 \leq 12$ Temporary closure and detour 0-50 miles
1.5	$12 < S3 \leq 15$ Delays/number of lanes reduced
0	N/A

3.4.2 Resilience Indicators

3.4.2.1 Proposed Indicators

The proposed approach weights and ranks five indicators of structural resilience and calculates a total resilience rating. Resilience "indicators" are synonymous with an unforeseen event (can also be thought of as the "demands" on resilience) and are compared with applicable capacity measures. Each resilience indicator is mapped and ranked against a respective capacity measure. As such, the five proposed resilience indicators and respective capacity measures are shown in Figure 3-3. These indicators and capacity measures are explained in detail in subsequent sections.

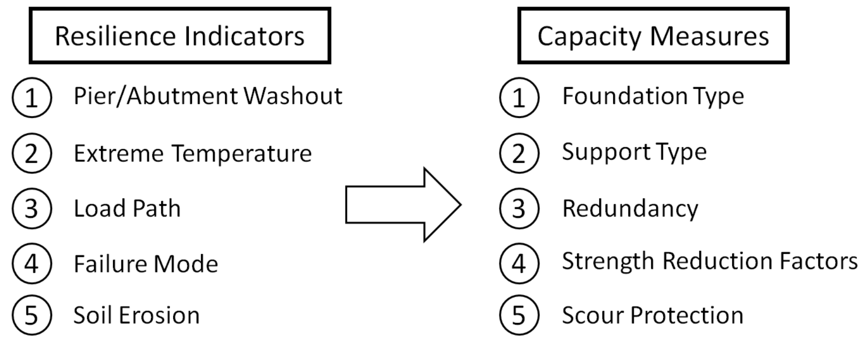

Figure 3-3: Resilience Indicators and Capacity Measures

3.4.2.2 Pier/Abutment Washout

Extreme precipitation events can cause flash flooding. Across Arizona, the summer monsoon season brings more precipitation than is seen the rest of the year. Commonly, precipitation events during monsoon season in Arizona have large amounts of rainfall over a short period of time. Changes in climate are making these events more intense and more frequent [Castro et al., 2014]. These new demands potentially cause unpredictable demand to piers and abutments relative to historical precedent. For example, as seen in the Tex Wash Bridge (Section 2.2.3.2), intense flash flooding can washout the entire structure and cause a total collapse. A more resilient foundation would resist such excessive amounts of soil loss. For the proposed method detailed in this chapter, the resilience indicator of *pier or abutment washout* is ranked against the *foundation type* capacity measure.

3.4.2.3 Extreme Temperature

Extreme temperatures can also affect the structural integrity of a bridge. With current changes in weather patterns, bridges are experiencing more extreme variations in temperature. Higher temperature variations can cause excessive longitudinal expansion, thereby placing higher demand on the support bearings and expansion joints. Thus, for the method proposed in this chapter, the resilience indicator of *extreme temperature* is ranked against the *support type*.

3.4.2.4 Load Path

A structure's load path is a critical component of its resilience because alternative load paths allow for load redistribution in the event of a local failure. By providing alternative load paths, a bridge can remain functional even if a member fails. In contrast, non-redundant structures are not resilient because a single member failure can cause the entire structure to collapse. For the method proposed in this chapter, the resilience indicator of *load path* is ranked with the bridge's *redundancy*.

3.4.2.5 Failure Mode

Typically, bridges are designed so that a weakening or failing member gives warning or indication of yielding before failure. In design, this is typically accomplished through the calibration of strength reduction factors. This calibration compels a bridge's capacity to be controlled by a ductile failure mode. In contrast, if a bridge were controlled by a brittle failure mode, there would be little to no indication of yielding. As a result, if the chance of a brittle failure mode occurring were such that it happened before a ductile failure mode, then an intense or unpredictable loading event could result in a sudden, unexpected failure. For the proposed method described in this chapter, the *failure mode* resilience indicator is ranked against the *strength reduction factors*.

3.4.2.6 Soil Erosion

Similar to abutment and pier washout, soil erosion can result from intense flooding or flows. Erosion of the soil surrounding the foundation could result in a stability failure (e.g. overturning, sliding, bearing, etc.). Bridges typically have scour protection measures installed in the channel to prevent excessive soil erosion. Thus, the type and amount of scour protection can make the bridge more or less resilient. For the proposed method detailed in this chapter, the *soil erosion* resilience indicator is ranked against the *scour protection*.

3.4.3 Capacity Measures

3.4.3.1 Resilience Indicators versus Capacity Measures

As previously mentioned, capacity measures are synonymous with the protection of the bridge to withstand an unforeseen event and resilience indicators categorize these events. Capacity measures are ranked with scores between 0 and 15. These values are adapted from the recommendations of Ikpong and Bagchi [2015]. According to Ikpong and Bagchi [2015], the values were determined from a combination of expert interviews and procedures found in Sinha et al. [2009], Patidar et al. [2007], and Thompson et al. [2008].

3.4.3.2 Foundation Type

The foundation type capacity measure is used to rate the resilience of the pier or abutment against washout. The ratings and capacity measures, as seen in Table 3-8, are based on foundation type given in the SI&A report.

Table 3-8: Foundation Type Rating

Rating	Foundation Type
15	9. Drilled shaft or caisson
13	8. Timber piles
11	7. Precast concrete piles
8	4 to 6. CIP fluted shell, CIP pipe shell, Steel H piles
6	3. Spread on bedrock
4	2. Spread on cemented soil
2	1. Spread on uncemented soil

Bridge foundation types are listed in the SI&A report with two numbers. The first number identifies the type of foundation for the abutments and the second number identifies the type of foundation for any applicable pier(s). For the proposed resilience rating method, the lower of the two numbers is used to rate the foundation type.

3.4.3.3 Support Type

The support type capacity measure is used to rate the resilience of the superstructure against extreme temperature variations. The two categories of supports considered are expansion joints and integral supports. Expansion joints allow for thermal movement due to an increase in temperature. This is beneficial because it allows the superstructure to expand without inducing stress. However, a disadvantage to expansion joints is they are more susceptible to corrosion and degradation, thereby requiring added maintenance. The required width of the expansion joint is a function of span length, change in temperature and the coefficient of thermal expansion. For this study, an extreme temperature differential between maximum lows and highs is 125° F. This value, found in the AASHTO LRFD Design Specifications section 3.12, comes from the most extreme temperature differential in Arizona, regardless of material or climate. The width of expansion joint required for this temperature difference is compared to the expansion joint as constructed. In this way, the capacity of the expansion joint is measured against an extreme change in temperature. The required expansion joint width calculated per Equation 3-4.

$$\delta_{temp125} = \alpha * \Delta T * L$$

Equation 3-4

Where:

$\delta_{temp125}$= thermal deformation for 125° F temperature difference (in.)

α= 6.0×10^{-6}/°F

ΔT= change in temperature (125° F)

L= span length (in)

This is then expressed as a ratio of the thermal deformation over the actual width of the expansion joint to determine the over-strength capacity of the expansion joint (Equation 3-5).

$$R_{\delta} = \frac{L_e}{\delta_{temp125}}$$

Equation 3-5

Where:

R_{δ}= deformation over-strength ratio

L_e= length of expansion joint (in.)

$\delta_{temp125}$= thermal deformation for 125° F temperature difference (in.)

If the thermal deformation is greater than the expansion joint or if there is no expansion joint (i.e. the superstructure is restrained from expanding), then the structure must resist the stresses caused by the deformation in Equation 3-4. The stress caused by thermal expansion is shown in Equation 3-6.

$$\sigma_{temp125} = \frac{\delta_{temp125}}{L} * E_c$$

Equation 3-6

Where:

$\sigma_{temp125}$= stress induced at a temperature difference of 125° F (ksi)

$\delta_{temp125}$= thermal deformation for a 125° F temperature difference (in.)

L= span length (in.)

E_c= modulus of elasticity of concrete (ksi)

This is also expressed as a ratio between the stress induced at a temperature differential of 125° F and 50% of the compressive strength of concrete to determine the over-strength of the concrete (Equation 3-7). A stress in excess of 50% of the compressive strength indicates the risk of concrete crushing due to service loading is assumed unacceptably high.

$$R_\sigma = \frac{0.5 * f'_c}{\sigma_{temp125}}$$

Equation 3-7

Where:

R_σ= stress over-strength ratio

f'_c= compressive strength of concrete (ksi)

$\sigma_{temp125}$= stress induced at a temperature difference of 125° F (ksi)

Table 3-9 shows the rating and respective capacity measures for support type. If the ratio is greater than 1, than there is over-strength in the supports. If the ratio is less than 1, than there is under-strength in the supports.

Table 3-9: Supports Rating

Rating	Support Type
15	Single span with integral abutments and $R_\sigma > 1$
13	Single span with expansion joints and $R_\delta > 1$
11	Multi span with integral abutments/piers and $R_\sigma > 1$
9	Multi span with expansion joints and $R_\delta > 1$
7	Single span with integral abutment and $R_\sigma \leq 1$
5	Multi span with integral abutments/pier and $R_\sigma \leq 1$
3	Single span with expansion joint and $R_\delta \leq 1$
1	Multi span with expansion joint and $R_\delta \leq 1$

Figure 3-4 shows a diagram of a bridge with the abutments and piers integral with the superstructure.

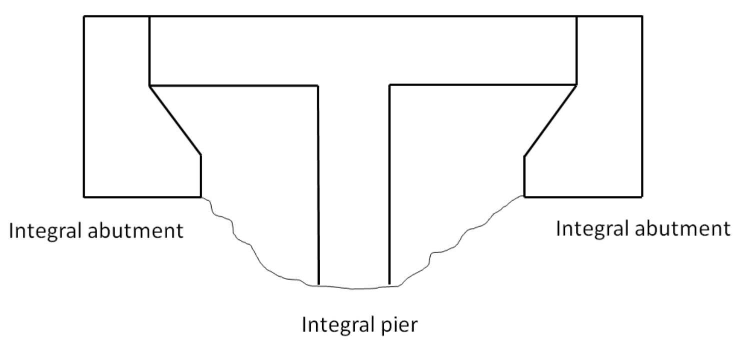

Figure 3-4: Bridge with Integral Abutments and Piers

Figure 3-5 shows a depiction of a bridge with expansion joints in the superstructure at the abutments and piers.

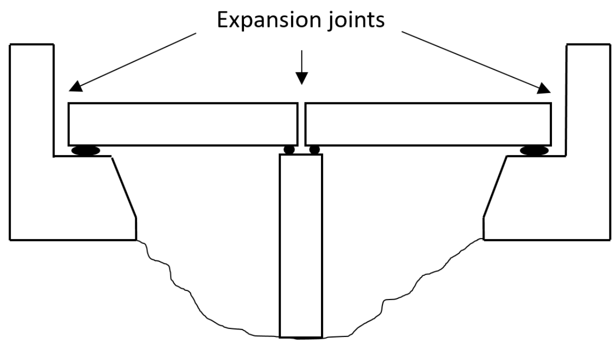

Figure 3-5: Bridge with Expansion Joints

3.4.3.4 Redundancy

To quantify the redundancy of a bridge, the capacity of the entire structure is compared to the capacity of its weakest component. This method is a ratio of the difference between the capacity of one component and capacity of the entire bridge over the capacity of the entire bridge and is expressed in Equation 3-8:

$$R = \frac{V_{bridge} - V_{element}}{V_{bridge}} \ or \ \frac{M_{bridge} - M_{element}}{M_{bridge}}$$

Equation 3-8

Where:

R= redundancy ratio

V_{bridge}= shear capacity of the entire bridge

$V_{element}$= shear capacity of the weakest element

M_{bridge}= flexural capacity of the entire bridge

$M_{element}$= flexural capacity of the weakest element

If the ratio equal to zero, then the bridge is fracture critical; that is, failure of a single component will result in a partial or complete failure of the bridge. The closer the ratio is to one, the grater the bridge's redundancy. Table 3-10 shows the redundancy rating scores and respective capacity measures.

Table 3-10: Redundancy Rating

Rating	Redundancy Ratio
15	0.85< Ratio $\leq$1.0; highly indeterminate
13	0.68< Ratio $\leq$0.85
11	0.51< Ratio $\leq$0.68
7	0.34< Ratio $\leq$0.51
5	0.17< Ratio $\leq$0.34
3	0< Ratio $\leq$0.16
1	Ratio = 0; fracture critical structure

The information on capacity for the entire bridge as well as the capacity of a single member is determined from the as-built construction drawings. The calculations to determine flexural and shear capacity are performed by hand using the procedures in the AASHTO LRFD Bridge Design Guidelines.

3.4.3.5Strength Reduction Factor

The resilience of a bridge in terms of its failure mode is rated by comparing the strength reduction factors for the two most critical failure mechanisms. The ratio of strength reduction factors is compared with the ratio of the load that causes the respective failure modes as given by Equation 3-9.

$$\%_{diff} = \left(\frac{FM_2}{FM_1} - \frac{\varphi_1}{\varphi_2}\right) \Bigg/ \left(\frac{\frac{FM_2}{FM_1} + \frac{\varphi_1}{\varphi_2}}{2}\right)$$

Equation 3-9

Where:

%diff= percent difference between failure modes and strength reduction factors

FM_1= load of causing failure mode 1

FM_2= load of causing failure mode 2

φ_1= strength reduction factor associated with failure mode 1

φ_2= strength reduction factor associated with failure mode 2

To illustrate the above equation, consider the scenario where failure mode 1 and 2 coincide with a strength reduction factor of 0.90 and 0.75, respectively. If both failure modes occur at the same load, the percent difference would be -18%. A percent difference above -18% means the load causing a ductile failure is sufficiently lower than the load causing a brittle failure. In other words, a ductile failure will most likely occur along with indications signs of yielding. A percent difference below -18% means the load causing a brittle failure is lower than the load causing a ductile failure. In this case, a brittle failure is more likely to occur before a ductile failure. The ratings for this capacity measure are given in Table 3-11.

Table 3-11: Strength Reduction Factor Rating

Rating	Strength Reduction Factor
15	$\%_{diff} > 50\%$
13	$33\% < \%_{diff} \leq 50\%$
11	$16\% < \%_{diff} \leq 33\%$
8	$-1\% < \%_{diff} \leq 16\%$
6	$-18\% < \%_{diff} \leq -1\%$
4	$\%_{diff} = -18\%$
2	$\%_{diff} < -18\%$

3.4.3.6 Scour Protection

The resilience of a bridge against soil erosion is rated based on its scour protection. The ratings, as seen in Table 3-12, are based on varying levels of scour protection. The rating values are taken from the SI&A report's "Scour protection countermeasures".

Table 3-12: Scour Protection Rating

Rating	Scour Protection		
	Flow Control Type	**Floor Protection Type**	**Bank Protection Type**
15	8. N/A	8. Grouted rock	8. Masonry
13	7. A combination	7. A combination	7. A combination
11	6. Retard	6. Aprons	6. Wire tied rock (Gabions)
9	5. Groins or training dikes	Wire tied pier pads	Dumped rock riprap
7	4. Outlet drop structure	Dumped rock floor	Dumped rock riprap
5	3. Wire tied riprap and rail check dam	Wire tied riprap	Rail bank
3	2. Concrete check dam	Soil cement floor	Soil cement
1	1. Spur dikes	Concrete floor	Concrete slope paving
0	0. No protection/ Scour Critical Designation	0. No protection/ Scour Critical Designation	0. No protection/ Scour Critical Designation

Scour protection countermeasures are listed in the SI&A report as a three digit field coded for flow control (first digit), floor protection (second digit), and bank protection (third digit). The codes are associated with the type of protection, given in Table 3-12. The score used to rank soil erosion for the calculation of the resilience rating is taken as the lowest of the three digits.

3.4.4 Resilience Rating Equation

The resilience rating for a bridge, Equation 3-2, is duplicated as follows for the convenience of the reader:

$$RR = \sum_{i=1}^{n} W(x_i) * R(x_i)$$

The resilience rating equation culminates in combining all of the above outlined steps. First, the resilience indicators are compared to their associated capacity measures (valued between 0 and 15), as seen in Table 3-13.

Table 3-13: Resilience Indicator Rating

Capacity Measure	Rating, R(x)
Foundation Type	-/15
Supports	-/15
Redundancy	-/15
Strength Reduction Factor	-/15
Scour Protection	-/15

Second, resilience indicators are weighted based on the weight factors for replacement cost, consequence of event, and user cost, as seen in Table 3-14.

Table 3-14: Resilience Indicator Weight Calculation

Resilience Indicator Weights					
Resilience Indicator	Replacement Cost (α)	Consequence of Event (β)	User Cost (φ)	Weight Factor, WF (α*β*φ)	Weight, W(x) (WF/ΣWF_i)
Pier/Abutment Washout					
Extreme Temperature					
Load Path					
Failure Mode					
Soil Erosion					

Finally, the resilience rating will be calculated by multiplying each resilience indicator rating with its corresponding weight percentage and summing the value, as seen in Table 3-15.

Table 3-15: Resilience Rating

Resilience Rating Calculation	
Resilience Indicator	W(x)× R(x)
Pier/Abutment Washout	
Extreme Temperature	
Load Path	
Failure Mode	
Soil Erosion	
Resilience Rating, RR=	=[ΣW(x)×R(x)]

3.5 Selection of Sample

A summary of the bridges chosen for the comparative case study can be seen in Table 3-16:

Table 3-16: Selected Sample Bridges

	High Sufficiency Rating	Low Sufficiency Rating
High Resilience Rating	Coyote Wash Bridge	Earp Wash Bridge
Low Resilience Rating	Tanner Wash Bridge	Midgley Bridge

Figure 3-6: Coyote Wash Bridge

The Coyote Wash Bridge in Prescott, AZ was chosen for the high sufficiency and an expected high resilience rating. The Coyote Wash Bridge was built in 2012 and the sufficiency rating is 99.90 from the last inspection on September 21, 2017. A high resilience rating is anticipated because of the bridge's high ratings in various categories in the SI&A report. For example, Coyote Wash Bridge has a scour critical rating of 8 out of 9, rock riprap along the bank, drilled shaft foundations, and expansion joints.

a) 

b) 

Figure 3-7: Earp Wash Bridge a) view looking north and b) looking west

The Earp Wash Bridge in Tucson, AZ was chosen for its low sufficiency and high anticipated resilience rating. The Earp Wash Bridge was built in 1958 and expanded in 1985 and has a sufficiency rating of 49.50. This bridge was chosen for a high expected resilience rating due to various categories found in the SI&A report. For example, the Earp Wash Bridge has a scour critical rating of 8 out of 9, rail bank protection, a short detour length of 1 mile, and a reinforced concrete slab superstructure integral with the substructure.

Figure 3-8: Tanner Wash Bridge

The Tanner Wash Bridge in Page, AZ was chosen for the high sufficiency and an anticipated low resilience rating. Tanner Wash Bridge was built in 1980 and has a sufficiency rating of 77.10. This bridge is expected to have a low resilience rating due to its scour data. Although the bridge is not scour critical, the scour critical rating is a 5. The bridge also has no scour countermeasures installed and the foundation is a spread footing on bedrock.

Figure 3-9: Midgley Bridge

Midgley Bridge in Sedona, AZ was chosen for the low sufficiency and an anticipated low resilience rating. This bridge was constructed in 1938 and has a sufficiency rating of 50.70. This bridge was chosen for the expected low resilience due to its classification as a fracture critical structure. In addition, it also has a large detour length of 38 miles with an ADT of 3858.

4 Results

4.1 Overview

The purpose of this chapter is to present the resilience rating results for the four case study bridges. First, information was gathered, calculations performed, and the results for the bridge ratings and weights are summarized. Next, the resilience ratings for each bridge are presented. Then, the resilience and sufficiency ratings are compared and contrasted. Results of a susceptibility analysis are presented demonstrating the practicality of the proposed approach. Finally, implications and further applications of the results are presented.

4.2 Bridge Ratings and Weights

4.2.1Coyote Wash Bridge

The Coyote Wash Bridge was selected for its high sufficiency rating of 99.9 and an anticipated high resilience rating. The following section provides details of the ratings and weights for the Coyote Wash Bridge.

Figure 4-1: Coyote Wash Bridge

4.2.1.1 Resilience Indicators Ranked by Capacity Measures

The following table shows all five resilience indicators and their rating for each capacity measure.

Table 4-1: Coyote Wash Bridge Resilience Indicator Ratings

Resilience Indicator	Capacity Measure Type	Rating	R(x)
Pier or Abutment Washout	9: Drilled shaft or caisson	15	15/15
Extreme Temperature	Single span with expansion joints and $R_\delta > 1$	13	13/15
Redundancy	0.87	15	15/15
Failure Mode	0.38	8	8/15
Soil Erosion	0: No protection	0	0/15

4.2.1.1.1 Pier or Abutment Washout versus Foundation Type

In order to rate Coyote Wash Bridge for its foundation type, the SI&A report was used. From the report, the foundation is listed as a drilled shaft or caisson type. The as-built construction plans confirm the bridge has 66-inch diameter drilled shaft foundations at the abutments. Using the rating scale given in Table 3-8, the bridge was rated a 15, as seen in Table 4-1.

4.2.1.1.2 Extreme Temperature versus Support Type

The Coyote Wash Bridge's superstructure is a single span and has expansion joints at the abutments. Since there are expansion joints, the change in length caused by a temperature differential of 125° F is compared to the actual length of the expansion joints. Using the rating scale given in Table 3-9, the Coyote Wash Bridge is ranked as seen in Table 4-1. Detailed calculations are provided in Appendix A.

4.2.1.1.3 Load Path versus Redundancy

In order to quantify redundancy, the capacity of one component of the bridge is compared to the overall capacity of the bridge. For the Coyote Wash Bridge, the flexural capacity of a single AASTHO type VI girder is calculated. Then, the flexural capacity all six girders and the 8" slab was calculated. Detailed calculations are given in Appendix B. The redundancy ratio was determined to be 0.8675, which, according to Table 3-10 results in a rating of 15 (Table 4-1).

4.2.1.1.4 Failure Mode versus Strength Reduction Factor

The resilience indicator for load path is quantified with the strength reduction factor capacity measure using Equation 3-9. For the Coyote Wash Bridge, the load that causes a flexural failure is calculated and compared to the load that causes a shear failure.

Detailed calculations are given in Appendix C. The load causing a flexural failure is 534 kip and the load causing shear failure is 937 kip. This leads to a percent difference between the failure loads and their respective strength reduction factors (0.90 and 0.75) of 0.38. According to Table 3-11, this value results in a rating of 8 as listed above in Table 4-1.

4.2.1.1.5 Soil Erosion versus Scour Protection

To rate Coyote Wash Bridge for scour protection, information from the SI&A report is used. From the report, the scour protection countermeasure described as 'no scour protection' for floor protection or flow control but bank protection is 'dumped rock riprap'. Since the lowest of the three scores is used. According to Table 3-12 a rating of 9 is calculated (Table 4-1).

4.2.1.2 Weight Factors

The following section presents the weight factors for each resilience indicator for the Coyote Wash Bridge.

4.2.1.2.1 Replacement Cost

The replacement cost rating is determined according to Table 3-5 based on the ratio of the size of the component to the size of the entire bridge (Table 4-2).

Table 4-2: Coyote Wash Bridge Replacement Cost Weigh Factor

Resilience Indicator	Replacement Cost (%)	Weight Factor
Pier/Abutment Washout	25+	5
Extreme Temperature	14	3
Load Path	4	1.5
Failure Mode	4	1.5
Soil Erosion	25+	5

Since Coyote Wash Bridge does not have piers, only the abutment sizes are compared to the entire bridge. The abutments are over 25% of the structure. For extreme temperature, the superstructure is considered. Since this bridge is wide, at 75.75 feet, the superstructure is 14% of the bridge. A single girder is used to represent a failure due to load path and failure mode. A girder is 4% of the structure. For soil erosion, the abutments are considered since these are the only components affected by scour.

4.2.1.2.2 Consequence of Event

The weight factors are determined based on Table 3-6, as shown in Table 4-3.

Table 4-3: Coyote Wash Consequence of Event Weight Factor

Resilience Indicator	Consequence of Event	Weight Factor
Pier/Abutment Washout	Out of service	5
Extreme Temperature	N/A	-
Load Path	Requires immediate repair/replacement	3
Failure Mode	Requires immediate repair/replacement	3
Soil Erosion	Triggers deterioration	2

The consequence of event weight factor is determined by identifying the state of the bridge after damage from the respective resilience indicator. For example, if a pier or abutment washed out on Coyote Wash Bridge, the bridge would be out of service. If the bridge were to experience extreme temperature differentials, the change in length would be less than the length of the expansion joints. There would be no consequence from this because there is no worry about inducing stress in the reinforcement. If a single component were to fail on the bridge, the load path would change and immediate repairs and/or replacement would be needed. This would also apply to failure mode if a component were to fail suddenly and without warning. Erosion of the soil would trigger deterioration.

4.2.1.2.3 User Cost

The user cost weight factor is determined according to Equation 3-1. Detailed calculations are provided in Appendix D. The calculation of the user cost assumes the bridge is out of service for each resilience indicator. Several items from the SI&A report are used to calculate a user cost value of 14.95, which results in a rating of 1.5 (Table 3-7) for all resilience indicators. However, because the consequence of pier or abutments washout is the bridge being "out of service", the user cost weight factor goes up to 2 since there is a 1 mile detour. Since extreme temperature would only warrant further testing or investigation, traffic on the bridge would not be affected. The weight factor of 1.5 remains. Damage due to load path and failure mode would result in immediate repairs, which could cause lane closures and delays. This results in a weight factor of 1.5, which is equal to the *S3* rating. Soil erosion would trigger deterioration and would not immediately affect traffic, so the rating of 1.5 from *S3* remains. The user cost weight factors for each resilience indicator are shown in Table 4-4.

Table 4-4: Coyote Wash Bridge User Cost Weigh Factor

Resilience Indicator	**User Cost (*S3*)**	**Weight Factor**
Pier/Abutment Washout	Detour 0-50 miles	2
Extreme Temperature	14.95	1.5
Load Path	14.95 Delays/number of lanes reduced	1.5
Failure Mode	14.95 Delays/number of lanes reduced	1.5
Soil Erosion	14.95	1.5

4.2.1.3 Resilience Indicator Weights

Taking the weight factors from Tables 4-2 to 4-4, the weights for each resilience indicator for the Coyote Wash Bridge are calculated, as shown in Table 4-5.

Table 4-5: Coyote Wash Bridge Weights

Resilience Indicator	**Composite Weight Factor, WF** *($\alpha\times\beta\times\varphi$)*	**Weight, W(x) %** *($WF/\Sigma WF_i$)*
Pier/Abutment Washout	50	60.2
Extreme Temperature	4.5	5.4
Load Path	6.8	8.1
Failure Mode	6.8	8.1
Soil Erosion	15	18.1

4.2.2 Tanner Wash Bridge

The Tanner Wash Bridge is selected for its high sufficiency rating of 77.10 and an anticipated low resilience rating. The following section details and provides explanation how the ratings and weights for the Tanner Wash Bridge are determined.

Figure 4-2: Tanner Wash Bridge

4.2.2.1 Resilience Indicators Ranked by Capacity Measures

The following table shows all five resilience indicators and their rating for each capacity measure.

Table 4-6: Tanner Wash Bridge Resilience Indicator Ratings

Resilience Indicator	**Capacity Measure Type**	**Rating**	**R(x)**
Pier or Abutment Washout	3: Spread on bedrock	6	6/15
Extreme Temperature	Multi span with expansion joint and $R_\delta \leq 1$	1	1/15
Redundancy	0.87	15	15/15
Failure Mode	1.35	8	8/15
Soil Erosion	0: No protection	0	0/15

4.2.2.1.1 Pier or Abutment Washout versus Foundation Type

In order to determine the type of foundation for the Tanner Wash Bridge, the SI&A report was consulted. In the report, foundation type was coded '33', which means that both the piers and abutments have spread footings on bedrock. The rating for the Tanner Wash Bridge foundation type is found in Table 3-8, as shown in Table 4-6.

4.2.2.1.2 Extreme Temperature versus Support Type

The Tanner Wash Bridge is a three span bridge that has a 2-in. expansion joint. The length of the expansion joint will be compared with the change in length caused by an extreme temperature. As seen in Appendix A, the change in length is 2.11 in. and was calculated using Equation 3-3. Since the actual length of the expansion joint is less than the change in length, stress will be induced in the slab. This means the bridge fits into the 'Multi span with expansion joint and $R_{\delta} \leq 1$' category and was rated a 1 in accordance with Table 3-9 and shown in Table 4-6.

4.2.2.1.3 Load Path versus Redundancy

To determine the redundancy ratio of Tanner Wash Bridge, the flexural capacity of a single AASHTO Type III girder is compared to the flexural capacity of all five Type III girders and the 9-in. thick slab. As seen in Appendix B, the flexural capacity of a single girder is 3,300 k-ft and the flexural capacity of the entire bridge is 27,220 k-ft. Given these flexural capacities, the redundancy ration is calculated per Equation 3-8 resulting in a value of 0.867. In accordance with Table 3-10, this ranks the bridge as a 15, as shown in Table 4-6.

4.2.2.1.4 Failure Mode versus Strength Reduction Factor

The failure mode for the Tanner Wash Bridge is quantified based on the percent difference between the ratio of the loads causing flexural and shear failures and the ratio of their respective strength reduction factors. As seen in Appendix C, the load causing flexural failure is 752 kip and the load causing shear failure is 1,982 kip. Using Equation 3-9, the percent difference between the failure modes and strength reduction factors is 0.75. This results in a rating of 8 using Table 3-11, as shown in Table 4-6.

4.2.2.1.5 Soil Erosion versus Scour Protection

The SI&A report is used to determine the type of scour protection for the Tanner Wash Bridge. In the report, the scour protection countermeasure is coded a '000', which means there is no protection for flow control, the waterway floor, or the bank. The resulting rating is a 0, as found in Table 3-12 and shown in Table 4-6.

4.2.2.2 Weight Factors

The following section presents the weight factors for each resilience indicator for the Tanner Wash Bridge.

4.2.2.2.1 Replacement Cost

The replacement cost weight factor is calculated by comparing the volume of a single component of the bridge to the entire structure. The resilience indicators are rated using Table 3-5, as shown in Table 4-7.

Table 4-7: Tanner Wash Bridge Replacement Cost Weight Factor

Resilience Indicator	Replacement Cost (%)	Weight Factor
Pier/Abutment Washout	25+	5
Extreme Temperature	16	4
Load Path	5	1.5
Failure Mode	5	1.5
Soil Erosion	25+	5

As seen in Appendix E, the superstructure accounts for 16.3%, the piers account for 5.7% each, the barriers account for 5.3%, the abutments account for 20.1% for each side, and the girders account for 5.3% each of the structure. When determining the weight factor for pier or abutment washout, the higher of the percentages for a single abutment or pier is used. If the bridge were to experience an extreme temperature differential, the superstructure alone would be affected. If there were a failure due to load path or an unexpected sudden failure, a single girder is most likely to fail. The rating for soil erosion also uses the higher of the two percentages for a single pier or abutment.

4.2.2.2.2 Consequence of Event

The consequence of event weight factor describes the state of the bridge's functionality after a complete failure due to the specified resilience indicator. The resilience indicators were rated based on Table 3-6, as shown in Table 4-8.

Table 4-8: Tanner Wash Bridge Consequence of Event Weight Factor

Resilience Indicator	Consequence of Event	Weight Factor
Pier/Abutment Washout	Requires immediate repair/replacement	3
Extreme Temperature	Warrants testing or investigation	1.5
Load Path	Requires immediate repair/replacement	3
Failure Mode	Requires immediate repair/replacement	3
Soil Erosion	Triggers deterioration	2

Since the bridge has two piers and abutments, the washout of one would not destroy the entire structure. However, a washout would require immediate repairs or replacement in order to regain functionality. If the bridge were to experience an extreme temperature differential and the deformation produced stresses, an investigation would need to be performed to determine if stresses would be induced in the reinforcement that could lead to yielding. Due to the bridges multiple girders, load path and failure mode were analyzed as if one were to fail. In both cases, it would require immediate repairs or replacement. If the bridge were to experience scour due to soil erosion, the piers and abutments would experience deterioration.

4.2.2.2.3 User Cost

The user cost weight factor, *S3*, is calculated using Equation 3-1. *S3* was determined to be 4.70 and is weighted according to Table 3-7 (as shown in Table 4-9).

Table 4-9: Tanner Wash Bridge User Cost Weight Factor

Resilience Indicator	User Cost	Weight Factor
Pier/Abutment Washout	4.70 Detour over 98 miles	3
Extreme Temperature	4.70	3
Load Path	4.70 Detour over 98 miles	3
Failure Mode	4.70 Detour over 98 miles	3
Soil Erosion	4.70	3

Several items from the SI&A report are used in this calculation. Due to its large detour length, the Tanner Wash Bridge is very costly to users and reduces its resilience. The user cost, *S3*, of 4.70 results in a rating of 3 for the entire bridge. Even though pier/abutment washout, load path, and failure mode have a consequence of event of immediate repairs or replacement, the detour length of 99 miles gives it the same rating as *S3*.

4.2.2.3 Resilience Indicator Weights

After the replacement cost, consequence of event, and user costs were rated, the composite weight factors (WF) are calculated. Multiplying the values from Tables 4-7 through 4-9 together for each respective capacity measure did this. The weight, W(x), is determined using the ratio of each weight factors and the sum of all the weight factors (Equation 3-3). The weights for each resilience indicator for Tanner Wash Bridge are shown in Table 4-10.

Table 4-10: Tanner Wash Bridge Weights

Resilience Indicator	**Composite Weight Factor, WF** *($\alpha \times \beta \times \varphi$)*	**Weight, W(x) %** *($WF/\Sigma WF_i$)*
Pier/Abutment Washout	45	37.5
Extreme Temperature	18	15
Load Path	13.5	11.25
Failure Mode	13.5	11.25
Soil Erosion	30	25

4.2.3 Earp Wash Bridge

The Earp Wash Bridge is selected for its low sufficiency rating of 49.5 and an anticipated high resilience rating. The following section details and provides a detailed explanation why the ratings and weights for the Earp Wash Bridge are determined.

Figure 4-3: Earp Wash Bridge

4.2.3.1 Resilience Indicators Ranked by Capacity Measures

The following table shows all five resilience indicators and their rating for each capacity measure.

Table 4-11: Earp Wash Bridge Resilience Indicator Rating

Resilience Indicator	**Capacity Measure Type**	**Rating**	**R(x)**
Pier or Abutment Washout	4: Steel H piles	8	8/15
Extreme Temperature	Multi span with integral abutments/pier and $R_\sigma \leq 1$	3	3/15
Redundancy	0.98	15	15/15
Failure Mode	-0.09	6	6/15
Soil Erosion	0: No protection	0	0/15

4.2.3.1.1 Pier or Abutment Washout versus Foundation Type

The SI&A report is used to determine the type of foundations for the piers and abutments of Earp Wash Bridge. In the report, the foundation type was coded '44', meaning that both the abutments and the piers have steel H pile foundations. The as-built construction drawings confirmed that the piles are steel H piles, with five at each abutment and six at each pier. The rating for foundation type is determined as detailed in Table 3-8 and is shown in table 4-11.

4.2.3.1.2 Extreme Temperature versus Support Type

Earp Wash Bridge is a four span bridge with integral abutments and piers. The bridge does not have expansion joints. The total length of the bridge is 92 feet, which results in a stress induced by extreme temperature change of 2810 psi. The bridge was built in 1958, so the compressive strength of the concrete is only 3000 psi according to the as-built construction plans. This means that the stresses induced by an extreme change in temperature are higher than 50% of the compressive strength. The rating for the support type is given in Table 3-9 and shown in Table 4-11.

4.2.3.1.3 Load Path versus Redundancy

In order to quantify redundancy, a ratio between the difference between the capacity of a single member and the entire structure over the capacity of the entire structure is used. Since the Earp Wash Bridge is a slab bridge, this is determined by comparing the flexural capacity of a 12-in. wide section of the 10.5-in. thick slab to the flexural capacity of the entire 40.7-ft. wide structure. As shown in Appendix B, the flexural capacity of a 12-in. wide section of the bridge was found to be 294 k-in and the entire bridge to be 11,940 k-in. Thus, according to Equation 3-8, the redundancy ratio is 0.975, resulting in a rating of 15 in accordance with Table 3-10 and shown in Table 4-11.

4.2.3.1.4 Failure Mode versus Strength Reduction Factor

The Earp Wash Bridge is a slab bridge with transverse and longitudinal reinforcement. However, there are no stirrups in the slab to resist shear. Therefore, the only component of the bridge that is able to resist shear is the concrete slab. As shown in Appendix C, the load causing shear failure of the bridge is 4.3 kip and the load causing flexural failure is 3.9 kip. Using Equation 3-9, this results in a percent difference between failure loads and strength reduction factors of -0.09. The resulting rating is a 6 using Table 3-11 and shown in Table 4-11.

4.2.3.1.5 Soil Erosion versus Scour Protection

To determine the type of scour protection countermeasures used on the Earp Wash Bridge, the SI&A report is consulted. In the report, scour countermeasure is coded '003'. This means there is no scour protection for flow control or the waterway floor, but there is a rail bank for the bank protection.   Although there is bank protection provided, the lowest of the three digits in the code controls. Therefore, the scour protection is rated a 0 based on Table 3-12 and shown in Table 4-11.

4.2.3.2 Weight Factors

The following section presents the weight factors for each resilience indicator for the Earp Wash Bridge.

4.2.3.2.1 Replacement Cost

The replacement cost weight factor is determined by comparing the volume of each component to the entire volume of the bridge. All of the replacement costs weight factors were determined based on Table 3-5 and shown in Table 4-12.

Table 4-12: Earp Wash Bridge Replacement Cost Weight Factor

Resilience Indicator	Replacement Cost (%)	Weight Factor
Pier/Abutment Washout	17	4
Extreme Temperature	25+	5
Load Path	25+	5
Failure Mode	25+	5
Soil Erosion	17	4

As seen in Appendix E, the superstructure accounts for 68%, the piers account for 5.3% each, the barriers account for 11.36% and the abutments account for 5.6% each of the entire structure. When rating the pier or abutment washout, the replacement cost for either a single abutment or pier is used. The controlling replacement cost is an abutment, at 5.6%. If the bridge were to experience failure due to extreme temperature, the superstructure with a replacement cost of 68% would most likely be the cause of failure. If there were failure due to a change in load path or an unexpected sudden failure, the superstructure would again be the likely source of failure. The replacement cost of either a single abutment or pier is used to rate the soil erosion.

4.2.3.2.2 Consequence of Event

The level of functionality of the Earp Wash Bridge is used codetermine the consequence of event weight factor, which was determined in accordance with Table 3-6 and as summarized in Table 4-13.

Table 4-13: Earp Wash Bridge Consequence of Event Weight Factor

Resilience Indicator	Consequence of Event	Weight Factor
Pier/Abutment Washout	Requires immediate repair/replacement	3
Extreme Temperature	Warrants testing or investigation	1.5
Load Path	Requires immediate repair/replacement	3
Failure Mode	Out of service	5
Soil Erosion	Requires immediate repair/replacement	3

If the bridge were to experience the washout of a single abutment, immediate repairs or replacement of the washed out component would be required to regain full functionality. If the bridge were subjected to an extreme temperature differential, stresses would be induced in the reinforcement steel. This would result in the need for further testing or investigation to determine whether or not the newly induced stresses significantly affect the integrity of the bridge. If there was a change in load path in the bridge, immediate repairs would be required. If sudden, unexpected failure were to occur, the deck would most likely be the source of failure and the bridge would be out of service. If the bridge were to experience soil erosion, a single pier or abutment would be the source of failure. This would result in the need for immediate repairs or replacements in order to regain full functionality.

4.2.3.2.3 User Cost

To determine the user cost weight factor, *S3*, Equation 3-1 is used. The user cost weight factor is determined in accordance with Table 3-7 and the ratings are shown in Table 4-14.

Table 4-14: Earp Wash Bridge User Cost Weight Factor

Resilience Indicator	User Cost	Weight Factor
Pier/Abutment Washout	9.95 Temporary closure and detour 0-50 miles	2
Extreme Temperature	9.95	2
Load Path	9.95 Temporary closure and detour 0-50 miles	2
Failure Mode	Out of service	5
Soil Erosion	9.95 Temporary closure and detour 0-50 miles	2

Earp Wash Bridge has a condition rating of '4' for the superstructure, which means it is in poor condition. This resulted in a low score for structural adequacy and safety. The bridge also scored low in serviceability and functional obsolescence. These two attributes make the bridge costly for users. Although the bridge has a very large ADT of 34,500, the detour length is 1 mile. The high ADT did not have a large effect on the user cost. The user cost, *S3*, of 9.95 gives the entire bridge a rating of 2. Although pier/abutment washout, load path, and soil erosion would all results in repairs or replacements, the rating did not change because it is the same as the *S3* rating for the entire bridge. However, since the consequence of event for failure mode is out of service, the user cost is also out of service, giving it a rating of 5.

4.2.3.3 Resilience Indicator Weights

After the replacement cost, consequence of event, and user cost weight factors were determined, the composite weight factors (WF) were found. Multiplying the values from Tables 4-12 through 4-14 together for each respective capacity measure did this. The weight, W(x), is calculated using the ratio of each weight factors and the sum of all the weight factors (Equation 3-3). The weights for each resilience indicator for Earp Wash Bridge are given in Table 4-15.

Table 4-15: Earp Wash Bridge Weights

Resilience Indicator	**Composite Weight Factor, WF** *($\alpha \times \beta \times \varphi$)*	**Weight, W(x) %** *($WF/\Sigma WF_i$)*
Pier/Abutment Washout	24	11
Extreme Temperature	15	6.9
Load Path	30	13.8
Failure Mode	125	57.3
Soil Erosion	24	11

4.2.4 Midgley Bridge

The Midgley Bridge is selected for its low sufficiency rating of 50.7 and an anticipated low resilience rating. The following section details and provides explanation how the ratings and weights for the Coyote Wash Bridge are determined.

Figure 4-4: Midgley Bridge

4.2.4.1 Resilience Indicators Ranked by Capacity Measures

The following table shows all five resilience indicators and their rating for each capacity measure.

Table 4-16: Midgley Bridge Resilience Indicator Ratings

Resilience Indicator	**Capacity Measure Type**	**Rating**	**R(x)**
Pier or Abutment Washout	3: Spread on bedrock	6	6/15
Extreme Temperature	Multi span with expansion joints and $R_\delta > 1$	9	9/15
Redundancy	Fracture critical structure	1	1/15
Failure Mode	1.50	8	8/15
Soil Erosion	N/A	-	-/15

4.2.4.1.1 Pier or Abutment Washout versus Foundation Type

The SI&A report is consulted to determine the type of foundation on Midgley Bridge. The foundation type is coded as '33' in the report, which means the foundations for both the piers and abutments are spread footings on bedrock. According to Table 3-8, this results in a rating of 6 as presented in Table 4-16.

4.2.4.1.2 Extreme Temperature versus Support Type

Seeing as Midgley Bridge is a four span bridge with expansion joints in the deck, the support type rating is determined by comparing the required expansion joint length for a temperature differential of 125° F to the actual width of the expansion joint (Equation 3-4). The length between expansion joints is 240 ft., which results in a required expansion joint width of 2.59 in. As seen in Appendix A, the actual width of the expansion joint is larger than required. Based on Table 3-9, the support type of 'Multi span with expansion joints and $R_\delta > 1$' results in a rating of 9, as summarized in Table 4-16.

4.2.4.1.3 Load Path versus Redundancy

Since the Midgley Bridge is designated as fracture critical in the SI&A report, no capacity calculations are needed. From Table 3-10, the redundancy rating for a fracture critical structure is 1, as summarized above in Table 4-16.

4.2.4.1.4 Failure Mode versus Strength Reduction Factor

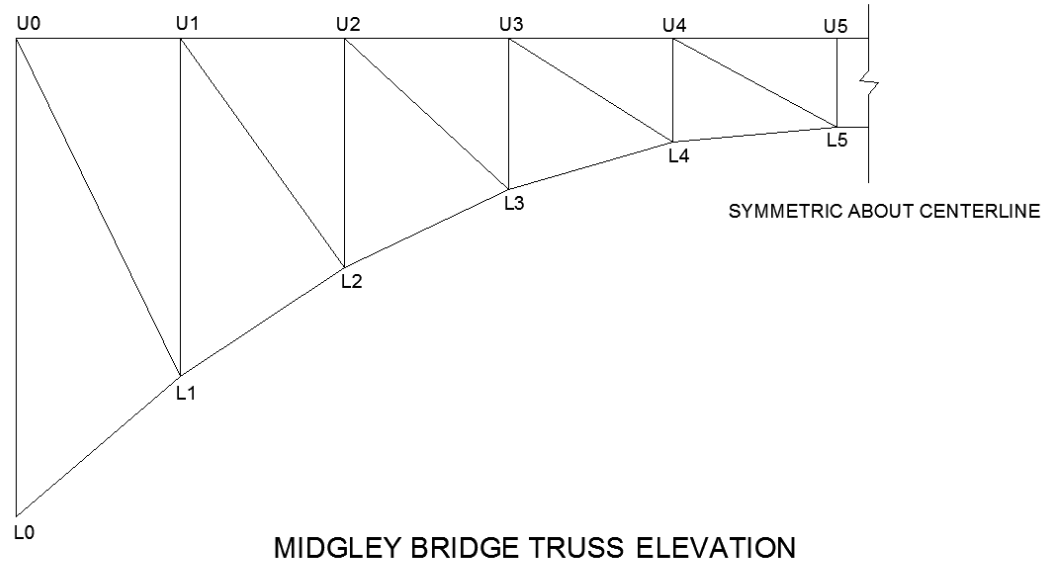

Figure 4-5: Truss Layout of Midgley Bridge

By comparing the gross area of cross sections provided versus required for each member, it was found that L1-L2 and L3-L4 are the highest stressed members in the truss. The bearing strength and block shear are analyzed for each connection in the two highest stressed members, as seen in Appendix F. Comparing the capacities for the two highest stressed members, the connections in member L3-L4 controlled. As shown in Appendix G, the load causing a failure in bearing strength is 46 kips and the load causing a block shear failure is 325 kips. This results in a percent difference in the strength reduction factors of 1.5, using Equation 3-9. The resulting rating is an 8 using Table 3-11 and shown in Table 4-16.

4.2.4.1.5 Soil Erosion versus Scour Protection

As seen in Figure 3-9, Midgley Bridge is over a canyon and the foundations sit well above the waterway below. There are no scour protection countermeasures because the bridge is not subjected to scour. Therefore, the scour protection capacity measure does not apply.

4.2.4.2 Weight Factors

The following section presents the weight factors for each resilience indicator for Midgley Bridge.

4.2.4.2.1 Replacement Cost

The replacement cost weight factors for Midgley Bridge were found by comparing the weight of each component to the weight of the entire bridge. The weight is compared instead of the volume due to the age of the construction drawings and the information it provides. These percentages are then rated based on the information given in Table 3-5, as shown in Table 4-17.

Table 4-17: Midgley Bridge Replacement Cost Weight Factor

Resilience Indicator	Replacement Cost (%)	Weight Factor
Pier/Abutment Washout	16	4
Extreme Temperature	16	4
Load Path	20	4
Failure Mode	20	4
Soil Erosion	N/A	-

As seen in Appendix E, one pier or abutment accounts for 16% of the structure. If the bridge were to experience a failure due to extreme temperature, the deck would be affected. This also accounts for 16% of the total structure. Failure due to load path would come from a failure in the truss, which is 20% of the structure. Failure from failure mode is also seen in the truss. Since the foundations of the bridge are well above the waterway, the bridge is not susceptible to scour. Soil erosion is not applicable.

4.2.4.2.2 Consequence of Event

The consequence of event weight factors are determined in accordance with Table 3-6 as shown below in Table 4-18.

Table 4-18: Midgley Bridge Consequence of Event Weight Factor

Resilience Indicator	Consequence of Event	Weight Factor
Pier/Abutment Washout	Requires immediate repair or replacement	3
Extreme Temperature	Warrants testing or investigation	1.5
Load Path	Out of service	5
Failure Mode	Out of service	5
Soil Erosion	N/A	-

Although the bridge is not susceptible to loss of a pier or an abutment due to scour, there is a loss potential to falling boulders. As seen in the Bridge Inspection Photographs, part of the sway frame is currently damaged from a boulder impact. If the bridge were to experience a loss of a pier or abutment, immediate repairs or replacement would be required. Since the expansion joints are larger than the change in length resulting from an extreme change in temperature, the consequence would at most require testing or investigation. If Midgley Bridge were to experience the loss of a member that resulted in a change in load path, the entire bridge could fail due to the fracture critical and non-redundant nature of the bridge. This would cause the bridge to be out of service. This is also true for a failure that occurs with little to no warning. If a member were lost due to a sudden, unexpected failure, the entire bridge could fail due to non-redundancy. This would cause the bridge to be out of service. Because the bridge is not susceptible to scour, the soil erosion resilience indicator is not applicable.

4.2.4.2.3 User Cost

The user cost weight factor, *S3*, was calculated using Equation 3-1. The results of this analysis are given in Table 4-19.

Table 4-19: Midgley Bridge User Cost Weight Factor

Resilience Indicator	User Cost	Weight Factor
Pier/Abutment Washout	6.90	2.5
Extreme Temperature	6.90	2.5
Load Path	Out of service	5
Failure Mode	Out of service	5
Soil Erosion	N/A	-

Several items from the SI&A report are used in this calculation. Although Midgley Bridge does not have a high ADT compared to Earp Wash Bridge, an average of 3858 vehicles per day is significant. This along with a detour length of 38 miles makes Midgley Bridge costly to users. The bridge also scored low on its serviceability and functional obsolescence. With a coding of '5' for structural evaluation, the bridge's load rating is slightly above the limit indicating obsolescence [US Department of Transportation Federal Highway Administration, 1995]. A coding of '2' for deck geometry means the size and number of lanes on the bridge are intolerable and have a high priority for corrective action [US Department of Transportation Federal Highway Administration, 1995]. These factors contribute to relatively low user cost of 6.90. However, the consequence of load path and failure mode would render the bridge out of service. Therefore, the user cost for these two resilience indicators results in a rating of '5'.

4.2.4.3 Resilience Indicator Weights

After the replacement cost, consequence of event, and user cost weight factors were determined, the composite weight factors (WF) were found. Multiplying the values from Tables 4-17 through 4-19 together for each respective capacity measure did this. The weight, W(x), is calculated using the ratio of each weight factors and the sum of all the weight factors (Equation 3-3). The weights for each resilience indicator for Midgley Bridge are summarized in Table 4-20.

Table 4-20: Midgley Bridge Weights

Resilience Indicator	**Composite Weight Factor, WF** *($\alpha \times \beta \times \varphi$)*	**Weight, W(x) %** *($WF/\Sigma WF_i$)*
Pier/Abutment Washout	30	12.2
Extreme Temperature	15	6.1
Load Path	100	40.8
Failure Mode	100	40.8
Soil Erosion	N/A	-

4.3 Resilience Rating Calculation

4.3.1 Coyote Wash Bridge

The resilience rating for the Coyote Wash Bridge is determined using the Indicators and User Cost ratings (Table 4-1) and Weights (Table 4-5). As seen in Table 4-21, the resilience rating for Coyote Wash Bridge is 77.4.

Table 4-21: Coyote Wash Bridge Resilience Rating

Resilience Indicator	Weight, W(x)	Rating, R(x)	W(x)×R(x)
Pier/Abutment Washout	58.1	15/15	58.1
Extreme Temperature	8.7	13/15	7.5
Load Path	7.9	15/15	7.9
Failure Mode	7.9	8/15	4.2
Soil Erosion	17.4	0/15	0.0
		Total (RR)=	77.4

4.3.2 Tanner Wash Bridge

The resilience rating for the Tanner Wash Bridge is determined using the Indicators and User Cost ratings (Table 4-6) and Weights (Table 4-10). As seen in Table 4-22, the resilience rating for Tanner Wash Bridge is 33.3.

Table 4-22: Tanner Wash Bridge Resilience Rating

Resilience Indicator	Weight, W(x)	Rating, R(x)	W(x) ×R(x)
Pier/Abutment Washout	32.9	6/15	13.1
Extreme Temperature	20.6	1/15	1.4
Load Path	12.3	15/15	12.3
Failure Mode	12.3	8/15	6.6
Soil Erosion	21.9	0/15	0.00
		Total (RR)=	33.3

4.3.3 Earp Wash Bridge

The resilience rating for the Earp Wash Bridge is determined using the Indicators and User Cost ratings (Table 4-11) and Weights (Table 4-15). As seen in Table 4-23, the resilience rating for Earp Wash Bridge is 43.9.

Table 4-23: Earp Wash Bridge Resilience Rating

Resilience Indicator	Weight, W(x)	Rating, R(x)	W(x) ×R(x)
Pier/Abutment Washout	6.2	8/15	3.3
Extreme Temperature	7.7	5/15	2.6
Load Path	15.5	5/15	15.5
Failure Mode	64.4	6/15	25.8
Soil Erosion	6.2	0/15	0.0
		Total (RR)=	43.9

4.3.4 Midgley Bridge

The resilience rating for the Midgley Bridge is determined using the Indicators and User Cost ratings (Table 4-16) and Weights (Table 4-20). As seen in Table 4-24, the resilience rating for Midgley Bridge is 31.7.

Table 4-24: Midgley Bridge Resilience Rating

Resilience Indicator	Weight, W(x)	Rating, R(x)	W(x) ×R(x)
Pier/Abutment Washout	7.7	6/15	3.1
Extreme Temperature	6.4	9/15	3.9
Load Path	42.9	1/15	2.9
Failure Mode	42.9	8/15	21.5
Soil Erosion	N/A	N/A	-
		Total (RR)=	31.7

5 Discussion of Results

5.1 Overview

The four bridges in this study were chosen to fit into categories of high and low sufficiency rating and high and low expected resilience ratings. The following discusses the resilience and sufficiency ratings, compares and contrasts the bridges, and a susceptibility analysis.

5.2 Comparing Resilience and Sufficiency Rating

Table 5-1 presents sufficiency and resilience ratings for the four bridges.

Table 5-1: Sufficiency and Resilience Ratings

Bridge	Sufficiency Rating	Resilience Rating
Coyote Wash Bridge	99.90	77.4
Tanner Wash Bridge	77.10	33.3
Earp Wash Bridge	49.50	43.9
Midgley Bridge	50.70	31.7

Figure 5-1 displays the sufficiency and resilience ratings for the four bridges in a bar graph. It is seen that there is no correlation between the sufficiency and resilience ratings. The line shows the percent difference between the sufficiency and resilience ratings.

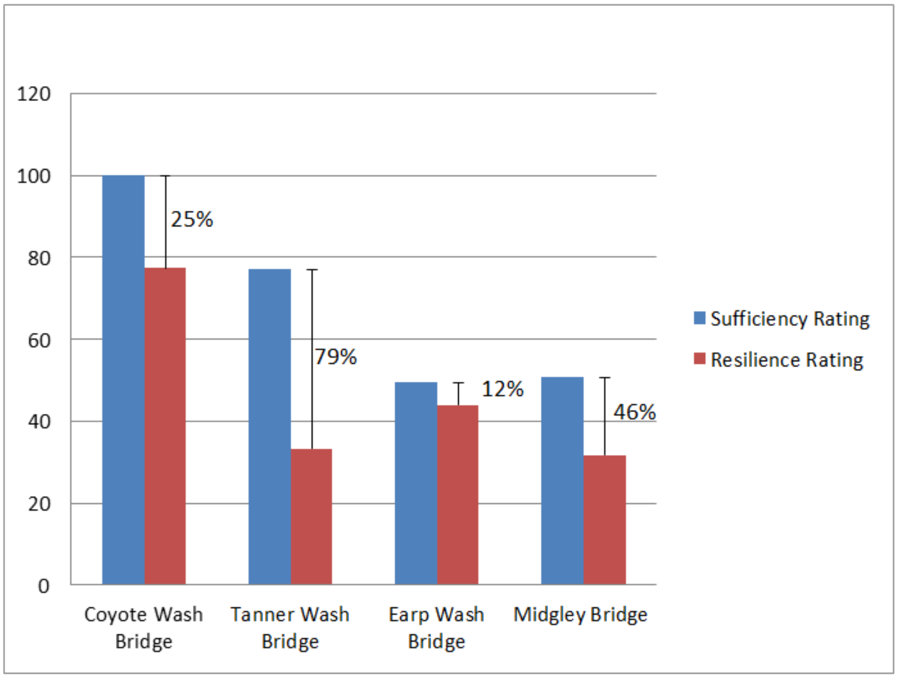

Figure 5-1: Bar Graph of Resilience and Sufficiency Ratings with Percent Differences

5.2.1 Coyote Wash Bridge

Coyote Wash Bridge has a sufficiency rating of 99.9 and was expected to have a high resilience rating, as well. The actual resilience rating of 77.4 is high and still fits into the category of high sufficiency and high resilience.

The high resilience came from several items. The bridge has drilled shaft foundations, making it very resilient in the case of an extreme flood event. Coyote Wash Bridge is also resilient to extreme temperature because the width of the expansion joints present on the structure are greater than the deformation caused by an extreme temperature differential. Having six girders and a concrete slab makes the bridge redundant. This redundancy makes the bridge resilient because loads can be redistributed with the loss of one girder. The percent difference between the nominal shear and moment capacities and their respective strength reduction factors was high, meaning the bridge is not as susceptible to a sudden unexpected failure that occurs with little to no warning. The bridge was built in 2012, so all of the condition ratings for the bridge are very high. The ADT is only 1000 and the detour length is only 1 mile. These factors all resulted in a high score for user cost.

A few factors decreased the resilience rating for Coyote Wash Bridge. The bridge has scour protection for the bank, but not for the waterway floor or flow control. Although the scour protection countermeasure for the bank is resilient, the lowest of the three item's scores is used. The bridge received a rating of 0 for soil erosion. Because the bridge has no piers, the loss of an abutment would likely result in a collapse of the bridge, making it less resilient. This caused the consequence of event for the bridge to be high. The varying ratings and weights showed a high sufficiency led to a high resilience in this case.

5.2.2 Tanner Wash Bridge

Tanner Wash Bridge has a sufficiency rating of 77.10 and was expected to have a low resilience rating. The actual resilience rating of 33.3 shows that Tanner Wash Bridge accurately fits into the category of high sufficiency and low resilience rating.

Several items led to the low resilience rating of Tanner Wash Bridge. The foundations of the abutments and piers are spread on bedrock. Although it is beneficial to have a foundation on bedrock, the lack of piles or drilled shafts makes it less resilient. The bridge also has no scour protection countermeasures for the flow control, waterway floor, or bank. This resulted in a rating of 0/15 for soil erosion, making it non-resilient in this category. The detour length of Tanner Wash Bridge is at least 99 miles as seen in the SI&A report, making the user cost for the bridge low and subsequently lowering the resilience.

Some factors helped increase the resilience rating of the bridge. The bridge is redundant, having five girders and a concrete slab. This makes the bridge more resilient because the load can be redistributed in the case of the loss of a single girder. The ratio between the flexural and shear failure modes and their accompanying strength reduction factors is also high. This means that the bridge will most likely experience a flexural failure, when warning signs will precede failure. The high ratio makes the bridge more resilient. In this case, the varying ratings and weights show that a high sufficiency rating is not indicative of a high resilience rating.

5.2.3 Earp Wash Bridge

Earp Wash Bridge has a sufficiency rating of 49.50 and was expected to have a high resilience rating. The actual resilience rating is 43.9, meaning that the bridge does not fit into the expected category of low sufficiency and high resilience rating. However, the resilience rating is higher than those for Tanner Wash and Midgley Bridges.

Several factors led to the low resilience rating of Earp Wash Bridge. The bridge is not resilient to extreme temperature. The four-span bridge is integral with its supports and the extreme temperature change induces a stress of 2810 psi. However, due to the bridge's age, the compressive strength of the concrete is only 3000 psi. Thus, the stresses induced from an extreme change in temperature are more than 50% of the compressive strength. In addition, the bridge has no scour protection for the flow control or waterway floor. The controlling scour protection countermeasure rated 0, making the bridge non-resilient to scour. The load rating for the bridge is not up to the HL-93 standards and had an ADT of 34,500. This makes the user cost rating low.

The bridge demonstrates resilient characteristics. The foundations are steel H piles and are driven to a depth of approximately 30 feet into the ground. Thus, the piers and abutments are not susceptible to washout. In addition, the structure is redundant. Loads can be redistributed in the case of a loss of a section of slab reinforcement, making it more resilient to a change in load path. The strength reduction factor ratio is also high, which means the bridge is not likely to experience a sudden, unexpected failure due to shear. In this case, it is seen that a low sufficiency rating is correlated with a low resilience rating.

5.2.4 Midgley Bridge

Midgley Bridge has a sufficiency rating of 50.70 and was expected to have a low resilience rating. The actual resilience rating is 31.7, meaning this bridge did meet the expectation of having a low resilience rating.

The biggest factor leading to the low resilience rating for Midgley Bridge is its designation as a fracture critical structure. The non-redundancy of the structure means that the loss of a fracture critical member could result in a partial or complete loss of the entire structure. This gave the bridge a rating of '1' for load path, which had a 40.8% weight on the resilience rating. The fracture critical designation also means that failure through load path or failure mode could cause a collapse of the entire structure. The consequence of these two resilience indicators would render the bridge out of service, making it costly to the user. In addition, the bridge had a low rating for the foundation type due to the spread footing foundation. Although the bridge is not susceptible to scour, it is susceptible to falling boulders and seismic activity. The user cost rated low due to the detour length of 38 miles and the ADT of 3858.

The highest rating Midgley Bridge received was for the extreme temperature resilience indicator. The bridge has two expansion joints that are each 2 in. long. The change in length of the structure due to an extreme temperature differential is 2.59 in. This means that if the bridge were to experience an extreme change in temperature, the expansion joints would be able to accommodate the change in length without inducing any additional stress in the structure.

5.3 Comparison

5.3.1 Weight Factors

The weights for all four bridges are compared in this section and Table 5-2 presents the weights for the capacity measures for each bridge.

Table 5-2: Weights (%) for All Bridges

Bridge	Foundation Type	Support Type	Redundancy	Strength Reduction Factors	Scour Protection
Coyote Wash Bridge	60.2	5.4	8.1	8.1	18.1
Tanner Wash Bridge	37.5	15	11.3	11.3	25
Earp Wash Bridge	11	6.9	13.8	57.3	11
Midgley Bridge	12.2	6.1	40.8	40.8	-

It is seen for Tanner and Coyote Wash bridges, the lowest weights were for redundancy (load path) and strength reduction factor (failure mode). This is likely because both these two bridges have AASHTO type prestressed concrete girders and a concrete deck. The highest weight for both the Tanner and Coyote Wash bridges is for foundation type (pier or abutment washout). However, the weight is much higher for Coyote Wash Bridge at almost a 50% weight. This is because Coyote Wash Bridge does not have piers, while Tanner Wash Bridge does. Thus, the loss of an abutment on Coyote Wash Bridge would result in a complete failure.

In contrast, the highest weight for Earp Wash Bridge and Midgley Bridge is redundancy (failure mode), which was the lowest for both Tanner and Coyote Wash

Bridges. This high weight factor for Earp Wash Bridge is because of a high rating for consequence of event and replacement cost, which both received a 5. Thus, damage to the deck, which is the entire superstructure, would result in a sudden failure and require a significant replacement cost. Similarly, if the deck were to fail with little to no warning, the bridge would be out of service. The lowest weights for Earp Wash Bridge are pier and abutment washout and soil erosion. These are low because the bridge has piers and the failure of one would redirect load to the other piers and abutments.

The high weight for redundancy in Midgley Bridge is because a loss of a fracture critical member could result in a complete failure of the bridge. The lowest weights for Midgley Bridge were foundation type and support type. The foundation type had a low weight because it is not subjected to scour, making the replacement cost, consequence, and user cost very low. The low weight for support type is because the deck allows for thermal expansion without adding stress into the bridge.

5.3.2 Capacity Measures

The following section compares the capacity measure ratings for each bridge. Table 5-3 displays the ratings for all four bridges.

Table 5-3: Capacity Measure Ratings for All Bridges

Capacity Measure	Coyote Wash Bridge	Tanner Wash Bridge	Earp Wash Bridge	Midgley Bridge
Foundation Type	15/15	6/15	8/15	6/15
Support Type	13/15	1/15	3/15	9/15
Redundancy	15/15	15/15	15/15	1/15
Strength Reduction Factor	8/15	8/15	6/15	8/15
Scour Protection	0/15	0/15	0/15	N/A

As shown in Table 5-3, all bridges susceptible to scour received a rating of 0 for scour protection. Even though Coyote and Earp Wash bridges had scour protection countermeasures for the bank, there were none for flow control or the waterway floor. This caused these two bridges to receive the same score as Tanner Wash Bridge, which had no scour protection countermeasures.

Coyote Wash, Tanner Wash, and Midgley Bridges also all received a rating of 8 for strength reduction factor. The bridges had a strength reduction factor rating between -0.09 and 1.50 (Appendix C), which means the load causing a ductile failure is likely to occur before the load causing a brittle failure.

Coyote Wash Bridge has more capacity measure values of 15 than any of the other bridges. This is because it is founded on drilled shaft foundations and a high level of redundancy. Tanner and Earp Wash Bridges also received a rating of 15 for load path due to their relatively high redundancies. Other than these, no other capacity measures were rated as high as 15.

Midgley Bridge scored the lowest for redundancy with a rating of 1. Other than the ratings of 0 for scour protection, this is the lowest rating for any bridge's capacity measure.

The reason for the low rating is because of the lack of redundant load paths. In other words, since Midgley Bridge is a fracture critical structure, the loss of one member will likely result in the collapse of the entire structure.

Midgley and Tanner Wash Bridges both received a rating of 6 for the foundation type of spread footing on bedrock. Spread footings receive a low score relative to drilled shaft or pile foundations. Drilled shaft and pile foundations are more resilient to the effects of an unforeseen event since they are driven several feet into the ground. Earp Wash Bridge received a rating of 8 for foundation type since the bridge is founded on steel piles, which are more resilient than spread footings.

Earp Wash Bridge received a rating of 3 for support type. This is a result of the low compressive strength of concrete used in the bridge. Due to the bridge's age, the compressive strength of the concrete is only 3000 psi. The stress induced by a temperature differential of 125° F would exceed 50% of the compressive strength. Tanner Wash Bridge received a rating of 1 for support type because the ration between the actual expansion joint width and the deformation due to temperature change is less than one. This means if the bridge were subjected to an extreme change in temperature, stresses would be induced once the expansion joint length was filled.

5.3.3 Resilience Ratings

The following section compares the resilience ratings for all four bridges, as shown in Table 5-4.

Table 5-4: Resilience Ratings for All Bridges

Resilience Indicator	Coyote Wash Bridge	Tanner Wash Bridge	Earp Wash Bridge	Midgley Bridge
Pier/Abutment Washout	60.24	15	5.9	4.9
Extreme Temperature	4.7	1	1.4	3.7
Load Path	8.1	11.3	13.8	2.7
Failure Mode	4.3	6	22.9	20.4
Soil Erosion	0.00	0.00	0.00	-
Total (%)= Resilience Rating (RR)	**77.4**	**33.3**	**43.9**	**31.7**

It is logical that the highest resilience rating was for Coyote Wash Bridge. This bridge has drilled shaft foundations, redundancy, and the ability to accommodate a change in length due to an extreme temperature differential. Being built in 2012, this bridge is also in very good condition. It was built to current design and loading standards. It does not come as a surprise that this bridge had the highest resilience rating of the four bridges.

Although Earp Wash Bridge was anticipated to have a high resilience rating, a few factors logically bring the resilience down. The bridge had a high weight for failure mode because the bridge would be rendered out of service. Even though the bridge has pile foundations, they receive an average rating. The weight for pier or abutment washout was also fairly low. This bridge was also built in the 1950's and not designed to the current loading standards. The compressive strength of the concrete is also fairly low to what is typically used in new construction. It is reasonable that the bridge received such a low resilience rating.

It was anticipated to see a low resilience rating for Tanner Wash Bridge and it is rational. The bridge has only a spread footing, which is less resilient than other types of foundations typically used in bridges. The pier or abutment washout also had a high weight because it could render the bridge out of service, reasonably bringing the resilience rating down. The large detour length also means that there is a high cost to the user if the bridge were rendered out of service, so it is logical that this factor would decrease the resilience rating.

It is understandable that Midgley Bridge had the lowest resilience rating than any other bridge in the study. The bridge is classified as fracture critical, which negatively affects the bridge's resilience. This is logical because a non-redundant structure can experience a partial or complete failure from the failure of one component. This bridge was also built in the 1940's and not up to current loading standards. It is clear that these factors would decrease the resilience rating for Midgley Bridge.

5.4 Susceptibility Analysis

5.4.1 Overview

An analysis is performed to determine the susceptibility of the resilience rating to bridge improvements or deteriorations. It is expected that improvements would increase and deterioration would decrease the resilience ratings. The following presents section several hypothetical scenarios, which demonstrate how receptive the resilience rating is to change.

5.4.2 Changes Resulting in a Lower Resilience Rating

The Coyote Wash Bridge had the highest resilience rating of any other bridge in the study. Suppose, over time, the bridge underwent deterioration without repair. The structure would remain the same, so the foundation type, support type, redundancy, and strength reduction factor ratings would not change. However, this could cause the bridge to become scour critical and subsequent condition ratings to decrease. If the bridge were to become scour critical, the rating for scour protection would remain a 0/15. However, this would change the consequence of event for soil erosion. If the bridge were to experience soil erosion in addition to being scour critical, loss of an abutment could happen, causing the bridge to be out of service. This would also increase the weight of the consequence of event for soil erosion, causing the rating to go from a 2 to a 5. This would also make the user cost increase in weight, causing the rate to change from 1.5 to 5.

In addition, if a condition rating, appraisal rating, or both were decreased, ADT increased, or combination, the user cost would have a lower weight. For example, if the condition rating for the deck condition decreased from a 7 to a 4, the condition rating for the superstructure decreased from an 8 to a 3, the appraisal rating for waterway adequacy dropped from an 8 to a 4, and the ADT increased from 1,000 to 32,000, the user cost would drop from a 14.95 to an 11.81. This would cause the user cost rating to increase from a 1.5 to 2. As a result, the resilience rating would decrease from a 77.4 to a 37.9, as shown in Table 5-5.

Table 5-5: Coyote Wash Bridge Susceptibility Analysis

Resilience Indicator	Weight, W(x)	Rating, R(x)	W(x)*R(x)
Pier/Abutment Washout	24.0	15/15	24.0
Extreme Temperature	7.2	15/15	7.2
Load Path	4.3	15/15	4.3
Failure Mode	4.3	8/15	2.31
Soil Erosion	60.1	0/15	0.0
		Total (RR)=	37.9

5.4.3 Changes Resulting in a Higher Resilience Rating

Tanner Wash Bridge had the second lowest resilience rating of any other bridge in the study. There are several ways this bridge could be improved. For example, suppose the bridge had scour protection countermeasures installed for flow control, floor protection, and bank protection. If an outlet structure were installed to control flow, rock riprap were installed on the floor, and the bank protection of the waterway remained the same, the coding in the SI&A report would go from a '0' to a '4' for all three items. This would cause the rating for scour protection would go from a 0/15 to a 7/15. Also, if the bridge had reconstruction to improve the foundations, the rating for pier and abutment washout would increase. Suppose that the spread footings were replaced with drilled shafts. This would cause the rating for pier or abutment washout to go from a 6/15 to a 15/15.

Also, suppose, additional routes were added to the existing transportation network near Tanner Wash Bridge. Currently, the detour length for Tanner Wash Bridge is coded '99' in the SI&A report, which means the detour is at least 99 miles long. If the detour length were reduced from 99 miles to, say, 20 miles, the user cost would increase from 4.70 to 12.92 and the rating for user cost would go from 3 to 1.5. The changes discussed above would cause the resilience rating of Tanner Wash Bridge to go from 33.33 to a 77.0, as shown in Table 5-6.

Table 5-6: Tanner Wash Bridge Susceptibility Analysis

Resilience Indicator	Weight, W(x)	Rating, R(x)	W(x)*R(x)
Pier/Abutment Washout	32.9	15/15	32.9
Extreme Temperature	20.5	11/15	15.0
Load Path	12.3	15/15	12.3
Failure Mode	12.3	8/15	6.6
Soil Erosion	21.9	7/15	10.22
		Total (RR)=	77.0

5.5 Implication of Results

The implication from this study is that the sufficiency rating alone is not enough of a measure to allocate maintenance and repair funds. While the sufficiency rating is an important measure, the resilience rating provides additional, useful information. The sufficiency rating results in a high score for new bridges with robust foundations and components that are in good condition. However, it is illustrated that the sufficiency rating is weakly correlated to the resiliency rating. Thus, current bridge assessment methodologies are not adequately adapting to the changing environment.

As seen from the results of this study, the sufficiency and resilience ratings are not correlated- a high sufficiency rating is not indicative of a high resilience rating. A bridge with a high sufficiency rating yet a low resilience rating may be functional and in good condition, but may not be able to quickly adapt to or recover from an extreme event. The resilience rating takes into account some aspects of condition, but the impacts on the rating are small. Although a sufficiency rating is still a good measure of the condition and functionality of a bridge, it is incomplete without a measure of resilience. Therefore, these two ratings should be viewed separately and in parallel when making funding decisions.

Adapting resilient practices into the design, construction, and maintenance of the nation's bridges is an integral step in improving infrastructure quality. With the increase in extreme and/or uncommon events, there is a need to determine how well a bridge can adapt, overcome, and minimize the impacts of these events. The proposed method quantitatively measures a bridge's level of resilience and, with the sufficiency rating, provides in-depth and complete information about the condition of a bridge. The method is also easily implementable due to its use of information already on hand. With the implementation of this resilience measure into routine bridge maintenance, disasters will be more easily prevented.

6 Conclusion

6.1 Summary

The overall objective of this study was to incorporate resilience into the design, construction, and maintenance of structures. Current bridge inspection procedures and condition assessments do not directly include resilience in their procedures. However, exceptionally intense and increasingly frequent weather events underscore a pressing need to improve the resilience of our infrastructure. In response, the goal of this study is to develop an approach to quantify resilience. This is accomplished by first reviewing the literature and state of the art of structural resilience and current sufficiency rating practices. Given these practices, a methodology for rating resilience was developed. Then, four case study bridges with varying combinations of high and low sufficiency and resilience ratings were selected for a comparative case study. These bridges were analyzed and the method to quantify resilience was adjusted as needed. All four bridges are then rated for resilience and the resilience ratings are compared to their sufficiency ratings. The impacts of maintenance and improved design were examined to determine their effect on the resilience rating. The final product was an easily adaptable design methodology. As predicted, it is shown that a high sufficiency rating is not indicative of a high resilience rating. In other words, there is no direct correlation between the sufficiency and resilience ratings.

6.2 Implications of Results

The results from this study have implications for improvement of transportation infrastructure design, construction, and maintenance. The results show that current sufficiency rating scores (Table 5-1) are not an adequate indicator of a bridge's resilience. In short, the sufficiency rating does not measure a bridge's resilience. Quantifying a bridge's condition, functionality, and usefulness does not determine whether a bridge can adapt to, quickly recover from, or reduce the impacts of an unforeseen event. Clearly, these two ratings have separate but comparable importance. In other words, neither can stand alone when determining the overall state of a bridge.

Resilience can be suitably quantified using the methodology proposed in this thesis. As expected, the resilience rating is sensitive to the condition and/or design details of a bridge. The susceptibility study performed in this thesis showed if a bridge were to deteriorate over time, the resilience rating decreased. It also confirmed if a bridge received improvements, the resilience rating increased. This shows that the resilience rating is as fluid as the sufficiency rating. It can change over time and the difference in the resilience ratings will demonstrate how the resilience of the structure has either increased or decreased.

A resilient bridge will increase public safety, and reduce the occurrence of unpredictable maintenance or construction costs. For example, if the Tex Wash Bridge (Section 2.2.3.2) had drilled shaft foundations, it likely would have withstood the intense precipitation event that washed out its abutment. Furthermore, had the I-35W Bridge (Section 2.2.3.2) contained redundant load paths, it may have only sustained partial damage during an unusual loading event. Clearly, if these two bridges had been more resilient, the catastrophic losses would have been mitigated. The results of this study indicate that quantifying resilience is beneficial to improving bridge design and performance in the wake of rapid climatic changes and an aging infrastructure.

Since this approach aims to identify non-resilient bridges, it can be concluded that disasters like the Tex Wash and I-35W bridges could be prevented. Had the Tex Wash Bridge been rated for resilience, the foundations present before the 2015 collapse would have reduced the rating for pier/abutment washout and raised the weights for user cost and consequence of event. These would likely cause the bridge to have a low resilience rating similar to that of Tanner Wash Bridge. The I-35W Bridge would likely have had a resilience rating similar to the Midgley Bridge. Due to its non-redundant substructure, the I-35W Bridge is also fracture critical. This would cause the redundancy rating to be '1' and the user cost and consequence of event weights to rise. Since these two bridges would have likely had low resilience ratings before their collapse, preventative and/or corrective actions could have been taken to avoid these tragedies.

6.3 Applications of Results

An important goal of this study is to develop an approach for quantifying resilience that is easily adaptable by state departments of transportation. As a result, much of the information needed to use the proposed methodology is collected directly from a bridge's SI&A report. Other information is found on the as-built construction drawings or existing inspection documents. By utilizing existing information, the implementation of the resilience rating approach is straightforward.

Additionally, the development of resilience rating procedure will aid in implementing resiliency into design practice. That is, components of a bridge that are affected by changes in resilience are more easily identified. This allows decision makers and designers to identify those portions of a bridge where improvements will most effectively benefit the transportation system as a whole.

6.4 Limitations of Study

It is important to emphasize a limitation of this study because this is an area for future development. Given the scope of this study, the capacity measures are simplified. For example, the rating for foundation type is simply based on the foundation code listed in the SI&A report. It does not take into account the foundation embedment depth, typical flow conditions, or flood event flow conditions. Another example is the capacity measure for scour protection. This was rated solely based on the countermeasures listed in the SI&A report for the floor, bank, and flow control. In other words, there was no in-depth scour analysis performed to determine if the countermeasures were sufficient.

Given the limitation highlight above, the resilience rating approach developed in this thesis can be further adapted, modified, and refined. One recommendation is to refine the procedure for rating the capacity measures. For example, the rating for foundations could incorporate the embedment depth and flow conditions. In addition, the rating for scour protection has the ability to be improved through an in-depth scour analysis. By comparing the results of the in-depth analysis to the bridge's current scour protection countermeasures, the adequacy of those countermeasures can be determined.

6.5 Conclusions

Given the need for an approach for rating bridge resilience and the above results and limitations, the following conclusions have been reached:

- A sufficiency rating alone is not enough to make informed decisions on fund allocation for bridges.
- The sufficiency rating is not correlated to the resilience rating. Both quantities are needed to understand the condition of bridges.
- The approach given in this thesis is easily adaptable and sensitive to the primary factors known to influence the resilience of a bridge.
- The framework for deriving the approach to quantify resilience in bridges is detailed in full. Users can update this approach to meet their specific needs.

Increasing severity in weather events has created a need to include resilience in structures. Current aging infrastructure was not designed to meet new increased demands. Previous solutions to this problem are unsatisfactory. Current proposed methods to quantify structural resilience are insufficient because they are specific to certain events, (e.g. a combined instance of scour and earthquake), the approach is retroactive, unformulated approaches are utilized, or they are specific to certain regions of the country (e.g. those subjected to permafrost). Developing an approach to quantify bridge resilience will not only allow for more informed decisions when allocating repair and replacement funds, but will benefit network users, including residents and the trucking industry. Resilient bridges will be able to withstand, quickly recover from, and/or minimize the consequences of an extreme event. The approach proposed in this thesis incorporates components from previously proposed resilience quantification methods and resulted in a method that encompasses all bridges in the NBI and that is adaptable to meet the needs of the user.

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Appendices

Appendix A: Support Type Calculations

Earp Wash Bridge

$\delta_{temp150}$	9.94E-01	in
α	6.00E-06	1/°F
ΔT	150	°F
L	1104	in

$\sigma_{temp150}$	2809.8167	psi
$\delta_{temp150}$	0.9936	in
L	1104	in
E_c	3122018.6	psi

f'_c	3000	psi
$0.5*f'_c$	1500	psi

E_c	3122018.578	psi
f'_c	3000	psi

Midgley Bridge

$\delta_{temp150}$	2.59	in
α	6.00E-06	1/°F
ΔT	150	°F
L	2880.00	in

$\sigma_{temp150}$	3441.3086	ksi
$\delta_{temp150}$	2.592	in
L	2880	in
E_c	3823676.2	ksi

f'_c	4500	psi
$0.5*f'_c$	2250	psi

Actual width of joint	4	in

E_c	3823676.242	psi
f'_c	4500	psi

Coyote Wash Bridge

$\delta_{temp150}$	1.5444	in
α	6.00E-06	1/°F
ΔT	150	°F
L	1716	in

$\sigma_{temp150}$	3.4413086	ksi
$\delta_{temp150}$	1.5444	in
L	1716	in
E_c	3823.6762	ksi

f'_c	4500	psi
$0.5*f'_c$	2250	ksi

Actual width of joint	4	in

E_c	3823676.242	psi
f'_c	4500	psi

Tanner Wash Bridge

$\delta_{temp150}$	2.115225	in
α	6.00E-06	1/°F
ΔT	150	°F
L	2350.25	in

$\sigma_{temp150}$	3.44	ksi
$\delta_{temp150}$	2.115225	in
L	2350.25	in
E_c	3823.6762	ksi

f'_c	4500	psi
$0.5*f'_c$	2250	psi

E_c	3823676.242	psi
f'_c	4500	psi

Actual width of joint	2	in

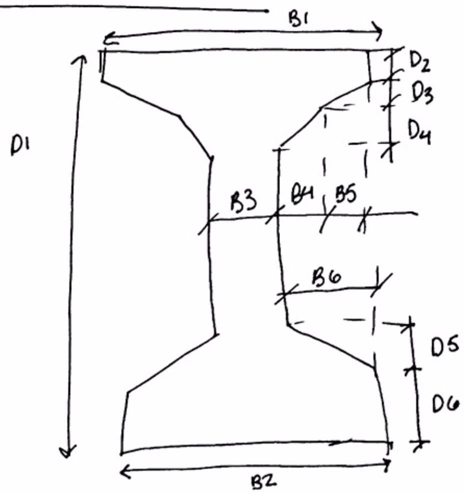
Appendix B: Load Path vs. Redundancy Calculation

Coyote Wash Bridge

64 strands: 21 harped & 43 straight

Capacity of single prestressed girder

AASHTO TYPE VI Girder Properties:

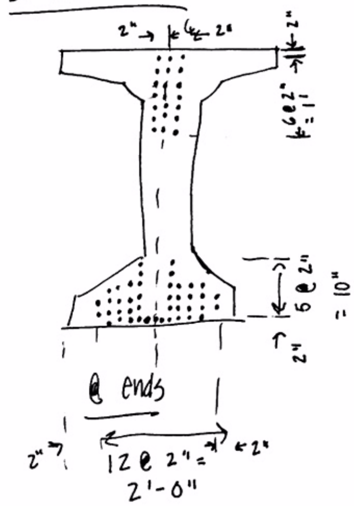


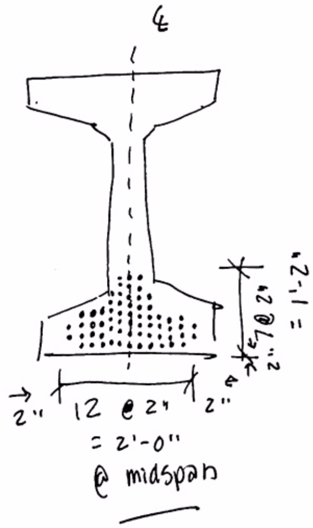
D1 = 72"
D2 = 5"
D3 = 3"
D4 = 4"
D5 = 10"
D6 = 8"

B1 = 42"
B2 = 28"
B3 = 8"
B4 = 4"
B5 = 13"
B6 = 10"

$A = 1085 in^2$
$\bar{Y}_{bot} = 36.38"$
$I = 733,320 in^4$
$w = 1.13 k/ft$

Strand Pattern:





Nominal Flexural Resistance : Ends

$M_n = A_{ps} f_{ps} (d_p - \frac{a}{2})$

0.19625

$A_{ps} = (21+43)(0.5in)^2(\frac{\pi}{4}) = \underline{12.56 in^2}$

$f_{ps} = f_{pu}(1 - k\frac{c}{d_p})$

$f_{pu} = 270$ ksi

$k = 2(1.04 - \frac{f_{py}}{f_{pu}}) = 0.28$ for low-relax strands

$d_p = h - y_{bs}$ 23.8125

$= 72'' - (36.38'' - e_c) = 48.1875''$

$c = \dfrac{A_{ps} f_{pu}}{0.85 f'_c \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}}$

* assume rectangular section behavior

3(70) + 3(

$e_c = 36.38'' -$

$e_c = 36.38'' - \dfrac{3(70+68+66+64+62+60+58) + 2(12+10) + 8(8) + 5(6+4) + 11(2)}{64}$

$e_c = 36.38 - \dfrac{1344 + 44 + 64 + 50 + 22}{64}$

$36.38 - 23.8125$

$e_c = 12.5675$

$f'_c = 6500$ psi

$\beta_1 =$

$b =$

$$Y_{bs} = \frac{3(2+4+6+8+10+12+14) + 2(62+60) + 8(64) + 10(66+68) + 11(70)}{64}$$

$$Y_{bs} = \frac{168 + 244 + 512 + 1340 + 770}{64} = 47.406$$

$d_p = h - Y_{bs} = 72'' - 47.406'' = 24.594$

$b = 28''$ $\quad f'_c = 6500$ psi $\quad \beta_1 = 0.85 - 0.05\left(\frac{6500-4000}{1000}\right) = 0.725$

$$c = \frac{12.56(270)}{0.85\left(\frac{6500}{1000}\right)(0.725)(28'') + 0.28(12.56)\left(\frac{270}{24.594}\right)} = \frac{3{,}391.2}{150.7655} = 22.49$$

38.608

$c = 22.49$

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) = 270\left(1 - 0.28\,\frac{22.49}{24.594}\right) = 200.878 \text{ ksi}$$

0.25604619

$a = \beta_1 c = 0.725(22.49) = 16.30525$

$$M_n = \frac{12.56 \text{ in}^2 (200.878 \text{ ksi})\left(24.594'' - \frac{16.30525}{2}\right)}{12} = 3{,}456.84 \text{ k-ft}$$

8.152625

---

midspan

$$\frac{11(2) + 13(4+6) + 11(8) + 9(10+12) + 3(14+16)}{64}$$

$22 \quad 130 \quad 88 \quad 110 \quad 90$

$6.875''$

$d_p = 72'' - \cancel{\ldots}$

$d_p = 65.125''$

$A_{ps} = 64(0.5\text{ in})^2\left(\frac{\pi}{4}\right) = 12.56 \text{ in}^2$ $\quad b = 42''$

$$c = \frac{12.56(270)}{0.85(6.5)(0.725)(42'') + 0.28(12.56)\left(\frac{270}{65.125}\right)} = \frac{3{,}391.2}{182.81625} = 18.55$$

$168.23625 \quad 14.58$

$$f_{ps} = 270\left(1 - 0.28\,\frac{18.55}{65.125}\right) = 248.4675 \text{ ksi}$$

0.07975

$a = \beta_1 \cdot c = 18.55 \cdot (0.725)$

$a = 13.44875$

$$M_n = \frac{12.56(248.4675)\left(65.125 - \frac{13.44875}{2}\right)}{12} = 15{,}187.82 \text{ k-ft}$$

6.724375

Entire Structure - PBR

$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2}\right)$

$A_{ps} = 12.56 in^2$

$f_{pu} = 270 ksi$

$$d_p = h - y_{bs} = 80" - \frac{11(2+8) + 13(4+6) + 5(10+12) + 3(14+16)}{64} = 73.125"$$

$$c = \frac{12.56(270)}{0.85(6.5)(0.725)(93") + 0.28\left[12.56\left(\frac{270}{73.125}\right)\right]} = \frac{3{,}391.2}{385.508} = 8.79670$$

12.985107

$b = \underline{93"}$

$\frac{1}{4} span = \frac{143 \times 12}{4} = 429"$

center to center beams = 7'-9" = 93"

12 (slab thickness) + $\frac{1}{2}$ beam top flange width = $12(8) + \frac{1}{2}(42) = 117"$

$a = \beta_1 \cdot c = 0.725(8.79670) = 6.3776$

$$f_{ps} = 270\left(1 - 0.28 \frac{8.79670}{73.125}\right) = 260.9064 ksi$$

0.03368

$$M_n = \frac{12.56(260.9064)\left(73.125 - \frac{6.3776}{2}\right)}{12} = 19098.3196 \text{ K-ft}$$

Shear Capacity

$V_n = 0.25 f'_c b_v d_v + V_p$ ✓ $f'_c = 6.5ksi$

$d_v = d_c - \frac{a}{2}$ $d_c = h_c - Y_{bs\,end} = 80" - \frac{\overset{22}{11(2)} + \overset{100}{10(4+6)} + \overset{64}{8(8)} + \overset{48}{2(10+12)}}{43}$

$= 67.834"$ $= 74.55814$ 5.44186

6.3776

$d_v = d_c - 0.5(a) = 74.55814 - 0.5(\cancel{\ldots}) = \cancel{\ldots}$ 71.369

$\geq 0.9 d_c = 67.102$

$\geq 0.72h = 57.6$ ✓

$b_v = 8"$

~~Vp= force/strand * # harped * sin ψ = 248.4675 / 64 x 21 * sin (tan⁻¹ ((72-4-14)" / 55.2x12))~~

=

per 5.8.3.4.3 $V_p = 0$ $A_v \geq 0.0316\sqrt{f'c}\frac{b_v s}{f_y}$ $s = 3"$ $b_v = 8"$ $f_y = 60ksi$

$0.31 in^2 \geq 0.03223 in^2$

$V_n = 0.25(6.5ksi)(8in)(\cancel{\ldots}) = \cancel{\ldots}$

71.369 927.8^K

Coyote Wash Bridge

M_{bridge}	114589.9	k-ft
$M_{element}$	15187.82	k-ft
R	0.867459	

Earp Wash Bridge

M_{bridge}	11936.18	k-in
$M_{element}$	293.5102	k-in
R	0.97541	

Tanner Wash Bridge

M_{bridge}	27222.65	k-ft
$M_{element}$	3303.67	k-ft
R	0.878643	

Appendix C: Failure Mode vs Strength Reduction Factor

Load causing Flexural Failure: Coyote Wash

M_{max} w/ pt load @ center span $= \frac{P\ell}{4} = \frac{P(143 \times 12)}{4} = 429P$

$\phi M_n \geq M_u$ $\quad$ $19{,}098.32$ k-ft ≥ 35.75 ft P $\quad$ $\underline{P \leq 534.2^k}$

Load causing shear failure: Coyote wash

V_{max} w/ pt load just next to support $= \frac{Pb}{\ell} = \frac{P(142.9)}{143} = 0.99P$

$\phi V_n \geq V_u$ $\quad$ $927.8^k \geq 0.99P$ $\quad$ $\underline{P \leq 937.17^k}$

Load causing Flexural Failure: Tanner Wash

M_{max} w/ pt load @ center span 2 $= \frac{P\ell}{4} = \frac{P(64 \times 12)}{4} = 192P$

$\phi M_n \geq M_u$ $\quad$ $12{,}033.92$ k-ft ≥ 16 ft P $\quad$ $\underline{P \leq 752.12^k}$

Load causing shear Failure: Tanner Wash

V_{max} w/ pt load just next to support $= \frac{Pb}{\ell} = \frac{P(63.9)}{64} = 0.99P$

$\phi V_n \geq V_u$ $\quad$ $1961.58^k \geq 0.99P$ $\quad$ $\underline{P \leq 1981.39^k}$

Load causing flexural failure: Earp Wash

M_{max} w/ pt load @ center span 2|3 $= \frac{P\ell}{4} = \frac{P(25)}{4} = 6.25$ ft P

$\phi M_n \geq M_u$ $\quad$ 24.46 k-ft ≥ 6.25 ft P $\quad$ $P \leq \underline{3.9136^k}$

Load causing shear failure: Earp wash

V_{max} w/ pt load just next to support $= \frac{Pb}{\ell} = \frac{P(24.9)}{25} = 0.99P$

$\phi V_n \geq V_u$ $\quad$ $4.27^k \geq 0.99P$ $\quad$ $P \leq 4.31^k$

Coyote Wash Bridge

FM1	3.9136	k-ft
FM2	4.31	k
ϕ1	0.9	
ϕ2	0.75	
%diff	-0.09	

Earp Wash Bridge

FM1	24.45918	k-ft
FM2	4.27	k
ϕ1	0.9	
ϕ2	0.75	
%diff	-1.49	

Tanner Wash Bridge

FM1	752.12	k-ft
FM2	1981.39	k
ϕ1	0.9	
ϕ2	0.75	
%diff	0.75	

Midgley Bridge

FM1	46.38	k
FM2	325.14	k
ϕ1	0.75	
ϕ2	0.75	
%diff	1.50	

Appendix D: User Cost

Coyote Wash Bridge

S3	14.953125
A	0.046875
k	1
ADT	1000
DL	1
B	0

S1	55
A	0
B	0

S2	30
J	0
G	0
H	0
I	0
A	0
B	0
C	0
D	
E	0
F	0
X	500
Y	21

Earp Wash Bridge

S3	9.950596107
A	3.049403893
k	0.53032906
ADT	34500
DL	1
B	2

S1	22.07797
A	25
B	7.92203

S2	23
J	7
G	0
H	0
I	0
A	3
B	2
C	2
D	
E	0
F	0
X	11500
Y	16.26667

Tanner Wash Bridge

S3	4.697447814
A	10.30255219
k	0.993658519
ADT	2206
DL	99
B	0

S1	54.46097
A	0
B	0.539026

S2	30
J	0
G	0
H	0
I	0
A	0
B	0
C	0
D	
E	0
F	0
X	1103
Y	22

Midgley Bridge

S3	6.895924486
A	8.104075514
k	0.847976119
ADT	3858
DL	38
B	0

S1	47.07797
A	0
B	7.92203

S2	25
J	5
G	0
H	0
I	0
A	0
B	1
C	4
D	
E	0
F	0
X	1929
Y	12

Appendix E: Replacement Cost

Coyote Wash

	Volume (ft^3)	% of Structure		
Superstructure	7155.958333	13.91946289		
Girders	12206.25	23.74307338	3.957179	*one girder
Foundation Piles	30239.082	58.81976389	29.40988	*one side
Barriers	649.22	1.262834868		
Wingwalls	1159.22	2.254864969		

Earp Wash

	Volume (yd^3)	% of Structure		
Superstructure	171.31	53.86171806		
Piers	79.58336728	25.02187199	8.340624	*one pier
Barriers	11.36	3.571706947		
Abutments	55.80184182	17.544703	8.772352	*one side

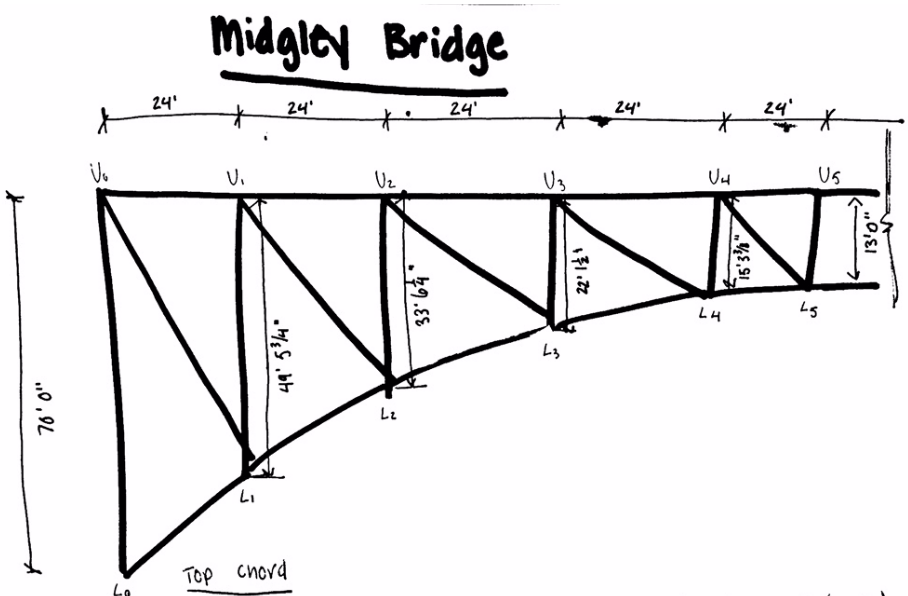
Tanner Wash

	Volume (yd^3)	% of Structure		
Superstructure	341.27	16.30408703		
Piers	240.2	11.47549361	5.737747	*one pier
Barriers	110.9471772	5.300473038		
Abutments	842.98	40.27315407	20.13658	*one side
Girders	557.7589699	26.64679225	5.329358	*one girder

Midgley

	Weight (lbs)	% of Structure	
pier/abutm	3199500	63.61467343	15.90367
Deck	810000	16.10498061	
Truss	1020000	20.28034596	

Appendix F: Midgley Bridge High Stressed Members



= FCM

= Highest stressed members

FCMs:

L_3L_4	U_0L_1
L_4L_5	U_1L_2
L_5L_4'	U_2L_3
$L_4'L_3'$	U_3L_4
	U_4L_5
	$U_4'L_5$
	$U_3'L_4'$
	$U_2'L_3'$
	$U_1'L_2'$
	$U_0'L_1'$

Floor beams in span 2 & e bent No 1

Top chord

U_0U_1, U_1U_2 → 2x C12X20.7 $A_g = 12.06\,in^2$ $A_{g,req'd} = 6.0\,in^2$ (U_0U_1), $9.7\,in^2$ (U_1-U_2) $\frac{A_g}{A_{g,req'd}} = \frac{12.06}{5.0} = 2.41$ $\frac{A_g}{A_{g,req'd}} = \frac{12.06}{9.7} = 1.24$

U_2U_3 → 2x C12 x 25 $A_g = 14.64\,in^2$ $A_{g,req'd} = 14.2\,in^2$ $\frac{A_g}{A_{g,req'd}} = \frac{14.64}{14.2} = 1.03$

U_3U_4 → 2x C12 x 20.7 + 2x 11" x 3/8" plate $A_g = 20.31\,in^2$ $A_{g,req'd} = 19.1\,in^2$ $\frac{A_g}{A_{g,req'd}} = \frac{20.31}{19.1} \doteq 1.06$

U_4U_5 → 2x C12 x 25 + 2x 11" x 5/16" plate $A_g = 21.52\,in^2$ $A_{g,req'd} = 21\,in^2$ $\frac{A_g}{A_{g,req'd}} = \frac{21.52}{21} = 1.02$

2x 14" x 8 pl
+
2x 14" x 5/16" pl

$L_1L_2 \rightarrow$ 2x C15 x 40 + 2x 14" x 3/8" pl — $A_g = 33.90\ in^2$ — $A_{g,req'd} = 33.9\ in^2$ — $\frac{A_g}{A_{g,req'd}} = \frac{33.9}{33.9} = 1.00$

$L_2L_3 \rightarrow$ 2x C15 x 33.9 + 2x 14" x 15/16" pl — $A_g = 28.55\ in^2$ — $A_{g,req'd} = 27.6\ in^2$ — $\frac{A_g}{A_{g,req'd}} = \frac{28.55}{27.6} = 1.03$

highest stressed member
L_1-L_2 L_3-L_4

$L_3L_4 \rightarrow$ 2x C15 x 40 — $A_g = 23.40\ in^2$ — $A_{g,req'd} = 23.2\ in^2$ — $\frac{A_g}{A_{g,req'd}} = \frac{23.4}{23.2} = 1.01$

$L_4L_5 \rightarrow$ 2x C15 x 33.9 — $A_g = 19.80\ in^2$ — $A_{g,req'd} = 18.7\ in^2$ — $\frac{A_g}{A_{g,req'd}} = \frac{19.8}{18.7} = 1.06$

Diagonals

$U_0L_1 \rightarrow$ 2x C12 x 20.7 — $A_g = 12.06\ in^2$ — $A_{n,req'd} = 5.6\ in^2$ — $\frac{A_g}{A_{n,req'd}} = \frac{12.06}{5.6} = 2.15$

$U_1L_2 \rightarrow$ W10 x 49 — $A_g = 14.40\ in^2$ — $A_{n,req'd} = 4.8\ in^2$ — $\frac{A_g}{A_{n,req'd}} = \frac{14.4}{4.8} = 3.00$

$U_2L_3 \rightarrow$ / U_3L_4 W10 x 33 — $A_g = 9.71\ in^2$ — $A_{n,req'd} = 5.5\ in^2 (U_2L_3)$ $6.3\ in^2 (U_3L_4)$ — $\frac{A_g}{A_{n,req'd}} = \frac{9.71}{5.5} = 1.77$ — $\frac{A_g}{A_{n,req'd}} = \frac{9.71}{6.3} = 1.54$

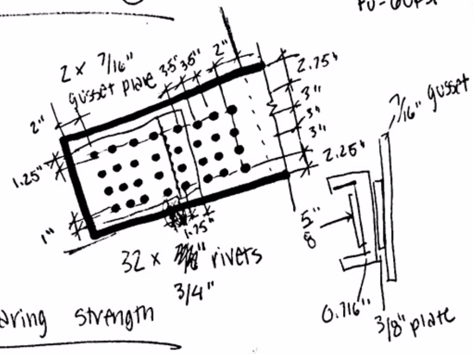
$U_4L_5 \rightarrow$ W10 x 49 — $A_g = 14.40\ in^2$ — $A_{g,req'd} = 6.0\ in^2$ — $\frac{A_g}{A_{g,req'd}} = \frac{14.4}{6} = 2.40$

Appendix G: Midgley Bridge Connections

31

Connection @ L1

$F_y = 33\text{ksi}$
$F_u = 60\text{ksi}$



Bearing strength

$\phi = 0.75$

$\phi R_n = 1.8 F_y A_{pb}$ (J7-1) → bearing strength of pin

$F_y = 33$ ksi

$A_{pb} = (\frac{5}{8} + 0.716 + 3/8 + 7/16) = 2.1535\text{in}$ ↳ $\phi = 3/4"$ ↳ 1.615in^2

$\phi R_n = 0.75(1.8)(33)(1.615) = \underline{71.95^K}$

Block Shear $\phi = 0.75$

$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$

$A_{nv} = (\frac{3}{8}" + 0.716")[(2" + 3.5" \times 14) - 15(3/4" + 1/8")] = 41.32\text{in}^2$

$A_{gv} = (\frac{3}{8}" + 0.716")(2" + 3.5" \times 14) = 55.64\text{in}^2$

$A_{nt} = (\frac{3}{8}" + 0.716")[3" \times 3 - 3(\frac{3}{4} + \frac{1}{8})] = 6.455$

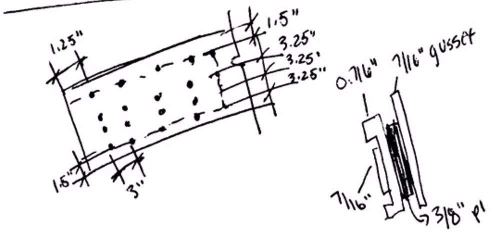
$0.6 F_u A_{nv} = 0.6(60\text{ksi})(41.32\text{in}^2) = 1487.52^K$

$0.6 F_y A_{gv} = 0.6(33\text{ksi})(55.64\text{in}^2) = 1101.67^K$ → governs

$U_{bs} = 1.0$

$\phi R_n = 0.75(1.0(60\text{ksi})(6.455\text{ in}^2) + 1101.67^K) = 1139.23^K$

Connection @ L2



Bearing Strength

$\phi = 0.75$

$\phi R_n = 1.8 F_y A_{pb}$

$F_y = 33$ ksi

$A_{pb} = 3/4[\frac{7}{16} + 0.716 + 7/16 + 3/8] = 1.474\text{in}^2$

$\phi R_n = 0.75(1.8)(33)(1.474) = \underline{65.67^K}$

Block Shear $\phi = 0.75$

$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$

$A_{nv} = (\frac{3}{8}" + 0.716")[(1.25" + 3"(8)) - 9(3/4" + 1/8")] = 18.96\text{ in}^2$

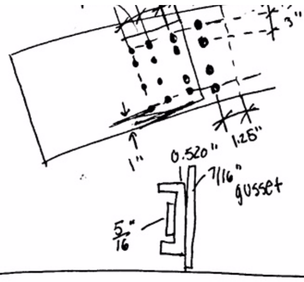
$A_{gv} = (\frac{3}{8}" + 0.716")[1.25" + 3"(8)] = 27.55\text{ in}^2$

$A_{nt} = (\frac{3}{8}" + 0.716")[3 \times 3.25" - 3(\frac{3}{4} + \frac{1}{8})] = 7.77\text{ in}^2$

$0.6 F_u A_{nv} = 0.6(60\text{ksi})(18.96) = 682.56^K$

$0.6 F_y A_{gv} = 0.6(33\text{ksi})(27.55\text{ in}^2) = 546.49^K$ → governs

$\phi R_n = 0.75[1.0(60\text{ksi})(7.77\text{in}^2) + 546.49^K] = \underline{758.77^K}$



Bearing Strength

$\phi = 0.75 \quad \phi R_n = 1.8 F_y A_{pb}$

$F_y = 33ksi$

$A_{pb} = 3/4"\left[\frac{5}{16}" + 0.520" + \frac{7}{16}"\right] = 0.953in^2$

$\phi R_n = 0.75(1.0)(33ksi)(0.953in^2)$

$= \underline{42.46^k}$

$A_{gv} = 0.520"\left[1.25" + 6(3")\right] = 10.01\ in^2$

$A_{nt} = 0.520"\left[3"(3) - 3\left(3/4 + \frac{1}{8}\right)\right] = 3.315in^2$

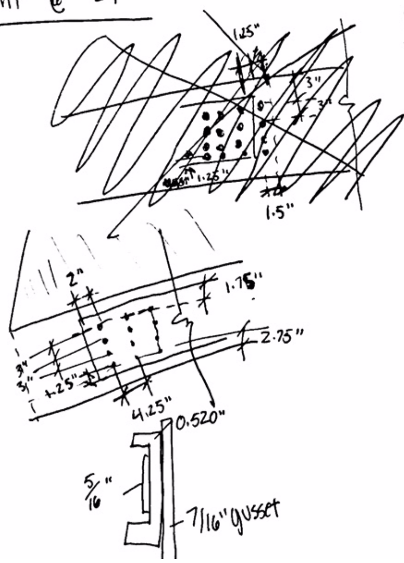
$0.6 F_u A_{nv} = 0.6(60)(6.825) = 245.7\ ^k$

$0.6 F_y A_{gv} = 0.6(33)(10.01) = 198.20^k \rightarrow$ governs

$\phi R_n = 0.75\left[1.0(60)(3.315) + 198.20^k\right] =$

$\underline{297.83^k}$

Joint @ L4



Bearing Strength

$\phi = 0.75 \quad \phi R_n = 1.0 F_y A_{pb}$

$F_y = 33ksi$

$A_{pb} = 3/4"\left[\frac{5}{16}" + 0.520" + \frac{7}{16}"\right] = 0.953in^2$

$\phi R_n = 0.75(1.0)(33ksi)(0.953in^2)$

$\phi R_n = \underline{42.46^k}$

Block Shear

$A_{nv} = 0.520\left[\left(2" + 4(4.25")\right) - 5\left(\frac{3}{4} + \frac{1}{8}\right)\right] = 7.605in^2$

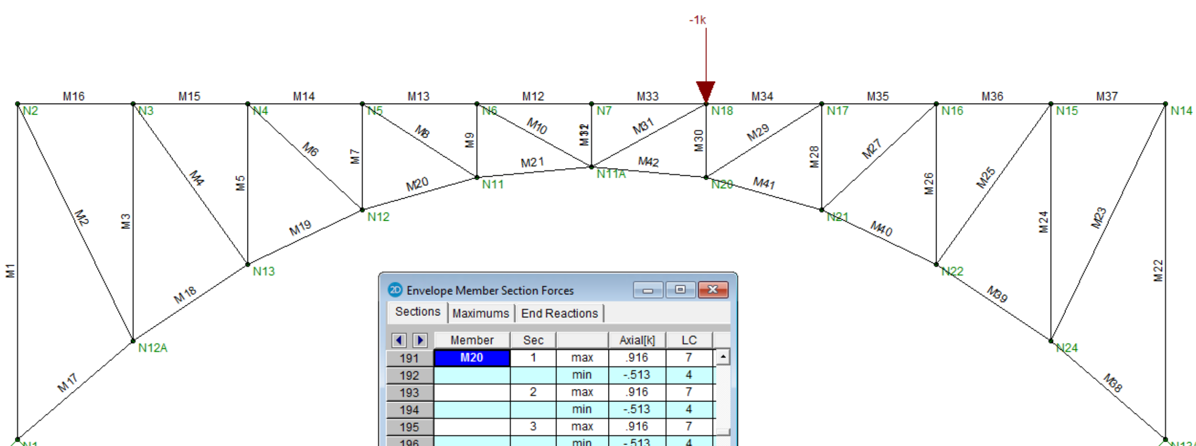
$A_{gv} = 0.520\left(2" + 4(4.25")\right) = 9.88\ in^2$

$A_{nt} = 0.520\left(3(3") - 3(3/4 + 1/8)\right) = 3.315in^2$

$0.6 F_u A_{nv} = 0.6(60)(7.605) = 273.78^K$

$0.6 F_y A_{gv} = 0.6(33)(10.01) = 198.20^K$ governs

$\phi R_n = 0.75\left[1.0(60)(3.315) + 198.20^K\right] = \underline{297.83^k}$



Envelope Member Section Forces

Sections | Maximums | End Reactions

	Member	Sec		Axial[k]	LC
191	M20	1	max	.916	7
192			min	-.513	4
193		2	max	.916	7
194			min	-.513	4
195		3	max	.916	7
196			min	-.513	4
197		4	max	.916	7
198			min	-.513	4
199		5	max	.916	7
200			min	-.513	4

L3-L4

Failure Loads : Midgley Bridge

Max axial force when pt load As seen in RISA $= 0.916P$

Bearing:

$\phi R_n \geq P_u$ $\quad 42.46^k \geq 0.916P$ $\quad \underline{P \leq 46.38^k}$

Block Shear:

$\phi R_n \geq P_u$ $\quad 297.83^k \geq 0.916P$ $\quad \underline{P \leq 325.14^k}$