A RESEARCH PLAN TO ASSESS THE SHEAR BEHAVIOR OF PRESTRESSED CONCRETE BEAMS WITH 0.7-INCH STRAND AND DIFFERING STRAND END ANCHORAGE CONDITIONS

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ABSTRACT

With the advent and use of high strength concrete, many states have developed new sections which position more prestressing strands in the bottom flange of a beam. Another way of gaining design efficiency without requiring prestressed concrete component manufacturers to replace forms is to use larger capacity strands in conjunction with high strength concrete and the current AASHTO sections.

This paper presents a research plan which will address the application of high capacity strand and anchorage devices in pretensioned concrete girders. Shear behavior will be examined using a variety of end anchorage and strand positioning conditions. As part of this effort, this study will use the strut-and-tie method to predict the capacity of rectangular and AASHTO Type 1 sections with high-capacity strands using 2-inch spacing. These results will be compared with small-scale rectangular section and full-scale AASHTO Type 1 section tests. Development and transfer lengths are to be measured and shear capacity testing is to be conducted under a variety of end conditions. These test results will then compared with an estimated capacity using strut-and-tie modeling and other shear capacity methods and each mathematic model will be assessed for use with high-capacity strand.

The study should give the designer a better understanding of shear behavior using the larger capacity strands and provide tested alternatives for end anchorage for prestressing strands.

Keywords: Prestressed Girders, High-Capacity Strand, Shear Capacity, Bridge Girder Design, Strut-and-Tie Modeling

INTRODUCTION

Prestensioned components, such as I-girders and Bulb-Tees, have been widely used in the construction of bridges. Currently, the prestressing strand diameters used in these components are predominantly 0.5- and 0.6-inch utilizing Grade 270 steel. This combination of section geometry and strand has worked well regardless of the particular shape when used with normal strength concrete.

With the development and use of high strength concrete (HSC), many states have developed sections with a wider bottom flange to allow the placement of more strands in the bottom flange. Increasing the number of strands will allow a more efficient use of the HSC materials and provide a longer span length in the section. Other states have elected to continue using the American Association of State Highway Transportation Officials (AASHTO) sections rather than requiring the prestressed girder manufacturers in the state to acquire new forms.

Within the last few years, researchers have been investigating the use of larger capacity strands in prestressed concrete components.[1,2] These strands, utilizing 0.62-inch diameter Grade 330 and 0.7-inch diameter Grade 270 steel, have been used in mining applications, and based upon preliminary findings, can be used in highway structural components. Bridges using these strands are being designed and manufactured currently. One example is the Pacific Street Bridge over I-680 in Omaha, Nebraska.[1]

Initial studies of the use of the higher-capacity strands indicate that they have the ability to introduce almost twice the prestressing force, when compared with 0.5-inch strand and 135% of the prestressing force when compared with 0.6-inch strand.[2] This could result in a significant span capacity increase of the current AASHTO sections. Form replacement would not likely be necessary. The improved capacity of the current section also could lead to other structural efficiencies such as wider girder spacing, reduced structural mass for seismic design, and reduced substructure and foundational requirements—all of which reduce costs.

To achieve the enhanced flexural capacity in a concrete girder, several conditions must exist. First, the longitudinal flexural reinforcement steel within the concrete girder must be developed. This development occurs longitudinally along the length of the component due to the bonding of the steel to the concrete. Once bonded, flexural capacity can be attained by the interaction of the longitudinal steel in tension and the compression of the concrete. Concrete has poor tensile capacity when compared to its compressive capacity and will fail brittlely when stressed in tension beyond its capacity. At maximum flexure capacity, the concrete is generally cracked and contributes little to the flexural capacity of the girder.

Additionally, a cracked section has serviceability issues. A cracked section is susceptible to environmental corrosion effects and has a poor appearance to the end user. Most codes restrict the usable flexural capacity at service conditions so that the flexure cracking in the concrete is not readily apparent.

Cracking will weaken the bond between the concrete and the steel. When this bond is broken, then the strands within the concrete girder can slip and the necessary bond to sustain its maximum flexural capacity may not be attained. This bond is usually developed at the end of the girder.

The second condition that must be met is that the girder must develop enough shear capacity at the girder end to develop the flexure capacity at the middle of the beam. Shear capacity is attained as a property of the concrete itself and through the use of steel reinforcement within the girder. The combination of shear and moment forces at the girder end can introduce tensile forces which can cause the concrete to crack. When this cracking occurs, the bond between the longitudinal reinforcement and the concrete will become weakened; the longitudinal strands may become debonded and the flexural capacity of the structural girder can become substantially reduced.

Due to its low tensile strength, concrete has a proclivity to crack which can result in an adverse effect in the development of the longitudinal reinforcement. Strand slippage due to cracking within the beam can be prevented in a number of different ways. In 2000, Ma et al. examined extending the strand end into an end diaphragm as method of end anchorage with 0.5-inch strands in an NU 1100 I-girder section.[2] A total of 5 shear tests were performed on two specimens, four of which included end anchorage and one did not. An end block was poured at the end of the girder and the longitudinal strands were cast inside this end block to simulate extending strands into an end diaphragm. The three of the testing specimens with end anchorage failed by the crushing of the web, and one specimen did not fail within the loading limits of the testing apparatus. The specimen without end anchorage failed with a bond-slip mode. This research demonstrates the importance of end anchorage in developing the strength of a girder.

Shear loading will be used in this study. The shear force will cause the end region to crack, provide an extreme case for strand debonding, and allow the examination of the effectiveness of end anchorage under extreme conditions. Along with the shear loading, several types of end anchorage will be examined under differing strand conditions.

We anticipate that using the larger capacity strands will result in higher than usual release stress in the girder end at maximum flexural capacity. This stress must then be alleviated. This can be accomplished by debonding, draping, or harping of the longitudinal strand. Debonding some of the strands may contribute to an end anchorage and strand bond-slip problem, but from a manufacturing standpoint, is easier to debond than alleviating the release stress by harping or draping. This research will explore end anchorage and its effect when using debonding and draping.

The objective of this research is to examine shear behavior in a concrete prestressed girder using 0.7-inch strand. Differing end anchorage and longitudinal strand positioning will be considered in this investigation. This study should provide a further knowledge in shear behavior, evaluate the change in shear capacity using a variety of end conditions, and provide the practicing designer with insight and tools necessary to design girders with 0.7-inch strand and HSC safely.

This paper presents a research plan that will be used to address application of high capacity strand and anchorage devices in pretensioned concrete girders.

PRESTRESSED GIRDER PARAMETERS AFFECTING SHEAR STRENGTH

Shear capacity in a concrete structural member is dependent upon many parameters which can influence capacity, and rarely can a pure shear test be developed without introducing bending stresses. Concrete also has differing properties in compression and tension and often the failure mode is not in shear, but in a normal tensile stress acting as a combination of shear and moment. This normal tension force results in cracking at relatively low tensile stresses.

Shear capacity in a given reinforced concrete girder with a known compressive strength is dependent upon contributions from two components, the shear capacity of the reinforcement used in the girder and the shear capacity within the concrete itself. Based upon our literature review, we have identified design parameters which have been known to influence concrete strength and have divided them into three categories.

The first category is geometric properties. This includes the type section being used (rectangular-shaped beam, I-beam, box-beam, or u-beam, as examples), the strength of concrete being utilized, type and position of longitudinal reinforcement, and the utilization of end anchorage. The second category is the reinforcement being used in the beam which can include vertical shear reinforcement, confinement steel for longitudinal reinforcement and bursting reinforcement, draping of longitudinal steel, and concrete splitting reinforcement. The third category involves the capacity of the concrete itself both in a cracked and a post cracked state. This capacity is dependent on the components of the concrete, type and strength of paste, aggregate used, curing age, and the type of cure, as examples.

Each of these parameters will be examined individually and their impact upon shear capacity is addressed.

GEOMETRIC PROPERTIES

Section Types

The geometry of the section can have a tremendous impact upon the shear capacity of a girder. From structural mechanics, we understand that shear stress at any point in a girder is dependent upon the statical moment area at the point of interest, the moment of inertia of the component, and the thickness of the member at the point of interest.[3] These are physical properties which can be modified to provide differing shear properties as required by design. These properties will be normalized by performing small-scale testing on a single rectangular section and full-scale testing on two AASHTO Type 1 girders.

Small-scale testing will be performed upon a rectangular section. The size of this section was based upon the stresses in the section with 10 strands at release and the calculated development length of high-capacity strand (approximately 10 feet). This approximation will be validated when testing the small- and full-scale specimens. For the small-scale rectangular-shaped specimens, the size was determined so that the release stress at the extreme compressive fiber of the section was at the 0.6 f'_{ci} limit prescribed by AASHTO

LFRD 2010. Using these criteria and a concrete release strength of 5,600 psi, a 12-inch by 45-inch specimen was selected.

It is intended that each small-scale specimen have this configuration. In this manner, a direct comparison of the shear capacity due to the changing of end conditions can be made. Each specimen will be approximately 30 feet long to allow the strand to be fully developed in the center and allow for two shear tests, one on each side of the girder. A total of five small-scale specimens will be manufactured for testing.

Concrete Strength

Two concrete strength ranges will be examined in the testing. Since shear capacity is dependent upon the strength of concrete, and girders designed with larger capacity strands gain efficiency with high-strength concrete, two concrete strength ranges will be examined during small-scale specimen testing.[4] They are 9000 to 12,000 psi and 12,000 to 15,000 psi. The first range is regularly used by prestressed girder manufacturers and will be used in conducting three of the small-scale tests. The second strength range can be attained by most manufacturers, but is not as commonly used. From previous research, it has been shown that by using the larger strand with higher strength concrete, a geometric section can attain a much higher efficiency than by using larger strands alone.[5] Therefore, we will examine two specimens using the higher strength concrete.

Position and Type of Longitudinal Reinforcement

Testing of 0.5- and 0.6-inch strand has shown that the position and size of longitudinal reinforcement can influence shear strength capacity.[6] To further quantify the shear capacity of girders using larger capacity strand, the position of the longitudinal reinforcement will be varied to test the impact that the variation has upon shear capacity. The strands in some of the specimens will be draped and debonded.

Three debonding percentages that will be used are 20, 40 and 60%. The impact of debonding upon shear capacity will be evaluated with and without strand end anchorage. AASHTO LFRD 2010 limits the percentage of strands to be debonded to 40%. Two specimens in this study will debond 60% of the strands, as a way to investigate this code requirement. The effect that end anchorage could have upon the tension tie and shear performance of the specimen will also be examined.

The debonding sleeves used in this investigation will be solid and will provide enough room to allow the prestressing strand to expand upon release. The ends of the sleeves will be sealed so that concrete paste is not allowed contact with the relaxed portion of the prestressing strand. Through contacts with prestressed component manufacturers, we determined that there are claims that split sleeves allow the concrete to interact with the relaxed portion of the prestressing strand and that undersized conduit can also interfere with the relaxation of strand. This will be monitored closely during the production of the test specimens.

Draping will be conducted on one specimen test and its impact upon shear capacity will be assessed. The impact of draping and debonding upon shear is important since larger strand capacity designs will likely require a mechanism to reduce end zone stress and cracking. Previous research has shown that end zone cracking can critically reduce a girder's shear and moment capacities.[6]

End Anchorage

Four types of end anchorage will be evaluated in this study. The first type places strand extensions at the end of the girder within an end diaphragm to guard against strand slip failure. To simulate this, an end block will be poured at the end of the girder and the cut strands will be poured into an end block. Bonded and debonded specimens will be examined using this type of end anchorage to assess any change in shear capacity. The large-scale testing samples will also include an evaluation of this type of end anchorage.

Another end anchorage detail, bond beads, as shown in Figure 1, will be used in a small-scale test. 60% of the strands will be debonded in one of the test specimens. These bond beads will be placed 9 inches from the end of the debonding point inside the specimen. The strands will be monitored for strand slip. We believe that the bond bead device can be beneficial in quickly developing the strand when shear and flexural capacity.

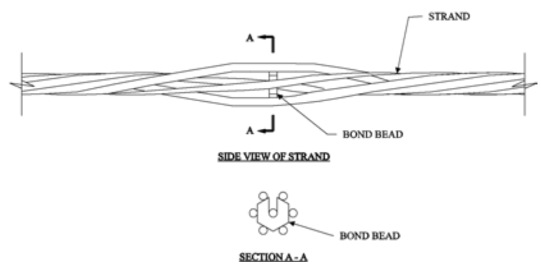

Figure 1 - Bond Bead Detail

The third type of anchorage system examined in the small-scale testing portion of this investigation is shown in Figure 2. This system uses a stiff plate mounted upon the end of the specimen and wedges to anchor the strands to the steel plate. At release, this system should more evenly distribute the release stresses across the cross-sectional area of the specimen, thereby reducing the eccentricity-related stress at release. This system will also aid in the prevention of strand slip under maximum loading. Development lengths with this end anchorage system may be shorter as the steel plate system should allow the strand to be developed to the end of the girder.

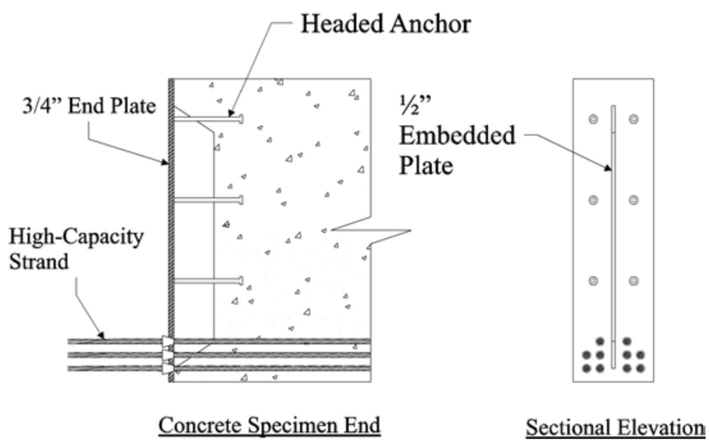

Figure 2 - End Plate Anchorage Detail

The fourth end anchorage is not a methodology, but a system. Moving the bearing pads from the end of the specimen in towards the center of the beam should increase the shear capacity of the beam. Both a debonded and a fully bonded test will be conducted during this investigation to assess the impact of this system.

REINFORCEMENT DETAILING

The reinforcement details can have a tremendous impact upon the shear capacity of the specimens. As this is an investigation of the impact upon shear using higher-capacity strand, the reinforcement detailing will remain similar for the small- and full-scale specimens.

Vertical Shear Reinforcement

The vertical shear reinforcement requirements are given by ACI 318-08 and AASHTO LRFD 2010. Prescribed amounts are provided in these codes so that a beam has enough shear capacity to develop its flexural strength, provide for a ductile failure, and prevent excessive cracking within the member. For this examination, only a minimal amount of vertical shear reinforcement will be used which is two vertical #4 bars at 30-inch spacing. Strut-and-tie modeling design of the specimens shows that the first node should be found at approximately 15 inches. With this, the vertical shear reinforcement shall be positioned at 15 inches on center for all small-scale specimens. Using this spacing, the shear capacity of the small-scale specimens is predicted to be approximately 275 kips.

Confinement Steel

Patzlaff and Tadros et al. have performed a number of studies regarding confinement reinforcement in the bottom flange.[7] Girders without confinement reinforcement experience considerably more cracking than those with confinement steel. Cracking, particularly in the end zone regions of the beam can reduce the shear capacity of a prestressed girder. No. 3 deformed bars with a 6.0-inch minimum spacing will be used as the confining reinforcement at the ends and the debonding point for the small-scale specimens.

Draping of Longitudinal Strands

According to AASHTO LRFD, strand draping will not only decrease the strand release stresses at the end zone of the girder, but will also increase the shear capacity of the beam.[8] As a result, the testing plan will include one specimen which includes draped strands. Shear capacities of the draped specimens will be compared to the shear capacity of an undraped section with large capacity strands.

Concrete Splitting Reinforcement

Concrete splitting reinforcement will be included in the small-scale specimens to avoid cracking in the end zone regions of the beam. The reinforcement will be in accordance with AASHTO LRFD 2010 Article 5.10.10.1. It is expected that the larger capacity strands will require splitting reinforcement especially when the number of strands is maximized for structural efficiency. According to AASHTO LRFD 2010, the small-scale specimens shall have 3 sets of #4 rebar pairs within the first 12 inches at the beam end. Additionally, this same splitting reinforcement will be positioned at each debonding point.

MATERIAL CAPACITIES

Concrete Shear Capacity

As stated, one key component in shear capacity is the concrete itself. For the purposes of this study, the capacity due to a specific concrete mix is not to be explicitly examined. Aggregate type used for each specimen will be the same and the same mix design will be used at each strength range. Estimates for determining the concrete contribution to the shear capacity will be performed using AASHTO LRFD 2010.

Two mix strength ranges will be examined during this investigation. The full-scale specimens have already been manufactured using a high-strength-range concrete mix. The middle-strength concrete mix has not been designed at the writing of this research plan.

The design listed in Table 1 was used for both full-scale girders. The slump of the concrete was 7 inches and the temperature was 75 ° F at the time of pour.

Table 1: High-Strength-Range Concrete Mix Design

Materials	Quantity
Cement Type I	800 lbs.
Coarse aggregate (Lime Stone) 8P , ½”	1814 lbs
Fine aggregate (Sand)	1390 lbs
Silica fume	56 lbs
Water	28 Gallons
HRWR	125 oz
Water reducer	25 oz
w/c	0.292

Two methods of curing were performed for the girders. The girder with 0.7-inch strand was water cured. And the girder with 0.62-inch diameter strand was cured with steam.

Thirty-nine concrete cylinders were prepared for the full-scale specimens, and concrete compression tests were performed at various times when strain measurements were performed. Table 2 presents the concrete testing results for the two full-scale specimens. The concrete cylinder test were performed at transfer of prestress force and 24 hrs, 3 days, 7 days, 14 days, 28 days from transfer. These tests were performed on 4x8 inch cylinders at the prestressed component manufacturing plant.

Table 2: Average Concrete Strength for Full-Scale Specimens

Testing Time	Cylinder Strength, psi	
	Specimen FST1-270 (0.7-inch Strand)	Specimen FST1-330 (0.62-inch Strand)
24 hrs from concrete pour	7,586	-
3 days from concrete pour	9,072	11,048
At Detensioning of Strands	10,252	11,592
24 hrs from Detensioning	10,929	11,618
3 days from Detensioning	10,438	11,724
7 days from Detensioning	11,791	11,877
14 days from Detensioning	12,374	12,255
28 days from Detensioning	14,191	12,295

Strand and Mild Reinforcement Used

Two different types of prestressing strand are to be used in this experimental investigation: 0.62-inch diameter (330 ksi) and 0.7-inch diameter (270 ksi). The 0.62-inch diameter strands were provided by Sumiden Wire Products Corporation, and the 0.7-inch diameter strands were provided by MMI Strand Co. These strands were manufactured to meet ASTM A-416-05 specifications. The surface conditions of both types of strand were similar and were without any rust.

The 0.62-inch diameter strands are made of high strength steel (330 Grade), uncoated, low-relaxation strand. The physical properties of the strand provided in Table 3 are as reported by the strand manufacturer. The 0.7-inch-diameter strands were uncoated seven wire low-relaxation strands. All strands were grade 270 ksi. The physical properties of the strand provided in Table 3 are as reported by the strand manufacturer. They were wound similarly to 0.5-inch and 0.6-inch diameter strands.

Table 3: Strand Properties

Grade	270 ksi	330 ksi
Nominal diameter	0.7”	0.62”
Diameter tolerance	+0.026”, -0.006”	Not Available
Nominal cross sectional area	0.294 in^2	0.2227 in^2
Elastic modulus	28,800 ksi	28,500 ksi
Minimum breaking strength	79,400 lbs.	76,418 lbs.
Minimum load at 1% extension	71,500 lbs.	72,576 lbs.
Minimum ultimate elongation in 24” gauge length	3.50%	5.20%

TESTING SPECIMENS

As discussed, this investigation will examine the shear capacity of prestressed concrete girders when using 0.7-inch Grade 270, and 0.62-inch Grade 330 strand. Both small-scale and large-scale tests will be conducted to assess and quantify shear behavior when using 0.7-inch strand. The specimens will include the use of rectangular-shaped small-scale specimens and two full-scale AASHTO Type 1 girders. Each specimen will be tested on each end yielding two tests per specimen. Table 4 presents a tabulated matrix for each specimen with the given test parameters for each individual tests.

Five small-scale specimens are included in this matrix. The specimen numbers were given to uniquely identify each specimen and each section tested. For example, the first specimen number given is 1R. This indicates that this is specimen #1 and is located at the right end of the specimen. Similarly, 1L refers to the left side of specimen #1. All of the small-scale specimens utilize 0.7-inch, Grade 270 strand. One full-scale specimen uses 0.7-inch, Grade 270 strand and the other uses 0.62-inch, 330 ksi strand.

0.7-inch, Grade 270 steel was selected as the primary strand to be used in this testing because it has a slightly higher capacity than 0.62-inch, Grade 330 strand. The full-scale testing samples should render an adequate comparison of these two types of strand. Two small-scale specimens utilize higher-strength concrete.

SMALL-SCALE SPECIMENS

Individual elevation and cross-sectional details are given for each specimen. These are labeled as Figures 3 to 5, and 7 and 8. These figures show the strand configuration, loading points, bearing points, and cut lines for each sample being investigated. Test specimen 1L is the control specimen for each of the other specimens. This specimen will be loaded at five feet away from the bearing point. It is not draped or debonded and has no specific end anchorage condition. It will be tested to failure and the results obtained from this specimen will be compared to the others as the baseline shear capacity for the specimen.

Specimens 1 through 3 will be cast at the same time using the same casting bed. This should normalize any potential differences in strand release stress and perhaps compressive strength. It will require 12.5 yards of concrete to pour specimens 1 through 3--approximately one truck load.

Specimen 1R will be debonded at 3 and 6 feet from the bearing pad. The debonding is in

Table 4 – Specimen Testing Matrix

Specimen Number	Geometric Properties				Reinforcement			
	Shape[1]	f'_c Range[2] (ksi)	%Debonding	Anchorage Type	VSR[3]	Confinement Steel	Draping	SS[4]
1L	R	MIDDLE	0	None	#4 @ 15” OC	#3 @ End - 6" Spacing	None	@ End
1R	R	MIDDLE	60/20	End Block	#4 @ 15” OC	#3 @ End & Dbnd Pt - 6" Spacing	None	@ End
2L	R	MIDDLE	60	Bond Bead	#4 @ 15” OC	#3 @ End & Dbnd Pt - 6" Spacing	None	@ End
2R	R	MIDDLE	0	End Block	#4 @ 15” OC	#3 @ End - 6" Spacing	None	@ End
3L	R	MIDDLE	40	End Plate	#4 @ 15” OC	#3 @ End & Dbnd Pt - 6" Spacing	None	@ End
3R	R	MIDDLE	0	End Plate	#4 @ 15” OC	#3 @ End - 6" Spacing	None	@ End
4L	R	HIGH	0	None	#4 @ 15” OC	#3 @ End - 6" Spacing	None	@ End
4R	R	HIGH	0	None	#4 @ 15” OC	#3 @ End - 6" Spacing	Ctr Stands	@ End
5L	R	HIGH	40	Pad Position	#4 @ 15” OC	#3 @ End & Dbnd Pt - 6" Spacing	None	@ End
5R	R	HIGH	0	Pad Position	#4 @ 15” OC	#3 @ End - 6" Spacing	None	@ End
FST1-270L	Type 1	HIGH	0	End Block	#4 @ 1.5,3,6,10	#3 @ End - 6" Spacing	None	@ End
FST1-270R	Type 1	HIGH	0	End Block	#4 @ 1.5,8,10	#3 @ End - 6" Spacing	None	@ End
FST1-330L	Type 1	HIGH	0	End Block	#4 @ 1.5,3,6,10	#3 @ End - 6" Spacing	None	@ End
FST1-330R	Type 1	HIGH	0	End Block	#4 @ 1.5,8,10	#3 @ End - 6" Spacing	None	@ End

Notes:

[1]The abbreviations for this column are as follows: R - Rectangular-shaped, Type 1 - AASHTO Type 1 Girder

[2]The terms in this column are as follows: MIDDLE - 9000 to 12,000 psi, HIGH - 12,000 to 15,000 psi

[3]VSR - Vertical Shear Reinforcement

[4]SS – Splitting Steel

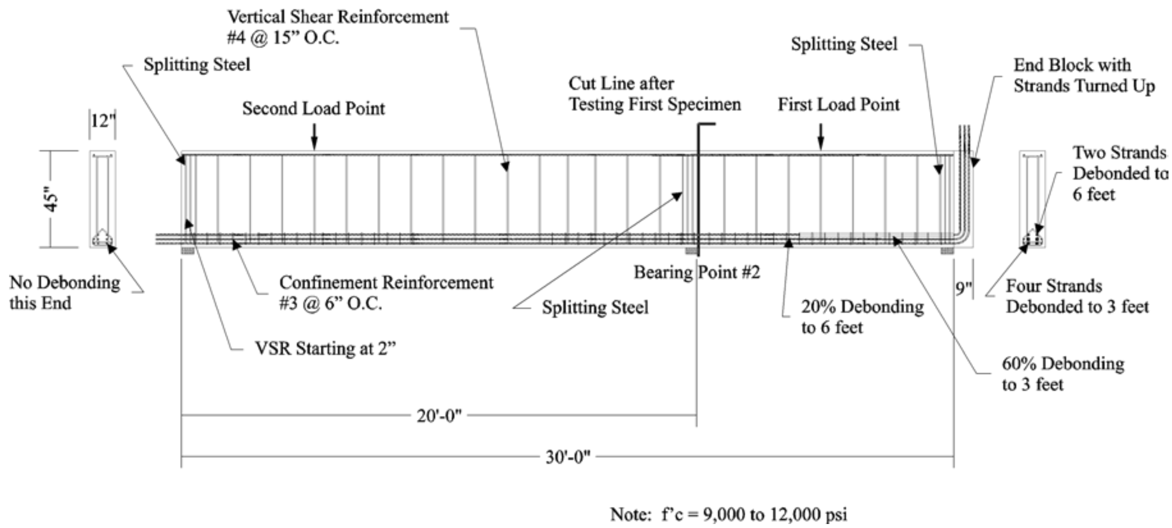

Figure 3 - Diagram of Specimens 1L and 1R

excess of the 40% AASHTO LRFD 2010 limit, but is not greater than the debonding percentages seen in some current bridge designs. It is the belief of the researchers that as larger strands are being utilized, that higher debonding percentages will become necessary to limit end zone release stresses. This sample could show a reduction of shear strength when compared to the tested shear strength of specimen 1L and 2R. This testing may give an initial indication whether the debonding percentage could be raised under AASHTO LRFD 2010 specifications.

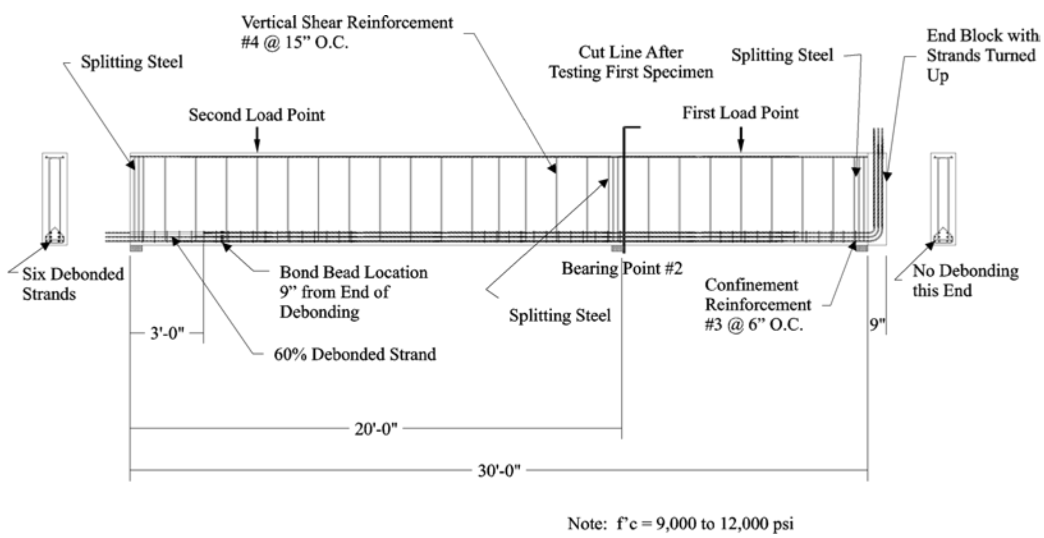

Figure 4 - Diagram of Specimens 2L and 2R

Specimen 2L explores the use of a bond bead and an end block to provide end anchorage and strand slip prevention to attain maximum shear capacity in the section. Specimen 2R will have a poured end block of 9 inches wide by the cross-sectional area of the specimen. The intention is to bend the strands vertically and allow them to protrude from the end block section. The results from this specimen will be compared with the results from specimen 1L and 1R to monitor for any change in shear capacity. This specimen will also be compared to specimen 3R to determine the differences in those end anchorage conditions.

Specimen 2L will use bond beads placed at a distance of 9 inches from the end of the debonding point inside the specimen. The strands will then be debonded and the strands will extend beyond the end of the specimen. Instrumentation to monitor strand slip will be attached to the end strands protruding from the end block. The bond beads may require staggering to provide proper concrete bonding and prevent interference between adjacent strands.

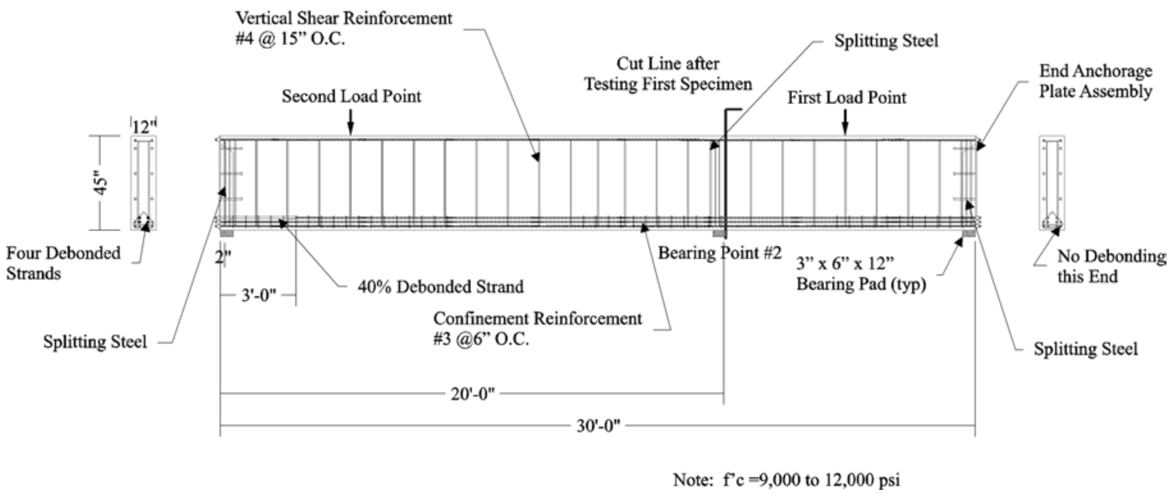

Figure 5 - Diagram of Specimens 3L and 3R

Specimens 3L and 3R use an end plate embedded into the side of the specimen to distribute forces more uniformly across the concrete cross-section, provide for end anchorage to prevent bond slip, and to eliminate shock forces applied to the bond between the concrete and strand when the strand is torch-cut. The strands are attached to the plate using a wedge mechanism.

A detail of the plate and slip mechanism is shown in Figure 6. Specimen 3L will have 40% of its strands debonded and specimen 3L will be completely bonded.

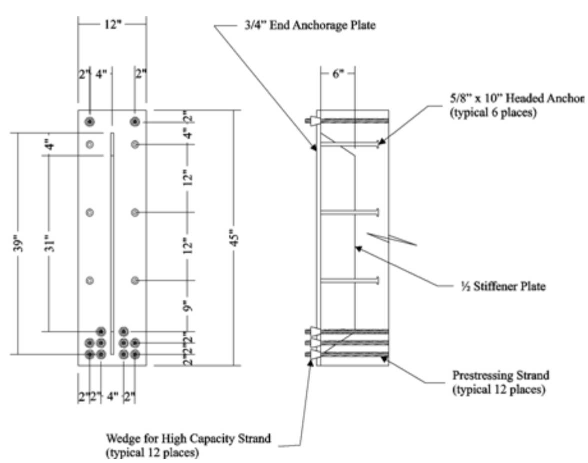

Figure 6 - End Anchorage Plate Detail

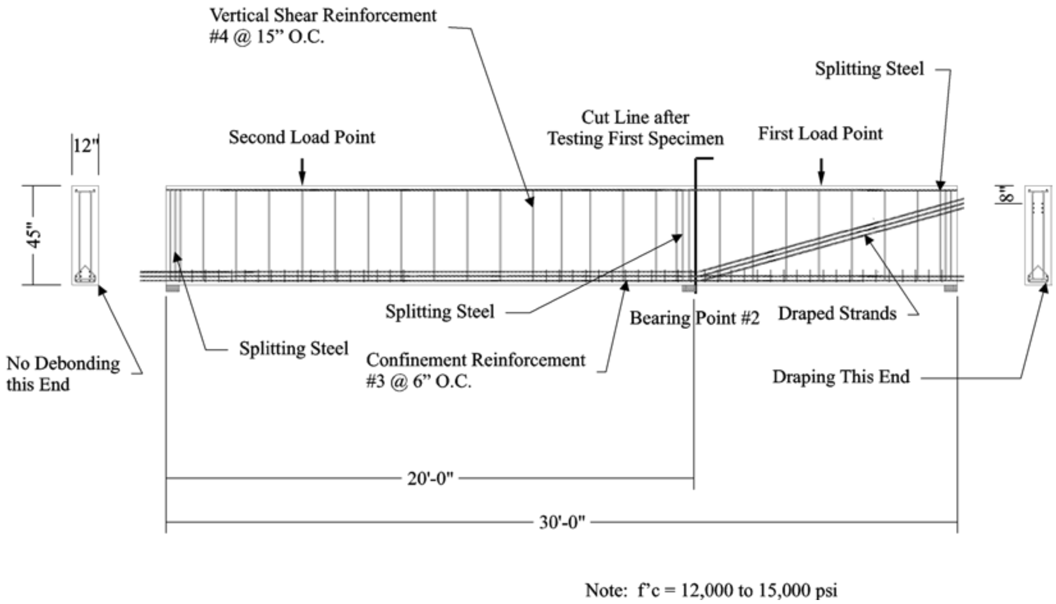

Figure 7 - Diagram of Specimens 4L and 4R

Comparisons will be made to each other and to the results from specimens 1 and 2.

Specimen 4L will be used to explore the effect that using higher-strength concrete in conjunction with 0.7-inch strand has upon shear capacity. The testing results from this specimen will be compared to the results from specimen 1L. These results will be compared to shear capacity calculations as predicted by various methods. This specimen will also serve as the baseline for the three other additional test specimens using higher-strength concrete.

Specimen 4R will explore the shear capacity of strand draping under load. 60% of the strands will be draped from the center of the strand pattern to within 8 inches of the top of the specimen. The drape point will be located at approximately 33% of the span length. The

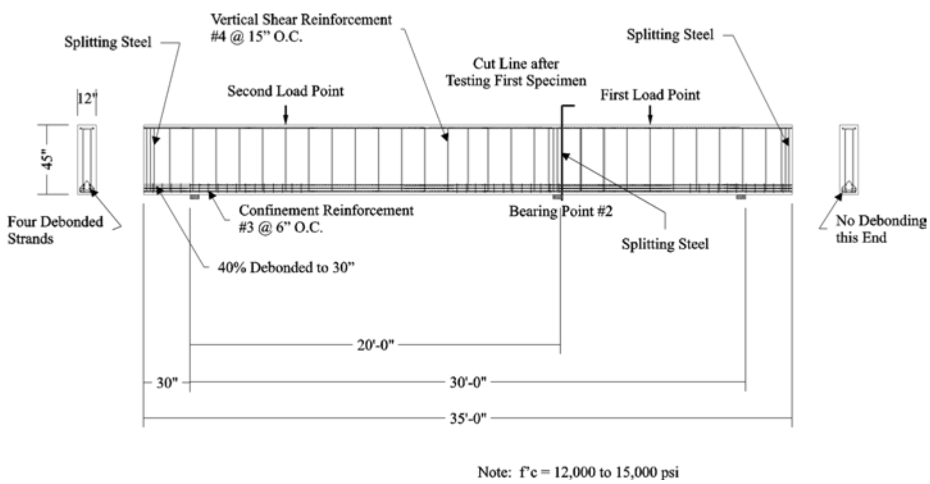

Figure 8 - Diagram of Specimens 5L and 5R

shear capacity will then be compared with the results from specimen 4L and the calculation of shear capacity using AASHTO LRFD 2010. The draped strands are predicted to add shear capacity due to being inclined.

Specimen 5L and 5R will explore the shear capacity of a debonded section and a fully bonded specimen when the bearing pad is move inward. Moving the bearing position inward is predicted to increase the shear capacity of the specimen by allowing a greater length for strand development. The results from this testing will be compared to the results from specimen 4L to assess the change in shear capacity.

FULL-SCALE TESTING

Once the small-scale testing is completed, the testing results will be analyzed and compared to the shear capacity predictions. The full-scale specimens have already been manufactured under previous research funding. The design of these girders was performed in sections. One end of each specimen was designed using AASHTO LRFD 2010 and the other using Strut-and-Tie Modeling. Specimen FST1-270 was designed using the 0.7-inch diameter, seven-wire, low-relaxation strand with the ultimate strength of 270 ksi and specimen FST1-330 was designed using the 0.62-inch diameter, seven-wire, low-relaxation strand with the ultimate strength of 330 ksi.

The span of the two specimens was 56 feet, which was determined based on the maximum span which could be tested at the University of Tennessee lab. A total of 12 strands were provided in the bottom flange of the specimens. All 12 strands were straight and spaced at

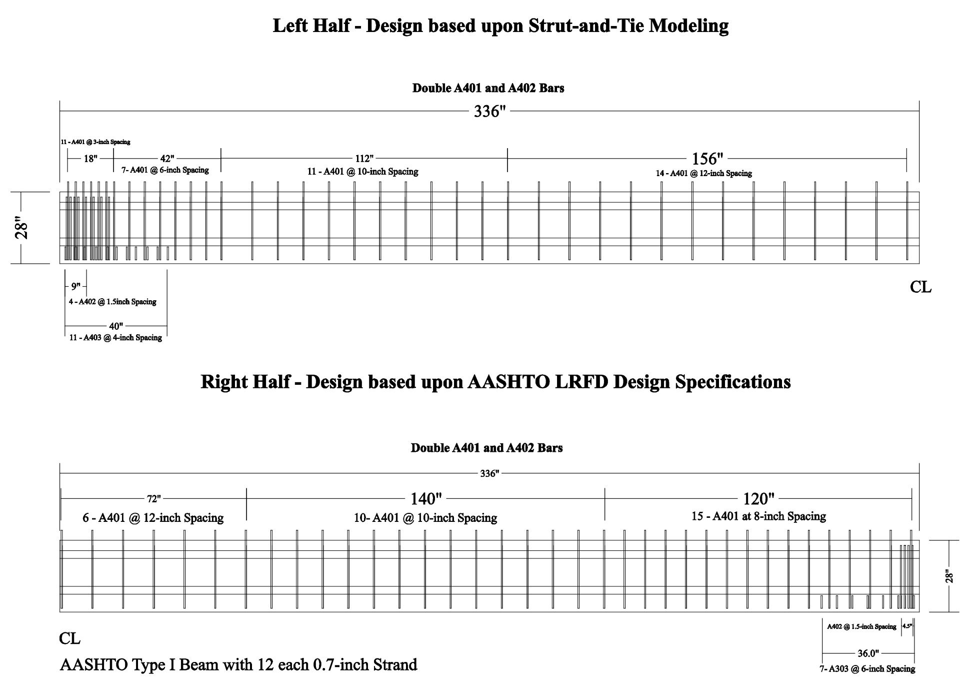

Figure 9 - Type 1 Girder Strand Arrangement and Shear Reinforcement Details

2.0 inches in both horizontal and vertical directions. The shear reinforcement, top flange reinforcement and the anchorage zone reinforcements are designed based on both AASHTO LRFD 2010 and Strut-and-Tie Modeling. The concrete strength was 10.0 ksi at transfer and 12.0 ksi at service. Figures 9 through 10 show the cross-sectional properties, strand and mild steel placement, and dimensions of the full-scale specimens.

A deck will be cast upon the full-scale specimens. The deck will be 36 inches wide, and will be designed using high-strength concrete. The compressive strength will be determined to give an equivalent deck width of 10 feet using 4,000 psi concrete. End block, end anchorage will be provided for the strands at both ends of the specimens. Maintaining the same size deck will normalize the effect of end anchorage on the shear capacity for each design method. Thus a direct comparison of AASHTO LRFD 2010 and Strut-and-Tie modeling can be performed for each higher-capacity strand size.

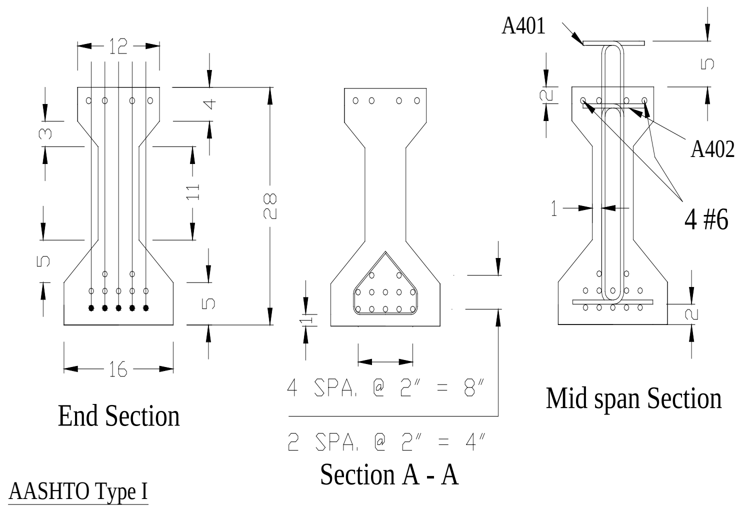

Figure 10 - Full-Scale Specimen Sections

INSTRUMENTATION AND MEASUREMENT PROCEDURES

This section describes the instrumentation and measurement procedures which will be used during the experimental examination of the specimens. It is subdivided by the parameter being measured. The instrumentation/measurement method chosen is then described in each subdivision.

PRESTRESS TRANSFER LENGTH

An accurate estimate of the transfer length is important for several reasons: calculation of the concrete stresses at transfer and service loads, design of anchorage zone reinforcement for strut and tie models, and design of shear reinforcement which requires knowledge of the level of precompression in the concrete. Two different types of instrumentation will used to determine the transfer length.

One method uses the DEtachable MEChanical (DEMEC) strain measurement system, which involves the measurement of the surface strain of concrete. The other method for calculating transfer length is strand drawn-in, in which the distance slipped by the strand into the concrete is measured.

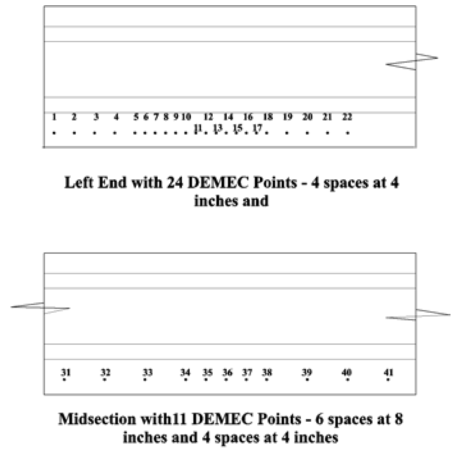

Figure 11 - DEMEC Points along the Centroid of the Strands and Web of the Specimen

Concrete Surface Strain Measurement

Transfer length is the distance required to transfer the effective prestressing force from the strand to the concrete. To determine the transfer length for the girders, a series of DEMEC points are placed on the surface of the concrete. These points have small metallic discs of 1/4th inch in diameter, which are placed at the centroid of the prestressing strands on sides/flange of the girder.

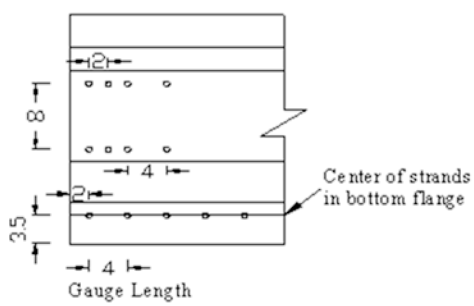

Figure 12 - DEMEC Points along the Centroid of the Strands and Web of the Specimen

Figure 13 - DEMEC Points at the end of the Specimen

For these specimens the DEMEC points will be spaced at 4 inches starting at 2 inches from the end of the girder for a distance of 20 inches, at a spacing of 2 inches for a distance of 26 inches and then at a spacing of 4 inches for a distance of 12 inches (60 inches total). This spacing was determined based on the gauge length (3.937 inches) of the DEMEC gauge and the DEMEC points were concentrated near the estimated transfer length distance. Thus, there are 22 points proving 20 readings on each DEMEC line. These lines are to be placed on the side of the girder yielding a total of 4 lines per specimen.

DEMEC points will be set at the midspan section of the girder on both sides in order to estimate the short- and long-term loss of prestress in the girders. An orthogonal grid of DEMEC points will also be set on the side of the rectangular sections and the web of the I-sections at the end zones to estimate the concrete strain in the vertical direction and to map the principal tensile stresses in the disturbed end zone region of the girder during loading. Some examples of the DEMEC points are shown in Figures 11, 12 and 13.

Strand Drawn-in Measurement

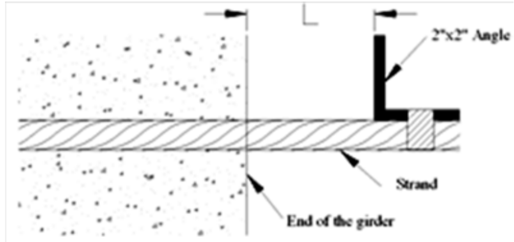

Figure 14 - Strand Drawn-in Measurement Set-up

Strand drawn-in is a measurement of how far the strand at the face of the concrete is pulled into the beam after the prestress is released. Strand drawn-in helps in determining the effectiveness of the bond between the concrete and prestressing strand after the prestress is released. To measure the drawn-in, 2” x 2” x 0.5” angle sections will be attached at a distance of 3 inches from the end face of the girder. A typical set up is shown in Figures 14 and 15. Two sets of readings of the distance between the end of the angle and the face of the beam end were taken to measure the drawn-in. The first reading will be obtained before the pretensioning is released and the second reading taken after the release. These readings will be measured using a digital caliper.

Figure 15 - Strand Drawn-in Measurement Set-up

CAMBER MEASUREMENT

The instrumentation for the measurement of the camber for the girder will be performed by running a thin wire at the centroid location of the girder from one end of the specimen to the

Figure 16 - Deflection Measurement Setup

other. The ends will be tied with tensioning applied to the wire. Rulers affixed at the midspan, quarter points and load position will be monitored to read the camber and the change in camber during specimen loading. In order to prevent a parallax error during the reading process a mirror will be attached directly behind the wire as shown in Figure 16.

TENSILE STRESS MAPPING

During this investigation, it will be important to measure and monitor the development of tensile stress in the disturbed region at the end of each specimen. This will be accomplished using two different methods. First, these measurements will be obtained by using surface-mounted strain gages which monitor strain in three directions. These gages will be placed at 6-inch centers for a distance 1.5*h* beyond the load point.

In addition to the strain gages, a grid of DEMEC gages will be positioned in a 6-inch vertical by 6-inch horizontal grid pattern centering the two-dimensional strain gages within the grid. These two sets of measuring devices will be used to measures the strain in two different directions. Strain measurements in the two directions will be used to calculate the stress in two orthogonal directions and then the principal compressive, tension, and shear stresses, along with the primary stress rotation angle will be calculated.

Also, 3-dimensional gauges will be embedded within the web of the specimens. The gauges will be centered along the cross-section and placed at a top, bottom, and middle line of the web section. Embedding strain gauges within concrete is difficult. The gauges are easily damage by placing and vibrating the concrete. This will be attempted on the initial specimens and depending upon the results, will be continued. It is our hope that a 3-dimensional stress evaluation will be attained within the web of the specimen using these gauges and that stress measurement is not solely relied upon surface strain measurements.

Strain data will be measured at different times and under different loading to give a progression of stress within the specimen. A stress progression map will then be developed to understand the critical locations for shear cracking under loading. These data primarily apply to the pre-cracked state of the specimen. After cracking, stress redistributes itself from the concrete to the mild steel and longitudinal strand reinforcement of the prestressed girder. The strain gages should show this stress redistribution provided the gage does not cross developed cracks.

The DEMEC gauge grid will be used to estimate the width of cracks developing between grid points. These data will be beneficial in the development of future fracture mechanics modeling for prestressed concrete girders and used to provide a case study for the evaluation of modified compression field theory.

The strain gauges provide an advantage in that the data received can be continuously monitored and stored automatically during the course of testing; whereas, the DEMEC gage will require physical measurement and manual recording. The strain gages have a disadvantage in that it is difficult to assure a good bond between the concrete and the gage that will accurately represent the actual strain in the concrete. Both will be used initially, and the data from each method will be compared to each other. After initial testing, the reliability of each method will be assessed and a decision as to which method/methods will be used for additional specimens will be made.

VERTICAL DISPLACEMENT MONITORING

Vertical displacement monitoring will be accomplished by using a series a linear vertical displacement transducers (LVDTs) connected to a data recording system to measure the vertical displacement of the specimen during loading. The LVDTs will be mounted onto a channel and placed underneath the specimen. LVDTs will be placed at two-foot spacing from the end of the specimen for ten feet--through the disturbed region--and at the midpoint of each specimen.

The data obtained from the LVDTs can be compared with data obtained from the camber measurement as a comparison of reasonableness. The vertical displacement measurement can then be used to compare displacement models using 0.7-inch strand to existing models for smaller strand.

STRAND SLIP MONITORING

After pouring and prior to strand release, several u-bolts will be attached to the strands to prevent the strand unraveling at release. Approximately five feet of strand will be left to extend beyond the specimen to enable strand slip monitoring or positioning of the strands within an end block. The setup is shown in Figure 14 will be attached to the strands. LDVTs will be mounted onto the strand and affixed to the framework. Any differential movement of the strand will be recorded by the LVDTs continuously during testing.

As an added measure, masking tape will be applied to the strand and the LDVT measurements will be validated using calipers to measure the distance from the end of the framework to the masking tape position. This will also be a precaution against instrumentation slippage or failure during the course of the testing.

LOAD MEASUREMENT

The load will be applied using a load frame equipped with a 500 kip capacity high-tonnage hydraulic cylinder and power unit. A 12-inch by 12-inch by ½-inch thick steel plate will be placed upon the top flange of the concrete specimen. A load cell with a 500 kip capacity will be placed on this plate. Another ½-inch plate will be placed upon the load cell. Then the hydraulic cylinder, suspended from the load frame, will rest upon the top plate.

The load cell will be used to measure and monitor the amount of force being applied to the specimen over time. All electronic measurements will be collected using a LabView data acquisition device and will be recorded in real time as the loading progresses. Loading position points are shown in Figures 3,4,5,7, and 8 for each small-scale specimen test.

STRAIN MONITORING IN EXTENDED STRANDS

To examine the stress/strain of strand in a poured end block, strain gages will be affixed to the surface of the strand. It is difficult to affix a strain gage to the rounded elements of strand, so a small portion of the strand will be flattened by filing. This will change the properties of the strand, but for the purpose of this examination, it should not change the shear capacity results of the girder.

Additionally, a DEMEC grid will be position on the side and end of the end block to monitor the surface strain of the concrete in response to loading. This measurement is important so that the additional stresses from strand anchorage can be quantified in end diaphragm and girder end anchorage design.

ANALYTICAL PROGRAM

For shear design, AASHTO LRFD recommends the use of either the sectional model or strut-and-tie modeling for flexural design. The sectional method assumes that the response at a particular section depends only on the calculated values of the sectional forces and does not consider how the forces were introduced into the member. The strut-and-tie method considers the flow of forces within a member both at disturbed and undisturbed regions of the beam.[9] In this investigation, both the sectional and strut-and-tie methods will be examined and a prediction of the shear strength of the beam will be determined. Variation of end conditions will be considered in the estimation of shear capacity.

STRUT-AND-TIE MODELING

Strut-and-tie models can be used to determine internal force effects near supports and at the application of concentrated loads. Sectional modeling assumes a linear strain distribution across the cross-section of the beam and that the longitudinal strains vary linearly across the length of the beam. The strut-and-tie modeling considers the flow of forces through the complete structure. This flow requires that both tensile and compressive forces be developed to resist a load condition.[8]

AASHTO LRFD 2010 section 5.6.3.6 specifies that crack control reinforcement be used when a strut-and-tie design is performed. This reinforcement consists of orthogonal bars in a grid pattern not exceeding a spacing of $d/4$ or 12 inches. Minimum steel requirements are given by equations 5.6.3.6-1 and 5.6.3.6-2. These provisions will be incorporated into the design of the small- and large-scale specimens in this investigation.

SECTIONAL SHEAR DESIGN

Section 5.8.3 of AASHTO LRFD 2010 specifications gives the sectional design approach for shear in concrete beams. This method estimates the resistance of members in shear and torsion by satisfying the conditions of equilibrium and compatibility of strains and by using experimentally verified stress-strain relationships for reinforcement and for diagonally cracked concrete. A nominal shear resistance is estimated by summing the shear contributions from the concrete material, mild reinforcement and prestressing strand.

The shear capacity from the concrete and steel in a concrete girder can be calculated by three different methods according to AASHTO LRFD 2010. The first method is used for non-prestressed sections where β, a factor indicating the ability of cracked concrete to transfer tension and shear is set to 2.0 and the angle of inclination of diagonal compressive stress, Θ, is set at 45 degrees.

A generalized procedure is given in section 5.8.3.4.2 where β and the angle Θ are given by algebraic equations derived from the Modified Compression Field Theory which is a comprehensive behavioral model for the response of diagonally cracked concrete subject to in-plane and normal stress. Once the calculated values for β and Θ are determined, the concrete contribution can be calculated using equation 5.8.3.3-3 and the mild steel reinforcement contribution can be calculated using equation 5.8.3.3-4.[8]

The third approach is a simplified approach which examines the nominal shear resistance provided by the concrete when inclined cracking is caused from the combination of shear and moment, and when the inclined cracking is caused from excessive principal stresses in the web. The smallest estimated capacity under these two conditions is used for the capacity of the concrete. The capacity of the steel and prestressing is calculated as the other methods.

The strut-and-tie model method, the generalized method, and the simplified methods will be used to calculate the shear capacity for each end anchorage condition. These estimated capacities will then be compared to the measured capacity results of testing and an evaluation of the effectiveness of each method will be conducted.

ANTICIPATED RESEARCH RESULTS

TENSILE STRESS MAPPING IN PRESTRESSED GIRDER END REGIONS

Through the testing of the small- and full-scale specimens, we anticipate being able to show a progression of tensile stress within the disturbed region of these prestressed girder ends. This progression is important as it will map the position and progression of stress and cracking with the larger strand with different end conditions.

The variety of end conditions used in this investigation will test the validity of each model with the use of 0.7-inch strand. Results from the tensile stress mapping will be compared with the predicted results from current mathematic models. Since shear capacity has been shown to correlate to end zone cracking, recommendations for the prevention of end zone cracking may be developed based on the results of this comparison.

EFFECTIVENESS OF TESTED END ANCHORAGE

One finding of this research will be a comparison of the effectiveness of each of the end anchorage conditions investigated. Mathematic predictions will be compared to actual testing data to assess the performance of these models using 0.7-inch and 0.62-inch strand. Shear capacities will be measured and each end anchorage condition will be assessed.

The effects of debonding and draping with end anchorage will be examined in this study. Several of the specimens will be debonded to assess the capacity with and without debonding. Draping of the strands will also be examined and the effect that draping has upon the shear capacity of the specimen will be examined. This study should aid the designer in selecting the best end anchorage for the girder design and provide the designer with an understanding of shear capacity change associated with their selection.

DETERMINATION OF TRANSFER LENGTH AND DEVELOPMENT LENGTH OF HIGH-CAPACITY STRAND

Surface concrete strain will be used to estimate the transfer and development lengths of the 0.7-inch and 0.62-inch strand. From DEMEC strain gage readings the distance where the effective stress of the strand is transferred to the concrete will measured and compared to results from previous research. Also, differing end conditions may have an impact on both transfer and development length.

Transfer length determination will not require a specific loading condition as these results can be obtained without dynamic loading on the specimen. Development Length tests can be performed upon these specimens after the shear/end anchorage tests are performed. The end of each girder can be removed and the load cell can be positioned at the center of the specimen. The small-scale specimens should be of sufficient length to yield the prestressing strand at the center. Surface strain measurements at the centroid of the prestressing strand will be used to measure the stress within the strand. Even if the concrete is cracked, the change in length between the DEMEC points will represent the change in length of the strand.

CAPACITY PREDICTION COMPARISON

Following testing and the analytical study, each mathematic model will be compared to the testing results with from this study. A shear capacity for each model will be estimated and that capacity compared to each mathematic model's prediction. An assessment of the model's ability to predict the shear capacity of each specimen with the varying end conditions will be made. The models will then be evaluated and a determination of the model's applicability for use with 0.7-inch or 0.62-inch strand will be made.

This assessment will be very beneficial to designers when selecting this strand to increase the span capacity of their prestressed concrete girders.

LONG-TERM PRESTRESS LOSS EXAMINATION WITH HIGH-CAPACITY STRAND

Specimens not used for transfer length evaluation can be monitored for a long-term evaluation of prestress loss of the 0.7-inch and 0.62-inch strand over time. The damaged ends of these girders will be removed and the specimens will be stored and monitored over time. The prestress loss will be measured using the DEMEC points once a month for the first two years and then semi-annually thereafter. After five years, these findings will be published for general use.

RECOMMENDATIONS FOR USE OF HIGH-CAPACITY STRAND

This study is designed to be a comprehensive study of the use of high-capacity strand using varying end anchorage conditions. As such, shear capacity is investigated under varying end conditions with the high-capacity strands. Differing parameters affecting shear were normalized and the shear capacity was predicted using three mathematic models. Transfer and development lengths of the high-capacity strand are to be evaluated during this study.

Recommendations from this study may include the use of specific end anchorage when debonding the strand, predictions of end zone cracking and methods of preventing it, the best mathematic models to use and revisions to existing ones for the use of high-capacity strands, and the effect of debonding upon shear capacity. Design practice guidelines should also be developed to enable the designer to use high-capacity strand confidently.

As with any research, other recommendations and findings will occur during the course of study and the assimilation of the data obtained. These findings alone will be extremely beneficial, but others may become apparent during the course of this study.

ACKNOWLEDGEMENTS

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