Ultimate Shear Capacity and Residual Prestress Force of Full-Scale, Forty-One-Year-Old Prestressed-Concrete Girders

Parry Osborn
Utah State University

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ULTIMATE SHEAR CAPACITY AND RESIDUAL PRESTRESS FORCE OF FULL-SCALE, FORTY-ONE-YEAR-OLD PRESTRESSED-CONCRETE GIRDERS

by

Parry Osborn

A thesis submitted in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

in

Civil and Environmental Engineering

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UTAH STATE UNIVERSITY
Logan, Utah

2010
ABSTRACT

Ultimate Shear Capacity and Residual Prestress Force of Full-Scale Forty-One-Year-Old Prestressed-Concrete Girders

by

Parry Osborn, Master of Science
Utah State University, 2010

Major Processor: Dr. Paul J. Barr
Department: Civil Engineering

The ultimate shear capacity of prestressed concrete beams is difficult to predict accurately, especially after being in service for an extended period of time. The Utah Department of Transportation asked researchers at Utah State University to experimentally determine the existing shear capacity of 41-year-old prestressed, decommissioned concrete bridge girders and then provide recommendations on how to increase that ultimate shear capacity. This thesis presents the research findings that relate to the existing shear capacity of the prestressed concrete girders.

Eight AASHTO Type II bridge girders were tested up to failure by applying external loads near the supports to determine their ultimate shear capacities. The measured results were then compared to calculated values obtained using the AASHTO LRFD bridge design code, and the ACI 318-08 design code. Prestress losses were also
measured by means of a cracking test and then compared to values calculated according to the AASHTO prestress loss equations. Both the ultimate shear capacities and the residual prestress forces were used to evaluate the girders after being in service for more than 40 years.
ACKNOWLEDGMENTS

I would like to thank all those who have helped to make this research possible. First of all, I would like to thank my wife and kids for their continued patience as I have spent many hours working in the lab and also writing. A big “Thank you” goes to Dr. Paul Barr whose continued advisement has made all of the physical research and theoretical methods possible. I would also like to thank all of the faculty and staff in the Civil Engineering Department at Utah State University who provided the proper setting to complete my thesis. I would like to thank the Utah Department of Transportation. This project would not have even been possible without the funding and cooperation of the Utah Department of Transportation. UDOT supplied all of the girders tested, along with the plans, and much more information and help along the way.

Parry Osborn
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CHAPTER 1

INTRODUCTION

1.1 Context

The ultimate shear capacity of prestressed concrete beams is difficult to predict accurately, especially after being in service for an extended period of time. The Utah Department of Transportation asked researchers at Utah State University to experimentally determine the existing shear capacity of 41-year-old prestressed, decommissioned concrete bridge girders and then provide recommendations on how to increase that ultimate shear capacity. This thesis presents the research findings which relate to the existing shear capacity of the prestressed concrete girders.

Eight AASHTO Type II bridge girders were tested up to failure by applying external loads near the supports to determine their ultimate shear capacities. The measured results were then compared to calculated values obtained using the AASHTO LRFD bridge design code, and the ACI 318-08 design code. Prestress losses were also measured by means of a cracking test and then compared to values calculated according to the AASHTO prestress loss equations. Both the ultimate shear capacities and the residual prestress forces were used to evaluate the girders after being in service for more than 40 years.
1.2 Ultimate Shear Capacity

There are three principle methods in which a reinforced prestressed concrete beam can fail in shear. The first type of shear failure is a web crushing failure. For a web crushing failure, the concrete compressive strength is exceeded and the web crushes typically at the top flange of an I-shaped section near the applied load. For a web crushing failure, the cracking is initiated in the web and then extends out in both directions.

The second type of shear failure is called a flexural shear failure. For this type of failure, the initial cracks form due to flexure at a 90-degree angle with respect to the longitudinal axis of the beam. As the externally applied load increases, shear forces and principal tensile stresses dominate the flexural effects causing the cracks to change direction (close to a 45 degree angle from the longitudinal axis) and continue until the principal stresses produce enough dilation of the crack to cause failure.

The third type of failure occurs in the discontinuity regions of the beam where plane sections don’t remain plane due to the load being applied so close to the support. Typical failure mechanisms occur due to arching action between the applied load and the support. Both the AASHTO LRFD and the ACI 318-08 design codes account for these three types of failure.

The ultimate shear capacity is a very complicated failure mechanism which is not fully understood or easy to quantify, despite significant advances over the past several years. There are several analytical methodologies which have been accepted as
accurate approximations of the overall shear behavior of reinforced prestressed concrete beams. This research focused on shear behavior produced by applying load at the near support (d-region) regions of AASHTO Type II girders. The deterioration that occurred over the service life of these girders added another level of uncertainty. Laboratory tests were performed on the eight girders to determine their existing shear capacities.

1.3 Research Objectives

The goal of this research was to obtain analytical and experimental values for the ultimate shear capacities of aged prestressed concrete bridge girders that had been subjected to corrosive conducive environments. The experimentally obtained results were compared to the calculated shear capacity obtained following the procedures outlined in the AASHTO LRFD bridge design code (2009) as well as the ACI 318-08 building code. Residual prestressing forces were experimentally obtained and compared to the values calculated using the AASHTO prestress loss equations. The measured and calculated values were used to determine the difference between the actual and predicted values.

1.4 Organization of Thesis

Ultimate shear tests were performed on eight AASHTO Type II bridge girders. The ultimate shear capacity was measured and compared to design equations according to two design codes. The residual prestress force was also experimentally obtained with a
cracking test, and then compared to the predicted residual force as calculated using the AASHTO predicted prestress loss equations. The organization of the research performed for this thesis is as follows:

1. Chapter 2 presents a summary of past research that had been performed on the shear capacities of prestressed concrete beams.

2. Chapter 3 presents a description of the experimental testing program. The different stages of the experimental process are described in detail, beginning with the test setup, then discussing the effective prestress tests, the shear tests, and finally presenting the results.

3. Chapter 4 introduces the design equations to calculate the ultimate shear capacity and prestress losses. A comparison between the measured results and the predicted results is also performed.

4. Chapter 5 summarizes the thesis and summarizes key conclusions. Recommendations for future research are also presented.
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

As prestressed concrete beams age and deteriorate, the tendency is to reinforce them in flexure leading to a very stiff beam that is more likely to fail in shear. This trend makes it more important than ever to understand the shear behavior of prestressed concrete beams. There are many factors that influence the overall shear capacity of prestressed concrete beams, many of which are dependent on the type of concrete, aggregate used, water content, and admixtures. According to ACI 318-08, there are two methods to determine the shear capacity of prestressed concrete beams: 1) the simplified method or 2) the detailed method where $V_c$ is the lesser of $V_{ci}$ or $V_{cw}$. These equations simplify the shear capacity calculations and overlook some of the contributors to the shear strength of the member. The actual shear capacity at failure depends on a combination of shear from the concrete, longitudinal mild reinforcement, prestressing strands, and the stirrups.

One contributing factor to the ultimate shear capacity that has changed drastically over the past 20-30 years is the strength of concrete. Since high-strength concrete is now being used more frequently in the design and construction of prestressed concrete beams, the effects of this higher strength concrete need to be considered when determining the shear capacity. Much of the completed research has been to determine the adequacy of the design codes’ specifications as they apply to
medium and high strength concrete, because the original design codes were developed based on regular strength concrete. With these considerations in mind, this chapter reviews some of the research that has been done to better understand the total shear strength of prestressed concrete beams.

2.2 Kordina, Hegger, and Teutsch (1989)

This research was done to gain a better understanding of the shear capacity of prestressed concrete beams with un-bonded prestressing tendons. Most of the research done prior to this investigation focused on quantifying the shear capacity of prestressed concrete beams with bonded tendons or the flexural capacity of prestressed concrete beams with un-bonded tendons. The goal of this research was to test prestressed concrete beams with un-bonded tendons in shear and to develop an accurate shear design methodology for this type of beam construction.

In their research, the authors conducted three series of tests. The first series utilized three monolithic beams. Each beam was simply supported and loaded at the mid-span. The beams spanned 13.12 ft (4 m). The second series of tests was carried out on two different beams. The first beam (SOV1) had a simple span of 19.69 ft (6 m), and the second beam (SOV2) was a continuous two span beam which was loaded at two points sequentially until shear failure occurred. Both of the beams used in the second series of tests were precast with joints that were carefully profiled. The third series of tests were performed on five beams simply supported over a 19.69 ft (6 m) span. These five beams, all containing stirrups, were loaded at different locations to cause up to
three failure zones. All of the beams that were used in these three series of tests were I-sections with the exception of one T-shaped cross section used in the third series. Straight tendons, having a diameter of 1.04 or 1.25 in (26.5 or 32 mm), or two unbonded single-strand tendons, harped at an angle of inclination of $\alpha=3.1$ degrees towards the support, were used.

Two analytical models were employed to analyze the behavior of the prestressed concrete beams, a truss analogy, and a tied-arch analogy. According to the truss analogy, the main factor governing shear was web reinforcement, whereas the tied arch analogy showed that the shear was controlled purely by the tension member. Therefore, the two main parameters looked at in this study were web reinforcement and tension reinforcement.

The initial formation of shear cracks in these test beams with unbonded tendons was similar to prestressed concrete beams with bonded tendons. The tension chords in the shear zone remained almost totally uncracked resulting in shear cracks forming independently from flexural cracks. After the initial cracking, the beams with unbonded tendons continued to crack due to “plate-action.” The shear cracking was the main observed difference between the bonded and unbonded prestressed concrete beams with regards to the shear carrying capacity.

The authors concluded that the most accurate shear model for use with prestressed concrete beams without bonded tendons is the truss analogy. The truss analogy can distinguish between tension-shear or flexural-shear failure and web-
crushing failure. This analogy was found to accurately predict the load-bearing capacity
and the failure mode. The tied-arch model only considered compression-arch failure
which was not consistent with the test results from six of the tested beams where the
obvious method of failure was yielding of the web reinforcement.

2.3 Oh and Kim (2004)

Several research projects have been conducted on prestressed concrete (PSC)
beams with an emphasis on flexural behavior. The shear behavior, however, is much
more complicated and less research has been conducted on this subject. As such, this
research focused on the shear capacity of prestressed, post-tensioned concrete beams.
The authors employed the use of large-scale, post-tensioned PSC girders made with
medium and high-strength concrete with compressive strengths of 40 and 60 MPa
respectively. Strain gages were used on the stirrups to analyze the strain behavior of
the shear stirrups, and surface concrete strain gages were attached to the side surfaces
on the beams to detect strain at that surface. Because of the deformation that occurs
during shear failure, many grids of sensors were needed, and the average strain was
used to describe the strain in the PSC beam during shear failure. With all of the data
collected during this study, more advanced design and analysis procedures of PSC beam
structures were proposed.

For this research two large-scale, post-tensioned PSC girders with grouted ducts
were constructed using normal and high-strength concrete. Each girder was a 1200 mm
deep and 10,600 mm long I-section. Girder 1 had a design compressive strength of 40
MPa, and Girder 2 was designed for a 60 MPa target compressive strength (42.8 and 62.1 MPa, respectively, at testing). The prestressing strands used were seven-wire strands with nominal diameter of 12.7 mm and nominal area of 98.71 mm$^2$ having a yield strength of 1620 MPa and an ultimate strength of 1890 MPa. Each girder encases three tendons consisting of six strands each. The girders each had mild steel reinforcements as stirrups (13 mm diameter) and as longitudinal steel bars (16 mm diameter), both having a yield strength of 345 MPa and ultimate strength of 540 MPa. Two different stirrup arrangements were used. The first was a 200 mm spacing on the right side and the second with a 400 mm spacing on the left side of each girder.

Girders 1 and 2 were loaded up to the ultimate load while strains in the stirrups and concrete surface were measured and compared. The cracking patterns were similar in both girders, but with some slight variation. Girder 2 exhibited more cracking, but with less dilation. This was a result of the high-strength concrete that was used in that girder. The strains on the surface of the girders remained small until diagonal shear cracks formed, and then a rapid increase was observed on the surface strains. Principal stresses along with their directions were calculated based on the strain and the deformations of the LVDTs that were attached to the sides of the girders. The principal directions were shown to rotate greatly as the load increased, and the principal directions approached 23 to 25 degrees at the ultimate load stage.

Oh and Kim concluded that: 1 - the high-strength concrete girder exhibited a more distributed cracking pattern, that is, there were more diagonal cracks with a
smaller crack width, 2 - the principal directions decreased as the load increased, and 3 -
the concept of average strains and the changing of principal directions according to the
applied load can be used for a more realistic shear analysis of PSC girders.

2.4 Kaufman and Ramirez (1988)

This paper presents research on high-strength prestressed concrete beams
loaded in shear and flexure. The focus of this paper was on the ultimate shear behavior
of high-strength, prestressed concrete beams. The authors employed the truss model
to obtain an accurate model which shows the behavior of the entire beam as opposed
to the segmental approach sometimes used. In their investigation, Kaufman and
Ramirez tested six full-scale AASHTO I-beams that included four Type I and two Type II.
The beams were cast at a local precast plant and designed according to ACI and AASHTO
bridge specifications. Each of the beams were loaded to failure and monitored for strain
and centerline deflection. Three different failure modes were observed: (1) flexural, (2)
web crushing, and (3) shear tension. The web crushing, flexural and shear tension
failures were all very explosive and brittle, however, if conservatively detailed following
either ACI or AASHTO specifications a more ductile failure was achieved.

High-strength concrete increased the capacity of the diagonal truss member
which allowed for smaller inclination angles. As the angle of inclination gets smaller, the
web reinforcement becomes more efficient through the mobilization of more stirrups.
The effectiveness of the truss model was contingent on the detailing of the members to
allow redistribution of internal forces and increased ultimate strengths. The amount of
reinforcement, both longitudinal and transverse and the proper development of these reinforcements have a great affect on the shear strength of prestressed, high-strength concrete beams. Proper development must be achieved by controlling the concrete stress in the web and flexural compression zone before web crushing occurs.

In order to prevent early tension shear failure, it is important to ensure that the transfer zone of the prestressing steel is behind the support region. If shear cracks develop that cross the transfer region of the prestressing steel, the bond will be damaged leading to a shear tension failure. The authors proposed that an alternative mechanical anchorage could be used to avoid this problem. Also noted was the fact that the ACI and AASHTO provisions are conservative in properly detailed members.

2.5 Elzanaty, Nilson, and Slate (1986)

This research investigated the effect of high-strength concrete on the shear capacity of reinforced concrete beams. The authors tested 18 beams with different concrete compressive strengths ranging from 6,000-12,000 psi (41-83 MPa). Of the 18 beams, only three had web reinforcement. Shear strength contribution from the concrete is essentially the “shear resistance of the still uncracked compression concrete above the top of the diagonal crack, aggregate interlock along the diagonal crack, and dowel resistance provided by the longitudinal reinforcement.” In high-strength concrete, the diagonal tension crack usually forms suddenly and typically has a much smoother shape than regular strength concrete leading to a decrease in aggregate interlock and subsequently reducing the shear capacity of the member.
The beams were reinforced with longitudinal ASTM Grade 60 deformed reinforcing bars with a yield strength of $f_y = 63$ ksi (434 MPa). The stirrups were smooth round bars $\frac{3}{4}$ inch in diameter (6.4 mm) with $f_y = 55$ ksi (379 MPa). The beams were all 7 inches (178 mm) wide by 12 inches (305 mm) deep. To identify the influence that $f'_c$, $a/d$, and $\rho_w$ had on the shear capacity, beams without web reinforcement were tested, whereas the beams with web reinforcement had a constant $a/d$ ratio of 4.0 while $f'_c$ varied. The tests on all of the beams were all loaded with symmetric concentrated loads. The loading was done in 4 kip (17.8 kN) increments up to a predicted load of 70 percent of the ultimate load where the load increments were reduced to 2 kips (8.9 kN). Strains, displacements, and crack development/propagations were measured at each load step. Material samples taken from each beam were tested to determine the compressive strength and modulus of rupture after each beam test.

Once flexural cracks formed in the shear spans the behavior of the beam varied depending upon the values of $f'_c$, $a/d$, and $\rho_w$, and was shaped by the presence or absence of web reinforcing steel. Beams without web reinforcing and with $\rho_w = 0.012$ failed suddenly in shear by forming a diagonal crack from the compression zone near the applied load towards the support. The beams without stirrups and with an $a/d = 4$ had an ultimate capacity in shear that was a little greater than the cracking load, but beams with an $a/d = 2$ showed significant shear capacity beyond the diagonal cracking load. Failure was observed to occur by either splitting along the flexural reinforcement
or sudden propagation of the critical inclined crack into the compression zone of the beam.

The authors concluded that the shear strength of beams without any web reinforcements increased with the increase of concrete compressive strength. They also stated that the current ACI codes for predicting shear capacity of concrete beams was unconservative for beams without web reinforcement and having high $f'_c$ and a/d, with low $\rho_w$. This was because the ACI code didn’t fully consider the effect of $\rho_w$ and a/d, yet overestimated the benefit of increasing compressive strength. Shear failures were more abrupt and the failure surfaces were smoother for beams with high-strength concrete.

2.6 MacGregor, Sozen, and Siess (1965)

In this study 104 simply supported, prestressed-concrete beams were tested in shear to determine the effects of web reinforcements on the overall shear capacity. All beams’ span was 9 ft with overall cross-sectional dimensions of 6 X 12 inches. Ten of the beams had a 2 X 24 inch deck cast on top after the prestressing strand was released. Five of the beams were rectangular, 43 were I-sections with 3 inch-thick webs, and 45 had a 1.75 inch-thick web. The strands had varying levels of prestress force ranging from 60-127 ksi, but with most beams’ strands prestressed to 120 ksi. Some of the beams had the prestressing tendons draped in the shear spans at an angle ranging from 1.5 to 10 degrees.

During the testing of the beams, different cracking patterns were observed for the inclined cracks. “Web-shear crack” was defined in this paper as an inclined crack
which occurs in the web before flexural cracks appeared in its vicinity. In contrast, flexural cracks occurred in the shear span before stresses were high enough to cause web-shear cracks. If an inclined crack occurred it was either an extension of a flexural crack or it occurred over or beside a flexural crack. The flexural crack that caused the inclined crack was referred to as an “initiating crack.”

For beams with draped tendons both web-shear and flexure-shear cracks were observed, however, the majority of the beams developed flexure-shear cracks. The test results indicate that draping the longitudinal reinforcement increases the inclined cracking load in the beams which developed web-shear cracks, and decreased the inclined cracking load for beams which developed flexure-shear cracks.

Two shear failures associated with the tied arch phenomenon were observed, namely tie rod connection failure and web distress failure. These failures were more prevalent in beams without web reinforcement therefore causing a large eccentricity of the compressive thrust. Shear compression failure also occurred where the inclined cracks reached the top of the beam under the loading point. In this type of failure the web reinforcement acts to restrain the opening of the inclined cracks and distribute the forces over a larger area.

The loads which caused flexure-cracking were found to correlate closely with the flexural cracking load near its point of origin. Web-shear cracking loads could be found by using an uncracked section analysis. These loads were increased for beams with web reinforcement and in general it was found that stirrups increased the shear
capacity of prestressed concrete beams. The web reinforcement was also found to increase the overall strength and ductility of the beams.

2.7 Saqan and Frosch (2009)

The shear strength of concrete is dependent on many factors such as concrete shear strength, the shear contribution of the prestressing steel and mild steel reinforcement. This research focused on the contribution of the flexural reinforcement with respect to the overall shear capacity of prestressed-concrete beams. Nine beams with varying amounts of mild reinforcement were tested to determine their effect on the shear capacity. All of the tested beams had the same prestressing force with identical cross-sectional dimensions (14 x 28 in. [336 x 711 mm]) and concrete strengths.

The testing was divided into three series, each with three beams for experimentation. Every series contained one beam with only prestressing strands and the other two beams had different quantities of mild steel reinforcement, as noted in the research, in addition to the prestressing strands. The prestressing strands were ASTM416, 0.5 inch (12 mm) seven wire Grade 270 low-relaxation prestressing strands. The mild steel used was ASTM A615, Grade 60 reinforcing bars. The concrete was specified as ASTM C150, Type I with nominal design strength of 6000 psi (41 MPa). The beams were tested as simply supported with a concentrated load applied at mid-span. The load was applied in 5 kip increments up to the calculated cracking load after which 2 kip load increments were used. A load cell was used to measure the load and LVDTs
were used at mid-span and at the supports to measure deflections. Strains were measured in the prestressing strands as well as in the mild reinforcement at mid-span by means of strain gauges.

The authors found that beams with mild reinforcement were much stiffer and the overall behavior of the beams was similar to that of a tied arch. All of the beams failed in shear-compression with the failure surface as the primary flexure-shear crack. For beams with only prestressing strand reinforcement the failure was more violent. It was also noted that by increasing the cross-sectional area of the prestressing steel the shear capacity also increased. Adding mild reinforcement (for larger moment capacity) increases the shear strength of the prestressed member.
CHAPTER 3
EXPERIMENTAL TESTING PROGRAM AND RESULTS

3.1 Introduction

Eight precast, prestressed concrete girders were experimentally tested for this research. Two different tests were performed on the salvaged girders. The first test was to determine the effective prestress force that remained in the girder after more than 40 years of service, and subsequently the second test was performed to determine the ultimate shear capacity at the end of the girder. Details about these two tests are presented in this chapter. Section 3.2 describes the test setup and all of the preparatory work completed prior to the experimental testing. Section 3.3 summarizes the determination of the effective prestress force. Section 3.4 details the shear tests. Section 3.5 presents the shear test results.

3.2 Test Setup

In order to apply the necessary external loads to the AASHTO girders, a steel reaction frame was designed and constructed. The completion of a new structural testing facility at Utah State University (USU) was finalized just prior to the commencement of this research. The Systems Materials and Structural Health Lab (SMASH Lab) at USU contains a strong floor which provided the means to anchor the reaction frame and develop the required external loads. A steel reaction frame was designed to be used in conjunction with this strong floor. The reaction frame was
designed to maximize the width of the strong floor and had a capacity of 1,000,000 lbs with a live load factoring of 1.6L according to American Institute of Steel Construction (AISC) manual (AISC, 2007).

The reaction frame consists of two I-shaped columns attached to stiffened base plates that can be anchored to the floor. The columns support a stiffened I-beam that can be bolted to the flanges of the columns as shown in Figure 3.1. The cross beam was designed so that a hydraulic ram could be attached to the bottom flange of the beam and easily positioned anywhere along the length of the beam according to the test requirements. Once the reaction frame was designed and fabricated, it was delivered to the USU SMASH Lab and installed on the strong floor by means of eight 2.5 inch diameter threaded rods which held the base plates of the columns to the strong floor. Figure 3.2 shows a closeup of the base plate and Figure 3.3 shows the connection of the beam to the column.

Eight American Association of State Highway and Transportation Officials (AASHTO) Type II girders were procured for this testing. The first six were shorter in length and the last two girders were longer and from a different bridge. All girders had the same cross-sectional dimensions as shown in Figure 3.4. A portion of the bridge deck was left over the top flange of each girder and the resultant structural properties are listed in Table A.1.
Figure 3.1 3-D view of reaction frame CAD model.

Figure 3.2 Base plate of reaction frame.
Figure 3.3 Beam to column connection.

Figure 3.4 AASHTO Type II girder dimensions.
Girders 1 through 6 were salvaged from Interstate 215 (I-215) in Salt Lake City, Utah at 45th South. The bridge was built in 1968 as a four span bridge with span lengths of 23-ft, 74.5-ft, 74.5-ft, and 67-ft shown in Figure 3.5. Figure 3.6 shows a picture of the bridge as it was being torn down showing the bridge’s in-service state. Figure 3.5 also shows that the bridge had a change of elevation of about 43 feet from one end to the other (for detailed bridge plans see Appendix A Figures A.3 through A.25). This slope caused the majority of the water and snow to run down into the expansion joint on the downward slope of the bridge causing degradation of the prestressed concrete girders due to corrosion of the steel reinforcements. It was the concern of UDOT that the deterioration that had occurred had reduced the shear capacity of the girders. Because they had several other bridges in similar states, UDOT was interested in evaluating the capacity of the girders. Each girder used in this testing was from Span 1 of this bridge where the most degradation had occurred. The center-to-center of bearing span length of these six girders was 22 feet 3 inches with an outside to outside dimension of 23 feet 7 inches. The girders were spaced at 9 feet on the bridge. The girders were made composite with an 8-in reinforced concrete cast-in-place deck. When the girders were delivered to USU, a portion of that deck was still intact. To prepare the girders for testing, the decking was squared up to provide a more uniform specimen (Figure 3.7).
Figure 3.5 Plan and profile view of bridge over I-215.
Figure 3.6 Bridge as it was being torn down.

Figure 3.7 Girders before testing.
Each of the girders was reinforced for shear with No. 4 bars used as stirrups. The first stirrup was placed 7.5 inches from the center of the bearing and then 23 inches on center afterwards. The bridge plans specified that “all reinforcing steel shall be intermediate grade billet steel conforming with AASHO designation M-31. Deformations shall conform with AASHO designation M-137” (Utah State Department of Highways Structural Division, 1967). Intermediate grade billet steel was specified as 33-ksi according to the state of practice up to the 1970s. A sample of shear reinforcing steel was removed from the girder and tested using a Tinius Olsen universal testing machine and the yield strength of the web steel was verified as being 33.4-ksi. Figure 3.8 shows the stress-strain curve for the web steel that was tested.

![Stress-strain curve](image)

**Figure 3.8** Stress-strain relationship for web shear steel.
The prestressing force after losses, according to the bridge plans, was specified as 176 kips at an eccentricity of 9 inches as measured from the bottom of the girder. The concrete compressive strength of the girder at transfer ($f'_{ci}$) was specified as 4,000-psi. Two concrete samples were removed from the control girder and tested in compression to determine the actual compressive strength according to ASTM standards. These tests yielded an average concrete compressive strength ($f'_{c}$) of 7,100-psi. A split-cylinder test was also conducted following ASTM C496-86 guidelines to determine that the concrete tensile strength ($f_{t}$) was 590-psi for these girders. This relationship resulted in the tensile capacity being equal to $7.0 f'_{c}^{0.5}$ which is close to the $7.5 f'_{c}^{0.5}$ reported in most codes.

The prestressing strands that were used in the girders were 7/16 inch diameter, 7-wire strands. These strands were tested in the lab to establish their ultimate stress, which was measured as 258.7-ksi. Figure 3.9 shows the stress vs. strain diagram for the prestressing strands. From these results and talking with UDOT officials it was assumed that the specified grade for the strands used in these girders was 250-ksi stress relieved strands. The plans did not give any exact criteria with respect to the grade and type of prestressing strands, but 250-ksi stress relieved strands were common during this time period. The first row of four tendons was located at 2 inches from the bottom, a second row of four tendons was located at 3.5 inches from the bottom, a third row of two tendons was located at 27 inches from the bottom, and a fourth row of two tendons was located at 28 inches from the bottom. This resulted in an eccentricity of 11 inches.
as measured from the bottom of the girder, which disagrees with the bridge plan specifications calling for a 9 inch eccentricity for the prestressing strands (Utah State Department of Highways Structural Division, 1967).

The two longer girders were also AASHTO Type II girders. Both were 34.5 feet in length. These girders were salvaged from a highway bridge in southern Utah that had been in service for about 40 years. These two girders were also used in UDOT project No. 81F15404, Determining Residual Tendon Stress in Pre-Stressed Girders.

![Stress-strain relationship for prestressing strands.](image)

Figure 3.9 Stress-strain relationship for prestressing strands.
The girders were stressed with fourteen 7/16-inch diameter, 7-wire prestressing strands which imparted a total prestressing force of 264,600 lbs onto the beams at an eccentricity of 9.46 inches as measured from the bottom of the beam. The ultimate capacity, \( f_{pu} \), of the strands was 250 ksi for these two girders. Two rows of four strands were placed 6 inches from the bottom of the girder followed by three rows of two strands. All strands were placed on a 2 inch center-to-center spacing. The residual prestress force in these longer girders was determined previously by means of a cracking test as 120,000-lbs. The compressive strength of these girders was specified as 5,000-psi, but was experimentally determined to be 9,300-psi. No information was available on the shear steel properties, but since these girders were fabricated approximately at the same time as the other six girders and detailed by the same organization (UDOT), it was assumed that the web steel was also intermediate grade steel with a yield stress of 33-ksi. In the two longer girders, the stirrups began at six inches from the center of the support and were then spaced at 17 inches throughout the length of the beam.

### 3.3 Determination of Effective Prestress Force

Even though the strands were horizontal and had no vertical component to contribute to shear, it was still of interest to determine the residual prestress force. Since the girders had been exposed to corrosive conditions during a large portion of their service lives as well as other deterioration, the prestress force was not likely to be
easily predicted. To this end, a simple test was performed to quantify the remaining prestress force after all losses had occurred during the 40 plus years of being in service.

Each beam was simply supported under the reaction frame so that a concentrated load could be applied directly at the mid-span of the girder. The external load was incrementally increased until there was a visibly clear crack across the bottom flange of the prestressed girder. Once this crack was identified, the load was held constant until the crack could be traced with a permanent marker to provide easy identification once the load was removed and the prestress force closed the crack.

After each girder had undergone this initial cracking, strain gauges were applied to each girder in three different locations with respect to the crack as shown in Figure 3.10. One 3.5 inch long foil strain gauge was placed across the crack, and two 2 inch long gauges were placed on either side of the crack. This strain gauge configuration was used to try to quantify the load that resulted in zero stress at the extreme tension fiber of the concrete. The value of the external load which resulted in the stress being equal to zero could be determined, and this value was used to calculate the effective prestress force.

Each of the girders had slightly different cross-sectional properties due to the deck portion that remained on top of the girders. Consequently, detailed section properties needed to be determined in order to accurately calculate the residual prestress force in the girders. For detailed information on each girder’s structural properties see Appendix A, Table A.1.
Once the strain gauges were applied and allowed to cure, section properties were determined, and the cracking tests were conducted. The applied load (decompression load) was recorded throughout the duration of the test. From the decompression load the prestressing force could then be calculated by means of Equation 3.1 and Equation 3.2 with careful attention given to the sign of each term.

\[
\sigma = \frac{P_e}{A_g} + \frac{Pe_{xy}y_g}{I_g} - \frac{M_{sw}y_g}{I_g} - \frac{M_{max}y_c}{I_c}
\]  

(3.1)

Equation 3.1 can be rewritten as Equation 3.2 to find the effective prestress force.

\[
P_e = \frac{M_{sw}y_g + M_{V_c}}{\frac{I_g}{I_c} + \frac{ey_g}{A_g}}
\]  

(3.2)
where:

\( \sigma \) = stress at the crack location (ksi)

\( P_e \) = the effective prestress force in the beam (kips)

\( I_g \) = moment of inertial of the girder at the crack location (in\(^4\))

\( I_c \) = moment of inertial of the composite section at the crack location (in\(^4\))

\( A_g \) = cross-sectional area at the crack location (in\(^2\))

\( e \) = eccentricity of the prestressing force at the crack location (in.)

\( y_g \) = neutral axis location of the girder measured from the bottom of the beam at crack location (in.)

\( y_c \) = neutral axis location of the composite section measured from the bottom of the beam at crack location (in.)

\( M \) = the total moment at the crack location (kip-in.)

\[
M = \frac{M_{\text{max}}}{L} x + M_{\text{sw}} \quad (3.3)
\]

\( x \) = distance from the crack to the nearest support (in.)

\( L \) = distance between supports (in.)

\( M_{\text{sw}} \) = moment at crack location due to self weight of the girder assuming a unit weight of .155 kip/ft\(^3\) (kip-in.)

\[
M_{\text{sw}} = \frac{1.55}{1728} A x^2 \quad (3.4)
\]

\( M_{\text{max}} \) = maximum moment in the beam due to externally applied load (kip-in.)

\[
M_{\text{max}} = \frac{P_a L}{4} \quad (3.5)
\]
\( P_a = \text{externally applied load (kips)} \)

A Vishay system 5000 data acquisition system was used to record the externally applied load measured with a Goekon strain gauge based load cell. Deflections were also measured using an LVDT placed on the top of the beam adjacent to the load cell. Three channels of strains were recorded from the strain gauges that were applied to each tested girder as described above. Measurements were recorded at a sampling rate of 10 hertz throughout each test. Tests were performed up to two different load steps. The first load was 70,000-lbs and the second was 80,000-lbs. This was done to ensure that the crack had sufficient dilation yet not enough dilation to destroy the strain gauges.

Once the test for each girder had concluded, the data was plotted as load vs. micro strain from the three strain gauges as well as plots of load vs. deflection. After analyzing the data, it was determined that the gauges that were placed directly over the crack showed the clearest point at which nonlinear behavior began. This determination came from examining others’ research findings, and also observing that the collected data was most consistent from the gauges that were placed over the crack. The strain gauges placed to either side of the crack were expected to produce a bilinear response when plotted versus load, but no such response was observed. The load vs. deflection theoretically would have produced a similar bilinear response as the crack opened to that of load vs. strain, but the LVDT that was available at the time of testing did not read with the precision needed to determine the magnitude of load at which the crack
opened, and therefore that data was not used in determining the decompression load.

Typical plots of load vs. micro strain are shown in Figure 3.11 for each of the three
gauge positions. The plots for the other girders are shown in Appendix B. The graphs
produced from the gauges located to the right and left of the crack were more difficult
to identify the exact load at which the crack opened up, and therefore the gauges over
the crack were consistently used to determine the decompression load.

The experimentally determined decompression load was obtained by fitting a
straight line to the initially “straight” portion of the load vs. strain plot produced from
the strain gauges placed directly over the crack. The decompression load was defined as
the load at which the strain enters the nonlinear portion and deviates from this initially
straight line. As illustrated in Figure 3.12 the decompression load was 30,500-lbs for
Girder 3. This definition comes from the reasoning that as soon as the externally
applied load reaches a magnitude large enough to overcome the prestressing force, the
section would crack and assume a different moment of inertia. With this changing
moment of inertia the load vs. strain relationship would become nonlinear as the
externally applied load was increased. Once the section was fully cracked, the load vs.
strain continues on a “straight” line, but with a different slope having a fully cracked
moment of inertia. The portion of the plot between the two “straight” lines is where
there is still aggregate interlock and the section properties (moment of inertia) were
changing with external load. For these reasons, the decompression load was taken as
the point where the load vs. strain behavior initially changes.
Figure 3.11 Strain gauges from girder 2 showing an example of the data recorded during the decompression tests.
The experimentally determined decompression load was obtained by fitting a straight line to the initially “straight” portion of the load vs. strain plot produced from the strain gauges placed directly over the crack. The decompression load was defined as the load at which the strain enters the nonlinear portion and deviates from this initially straight line. As illustrated in Figure 3.12 the decompression load was 30,500-lbs for Girder 3. This definition comes from the reasoning that as soon as the externally applied load reaches a magnitude large enough to overcome the prestressing force, the section would crack and assume a different moment of inertia. With this changing moment of inertia the load vs. strain relationship would become nonlinear as the externally applied load was increased. Once the section was fully cracked, the load vs. strain continues on a “straight” line, but with a different slope having a fully cracked moment of inertia. The portion of the plot between the two “straight” lines is where there is still aggregate interlock and the section properties (moment of inertia) were changing with external load. For these reasons, the decompression load was taken as the point where the load vs. strain behavior initially changes.

Table 3.1 lists each of the six girders’ externally applied loads with their corresponding calculated decompression load as determined using Equation 3.2. Table A.2 shows all of the calculations used to determine the effective prestress force in the girders. The first load listed comes from the initial test in which an externally applied load of 70,000-lbs was reached before the test was concluded, and the second load listed comes from the test in which an externally applied load of 80,000-lbs was reached
before the test was concluded. The average calculated residual prestress force shown in the fourth column is the force which was considered as the residual prestressing force in the girders. The bridge plans specified a prestress force after all losses of 176-kips.

Figure 3.12 Typical load vs. micro strain plot (Girder 3 shown with an 80,000-lb limit).
Table 3.1 Girder decompression loads with corresponding residual prestress force

<table>
<thead>
<tr>
<th>Beam</th>
<th>Decomp. Load (kips)</th>
<th>Residual Prestressing Force (kips)</th>
<th>Average Residual Prestressing Force (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>38.5</td>
<td>175.2</td>
<td>172.3</td>
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<td>1</td>
<td>37.0</td>
<td>169.4</td>
<td></td>
</tr>
<tr>
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<td>35.5</td>
<td>161.2</td>
<td>158.3</td>
</tr>
<tr>
<td>2</td>
<td>34.0</td>
<td>155.3</td>
<td></td>
</tr>
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<td>30.5</td>
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<td>145.3</td>
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<td>6</td>
<td>38.0</td>
<td>170.9</td>
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3.4 Shear Tests

After the conclusion of the cracking test to determine the effective prestress force, shear tests were performed on each end of the eight girders. This section will focus on the shear behavior of the girders with an emphasis on the ultimate shear capacities. Of the eight girders tested, two were tested in an unaltered condition for a total of four shear tests. The other six girders were retrofitted with carbon fiber reinforced polymer at the ends. These beams were tested in shear with the goal of quantifying the additional shear capacity that the carbon fiber wraps imparted to the girders. For further information on the carbon fiber reinforcement see Petty (2010).

Prior to testing, each girder was fitted with a high strength grout pad on the top of the deck at a distance equal to the depth of the beam (not including the deck) plus
one foot (d+1-ft). This was done to provide a flat surface for the hydraulic ram to react against as illustrated in Figure 3.13. The grout pad was not considered to provide any structural integrity and was not taken into account while determining the section properties of the girders (Table A.1).

During testing, all girders were simply supported with varying span lengths as listed in Table 3.2. The spans varied due to the fact that both ends were tested independently. Once one end had been tested through failure, it became necessary to place the support under a portion of the beam where the cross-section was still intact. The shear spans were kept close to a constant distance of 48 inches on the ends with the exception of Girder 6. Girder six was initially tested with a 48-inch shear span, but cracking occurred in the middle of the girder rather than at the end. This failure behavior was produced by the carbon fiber reinforcement providing an increase in shear strength on the end, which was more than the shear strength in the middle of the beam.

Figure 3.13  Grout pad fitted onto girders.
where the shear force was lower. In order to ensure that failure occurred at the desired location, the shear span was decreased slightly, and the expected failure was observed. Girders 7 and 8 had varying shear spans due to space limitations in the lab, however, the desired shear failures were still observed. In Table 3.2 the A or B denotes which end was tested according to the markings on the beams. Top or Bottom denotes the orientation of the beam when it was in service (e.g. top refers to the up-slope end of the girder).

The externally applied load was measured during the shear tests in two ways. The first was by means of a pressure transducer in line with the hydraulic ram, and the second was with a Geokon strain gauge based load cell. To calculate the load from the pressure transducer, the pressure (psi) was multiplied by the bore area of the ram (in$^2$) to get the load in pounds.

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<th>Span Length (inches)</th>
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</tr>
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</tr>
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<td>42.00</td>
</tr>
<tr>
<td>7L A</td>
<td>199.50</td>
<td>51.50</td>
</tr>
<tr>
<td>7L B</td>
<td>163.00</td>
<td>48.00</td>
</tr>
<tr>
<td>8L A</td>
<td>150.5</td>
<td>48.5</td>
</tr>
<tr>
<td>8L B</td>
<td>196.00</td>
<td>49.00</td>
</tr>
</tbody>
</table>
Because the failure load was the most important criterion considered in this testing, the redundancy was desirable to verify results. Deflections were also measured with an LVDT placed on top of the girder next to the load cell. Strains were also measured in various locations on the girder. The strain gauges were placed strategically so that strains could be measured in line with the fibers of the carbon fiber wraps (for details see Petty, 2010). Figure 3.14 shows a typical shear test setup.
Once the setup was completed and the span lengths were recorded, the load tests were performed. A hydraulic ram was used to gradually apply the increasing load up to and through failure. The applied load was continuously monitored and recorded during the test by means of an online display in the Vishay software which was used in conjunction with the System 5000 data acquisition system. Once the applied maximum load had dropped off significantly, the test was terminated by completely removing the load, after which the girder and data were examined. Table 3.3 shows the ultimate shear capacities from each test.

The ultimate shear capacity was determined using simple statics. The ultimate load was determined as the maximum recorded load from the data. This load was used along with Equation 3.6 to calculate the ultimate shear force. For the completed tests, the base line shear was determined as 163.6-kips for Girders 1 through 6 and 280.4-kips for Girders 7 and 8 from the average shear of the two tests on the control girder (Girders 1 and 7). In Petty (2010) the ultimate shear capacities from Girders 2 through 6 are compared with a baseline ultimate shear capacity of 163.6-kips and Girder 8 was compared to a baseline ultimate shear of 280.4-kips to determine the increase in shear capacity that resulted from the application of the carbon fiber wraps.
Table 3.3 Ultimate shear capacity of each girder

<table>
<thead>
<tr>
<th>Beam</th>
<th>Shear (kips)</th>
<th>Beam</th>
<th>Shear (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A(Bottom)</td>
<td>150.25</td>
<td>5A (Top)</td>
<td>225.07</td>
</tr>
<tr>
<td>1B(Top)</td>
<td>176.86</td>
<td>5B (Bottom)</td>
<td>244.94</td>
</tr>
<tr>
<td>2A(Bottom)</td>
<td>178.71</td>
<td>6A (Top)</td>
<td>261.48</td>
</tr>
<tr>
<td>2B(Top)</td>
<td>162.70</td>
<td>6B (Bottom)</td>
<td>151.79</td>
</tr>
<tr>
<td>3A (Top)</td>
<td>197.02</td>
<td>7LA</td>
<td>280.87</td>
</tr>
<tr>
<td>3B (Bottom)</td>
<td>209.50</td>
<td>7LB</td>
<td>280.02</td>
</tr>
<tr>
<td>4A (Bottom)</td>
<td>179.61</td>
<td>8LA</td>
<td>311.96</td>
</tr>
<tr>
<td>4B (Top)</td>
<td>174.80</td>
<td>8LB</td>
<td>307.97</td>
</tr>
</tbody>
</table>

\[ V_u = \frac{P_u (L - L')}{L} \quad (3.6) \]

where:

\( V_u \) = ultimate shear force at failure (kips)

\( P_u \) = maximum applied load (ultimate load) (kips)

\( L \) = span length (in.)

\( L' \) = shear span (in.)

For Girder 1, the failure mechanism was flexural shear where the cracks first developed at an angle of 90-degrees from the longitudinal axis. The cracks changed direction as the shear forces dominated the flexural effects, and the cracks’ directions changed to approximately 42-degrees. The cracks then dilated until there was not enough aggregate interlock or friction to hold the girder together at which point the girder experienced a significant failure as shown in Figure 3.15.
Girder 7 underwent a web crushing failure as it was loaded through its ultimate load. The cracks started in the web and extended toward the flanges as the load increased. Once the compressive strength of the concrete was reached, the top web of the top flange crushed causing the girder to fail. Figure 3.16 shows a closeup of the top flange after it had crushed.

Figure 3.15  Girder 1 after undergoing a flexural shear failure.
3.5 Shear Test Results

The experimental results will be presented in two formats. The first will be the load vs. deflection charts, and the second will be the load vs. (micro) strain charts. Due to some equipment failures, the load vs. deflection was not recorded on all beams. Load vs. deflection was of less interest in this study and consequently was not used in the determination of the results, but nevertheless will be shown herein for comparison purposes only. The load vs. strain charts are numbered in Appendix B according to the location of the corresponding strain gauge on the control girder (e.g. Gauge 2A.1 will correspond to strain gauge on beam 2 side A at location 1). Figure 3.17 shows the control beam ends with their respective strain gauge locations and numbers.
Each beam failed in a repeatable manner with a primary shear crack forming at an average of 42 degrees. This shear crack began at the support and moved diagonally up the girder towards the point of the applied load. The primary shear crack was accompanied by other shear cracks, but smaller in size. The accompanying cracks were generally parallel to the primary shear crack. The behavior of the beams with the carbon fiber failed in a slightly different way, but in general the failure mode was the same.
Figure 3.18 Strain gauge on control Girder 1B.

The strain was measured at the concrete surface. By measuring the strain in the concrete, compatibility can be used to assume that the strain in the steel was the same as in the concrete. As was shown in Figure 3.8 the yield stress of the shear steel was 33 ksi. This stress occurred at a strain of .003, yet the strain in the concrete was .004, showing that the stress in the concrete at failure was at, or a little above, the yield stress of the steel and therefore verifying that the girder failed as the steel yielded.
CHAPTER 4

COMPARISON OF MEASURED AND PREDICTED RESULTS

4.1 Introduction

For this research two different predictive methods were used to compare code practices with the measured results which were provided in Chapter 3. The first methodology was the current AASHTO LRFD Bridge Design Specifications (AASHTO, 2009). This is the preferred method for most state DOTs and for the Federal Highway Administration when designing bridges. Chapter 5 of this code, which describes the shear and torsion behavior of concrete beams, was the main section utilized in this research to determine the calculated capacity.

The second predictive method was from the American Concrete Institute’s (ACI) concrete building code ACI-318-08 (ACI, 2008). This design code is for structural concrete both in buildings and otherwise. Chapter 11 of the ACI code was the main portion utilized for this research. This chapter describes the shear strength design codes as they apply to prestressed concrete girders.

4.2 Predictive Method AASHTO LRFD Bridge Design Specifications

The AASHTO LRFD Bridge Design Specifications (AASHTO, 2009) provides two different methodologies for the determination of the design shear of a reinforced prestressed concrete girder. This code summarizes the components of shear from three
different factors including the tensile strength of the concrete \( V_c \), the shear resistance provided by the transverse reinforcement \( V_s \), and the vertical component of the prestressing force \( V_p \). The nominal or total shear capacity of the girder is taken as the lesser of the two values calculated using AASHTO Equations 5.8.3.3-1 and 5.8.3.3-2 which are provided in this research as Equations 4.1 and 4.2, respectively.

\[
V_n = V_c + V_s + V_p \tag{4.1}
\]

\[
V_n = 0.25f'_c b_v d_v + V_p \tag{4.2}
\]

The shear contribution from the vertical or transverse reinforcing steel is calculated using Equation 4.3.

\[
V_s = \frac{A_v f_y d_v (\cot \theta + \cot \alpha) \sin \alpha}{s} \tag{4.3}
\]

where:

\( V_p \) = component in the direction of the applied shear of effective prestressing force (kips)

\( f'_c \) = compression strength of concrete (ksi)

\( b_v \) = effective web width taken as the minimum web width within the depth \( d_v \) (in.)

\( d_v \) = effective shear depth as determined as follows (AASHTO Article 5.8.2.9) (in.)

\[
d_v = \text{effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not be taken to be less than the greater of } 0.9d_e \text{ or } 0.72h \text{ (in.)}
\]

in which:

\[
d_v = \frac{M_n}{A_s f_y + A_{ps} f_{ps}} \tag{4.4}
\]
\[ d_v = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y} \]  

(4.5)

\( s \) = spacing of the transverse reinforcement

\( A_v \) = area of transverse steel within a distance \( s \) (in\(^2\))

\( f_y \) = yield strength of transverse reinforcement (ksi)

\( \theta \) = angle of inclination of diagonal compressive stresses (°)

\( \alpha \) = angle of inclination of transverse reinforcement within distance \( s \) (°)

\( M_n \) = nominal moment at the section being considered (kip-in.)

\( A_s \) = area if longitudinal steel (in\(^2\))

\( A_{ps} \) = area of prestressing steel (in\(^2\))

\( f_{ps} \) = force in the prestressing steel (kips)

\( d_p \) = depth to the centroid of the prestressing steel (in.)

\( d_s \) = depth to the centroid of the longitudinal steel (in.)

According to the AASHTO specifications, there are two different methodologies of calculating the concrete contribution to shear \( (V_c) \). The first method (general procedure) comes from a modified compression field theory and assumes that the concrete shear stresses are uniformly distributed over an area \( b_v \) in width and \( d_v \) in depth. For this method, it is also assumed that the directions of the principal compressive stresses \( (\theta) \) remain constant over a length \( d_v \), and that the shear strength of the section can be determined by considering the biaxial stress conditions at just one location in the web. AASHTO Equation 5.8.3.3-3 provides the relationship for calculating the magnitude of \( V_c \) and is provided in this research as Equation 4.6. The values of \( \beta \) and
\( \theta \) are determined in one of two ways, namely the Empirical Method and the Iterative Method. For this research, the Empirical Method was used and will be presented with the following equations.

\[ V_{c} = 0.0316 \beta \sqrt{f'_{c}} b_{c} d_{v} \] (4.6)

For sections containing at least the minimum amount of transverse reinforcement specified in AASHTO Article 5.8.2.5 the value of \( \beta \) may be determined using Equation 4.7.

\[ \beta = \frac{4.8}{1 + 750 \varepsilon_{x}} \] (4.7)

When sections do not contain at least the minimum quantity of shear reinforcement, the value of \( \beta \) should be calculated using Equation 4.8.

\[ \beta = \frac{4.8}{1 + 750 \varepsilon_{x}} \frac{51}{39 + s_{x} e} \] (4.8)

In either case, the value of \( \theta \) is calculated using Equation 4.9.

\[ \theta = 29 + 3500 \varepsilon_{x} \] (4.9)

For Equations 4.7-4.9, the strain \( \varepsilon_{x} \) is defined as the strain in the non-prestressed longitudinal tension reinforcement. The code provides a simplified equation which may be used rather than performing a more detailed and involved analysis. This simplified equation provided as AASHTO Equation 5.8.3.4.2-4 provided in this research as Equation 4.10 to determine \( \varepsilon_{x} \).

\[ \varepsilon_{x} = \frac{M_{u}}{d_{v}} + 0.5N_{u} + |V_{u} - V_{e}| \frac{A_{ps} f_{p0}}{E_{s} A_{s} + E_{p} A_{ps}} \] (4.10)

where:
\( s_{xe} = \text{crack spacing parameter} \)

\[
S_{xe} = \frac{S_x}{\frac{138}{a_g + 0.63}}
\]  

\( A_c = \text{area of concrete on the flexural tension side of the member (in}^2) \)

\( A_{ps} = \text{area of prestressing steel on the flexural tension side of the member (in}^2) \)

\( A_s = \text{area of nonprestressed steel on the flexural tension side of the member at the section under consideration. (in}^2) \)

\( a_g = \text{maximum aggregate size (in.)} \)

\( f_{po} = \text{a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked in difference in strain between the prestressing tendons and the surrounding concrete (ksi). For the usual levels of prestressing, a value of 0.7} f_{pu} \text{ will be appropriate for both pretensioned and post-tensioned members} \)

\( N_u = \text{factored axial force, taken as positive if tensile and negative if compressive (kips)} \)

\( M_u = \text{factored moment, not to be taken less than (}V_u-V_p)\text{d}_v \text{ (kip-in.)} \)

\( S_x = \text{the lesser of either} \ dv \text{ of the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than 0.003} b_s S_x \)

\( V_u = \text{factored shear force (kips)} \)

The second method (simplified method) for calculating \( V_c \) is very similar to the ACI method presented in the next section. In this method, the values of \( V_c \) and \( V_s \) are calculated differently based on the way the shear cracks develop, namely, flexure-shear cracking or web-shear cracking. If flexure-shear cracks control the design, the value \( V_{ci} \)
should be used as $V_c$. However, if web-shear cracks control the design, $V_{cw}$ should be used. $V_c$ is defined to be the lesser of $V_{ci}$ and $V_{cw}$. In the AASHTO code the requirements are provided in Article 5.8.3.4.3 and provided herein as follows.

$$V_{ci} = 0.02\sqrt{f'_c b_y d_y} + V_d + \frac{V_t M_{cre}}{M_{max}} \geq 0.06\sqrt{f'_c b_y d_y} \quad (4.12)$$

where:

$V_d$ = shear force at section due to unfactored dead load including both DC and DW (kips)

$V_t$ = factored shear force at section due to externally applied loads occurring simultaneously with $M_{max}$ (kips)

$M_{cre}$ = moment causing flexural cracking at section due to external loads (kip-in.)

$$M_{cre} = S_c \left(f_{cpe} + f_{cpe} - \frac{M_{dnc}}{S_{nc}}\right) \quad (4.13)$$

$M_{max}$ = maximum factored moment at section due to externally applied loads (kip-in.)

$f_{cpe}$ = compressive stress in concrete due to effective prestress forces only (after all losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

$M_{dnc}$ = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.)

$S_c$ = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in$^3$)

$S_{nc}$ = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in$^3$)
The value $V_{cw}$ is to be calculated according to Equation 4.14. The component of shear resistance provided by the transverse steel shall be computed via Equation 4.3 with $\cot \theta = 1.0$ where $V_{ci} < V_{cw}$, and $\cot \theta = 1.0 + 3 \left( \frac{f_{pc}}{\sqrt{f_c^*}} \right) \leq 1.8$ where $V_{ci} > V_{cw}$.

$$V_{cw} = \left( 0.06 \sqrt{f_c^*} + 0.30 f_{pc} \right) b_v d_v + V_p$$

(4.14)

where:

- $f_{pc} = $ compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange (ksi). In a composite member, $f_{pc}$ is the resultant compressive stress at the centroid of the composite section, or at junction of web and flange, due to both prestress and moments resisted by precast member acting alone.

For Girders 1 through 6 the calculated shear capacity according to the AASHTO general procedure was calculated to be 47.79-kips. Using the simplified method, a shear value of 82.27-kips was calculated. For Girders 7 and 8 the calculated capacities for shear were calculated as 37.66-kips and 100.28-kips from the general and simplified procedures respectively.

Of the two methods provided by the AASHTO specifications, the simplified method provided a closer estimate of the ultimate shear capacity of the AASHTO Type II bridge girders tested for this research. In general, both methods are very conservative and rely on bending theory. The bending theory is believed to be less correct in the d-
regions (a distance equal to the depth of the girder from the face of the support) of concrete beams, especially thin webbed beams. In the d-regions, the shear stresses are not distributed linearly over the depth of the beam according to bending theory, and therefore St. Venant’s Principle does not apply. The AASHTO LRFD code, as well as the ACI code, allow for sectional analysis of beams. This is acceptable because the predicted values are conservative. As will be shown in the following section, the ACI 318-08 code for shear calculations outside the d-region is also derived from bending theory and accordingly the predicted shear is also conservative.

4.3 Predictive Method ACI 318-08

The ACI code presents two different methods for computing the ultimate shear capacity of prestressed concrete members. The first is described as the approximate method which estimates the contribution of shear strength from the concrete to be a function of the girder shape, applied loads, and concrete strengths. This method can only be used in prestressed members if the effective prestress force is equal to or greater than 40% of the tensile strength of the flexural reinforcement. The nominal shear capacity of a prestressed girder according to ACI Equation 11-9 is provided here as Equation 4.15.

\[ V_c = \left( 0.6 \lambda \sqrt{f_c^*} + \frac{700V_{ud}d_p}{M_u} \right) b_w d \]  

(4.15)

This value must be greater than or equal to Equation 4.16,

\[ V_c = 2\lambda \sqrt{f_c^*} b_w d \]  

(4.16)
but must be less than Equation 4.17.

\[ V_c = 5\lambda \sqrt{f'c b_w d} \]  \hspace{1cm} (4.18)

where:

\( \lambda \) = unit weight of concrete modification factor (1 for normal weight concrete)

\( V_u \) = the maximum design shear at the section being considered (kips)

\( M_u \) = the design moment at the same section occurring simultaneously with \( V_u \) (kip-in.)

\( d_p \) = the distance from the extreme compression fiber to the centroid of the prestressing strands (in.)

\( d \) = the distance from the extreme compression fiber to the centroid of the tension reinforcement (in.)

\( f'c \) = the compressive stress of the concrete (psi)

\( b_w \) = web width (in.)

The contribution of shear from the web shear reinforcing steel must be added to the shear contribution of the concrete. ACI Equation 11-15 is recommended to be used to calculate the shear contribution from the stirrups and is provided here as Equation 4.19.

\[ V_s = \frac{A_sf_yd}{s} \]  \hspace{1cm} (4.19)

where:

\( V_s \) = the shear resistance provided by the transverse shear steel (kips)

\( A_s \) = the area of transverse steel (in\(^2\))

\( f_y \) = the yield strength of the transverse (ksi)
The shear capacities were calculated using this approximate method for Girders 1 through 6 as 101.74-kips. This value is approximately 62% of the average measured value of 163.56-kips. For Girders 7 and 8 the approximate method resulted in a calculated shear capacity of 131.09-kips. For Girders 7 and 8 the approximate method underestimated the measured value, yielding only about 50% of the average measured value of 261.50-kips.

The second method recommended by the ACI code is the detailed method in which $V_c$ is taken as the smaller of the calculated values of $V_{ci}$ of $V_{cw}$. This method may be used for any beam, and must be used when the effective prestress force is less than 40% of the tensile strength of the flexural reinforcement. The term $V_{ci}$ is used to describe the concrete shear strength of a member when the diagonal shear cracks form due to a combination of shear and moment. $V_{cw}$ is used to define the nominal concrete shear strength of a member when the diagonal cracks form due to excessive principal tensile stress in the concrete. $V_{ci}$ can be approximated with ACI Equation 11.3.3.1 which is provided as Equation 4.20.

$$V_{ci} = 0.6\lambda \sqrt{f'c} b_w d_p + V_d + \frac{V_i M_{cre}}{M_{max}} \geq 1.7\lambda \sqrt{f'c} b_w d$$  \hspace{1cm} (4.20)

where:

$V_d = \text{the shear at the section in question due to service dead load (lbs)}$

$V_i = \text{the shear that occurs simultaneously with } M_{max} \text{ (lbs)}$

$M_{cre} = \text{the cracking moment (lb-in.)}$
\[ M_{cr} = \left( \frac{l}{yat} \right) \left( 6\lambda \sqrt{f'c} + f_{pe} - f_d \right) \text{ (lb-in.)} \quad (4.21) \]

\( I = \) the moment of inertia of the section that resists the externally applied load \( (\text{in}^4) \)

\( Y_t = \) the distance from the centroidal axis of the gross section (neglecting the reinforcing) to the extreme tension fiber \( (\text{in.}) \)

\( f_{pe} = \) the compressive stress in the concrete due to prestress after all losses at the extreme fiber of the section where the applied loads cause tension \( (\text{psi}) \)

\( f_d = \) the stress due to unfactored dead load at the extreme fiber where the applied loads cause tension \( (\text{psi}) \)

The equation for \( V_{cw} \) provides the shear capacity of the concrete beam in units of pounds as derived from a rather simplified principal tension theory and is provided as Equation 4.22 which comes from ACI Equation 11-22.

\[ V_{cw} = (3.5\lambda \sqrt{f'c} + 0.3f_{pc})b_w d_p + V_p \geq 1.5\lambda \sqrt{f'c}b_w d \quad (4.22) \]

where:

\( f_{pc} = \) the calculated compressive stress in the concrete at the centroid of the section resisting the applied loads due to the effective prestress after all losses \( (\text{psi}) \)

\( V_p = \) the vertical component of the effective prestress force at the section of interest \( (\text{lb}) \)

The value for \( f_{pc} \) is to be calculated at the centroid of the composite cross-section unless the centroid falls within the flange, in which case \( f_{pc} \) should be computed at the intersection of the web and the flange. ACI 11.3.3.2 states that \( V_{cw} \) may be taken as the concrete shear capacity that corresponds to a multiple of dead load plus live load. This
results in a calculated principal tensile stress equal to $4\sqrt{f'_{c}}$ at the point where $f_{pc}$ is calculated as described above.

For the detailed method, the total shear in a prestressed concrete member must be the sum of the shear contributions from the concrete, the vertical component of the prestressing, and the shear contribution from the web steel. If the effective prestress force is greater than or equal to 40% of the tensile strength of the flexural reinforcement Equation 4.23 (ACI Equation 11-14) is to be used to calculate the required area of shear steel, $A_{v}$.

$$A_{v,m} = \left(\frac{A_{ps}}{80}\right) \left(\frac{f_{pu}}{f_{yt}}\right) \left(\frac{s}{d}\right) \sqrt{\frac{d}{b_{w}}}$$  \hspace{1cm} (4.23)

where:

$A_{v,m}$ = the minimum area of shear steel (in$^2$)

$A_{ps}$ = the area of prestressing reinforcement in the tensile zone (in$^2$)

$f_{pu}$ = the ultimate stress of the prestressing reinforcement (psi)

$f_{yt}$ = the yield strength of the mild steel tension reinforcement (psi)

The detailed method provided a computed shear capacity of 90.98-kips for Girders 1 through 6, and 136.75-kips for Girders 7 and 8. These values are only 55.62% and 52.29% of the average measured values for Girders 1 through 6 and Girders 7 and 8 respectively.

As was described previously for the AASHTO method, the ACI design equations were developed using bending theory which has been found to be less accurate in the d-
regions of a prestressed concrete beam, and thus provides very conservative values for shear in this region.

4.4 Predictive Method Strut-and-Tie Model

Concrete girders can be divided up into B-regions and D-regions. The B-regions are regions in which Bernoulli bending theory applies. In B-regions it is assumed that strains are distributed linearly through the depth of the girder. D-regions are discontinuity or disturbed regions as defined by St. Venant’s Principle. Since the design code equations were developed based on bending theory another method needed to be examined which would better describe the types of failures observed in this research. One such method is called the strut-and-tie model. Both the AASHTO and the ACI design codes allow for a strut-and-tie model (STM) to be used in the design of prestressed concrete girders when the critical section is located within a D-region. The governing equations and recommendations on how to apply the STM are found in the appendices of each of the two design codes. STMs are rarely used in design of new girders, but this model does prove very useful in design as well as analysis of prestressed concrete girders. Analysis of concrete girders for shear in the D-region is easily and accurately handled by the strut-and-tie model.

The strut-and-tie model is an idealized model of a girder consisting of struts which are compression members made of concrete parallel to the expected cracks, ties or stirrups which are tension members made of steel analogous to the reinforcement, and nodes made of concrete which are connecting members.
Various types of struts may be used depending on the application. The most common struts are rectangles (prisms), bottles, and fans. The different shapes assume a distribution of the forces corresponding to the shape, and represent compressive stress fields. The compressive stresses act parallel to the longitudinal axis of the strut, causing transverse tension in the strut which can lead to failure.

The node sizes are determined by the bearing area of the load, the bearing area of the support, and the prism of concrete surrounding the tie. Nodes are sections of concrete which connect the strut to the ties. The nodes are idealized as pinned joints. The concrete in and surrounding the node is referred to as the nodal zone. There are three or more forces planer forces that all act through the node and satisfy equilibrium.

Ties act as the reinforcement, whether that is a single layer, or several different layers of reinforcement. The axis of the tie must coincide with the axis of the reinforcement. In a STM the tie consists of the reinforcement plus a prism of concrete concentric with the longitudinal reinforcement making up the width of the tie (Wright and MacGregor, 2009). The node dimensions are developed from the concrete surrounding the tie, which do not carry any load, but aid in transferring the loads.

For this research a strut-and-tie model was developed using two struts and one tie connected at three nodes. This configuration formed a simple triangular truss which was analyzed to obtain ultimate shear values of 138.56-kips and 258.7-kips for Girders 1 through 6 and for Girders 7 and 8, respectively. Some sample calculations are included in Appendix A, pages 89-92.
4.5 Comparison of Calculated to Measured Shear Capacities

The AASHTO and ACI procedures for calculating the shear capacities of prestressed concrete girders are both based on bending theory, and St. Venant’s Principle. This implies that it is assumed that the shear stresses are distributed linearly through the depth of the beam as long as the load is applied at a distance larger than the depth of the beam or outside the D-region. The eight beams that were tested for this research were tested in the D-region where the shear stresses were believed to not be evenly distributed through the depth of the beam. Having the load applied in the D-region causes the stresses in the girder to be concentrated in some regions, and almost non-existent in other areas due to arching action of the beam. The design codes examined in this research did not take into account the effects of unevenly distributed shear stresses through the depth of the beam. The codes allow the sectional analysis to be done along the length of the girder because the values computed using the design equations are conservative.

The strut-and-tie model used for this research was able to much more accurately predict the ultimate shear capacity of the girders. The strut-and-tie model was very simple while still yielding good results. When analysis is to be done, the STM is far better at predicting the actual strength in the D-regions of reinforced concrete beams.
Table 4.1 Calculated shear values

<table>
<thead>
<tr>
<th>Method</th>
<th>Girders 1-6 Shear (kips)</th>
<th>Percentage of Measured</th>
<th>Girders 7-8 Shear (kips)</th>
<th>Percentage of Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO General</td>
<td>47.79</td>
<td>29.22%</td>
<td>37.66</td>
<td>14.40%</td>
</tr>
<tr>
<td>AASHTO Simplified</td>
<td>82.27</td>
<td>50.30%</td>
<td>100.28</td>
<td>38.35%</td>
</tr>
<tr>
<td>ACI Simplified</td>
<td>101.74</td>
<td>62.20%</td>
<td>131.09</td>
<td>50.13%</td>
</tr>
<tr>
<td>ACI Detailed</td>
<td>90.98</td>
<td>55.62%</td>
<td>136.75</td>
<td>52.29%</td>
</tr>
<tr>
<td>Strut-and-Tie</td>
<td>138.56</td>
<td>84.72%</td>
<td>258.7</td>
<td>98.93%</td>
</tr>
<tr>
<td>Measured Value</td>
<td>163.56</td>
<td></td>
<td>261.5</td>
<td></td>
</tr>
</tbody>
</table>

The results calculated for this research are presented in Table 4.1 showing the predicted values as calculated using the methods presented above. The calculated values are also compared to the measured values as a percentage of the measured values.

4.6 ASHTO LRFD Predicted Prestress Losses

The AASHTO LRFD bridge design code provides two different methods for predicting the prestress losses in a prestressed concrete girder (AASHTO, 2009). The first is classified as the approximate method which can be used with gross section properties or transformed section properties. The second method is classified as the detailed method with transformed section properties. This section will provide both methods and present the values calculated from each method.

The approximate method, using transformed section properties, automatically takes into account the elastic shortening loss of prestress due to introduction of prestress to the concrete member, as well as any instantaneous gain due to the application of gravity loads. The long-term prestress losses are assumed to be the
negative prestress ($\Delta f_{pLT}$ in ksi) as calculated from Equation 4.24 multiplied by the area of the prestressing tendons ($A_{ps}$ in $\text{in}^2$). These long-term losses include losses that result from creep, shrinkage of the concrete, and relaxation of the prestressing steel.

$$\Delta f_{pLT} = 10.0 \frac{f_{pl} A_{ps}}{A_g} \gamma_h \gamma_{st} + 12.0 \gamma_h \gamma_{st} + \Delta f_{pR}$$ (4.24)

$$\gamma_h = 1.7 - 0.01H$$ (4.25)

$$\gamma_{st} = \frac{5}{(1 + f'_{cl})}$$ (4.26)

where:

$f_{pl}$ = prestressing immediately before transfer (ksi)

$A_g$ = gross area ($\text{in}^2$)

$H$ = average ambient humidity as a percent

$f'_{cl}$ = specified initial concrete compressive strength (ksi)

$A_{ps}$ = the total area of the prestressing steel ($\text{in}^2$)

$\Delta f_{pR}$ = an estimate of relaxation loss taken as 2.4 for low relaxation strand, 10.0 for stress relieved strand, and in accordance with manufactures recommendations for other types of strand (ksi)

This loss in prestress is applied at the centroid of the prestressing steel area and the transformed concrete section resulting in a prestress force at service. Using the effective prestress force at service, stresses can be easily calculated using the transformed section properties and compared against the design stress limits.

The detailed method is much more involved, but relatively easy to apply. This method entails calculating creep and shrinkage material properties independently.
Equation 4.27 should be used to calculate the elastic shortening loss and Equation 4.28 should be used to calculate the prestress losses from the detailed method. The subscript “id” is used to denote losses before the deck is made composite, and the subscript “df” is used to denote losses that occur after the deck has been made composite until the final time. The bottom fiber stress and the stress at the centroid of the steel can be calculated at different stages of construction. The elastic shortening loss due to initial prestress force and the girder self weight is automatically accounted for, as described above in the simplified method, if transformed section properties are used. Using all of the correction coefficients and the different factors, the prestress loss was calculated as 179.75- kips shown in table 4.2.

\[
\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp}
\]  

(4.27)

where:

\(E_p\) = modulus of elasticity of prestressing steel (ksi)

\(E_{ct}\) = modulus of elasticity of concrete at transfer or time of load application (ksi)

\(f_{cgp}\) = the concrete stress at the center of gravity of the prestressing strands due to prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi)

\[
\Delta f_{pLT} = (\Delta f_{PSR} + \Delta f_{PCR} + \Delta f_{pR1})_{ld} + (\Delta f_{PSD} + \Delta f_{PCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}
\]  

(4.28)

where:

\(\Delta f_{PSR}\) = prestress loss due to shrinkage of girder concrete between transfer and deck placement (ksi)
\[ \Delta f_{pCR} = \text{prestress loss due to creep of girder concrete between transfer and deck placement (ksi)} \]

\[ \Delta f_{pR1} = \text{prestress loss due to relaxation of prestressing strands between time of transfer and deck placement (ksi)} \]

\[ \Delta f_{pR2} = \text{prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time (ksi)} \]

\[ \Delta f_{psD} = \text{prestress loss due to shrinkage of girder concrete between time of deck placement and final time (ksi)} \]

\[ \Delta f_{pCD} = \text{prestress loss due to creep of girder concrete between time of deck placement and final time (ksi)} \]

\[ \Delta f_{pSS} = \text{prestress gain due to shrinkage of deck in composite section (ksi)} \]

### 4.7 Comparison of Measured and Calculated Prestress Losses

The effective prestress force which remained in Girders 1 through 6 was calculated as 188.42-kips using the AASHTO LRFD simplified method. Using the simplified method the effective prestress force was 114% of the average measured prestress force in the girders. When the effective prestress force was calculated using the AASHTO LRFD detailed method a value of 179.75-kips was found. This was 109% of the average measured prestress force from the cracking tests. Table 4.3 shows four different effective prestress forces. All of the effective prestress forces were compared to the average measured prestress force in Girders 1 through 6. These girders were
subjected to severe corrosion during their service life. This corrosion was believed to contribute to the lower than expected effective prestress force. The maximum difference was 23.42-kips, with the bridge plan specifications being the closest at a difference of only 11-kips.

Table 4.2 Factors used with AASHTO Detailed Method for $P_e$

<table>
<thead>
<tr>
<th>Factor</th>
<th>Value</th>
<th>Change in Concrete Stress at the Level of Prestressing Strands:</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_g$</td>
<td>1.17</td>
<td>$\Delta f_{cd} =$ 0.0924415 ksi</td>
</tr>
<tr>
<td>$K_{cd}$</td>
<td>0.55</td>
<td>$\Delta f_{pc} =$ 0.7855794 ksi</td>
</tr>
<tr>
<td>$K_{tr}$</td>
<td>1.00</td>
<td>$\Delta f_{cb} =$ -0.1191112 kips</td>
</tr>
<tr>
<td>$K_{hs}$</td>
<td>1.21</td>
<td>$\Delta f_{cb} =$ 3.293183</td>
</tr>
<tr>
<td>$\varepsilon_{bid}$</td>
<td>0.000321</td>
<td>$\Delta p =$ -29.183121 kips</td>
</tr>
<tr>
<td>$\psi_{bid}$</td>
<td>1.1726337</td>
<td>$\varepsilon_{bid} =$ 0.0008718</td>
</tr>
<tr>
<td>$f_{pu}$</td>
<td>250 ksi</td>
<td>$\varepsilon_{bud} =$ 0.000551</td>
</tr>
<tr>
<td>$f'_{cl}$</td>
<td>4 ksi</td>
<td>$\varepsilon_{dad} =$ 0.0008718</td>
</tr>
<tr>
<td>$f_{pl}$</td>
<td>175 ksi</td>
<td>$K_i =$ 1.9995044</td>
</tr>
<tr>
<td>$E_p =$</td>
<td>28500 ksi</td>
<td>$K_i =$ 0.748503</td>
</tr>
<tr>
<td>$E_c$ at trnc</td>
<td>4027.56 ksi</td>
<td>$\Psi_{bd}$</td>
</tr>
<tr>
<td>$E_c$ at svc</td>
<td>4830.55 ksi</td>
<td>$\Psi_{dd}$</td>
</tr>
<tr>
<td>$\varepsilon_{bid}$</td>
<td>0.000321</td>
<td>$K_i =$ 0.9317569</td>
</tr>
<tr>
<td>$n_i =$</td>
<td>7.08</td>
<td>Shrinkage Loss:</td>
</tr>
<tr>
<td>$n_d =$</td>
<td>0.83</td>
<td>$\Delta f_{psd} =$ 14.626124 ksi</td>
</tr>
<tr>
<td>$n_{service} =$</td>
<td>5.90</td>
<td>Creep Loss Due to Initial Loads:</td>
</tr>
<tr>
<td>$A_p =$</td>
<td>1.38 in²</td>
<td>$\Delta f_{psd1} =$ 5.5542193 ksi</td>
</tr>
<tr>
<td>$P_d =$</td>
<td>241.5 kips</td>
<td>$\Delta f_{psd2} =$ -14.907152 kips</td>
</tr>
<tr>
<td>$P_r =$</td>
<td>241.5 kips</td>
<td>Relaxation Loss:</td>
</tr>
<tr>
<td>$M_s =$</td>
<td>1193.0882 kip-in</td>
<td>$\Delta f_{rel} =$ -5.89 ksi</td>
</tr>
<tr>
<td>$f_{prep} =$</td>
<td>0.8780209 ksi</td>
<td>Prestress Gain Due to Shrinkage of the deck:</td>
</tr>
<tr>
<td>$\Delta f_{rel} =$</td>
<td>6.2130983 ksi</td>
<td>$\Delta f_{cdf} =$ -0.9133993 ksi</td>
</tr>
<tr>
<td>$\Delta f_{psr} =$</td>
<td>8.4930055 ksi</td>
<td>$\Delta f_{ssp} =$ -11.982886 ksi</td>
</tr>
<tr>
<td>$\Delta f_{psr} =$</td>
<td>8.4930055 ksi</td>
<td>Total Long-term Stress change between deck placement and final time:</td>
</tr>
<tr>
<td>$\Delta f_{PCR} =$</td>
<td>6.7641834 ksi</td>
<td>$\Delta f_{pf1} =$ -12.599695 ksi</td>
</tr>
<tr>
<td>$\Delta f_{PCR} =$</td>
<td>6.7641834 ksi</td>
<td>Relaxation Loss: The change in the concrete stress at the bottom fiber of the girder due to long-term losses is:</td>
</tr>
<tr>
<td>$\Delta f_{plt2} =$</td>
<td>5.89 ksi</td>
<td>$\Delta f_{plt} =$ -17.387579 kips</td>
</tr>
<tr>
<td>$\Delta f_{plt} =$</td>
<td>21.147189 ksi</td>
<td>Total Losses:</td>
</tr>
</tbody>
</table>

$\Delta f_{cb} =$ 0.0723921 ksi
Table 4.3  Effective prestress force comparison

<table>
<thead>
<tr>
<th>Method</th>
<th>$P_e$ (kips)</th>
<th>Difference from Measured (kips)</th>
<th>Percent of Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO Simplified</td>
<td>188.42</td>
<td>23.42</td>
<td>114.19%</td>
</tr>
<tr>
<td>AASHTO Detailed</td>
<td>179.75</td>
<td>14.75</td>
<td>108.94%</td>
</tr>
<tr>
<td>Bridge Plan Specifications</td>
<td>176</td>
<td>11</td>
<td>106.67%</td>
</tr>
<tr>
<td>Cracking Test (average)</td>
<td>165</td>
<td>0</td>
<td>100.00%</td>
</tr>
</tbody>
</table>
5.1 Summary

As bridges age and deteriorate, their capacities tend to decrease and are difficult to predict. This fact coupled with larger and larger loads being applied to the nation’s bridges has lead to much research and implementation of retrofitting and strengthening of in-service bridge girders. The tendency is to reinforce bridge girders primarily in flexure. This increase in flexural capacity leads to a stiffer girder which can result in shear being the failure mode of the girder. With added load and flexure strengthening, understanding of the shear behavior becomes increasingly more important. To this end, research was conducted on two different types of AASHTO Type II reinforced prestressed concrete bridge girders that were over 40 years old to determine their existing shear capacities at the end of the girders where water had damaged them extensively. The measured values were compared against the predicted values using the AASHTO LRFD shear design code as well as the ACI-318 shear design specifications.

Two separate groups of girders were tested from two different decommissioned bridges. Girders 1 through 6 had an in-service span length of 22-ft 3-in, and Girders 7 and 8 had an in-service span length of 34.5-ft. The girders were simply supported and loaded at a distance of 48 inches (d + 1-ft) from the supports with a single point load. Each end of each girder was tested independently of the other. This caused the overall span lengths to vary from end to end; as one end was tested through failure, it became
necessary to move the corresponding support, locating it under a section of the girder which was still intact. The measured shear capacities for Girders 1 through 6 and 7 and 8 respectively were 163.56-kips and 261.50-kips.

5.2 Comparison with AASHTO LRFD

Both the General and Simplified methods provided by the AASHTO LRFD bridge design code provided conservative values of the ultimate shear capacity. The AASHTO Design specifications were developed using bending theory with the assumption that plane sections remain plane. The shear load, as tested in this research, was right at the boundary and therefore St. Venant’s Principle was not likely valid. The AASHTO specifications allow for sectional design because it is known that the values calculated for the shear capacity near the supports will be conservative. The average measured shear for Girders 1 through 6 was 163.56-kips where the General Method produced a calculated shear capacity of 47.79-kips, and the Simplified Method resulted in a calculated shear capacity of 82.27-kips. For Girders 7 and 8 the General Method resulted in a calculated shear capacity of 37.66-kips and the Simplified Method resulted in a calculated shear capacity of 100.28-kips. When tested in the lab the average shear capacity of Girders 7 and 8 was 261.50-kips.

5.3 Comparison with ACI318-08

The ACI-318 design code was developed based on bending theory assuming that plane sections remain plane. As was described for the AASHTO specifications, these
assumptions have been shown to not be valid near the supports of a girder. The ACI-318 code also allows for sectional analysis near the supports of a beam with the understanding the calculated shear capacities will be conservative near the supports, which was shown from the results of this research. For Girders 1 through 6 the ACI Simplified Method and Detailed Method produced values of the shear capacity as 101.74-kips and 90.98-kips, respectively. The average measured value was 163.56-kips. For the longer girders, Girders 7 and 8, the Simplified Method resulted in a calculated value of 131.09-kips, and the Detailed Method gave a calculated value of 136.75-kips. The average measured shear value for Girders 7 and 8 was 261.50-kips.

5.4 Comparison with the Strut-and-Tie Model

The strut-and-tie model which is not based on bending theory or St. Venant’s Principle was developed for the girders tested in this research. The model consisted of two main compression struts and a tension tie connected at the nodes. This model was very simple in nature, yet yielded much more accurate results. For Girders 1 through 6 the STM produced an ultimate shear capacity of 138.56-kips which is 84.72% of the average measured value. For Girders 7 and 8 the STM gave an ultimate shear capacity of 258.7-kips. The STM was 98.93% of the average measured value of 261.50-kips for Girders 7 and 8.
5.5 Cracking Test

Cracking tests were carried out on Girders 1 through 6 to determine the residual prestressing force in the girders. This was done by initially cracking the simply supported beams by means of a single point load applied at mid-span. Once the crack was located and marked, strain gauges were placed across and to either side of the crack on the bottom flange of the girder. The girder was then reloaded at mid-span while load and strain were recorded. The strain was then plotted vs. load and the decompression load was obtained from the response. The decompression load was used to calculate the prestressing force. The average existing prestressing force for Girders 1 through 6 was 165.0-kips.

5.6 Comparison of Prestress Losses

AASHTO prestress loss equations were used to compare against the measured values. The AASHTO LRFD specifications (AASHTO, 2009) were used as a guide for these calculations. The two methods utilized herein were the Approximate Method using transformed section properties and the Detailed Method with transformed section properties. Using the Approximate Method, the effective prestress force at service was calculated as 188.42-kips. The Detailed Method produced a calculated effective prestress force of 179.74-kips. The bridge plans specified an effective prestress force after all losses of 176.00-kips. The effective prestress force obtained from the cracking
This research investigated the near support shear capacity of prestressed reinforced concrete AASHTO Type II girders with the load applied at a distance of $d + 1$-ft. The girders failed in a typical shear manner, but the design codes did not closely predict this kind of shear failure.

Future research needs to be done to determine equations for the shear capacity at near support regions of prestressed concrete AASHTO Type girders. Such equations could prove especially useful in bridges where large forces result from live loads being located near the support.

Other types of prestressed concrete girders should be tested in a similar manner to generalize the results and types of shear failures. By testing various shapes with differing levels of prestressing and shear reinforcement, it can be shown whether or not the design codes are adequate for specific shapes or if the results are all very conservative regardless of shape. Having a better understanding of the shear failures of different shapes of girders would provide useful information as methods are developed to increase the existing shear capacities of prestressed concrete girders.
REFERENCES


Petty, David. 2010. Full-scale testing of 40 year old prestressed AASHTO girders that have been retrofitted in shear by externally applied carbon fiber reinforced polymer wraps. Unpublished MS thesis. Utah State University, Logan, Utah. 182 p.


Appendix A

Section Properties and Bridge Plans
### Table A.1 Girder section properties

<table>
<thead>
<tr>
<th>Beam</th>
<th>Area (in²)</th>
<th>e (in)</th>
<th>Y (in)</th>
<th>I (in⁴)</th>
<th>Distance to Crack (in)</th>
<th>Length of Beam (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>433.5</td>
<td>8.405</td>
<td>19.41</td>
<td>79821</td>
<td>132.5</td>
<td>268.5</td>
</tr>
<tr>
<td>2</td>
<td>429</td>
<td>8.144</td>
<td>19.14</td>
<td>77629</td>
<td>121.5</td>
<td>268.5</td>
</tr>
<tr>
<td>3</td>
<td>441</td>
<td>8.837</td>
<td>19.84</td>
<td>83539</td>
<td>124</td>
<td>268.5</td>
</tr>
<tr>
<td>4</td>
<td>400</td>
<td>6.352</td>
<td>17.35</td>
<td>63725</td>
<td>120.5</td>
<td>268.5</td>
</tr>
<tr>
<td>5</td>
<td>429</td>
<td>8.144</td>
<td>19.14</td>
<td>77629</td>
<td>124</td>
<td>268.5</td>
</tr>
<tr>
<td>6</td>
<td>429</td>
<td>8.144</td>
<td>19.14</td>
<td>77629</td>
<td>121</td>
<td>268.5</td>
</tr>
</tbody>
</table>

### Table A.2 Calculation of effective prestress force

<table>
<thead>
<tr>
<th>Beam</th>
<th>Average flexural stress (psi)</th>
<th>Area Girder (in²)</th>
<th>Area Girder (in²)</th>
<th>Young's Modulus (ksi)</th>
<th>Erection Grade</th>
<th>Erection Grade</th>
<th>Total Stress (psi)</th>
<th>Prestress Force (psi)</th>
<th>Prestress Force (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>37.5</td>
<td>396.9</td>
<td>212.8</td>
<td>10,000</td>
<td>60</td>
<td>60</td>
<td>32.2</td>
<td>26.0</td>
<td>26.0</td>
</tr>
<tr>
<td>5</td>
<td>37.5</td>
<td>396.9</td>
<td>212.8</td>
<td>10,000</td>
<td>60</td>
<td>60</td>
<td>32.2</td>
<td>26.0</td>
<td>26.0</td>
</tr>
<tr>
<td>6</td>
<td>37.5</td>
<td>396.9</td>
<td>212.8</td>
<td>10,000</td>
<td>60</td>
<td>60</td>
<td>32.2</td>
<td>26.0</td>
<td>26.0</td>
</tr>
</tbody>
</table>
Figure A.1  Bridge plans page 1 of 23.

Figure A.2  Bridge plans page 2 of 23.
Figure A.5  Bridge plans page 5 of 23.

Figure A.6  Bridge plans page 6 of 23.
Figure A.7  Bridge plans page 7 of 23.

Figure A.8  Bridge plans page 8 of 23.
Figure A.9  Bridge plans page 9 of 23.

Figure A.10  Bridge plans page 10 of 23.
Figure A.11 Bridge plans page 11 of 23.

Figure A.12 Bridge plans page 12 of 23.
Figure A.15  Bridge plans page 15 of 23.

Figure A.16  Bridge plans page 16 of 23.
Figure A.17  Bridge plans page 17 of 23.

Figure A.18  Bridge plans page 18 of 23.
Figure A.19 Bridge plans page 19 of 23.

Figure A.20 Bridge plans page 20 of 23.
Figure A.21 Bridge plans page 21 of 23.

Figure A.22 Bridge plans page 22 of 23.
Node Zone

**Node A (C-C-T Node)**

\[ f_{ce} = \frac{1.85 \times 10^{-6} \times 1000}{329} = 4.52 \text{ ksi} \]

**Node B (C-C-C Node)**

\[ f_{ce} = \frac{1.85 \times 10^{-6} \times 1000}{329} = 5.63 \text{ ksi} \]

**Node C (C-C-T Node)**

\[ f_{ce} = \frac{1.85 \times 10^{-6} \times 1000}{329} = 4.52 \text{ ksi} \]

**Moment**

\[ M = 100 k \times (100 \text{ in}) = 94210 \text{ in-lb} \] at 100 in. from Plane

\[ M = 8 \text{ ksi} \text{ lever arm} \]
At node 2 horizontal force
\[ F = 603.5 \pi (h_2) (d) = 603.5 \pi (h_2) (6 \text{ in}) = 30.41 \text{ lb (lbs)} \]

\[ M = F \times \text{lever arm} \]

915.2 kips 25.00 1.0 \( h_6 \) \( (41 - 2.8115 - \frac{h_6}{2}) \)

\[ h_6 = 7.54 \text{ in.} \]

Force in HR AC
\[ F = 603.5 \left( \frac{41.5}{2} \right) (6) = 241.0 \text{ kN} \]

(compute capacity of progressing move)

Initial angle of sway \( \alpha_{2,0} \)
\[ \alpha = \arctan \left( \frac{41 - \frac{25}{2} - 2.8115}{43 - 4 - \frac{2}{2}} \right) = 40.69^\circ \]

Therefore force in Diagonal STAB
\[ \sin 49.14^\circ = \frac{115}{F_{AB}} \]

\[ F_{AB} = 228 \text{ kN} \]

Angle of sway \( \beta_{2,0} \)
\[ \beta = \arctan \left( \frac{41 - \frac{25}{2} - 2.8115}{23.55} \right) = 8.68^\circ \]

Therefore force in diagonal STAB on \( \eta \)
\[ \sin 8.68^\circ = \frac{2\pi M}{F_{\eta}} \]

\[ F_{\eta} = 12.93 \text{ kN} \]
Check reaction of nodal zones

**Node Zone A**

Face AC (should be satisfied)

\[
F = 1.25 \frac{P}{2} A_c = 1.25 \times 400 \times 8 \times 0.8 = 211.2 \text{ kN}
\]

Face BC

\[
F = 1.25 \sigma_x A_n 
\]

With \( \sigma_x = 0 \text{ kN/m}^2 \)

\[
= 1.25 \times 12 \times 700 \times 0.2 = 309 \text{ kN}
\]

**Reaction support** \( 1 \text{ kN} \leq \frac{309 - 120}{12} \leq 1 \text{ kN} \)

Check diagonal stress

\[
F_{\text{diag}} = F \frac{A_c}{b} \text{ (without reinforcement, assuming } A_e = 0.12)
\]

\[
F_x = 1.25 \sigma_x A_n
\]

\[
= 1.25 \times 12 \times 700 \times 0.2 = 302 \text{ kN}
\]

\[
C = \frac{400 \times 1}{8} = 16 \text{ in}
\]

\[
F_{\text{ab}} = 3 \times 11.7 \times \sin(30^\circ) \times 1.8 = 212.5 \text{ kN}
\]
\[ \sin(91.49) = \frac{\text{reaction}}{\text{side}} \]

Structural force = \( \frac{143}{\sin(91.49)} \)

= 243.97 K

Reaction = \sin(46.78)\times(20.5) = 138.56 K

Using minimum reinforcement

\[ \sigma_{ce} = \frac{1.885 \times (7100)}{9 \times 5} = 4524 \text{ psi} \]

\[ F_{st} = 4526 \times (9.98) \times (6) = 265.6 K \]

Reaction = \sin(36.45) \times (345.4) = 157.3 K
Appendix B

Load vs. Strain and Load vs. Deflection Charts
Figure B. 1 Load vs. time for Girder 1.

Figure B. 2 Load vs. strain gauge 1A.13.
Figure B. 3  Load vs. strain gauge 1A.14.

Figure B. 4  Load vs. strain gauge 1A.15.
Figure B. 5 Load vs. strain gauge 1A.16.

Figure B. 6 Load vs. strain gauge 1A.17.
Figure B. 7  Load vs. strain gauge 1A.18.

Figure B. 8  Load vs. strain gauge 1A.19.
Figure B. 9  Load vs. strain gauge 1A.20.

Figure B. 10  Load vs. strain gauge 1A.21.
Figure B. 11  Load vs. strain gauge 1A.22.

Figure B. 12  Load vs. strain gauge 1A.23.
Figure B. 13  Load vs. strain gauge 1A.24.

Figure B. 14  Load vs. strain gauge 1A.25.
Figure B. 15  Load vs. time test 1B.

Figure B. 16  Load vs. deflection test 1B.
Figure B. 17  Load vs. strain gauge 1B.1.

Figure B. 18  Load vs. strain gauge 1B.2.
Figure B. 19  Load vs. strain gauge 1B.3.

Figure B. 20  Load vs. strain gauge 1B.4.
Figure B. 21  Load vs. strain gauge 1B.7.

Figure B. 22  Load vs. strain gauge 1B.8.
Figure B. 23  Load vs. strain gauge 1B.9.

Figure B. 24  Load vs. strain gauge 1B.10.
Figure B. 25  Load vs. strain gauge 1B.11.

Figure B. 26  Load vs. strain gauge 1B.12.
Figure B. 27  Load vs. time test 2A.

Figure B. 28  Load vs. deflection test 2A.
Figure B. 29  Load vs. strain gauge 2A.7.

Figure B. 30  Load vs. strain gauge 2A.8.
Figure B. 31  Load vs. strain gauge 2A.11.

Figure B. 32  Load vs. strain gauge 2A.12.
Figure B. 33  Load vs. strain gauge 2A.10.

Figure B. 34  Load vs. time test 2B.
Figure B. 35  Load vs. deflection test 2B.

Figure B. 36  Load vs. strain gauge 2B.7.
Figure B. 37  Load vs. strain gauge 2B.8.

Figure B. 38  Load vs. strain gauge 2B.11.
Figure B. 39  Load vs. strain gauge 2B.12.

Figure B. 40  Load vs. strain gauge 2B.10.
Figure B. 41  Load vs. time test 3A.

Figure B. 42  Load vs. deflection test 3A.
Figure B. 43  Load vs. strain gauge 3A.8.

Figure B. 44  Load vs. strain gauge 3A.13.
Figure B. 45  Load vs. strain gauge 3A.22.

Figure B. 46  Load vs. strain gauge 3A.15.
Figure B. 47  Load vs. strain gauge 3A.21.

Figure B. 48  Load vs. strain gauge 3A.23.
Figure B. 49  Load vs. strain gauge 3A.12.

Figure B. 50  Load vs. time test 4A.
Figure B. 51  Load vs. deflection test 4A.

Figure B. 52  Load vs. strain gauge 4A.13.
Figure B. 53  Load vs. strain gauge 4A.14.

Figure B. 54  Load vs. strain gauge 4A.15.
Figure B. 55  Load vs. strain gauge 4A.16.

Figure B. 56  Load vs. strain gauge 4A.17.
Figure B. 57  Load vs. strain gauge 4A.18.

Figure B. 58  Load vs. strain gauge 4A.19.
Figure B. 59  Load vs. time test 4B.

Figure B. 60  Load vs. deflection test 4B.
Figure B. 61  Load vs. strain gauge 4B.13.

Figure B. 62  Load vs. strain gauge 4B.14.
Figure B. 63  Load vs. strain gauge 4B.15.

Figure B. 64  Load vs. strain gauge 4B.16.
Figure B. 65  Load vs. strain gauge 4B.17.

Figure B. 66  Load vs. strain gauge 4B.18.
Figure B. 67  Load vs. strain gauge 4B.19.

Figure B. 68  Load vs. time test 5A.
Figure B. 69  Load vs. deflection test 5A.

Figure B. 70  Load vs. strain gauge 5A.11.
Figure B. 71  Load vs. strain gauge 5A.2.

Figure B. 72  Load vs. strain gauge 5A.7.
Figure B. 73  Load vs. strain gauge 5A.5.

Figure B. 74  Load vs. strain gauge 5A.3.
Figure B. 75  Load vs. strain gauge 5A.1.

Figure B. 76  Load vs. strain gauge 5A.12.
Figure B. 77  Load vs. strain gauge 5A.4.

Figure B. 78  Load vs. strain gauge 5A.8.
Figure B. 79  Load vs. time test 5B.

Figure B. 80  Load vs. deflection test 5B.
Figure B. 81  Load vs. strain gauge 5B.7.

Figure B. 82  Load vs. strain gauge 5B.2.
Figure B. 83 Load vs. strain gauge 5B.11.

Figure B. 84 Load vs. strain gauge 5B.1.
Figure B.85  Load vs. strain gauge 5B.3.

Figure B.86  Load vs. strain gauge 5B.5.
Figure B. 87  Load vs. strain gauge 5B.8.

Figure B. 88  Load vs. strain gauge 5B.4.
Figure B. 89  Load vs. strain gauge 5B.12.

Figure B. 90  Load vs. time test 6A.
Figure B. 91  Load vs. strain gauge 6A.13.

Figure B. 92  Load vs. strain gauge 6A.14.
Figure B. 93  Load vs. strain gauge 6A.15.

Figure B. 94  Load vs. strain gauge 6A.16.
Figure B. 95  Load vs. strain gauge 6A.17.

Figure B. 96  Load vs. strain gauge 6A.18.
Figure B. 97  Load vs. strain gauge 6A.19.

Figure B. 98  Load vs. time test 6B.
Figure B. 99  Load vs. deflection test 6B.

Figure B. 100  Load vs. strain gauge 6B.16.
Figure B. 101  Load vs. strain gauge 6B.17.

Figure B. 102  Load vs. strain gauge 6B.18.
Figure B. 103  Load vs. strain gauge 6B.13.

Figure B. 104  Load vs. strain gauge 6B.14.
Figure B. 105  Load vs. strain gauge 6B.15.

Figure B. 106  Load vs. time test 7A.
Figure B. 107  Load vs. deflection test 7A.

Figure B. 108  Load vs. time test 7B.
Figure B. 109 Load vs. deflection test 7B.