

Anchorage of 0.6" Diameter Strands

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Anchorage of 0.6" Diameter Strands
NDOR Project Number SPR-1 (03) P551

Final Report

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| 16. Abstract In a recent investigation on shear limits in precast prestressed concrete girders for the Nebraska Department of Roads (NDOR), it was determined that the AASHTO LRFD limit of $0.25f_c b d_v$ for maximum shear reinforcement is attainable as long as an adequate number of strands is anchored into the abutment diaphragms. In addition, extending strands in prestressed concrete girders beyond member ends and bending them into cast-in-place pier diaphragms can be a cost-effective method of controlling creep and shrinkage effects in bridges designed as simple spans for girder and deck weights and continuous spans for additional loads. In this research project, the pullout capacity of 0.5 in. and 0.6 in. (12.7 and 15.2 mm) diameter strands is evaluated. A full-scale test of beam end anchorages was conducted. A proposed design procedure for a bent strand anchorage is included. In addition, a confinement reinforcement detail for bridge diaphragms is proposed. Two numerical design examples are included together with design recommendations for determining the required number and length of strands that need to be bent and embedded into the diaphragms. | | | |
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ABSTRACT

In a recent investigation on shear limits in precast prestressed concrete girders for Nebraska Department of Roads (NDOR), it was determined that the AASHTO LRFD limit of $0.25f'_c b d_v$ for maximum shear reinforcement is attainable as long as an adequate number of strands is anchored into the abutment diaphragms. In addition, extending strands in prestressed concrete girders beyond member ends and bending them into cast-in-place pier diaphragms can be a cost-effective method of controlling creep and shrinkage effects in bridges designed as simple spans for girder and deck weights and continuous spans for additional loads. In this research, the pullout capacity of 0.5 in. and 0.6 in. (12.7 and 15.2 mm) diameter strands is evaluated. Full-scale test of beam end anchorages is investigated, and a design procedure for bent strand anchorage is proposed. In addition, confinement reinforcement detail of bridge diaphragm is proposed. Two numerical design examples are included together with design recommendations for determining the required number and length of strands that need to be bent and embedded into the diaphragms.

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CHAPTER 1

INTRODUCTION

1.1 Problem Statement

The shear capacity of pretensioned concrete simple span I-girders can be significantly increased by extending and bending strands that already exist in the bottom flange into the end diaphragms ^(1, 2, 3). In a recent investigation on shear limits in precast prestressed concrete girders ⁽¹⁾ for the Nebraska Department of Roads (NDOR), all strands in the bottom flange of the girders were extended and bent into the end diaphragms. This provided sufficient anchorage for the strands to act as a “tension tie” to develop the strut-and-tie mechanism for shear resistance.

The developed tension tie can significantly enhance shear capacity. However, extending and bending all strands available in the bottom flange of the girders may cause steel congestion in the diaphragm. Knowledge of the anchorage capacity of non-prestressed bent strands would be useful in determining the number of strands required to be bent into the diaphragms.

In a long bed prestressing operation, several feet of strand already exists in the gap between girder ends. After the prestress is released and the girders are separated, strand extensions beyond the face of girder ends are generally removed and discarded. In Nebraska and several other states, it has been the practice for over two decades to remove all but four to twelve strand extensions.

These strands are then bent in the plant, as shown in Figure 1.1, using a simple strand-bending tool. This technique is rather simple and adds almost nothing to the cost

of girder fabrication. However, the number of bent strands and the embedment lengths has been selected by trial and adjustment based on observation of the behavior of actual bridges, rather than using design calculations.

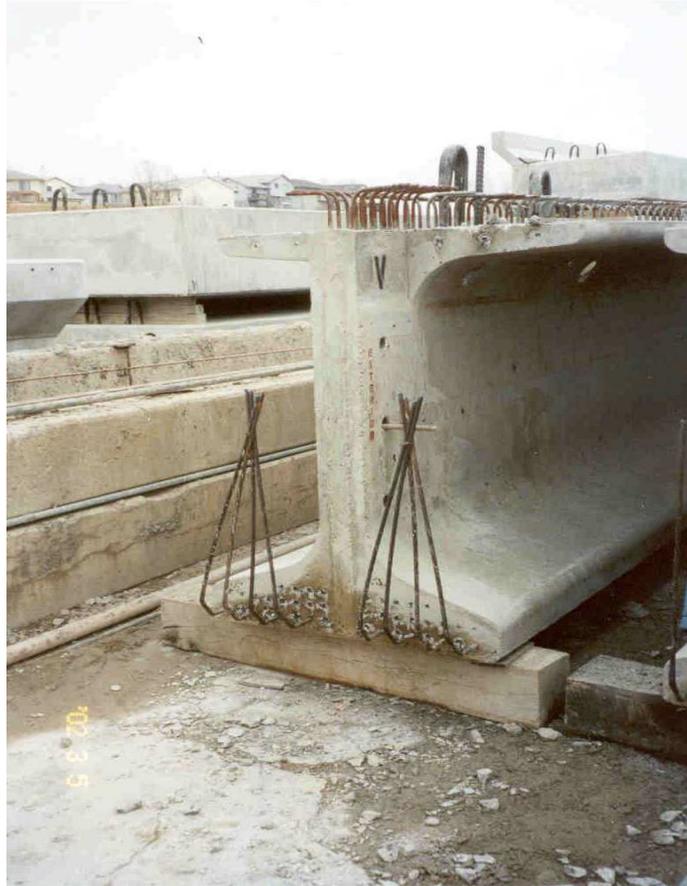


Figure 1.1 Prestressing Strands Being Extended and Bent

Another reason for extending strands in prestressed concrete girders beyond member ends and bending them into cast-in-place abutment pier is to control creep and shrinkage effects in bridges designed to be simple spans for girder and deck weights and continuous spans for additional loads ⁽⁴⁾. When the prestressed concrete girders are set on piers or abutments, and made continuous reinforcement in cast-in-place decks, the beams are restrained at their ends.

As a result, time-dependent movement occurring after the deck concrete is cured causes positive restraint moments over the piers. This behavior depends on the amount of girder creep that is being restrained; girders made continuous at a relatively young age experience large restraint moments.

There are a number of methods to estimate the time-dependent restraint moment over bridge piers. They include a time-step computer analysis as described by Ma et al. ⁽⁴⁾ and by Oesterle et al. ⁽⁵⁾, a closed form solution by Freyermuth ⁽⁶⁾ and a common-practice empirical recommendation by Mirmiran et al. ⁽⁷⁾. These methods vary in their consideration of whether the positive restraint moment should be calculated assuming a cracked or an uncracked section in the area over the piers. This research project does not address how the moment should be calculated. It demonstrates how, for a given calculated moment, the strand embedment requirement can be calculated. It is assumed that the moment is a serviceability limit state design parameter for which an allowable working design is specified. It is reasonable to assume the stress limit in the prestressing strands = 30 ksi (207 MPa) as a means of crack control. The stress-versus-embedment-length formula developed herein would permit other stress limits to be used by the designer.

Another benefit of embedding strands into end diaphragms is to enhance resistance to shear. There is adequate evidence ^(1, 2, 3) of the importance of anchorage of longitudinal reinforcement at member ends. This is generally the bottom reinforcement at the abutments. In simple span construction, used for example in Texas and Florida, it is also the bottom reinforcement at the piers. In continuous span construction, such as that

used in the Midwest, the tensile longitudinal reinforcement at the piers to be considered in shear design is the top continuity reinforcement. For that situation, extension of bottom strands into pier diaphragms is not relevant to shear design.

Design for shear is based on the strength limit state. Thus, checking of strand embedment at the abutments, and at the piers for simple span construction, should be based on factored loads and the stress in the strand that can be achieved at pull-out failure.

1.2 Research Objectives and Approach

The work presented herein discusses the evaluation of the pullout capacity of 0.5 in. and 0.6 in. (12.7 and 15.2 mm) diameter strands and gives recommendations for determining the required number and length of strands to be bent and embedded into the diaphragms.

Anchorage of the longitudinal reinforcement into bridge diaphragms is an effective and virtually no-cost means of achieving the maximum capacity of precast prestressed beams and avoiding unacceptable cracking due to restraint moments.

1.3 Scope and Layout

This study focuses on the behavior of the end zone of precast pretensioned concrete bridge girders. The proposed design methods will be based on available experimental results and empirical analysis. The design of regions of members near support, considered “disturbed regions” or D-regions using special procedures including the strut-and-tie method, is not the focus of this report.

Chapter 2 contains background information and a literature review of pullout tests and beam end anchorages for pretensioned concrete bridge girders.

Chapter 3 deals with the experimental program of pullout tests of non-prestressed 90-degree bent strands. A design equation to determine the embedment length of 0.5-in. and 0.6-in. diameter strands is proposed. Recommendations for determining the required number and length of strands to be bent and embedded into the diaphragms are proposed.

Chapter 4 covers the experimental investigation of full-scale shear tests of the anchorage at beam ends. The anchorage detailing to enhance the shear capacity of concrete bridge girders is proposed.

Chapter 5 contains a design procedure of a 90-degree bent strand anchorage. In addition, a confinement reinforcement detail in the end diaphragm is presented for practical use. Two design examples are illustrated in this chapter as well.

Chapter 6 summarizes the conclusions and recommendations of the research project.

CHAPTER 2

BACKGROUND INFORMATION

2.1 Introduction

The use of untensioned, bonded prestressing strand for concrete reinforcing is quite common in the precast prestressed concrete industry. This use includes lifting handles, reinforcing for crack control, and connection reinforcing between precast elements. The use of untensioned, bonded prestressing strands as the anchorage reinforcement at member end, by extending the strands beyond member end and bending them into the end diaphragm, can also result in also avoiding bond failure, which causes the girder to not reach its maximum shear capacity^{1,2}. The latter use provides justification for studying the behavior of the embedment length of a 90-degree bent untensioned strand. Much research has been conducted to determine the embedment length of pretensioned straight strands. In the Missouri study⁹, untensioned straight, frayed and bent strands were also investigated. However, the Missouri experimental program for 90-degree bent untensioned strands considered only 0.5-in. diameter strands. Use of 0.6-in. strands has been increasing in recent years. Therefore, pullout tests on 0.5-in. and 0.6-in. diameter strands were undertaken and are presented in Chapter 3.

2.2 Background

The bond characteristics and development length of strands were studied using mostly prestressed straight strands⁽⁸⁻¹¹⁾. One exception is the University of Missouri study performed in the 1970s⁽¹²⁾. The objective of the Missouri study was to examine the use of embedded prestressing strands to develop positive moment continuity of precast,

prestressed I-beam members. Most of the experiments in the Missouri study focused on comparison of the bond characteristics of untensioned bent strands, straight strands, and frayed strands. In this research project, only pullout tests of non-prestressed bent strands were performed to expand the scope of the Missouri tests.

The full tensile strength of a prestressing strand can usually be developed at a section, provided the strand extends in the concrete a sufficient distance beyond that section. The length of bar beyond the section required to develop the strength of the bar is known as the development length. When the straight length of strand available for anchorage is insufficient, the reinforcement should be bent to aid anchorage. 90-degree bent anchorages for prestressing strands have distinct advantages that have been recognized by concrete bridge engineers in some states.

The bond of untensioned strands in concrete differs from that of plain, deformed reinforcing bars, and tensioned prestressing strands. A 90-degree bent strand loaded in tension develops stresses in the manner shown in Figure 2.1. The stress in the strand is resisted by bond on the surface of the strand and by bearing on the concrete inside the bent strand. The horizontal part of the embedded strand moves inward, leaving a gap between the vertical part of the strand and the concrete outside the bend. Failure in the direct pullout bent strand testing involves splitting cracks of the concrete surrounding the strand. Because of the flexibility of non-tensioned prestressing strands, the vertical part of the strand near the tail does not tend to straighten and produce compressive stress on the outside of the tail, unlike mild steel.

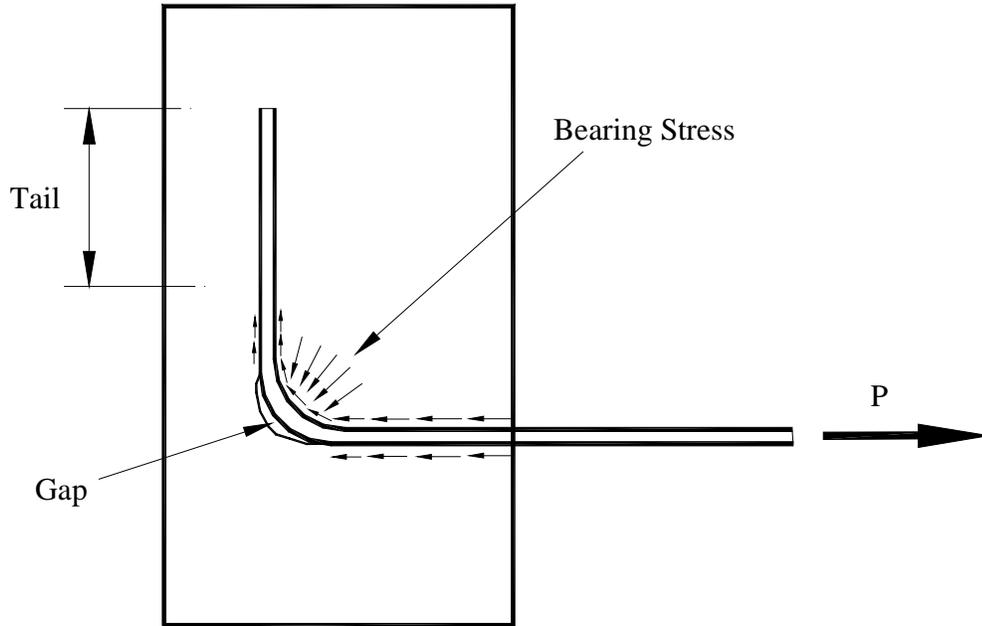


Figure 2.1 Stresses acting on pullout strand

2.3 Beam End Anchorage Enhances Shear Capacity

The traditional and simple 45-degree truss model clearly and correctly shows that the stresses in the longitudinal tensile reinforcement in the shear span are larger than those predicted from beam theory. If the longitudinal tensile reinforcement is not well anchored in the beam support region, premature shear failure is unavoidable. Bridge I-beams can take advantage of the existing beam end diaphragm, where the beam strands can be anchored.

In beams with a small shear span-depth ratio a/d , arch action is the predominant mode of shear resistance after the onset of diagonal cracking. Accordingly, the bottom strands are required to function as the tie of this arch. The straight strand anchorage for a pretensioned concrete beam is likely to reduce the ultimate shear strength due to strand

slip. In such situations, it is better to carry all of the bottom strands to the end zone and bend them up at the beam end.

A benefit of embedding strands into end diaphragms is to enhance resistance to shear. This method generally uses the bottom reinforcement at the abutments. In simple span construction, used for example in Texas and Florida, there is also bottom reinforcement at the piers. In continuous span construction such as that used in the Midwest, top continuity reinforcement at the piers is considered in shear design. For this situation, extension of the bottom strands into the pier diaphragms is not relevant to shear design.

CHAPTER 3

PULLOUT CAPACITY OF NON-PRESTRESSED BENT STRANDS

3.1 Introduction

Traditionally, the anchorage performance of various reinforcing bars embedded in concrete of different strengths is determined from pullout tests. This chapter covers the experimental program of pull-out tests of varied anchorage lengths for non-prestressed 90-degree bent strands in the end diaphragm. In this research project, only pullout tests of non-prestressed bent strands were performed in order to expand the scope of the Missouri tests.

3.2 Experimental Program

To study the behavior of a 90-degree bent untensioned strand embedded in a concrete mass, a series of 55 direct pullout tests was conducted on specimens that contained 22 different embedment lengths. The objectives of the experimental investigation were to:

- (1) Determine the pullout capacity of various embedment lengths of untensioned bent strands in a simulated concrete diaphragm.
- (2) Recommend a method for determining the required number and length of strands that need to be bent and embedded into the diaphragms.

In designing the pullout test specimens, the following parameters were considered:

(1) Strand horizontal embedment length, L_h , which is defined as the distance from the end face of the pretensioned I-girder to the centerline of the vertical leg of the extended strand, as shown in Figure 3.1;

(2) Strand vertical embedment length, L_v , which represents the vertical portion of the extended bent strand, as shown in Figure 3.1; and

(3) Diameter of strands.

The total embedment length (L_e , equals to $L_h + L_v$) was considered as the effective embedment length of the strand. The specified concrete compressive strength for all specimens was 4000 psi (28 MPa). This strength was believed to be at the low end of what is currently used in practice. The results of this investigation should be conservative for higher strength concrete.

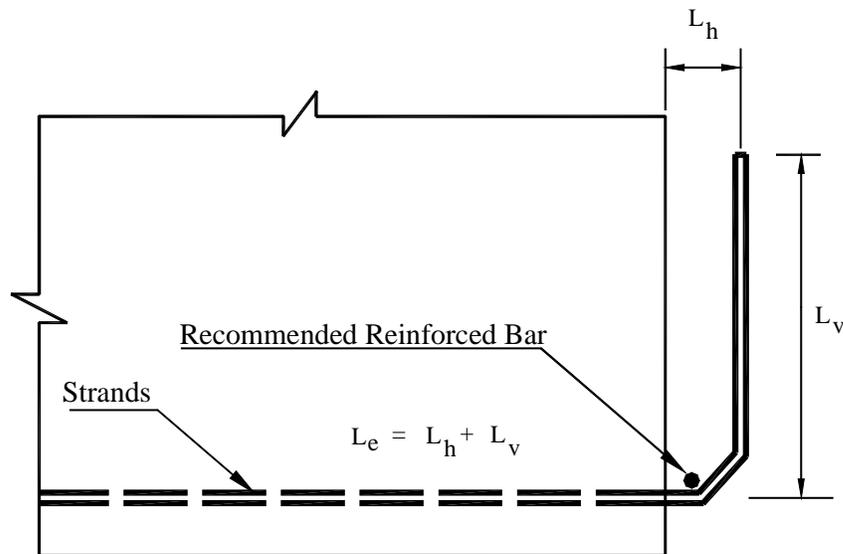


Figure 3.1 Bent Strand Details

Diaphragm reinforcement, diaphragm volume and continuity between girders aid in confining the embedded strands and improving their anchorage capacity. However, it

was more conservative and convenient to ignore these effects using a plain concrete mass in the experimental program. It should be noted, however, that placing a reinforcing bar on the inside corner of the strand bend in the diaphragm is a recommended detail as it significantly enhances strand anchorage.

3.3 Specimens and Test Procedures

Three specimens were designed and fabricated at the University of Nebraska Structures Laboratory. Table 3.1 shows the properties of Specimens 1, 2 and 3. The strand spacing shown in the table is centerline to centerline spacing. The horizontal embedment length for Specimen 1 was 6 in. (150 mm). The vertical embedment lengths varied from 4 to 25 in. (100 to 635 mm).

Table 3.1: The Properties of Specimens 1, 2 and 3

| Specimen | L_h (in.) | L_v (in.) | Strand Diameter (in.) | Strand Spacing (in.) | Specified Concrete Strength (psi) | Average Concrete Strength (psi) |
|----------|----------------|-----------------|-----------------------------|----------------------------|--|--|
| 1 | 6 | varies 4 to 25 | 0.5 | 4 | 4000 | 5350 |
| 2 | 10 | varies 4 to 25 | 0.5 | 4 | 4000 | 5350 |
| 3 | 6 | varies 12 to 46 | 0.6 | 6 | 4000 | 4063 |

Strands of 0.5 in. (13 mm) diameter Grade 270 (1860 MPa) low relaxation steel were used in this specimen. The total thickness of the specimen was 12 in. (305 mm). The cover of the vertical portion of the bent strand was 6 in. (150 mm). A 4 in. (100 mm) spacing of the bent strands was chosen to avoid the development of splitting cracks between strands that propagate to the surface. This action reduces the pullout capacity of the adjacent strands.

Specimen 2 was identical to Specimen 1, except that the total thickness of Specimen 2 was increased to 20 in. (510 mm). Therefore, the cover of the vertical portion of the bent strand was increased from 6 to 10 in. (150 to 250 mm). Also, the horizontal embedment length was increased to 10 in. (250 mm). These two specimens were designed to investigate the effect of the horizontal embedment length on pullout capacity.

For Specimen 3, the horizontal embedment length was 6 in. (150 mm). The vertical embedment length was varied from 12 to 46 in. (305 to 1170 mm). Strands of 0.6 in. (15 mm) diameter Grade 270 (1860 MPa) low relaxation steel were used. The total thickness of the specimen was 12 in. (305 mm).

The cover of the vertical portion of the bent strand was 6 in. (150 mm). A 6 in. (150 mm.) spacing of the bent strands was chosen to prevent cracks from propagating from the previously pulled strands and affecting the pullout capacity of the adjacent strands. The diaphragm width in a bridge would normally allow for a minimum of 6 in. (150 mm) horizontal strand embedment. Specimens 1 and 3 were designed to study the effect of strand diameters on pullout capacity.

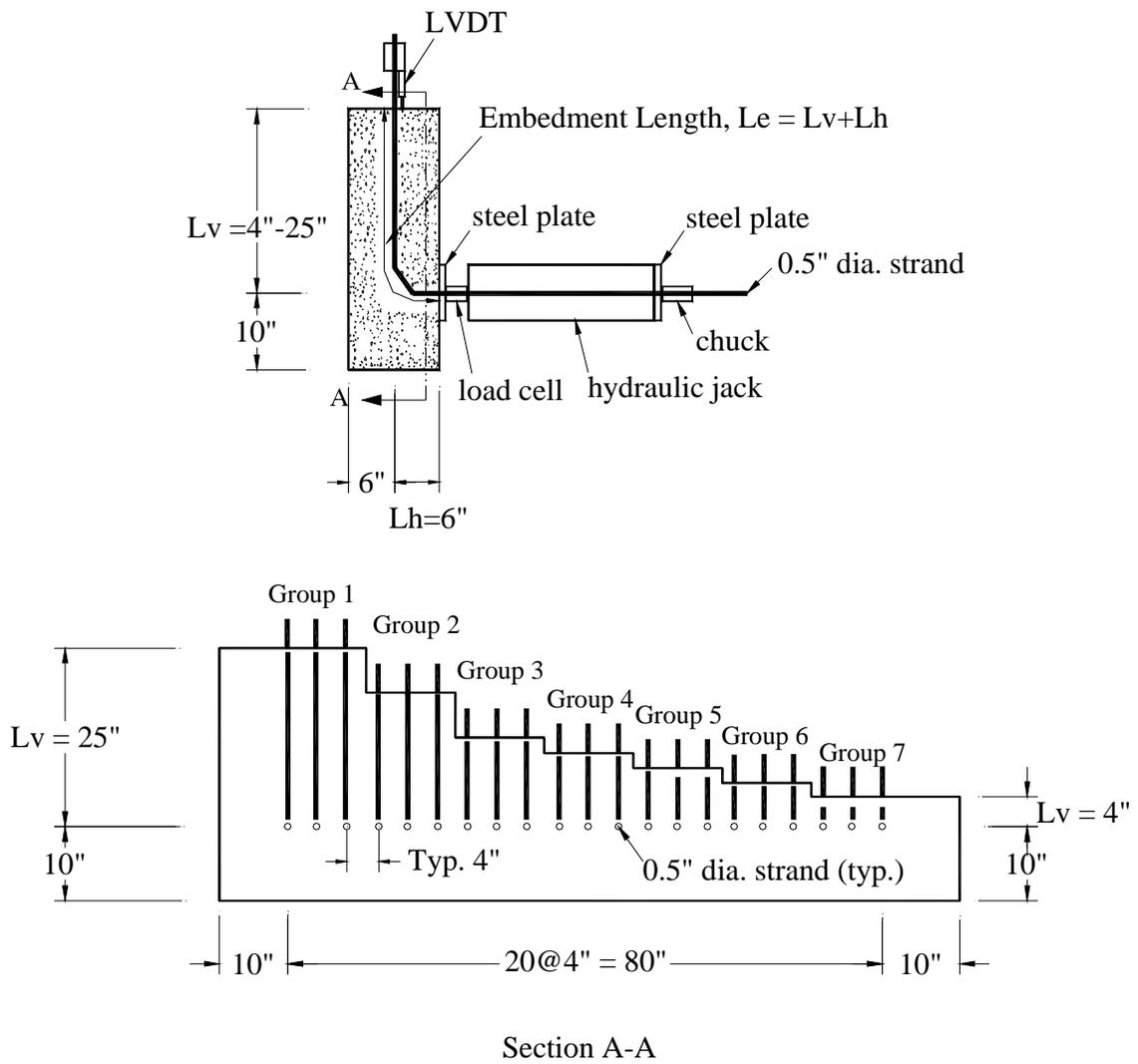


Figure 3.2 Specimen 1 and Loading Arrangement

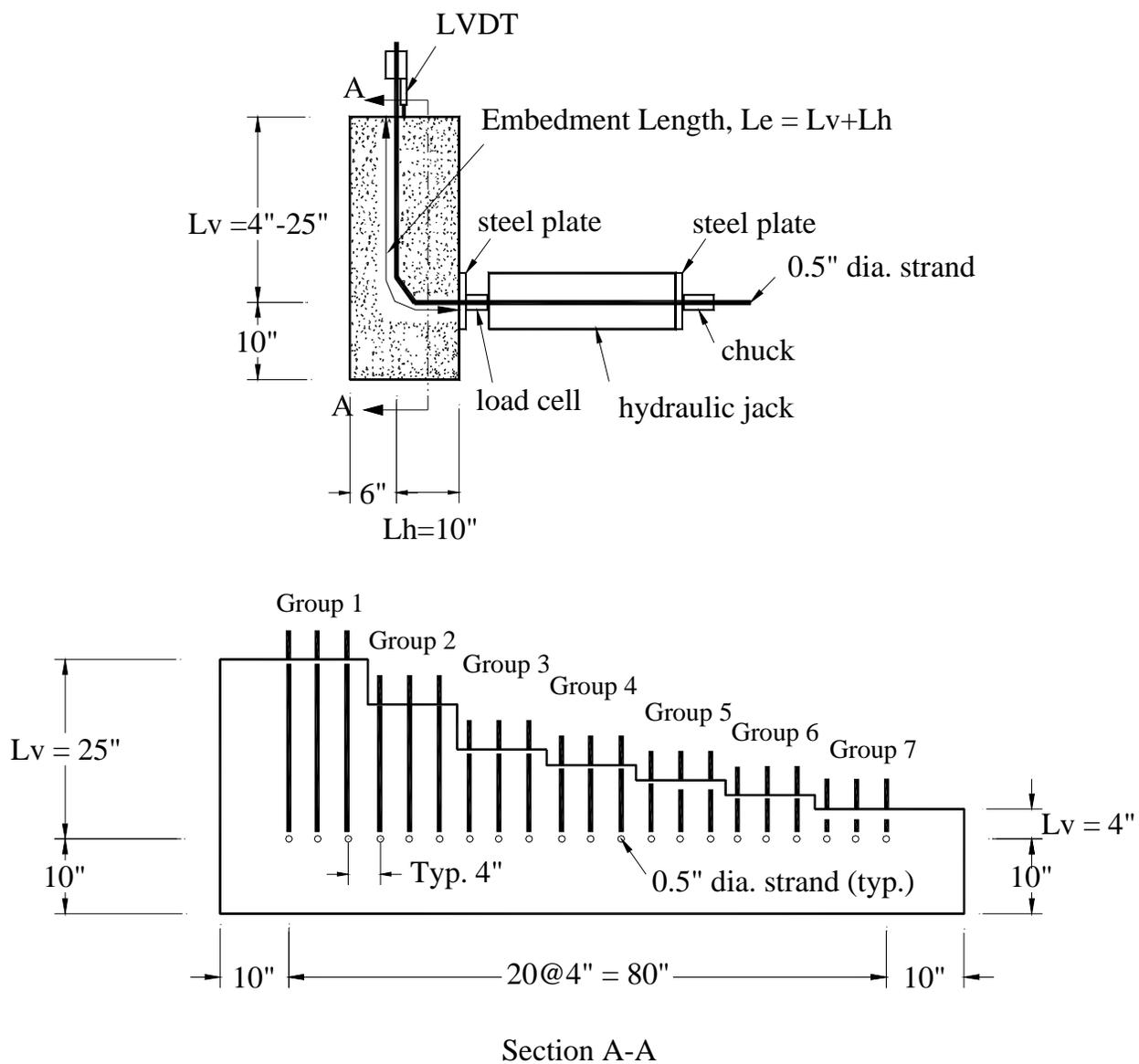


Figure 3.3 Specimen 2 and Loading Arrangement

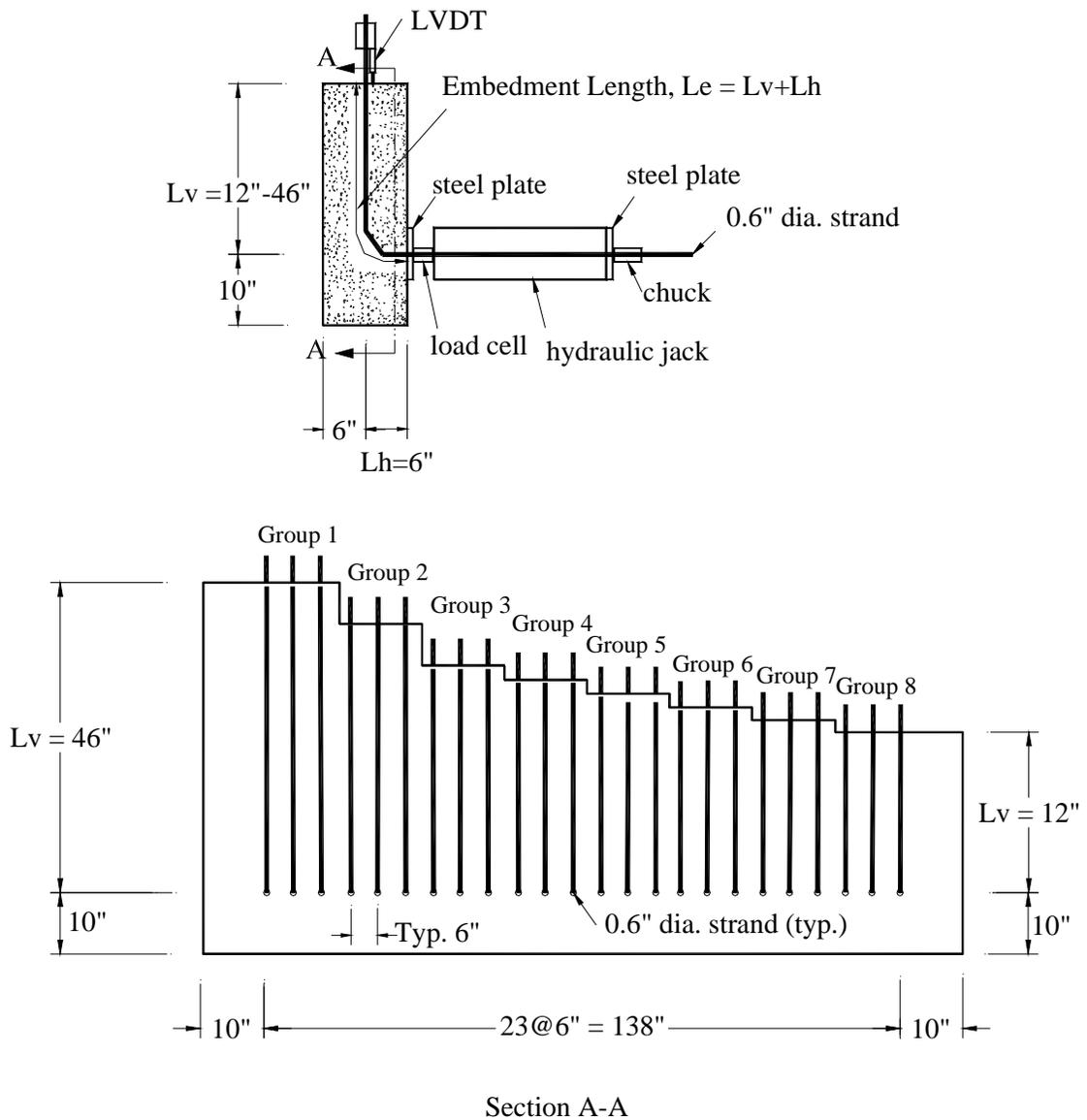


Figure 3.4 Specimen 3 and Loading Arrangement

Figures 3.2, 3.3 and 3.4 show sketches of the specimens and loading arrangement. In order to vary the vertical embedment length, step-like plywood forms were built to cast the specimens (see Figure 3.5). No confining reinforcement was used. This approach would allow test results to be valid in practice, regardless of the level of confinement

reinforcement used in a diaphragm of an actual bridge. This conservative approach was used in order to keep the number of experimental variables at a manageable level.



Figure 3.5 Specimen Fabrication

Figure 3.6 shows a typical setup of the pullout test. Load and strand slip were recorded during the testing. The load was monitored with a pressure gauge. The strand slip was carefully measured using a Linear Variable Differential Transducer (LVDT) with a range of ± 2 in. (± 51 mm).

The method of loading was to apply the load for two seconds and then allow the strand being pulled to slip for two seconds before applying the next load increment. The average rate of loading was about 495 lbs per second. The tests were stopped when the load significantly dropped after reaching a peak value.



LVDT

Figure 3.6 Pullout Test Set-Up

Two important relationships were developed and plotted from the test results:

(a) The relationship between the pullout force and slip for the various vertical embedment lengths; and

(b) The relationship between maximum strand stress, in terms of percent of specified strand strength and total embedment length.

To obtain a unified embedment length equation, relationships between the ratio of maximum steel stress to specified ultimate strand strength (f_{ps}/f_{pu}) and the ratio of vertical embedment length to nominal strand diameter (L_v/d_b) were plotted.

3.4 Test Results and Discussion

The average cylinder strength of Specimens 1 and 2 on the testing date was 5350 psi (37 MPa). The average concrete strength of Specimen 3 on the testing date was 4063 psi (28 MPa). Figures 3.7, 3.8 and 3.9 show the relationship between the pullout force and slip for different vertical embedment lengths. A summary of test results is listed in Table 3.2.

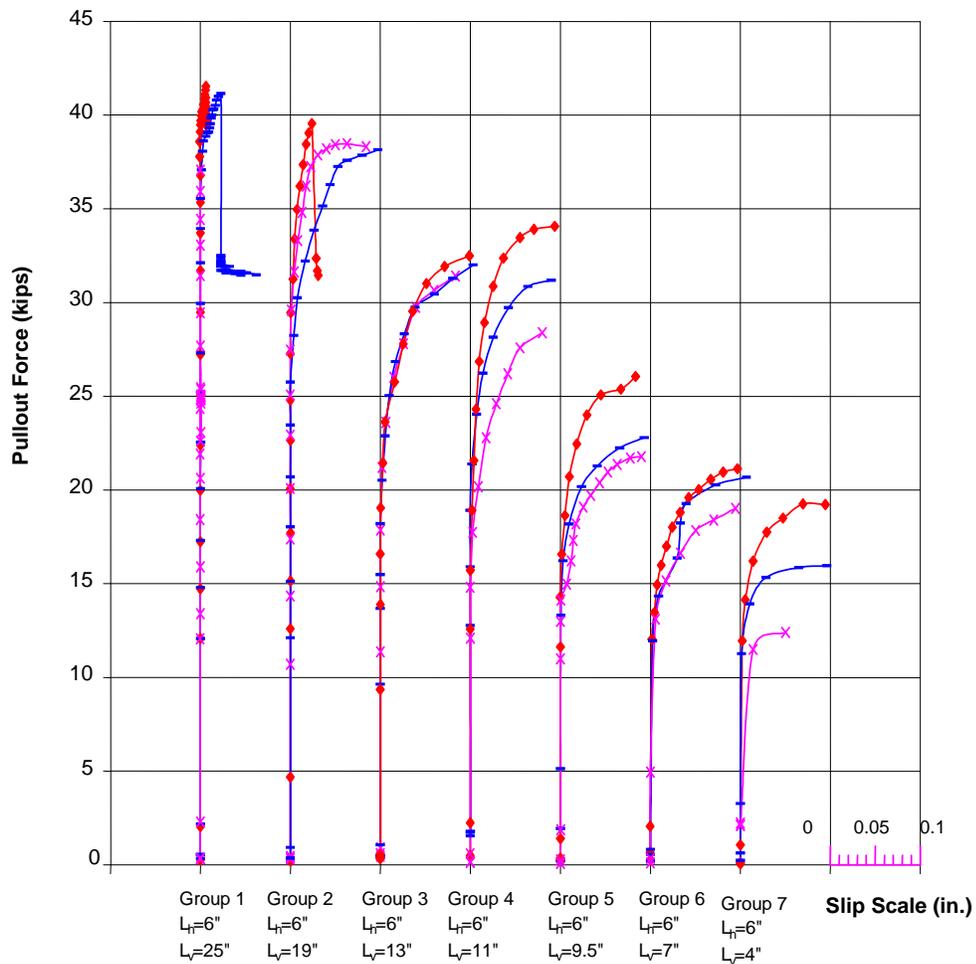


Figure 3.7 Relationship of Pullout Force and Strand Slip (Specimen 1)

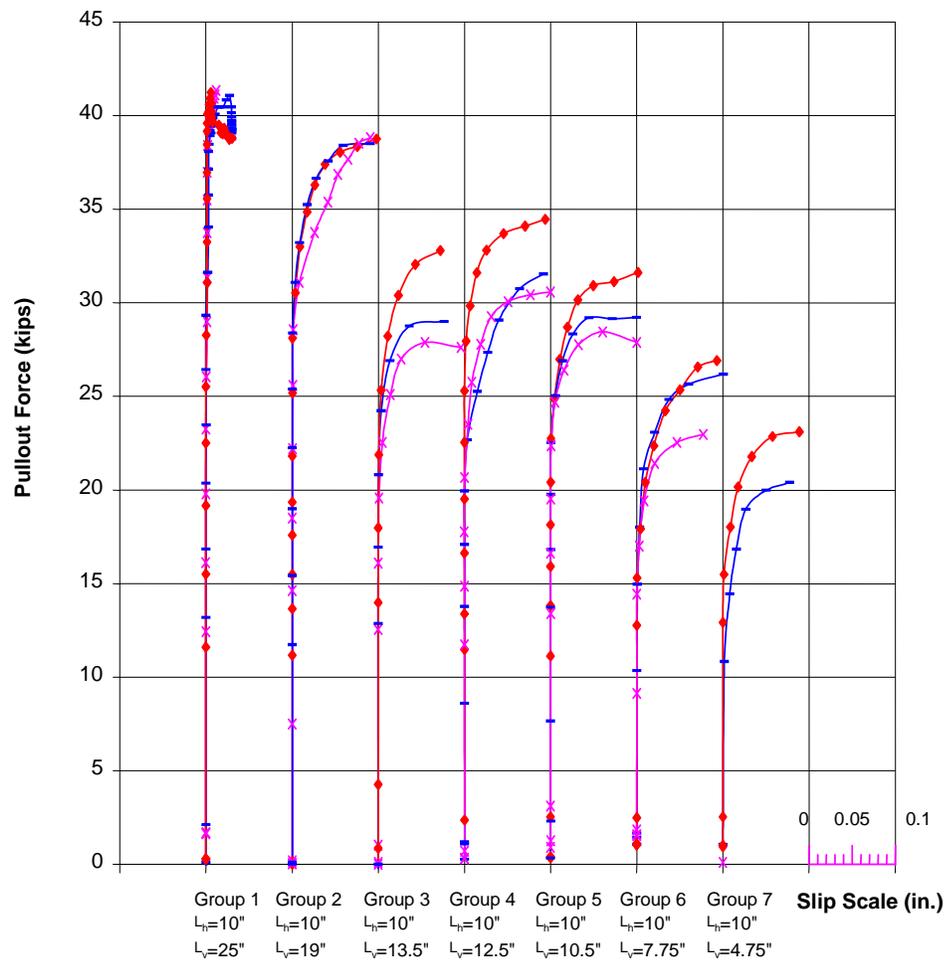


Figure 3.8 Relationship of Pullout Force and Strand Slip (Specimen 2)

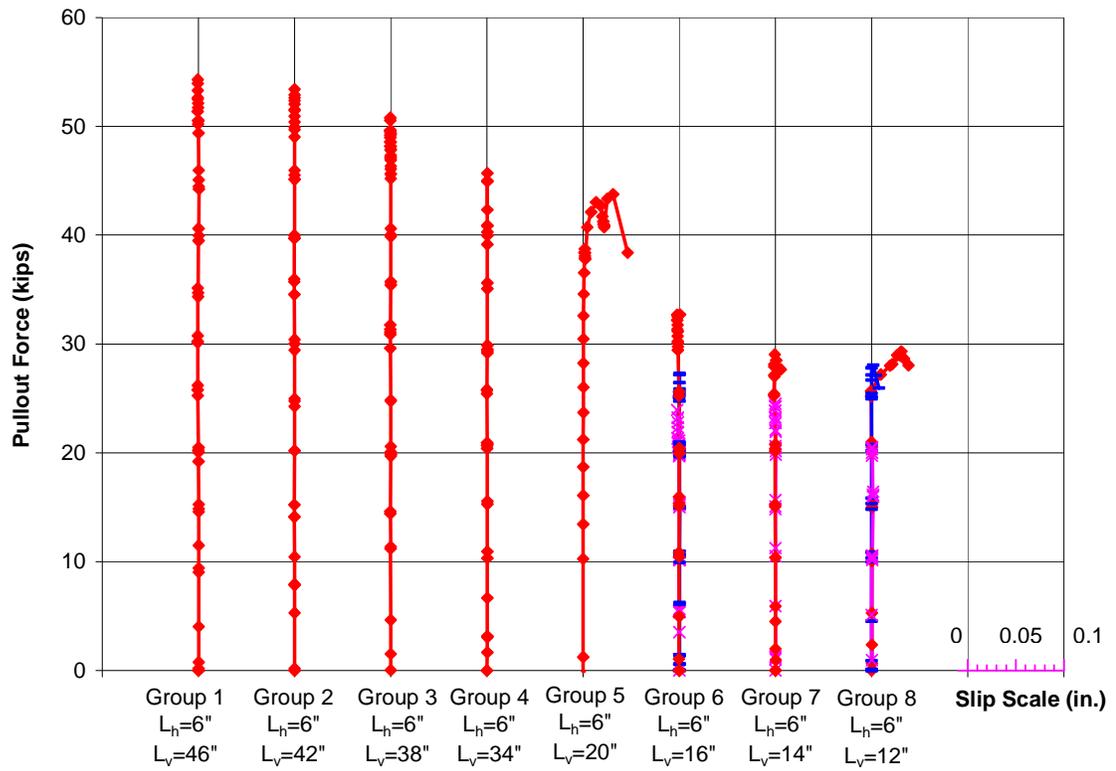


Figure 3.9 Relationship of Pullout Force and Strand Slip (Specimen 3)

Table 3.2 Summary of Test Results

| Specimen | Lh (in.) | Lv (in.) | Max. Pullout Force (kips) | Specimen | Lh (in.) | Lv (in.) | Max. Pullout Force (kips) | Specimen | Lh (in.) | Lv (in.) | Max. Pullout Force (kips) |
|-----------------|----------|----------|---------------------------|-----------------|----------|----------|---------------------------|-----------------|----------|----------|---------------------------|
| 1 | | | | 2 | | | | 3 | | | |
| 0.5-in diameter | 6 | 4 | 12.4 | 0.5-in diameter | 10 | 4.75 | 27.27 | 0.6-in diameter | 6 | 12 | 20.4 |
| | 6 | 4 | 18 | | 10 | 4.75 | 31.90 | | 6 | 12 | 28.1 |
| | 6 | 4 | 22.7 | | 10 | 4.75 | N/A | | 6 | 12 | 29.3 |
| | 6 | 7 | 21.9 | | 10 | 7.75 | 34.80 | | 6 | 14 | 24.5 |
| | 6 | 7 | 28 | | 10 | 7.75 | 36.30 | | 6 | 14 | 29 |
| | 6 | 7 | 30 | | 10 | 7.75 | 36.86 | | 6 | 14 | N/A |
| | 6 | 9.5 | 34.8 | | 10 | 10.5 | 39.70 | | 6 | 16 | 22.7 |
| | 6 | 9.5 | 36.2 | | 10 | 10.5 | 40.30 | | 6 | 16 | 27.3 |
| | 6 | 9.5 | N/A | | 10 | 10.5 | 40.33 | | 6 | 16 | 32.7 |
| | 6 | 11 | 32.2 | | 10 | 12.5 | 36.19 | | 6 | 20 | 43.8 |
| | 6 | 11 | 36.7 | | 10 | 12.5 | 37.50 | | 6 | 20 | N/A |
| | 6 | 11 | 37.7 | | 10 | 12.5 | 38.50 | | 6 | 20 | N/A |
| | 6 | 13 | 33.9 | | 10 | 13.5 | 34.60 | | 6 | 34 | 45.7 |
| | 6 | 13 | 36.5 | | 10 | 13.5 | 35.20 | | 6 | 34 | N/A |
| | 6 | 13 | 38.5 | | 10 | 13.5 | 36.83 | | 6 | 34 | N/A |
| | 6 | 19 | 38.4 | | 10 | 19 | 38.60 | | 6 | 38 | 50.8 |
| | 6 | 19 | 38.5 | | 10 | 19 | 38.70 | | 6 | 38 | N/A |
| | 6 | 19 | 39.5 | | 10 | 19 | 39.18 | | 6 | 38 | N/A |
| 6 | 25 | 41.2 | 10 | 25 | 41.07 | 6 | 42 | 53.4 | | | |
| 6 | 25 | 41.5 | 10 | 25 | 41.24 | 6 | 42 | N/A | | | |
| 6 | 25 | N/A | 10 | 25 | 41.30 | 6 | 42 | N/A | | | |
| | | | | | | | | 6 | 46 | 54.3 | |
| | | | | | | | | 6 | 46 | N/A | |
| | | | | | | | | 6 | 46 | N/A | |

The maximum pullout capacities of 0.5 and 0.6 in. (13 and 15 mm) diameter strands were 41.3 and 58.6 kips (184 and 261 kN), respectively, for 270 ksi (1860 MPa) specified strand strength. However, for 0.5 in. (13 mm) diameter strand, a total embedment length of 35 in. (890 mm) was needed to reach f_{pu} , the specified strand strength. For 0.6 in. (15 mm) diameter strand, the total embedment length of 48 in. (1220 mm) was needed to reach $0.9f_{pu}$.

The specified strand strength of the 0.6 in (15.2 mm) diameter strands could not be attained even with a total embedment length of 52 in. (1320 mm). For a total embedment length greater than 44 in.(1120 mm), the pullout force did not significantly increase (see Figure 3.9).

plotted (see Figure 3.11) between the ratio of the maximum steel stresses to the specified ultimate strand strength and the ratio of the vertical embedment lengths to nominal strand diameters to obtain a representative vertical embedment length equation.

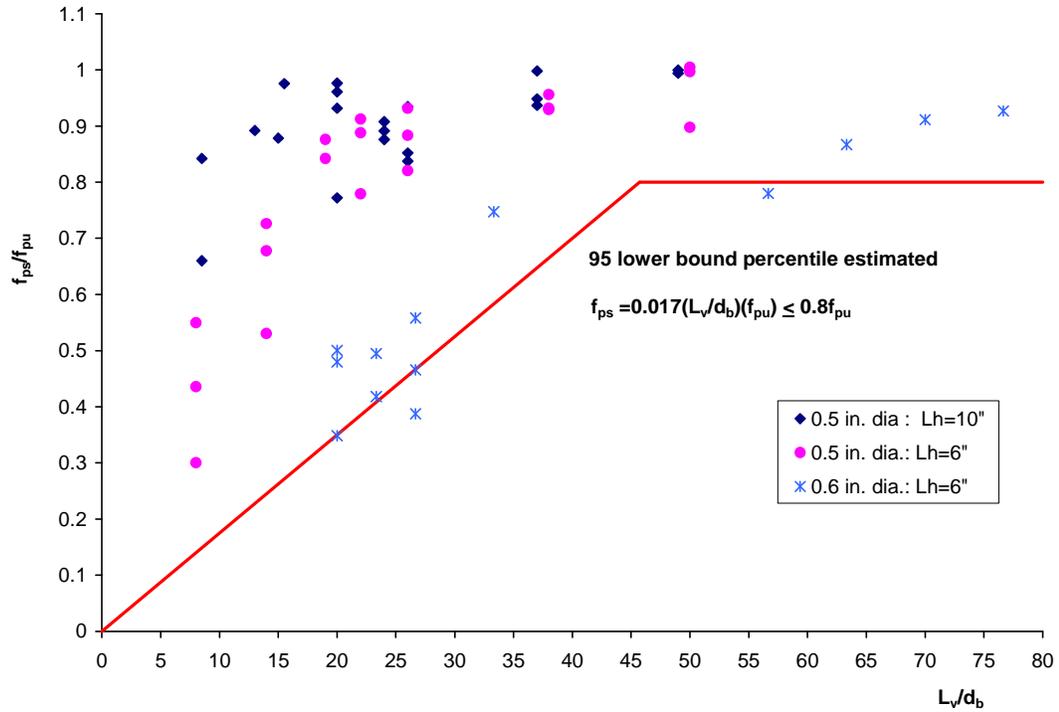


Figure 3.11 Vertical Embedment Length-Nominal Diameter Ratio vs Strand-Stress-to-Specified-Strength Ratio

The vertical part of the embedment length was considered to be the only independent variable in the equation because the 6 in. (150 mm) horizontal length is fixed to reflect common diaphragm dimensions in practice. An empirical embedment length equation using the fifth percentile value from lower bound of the test results was developed:

$$f_{ps} = 0.017f_{pu} L_v/d_b \leq 0.8f_{pu} \quad (3.1)$$

where f_{ps} = developed strand stress

L_v = vertical embedment length of non-prestressed bent strand

f_{pu} = specified tensile strength of prestressing tendons

d_b = nominal diameter of strand

Equation 3.1 gives the developed stresses in the strand corresponding to the vertical embedment length. At the upper limit of Equation 3.1, $(0.8f_{pu})$ the vertical embedment lengths, L_v , for 0.5 in. and 0.6 in (13 and 15 mm) diameter strands are 24 in. and 29 in. (610 and 737 mm), respectively.

It is therefore recommended that the total embedment length, L_e , be at least 30 in. (760 mm) for 0.5 in. (13 mm) diameter strands, and at least 36 in. (914 mm) for 0.6 in. (15 mm) diameter strands, to attain a strand stress of $0.8f_{pu}$. These values are based on the test results with concrete strength of at least 4000 psi (28 MPa).

From Equation 3.1, assuming a service level strand stress of 30 ksi (210 MPa), the vertical embedment length is about 4 in. (100 mm). This length was the smallest vertical length used in the testing. The test results of the three specimens tested at that length showed high variability as shown in Figures 3.8 and 3.11. Specimens with longer vertical strand embedment had a narrow range of variability. Because of the minimal cost involved, it is recommended that the minimum vertical embedment used in design not be less than 10 in. (250 mm), for a total horizontal plus vertical embedment of not less than 16 in. (406 mm). With the recommended minimum length, the strand is guaranteed to develop much higher than the 30 ksi (210 MPa).

Table 3.3 summarizes the recommended embedment lengths for both 0.5 in. and 0.6 in. (13 and 150 mm) diameter strands. These lengths are recommended based on 4 in. (100mm) strand spacing for 0.5 in.(13 mm) diameter strands and 6 in.(150 mm) strand spacing for 0.6 in.(15 mm) diameter strands.

Table 3.3 Recommended Embedment Lengths

| Diameter of Strand (in.) | Total Embedment Length for Shear Design (in.) | Total Embedment Length for Time-Dependent Design (in.) |
|-----------------------------|--|---|
| 0.5 | 30 | 16 |
| 0.6 | 36 | 16 |

CHAPTER 4

FULL SCALE TEST OF BEAM END ANCHORAGE

4.1 Introduction

Design for shear is based on the strength limit state. Thus, checking strand embedment at the abutments, and at the piers for simple span construction, should be based on factored loads and the stress in strand that can be achieved at pull-out failure. In chapter 3, the pullout capacity of non-tensioned prestressing strands was experimentally investigated. However, the pullout capacity proposed in chapter 3 is based on a single strand pullout without considering the group effect of the tension tie. In a concrete bridge girder, the bottom flange contains tens of prestressed strands. When the girder is loaded, all bottom reinforcement is expected to resist the tension force at the same time. However, due to the extra wide bottom flange of an I-girder, the strands at the flange tip may carry less tension than the strands in the line of the web. To investigate the pullout capacity of the non-tensioned prestressed strands at the bottom flange, and to propose an appropriate detail of anchorage in the beam end, a full- scale test of an NU I-girder was performed.

4.2 Experimental Program

4.2.1 Testing Specimens

Four non-pretensioned NU 1100 I-beams were designed as shown in Figures 4.1 and 4.2. All beams were designed for 28-day concrete strength of 8,000 psi. All end blocks were designed for 4000 psi concrete strength. The shear reinforcement for all girders and the confinement reinforcement in all end blocks were deformed bars having a

specified yield strength of 60 ksi. 2#4 bars at a spacing of 4 in. were provided for shear reinforcement as shown in Figures 4.1b) and 4.2. The specimens were fabricated in the Bellevue plant of the Rinkers Materials Company. Figure 4.3 presents the typical NU1100 I-section with 26 prestressing strands. All specimens had 22 straight strands with a spacing of 4 in. in the bottom flanges. The bottom strands were 0.6-in. diameter, Grade 270 ksi, low-relaxation 7-wire strands. Four straight strands of 0.5-in. diameter, Grade 270 ksi, low-relaxation 7-wire strands were provided in the top flanges. The bottom and top strands were pulled only to a stress level of 13,100 psi in order to straighten and keep the strands in the required positions.

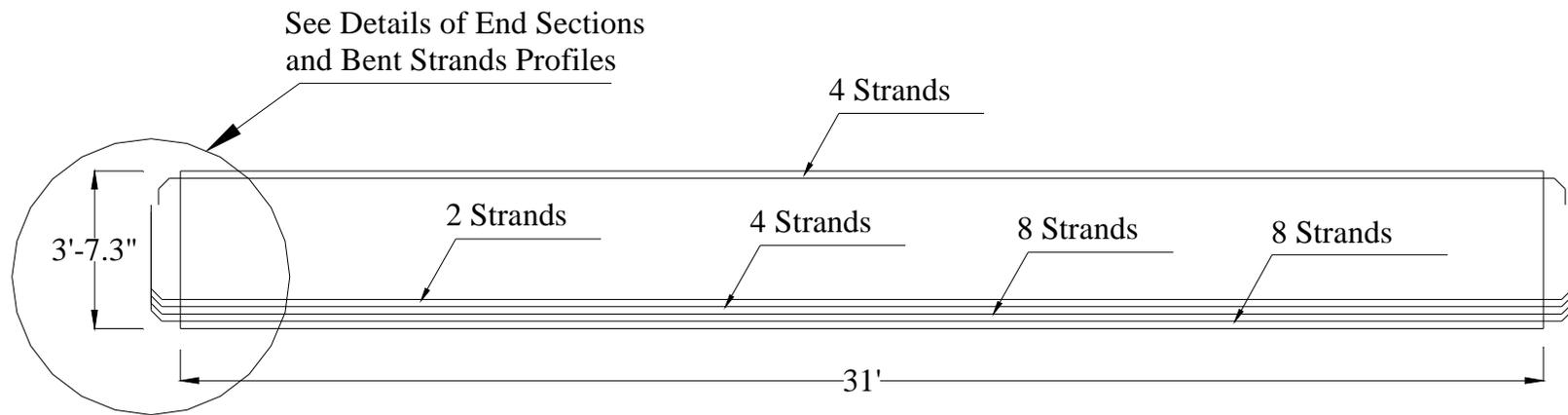


Figure 4.1 a) Typical Longitudinal Reinforcement

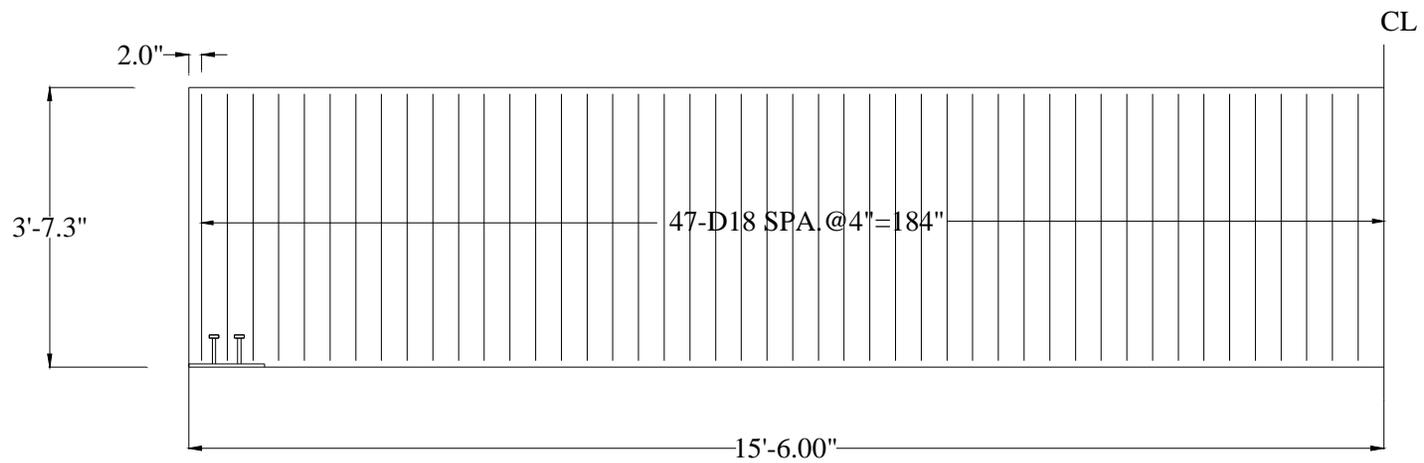


Figure 4.1 b) Typical Shear Reinforcement

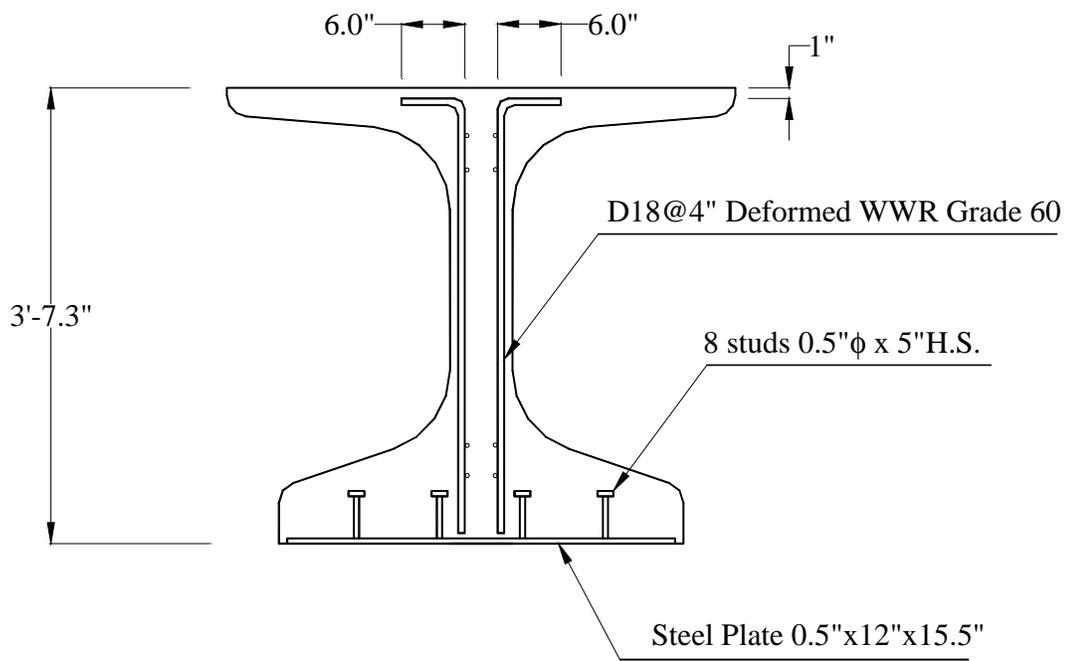


Figure 4.2 Shear Reinforcement at End Section



Figure 4.3 Beam End without End Block of Specimen B4E2

The experimental program was to study two variables used to detail the anchorage blocks at the beam ends. These variables are a number of bent non-tensioned strands in the end blocks and the total strand length embedded into the end blocks. Table 4.1 summarizes the properties of the tested specimens. Eight configurations of the end blocks were designed by varying the embedment length and numbers of bent strands as shown in Table 4.1. Figures 4.4 a) and b) show an end view of the bent strands in specimen B1E1. Figures 4.5 a) and b) show an end view of the bent strands in specimen B3E2. Figures 4.6 and 4.7 show the reinforcement details of the anchorage block used in this study. #5 U-bars as confinement reinforcement are provided in all anchorage end blocks.

Table 4.1 Properties of Specimens

| Specimen | Concrete Strength of Girders at Time of Testing (psi) | | Concrete Strength of Blocks at Time of Testing (psi) | | Embedment Length (in.) | | Number of Bent Strands |
|----------|---|--------|--|--------|------------------------|----------------|------------------------|
| | Specified | Actual | Specified | Actual | L _h | L _v | |
| B1E1 | 8,000 | 9379.0 | 4,000 | 4737 | 6 | 30 | 22 |
| B1E2 | 8,000 | 9379.0 | 4,000 | 5004 | 6 | 10 | 22 |
| B2E1 | 8,000 | 9671.9 | 4,000 | 5665 | 6 | 30 | 16 |
| B2E2 | 8,000 | 9671.9 | 4,000 | 5024 | 6 | 10 | 16 |
| B3E1 | 8,000 | 9604.3 | 4,000 | 4515 | 6 | 30 | 10 |
| B3E2 | 8,000 | 9604.3 | 4,000 | 4546 | 6 | 10 | 10 |
| B4E1 | 8,000 | 9964.4 | 4,000 | 6389 | 6 | 30 | 6 |
| B4E2 | 8,000 | 9964.4 | 4,000 | NA | 0 | 0 | 0 |

Note: 1 ksi = 6.9 MPa

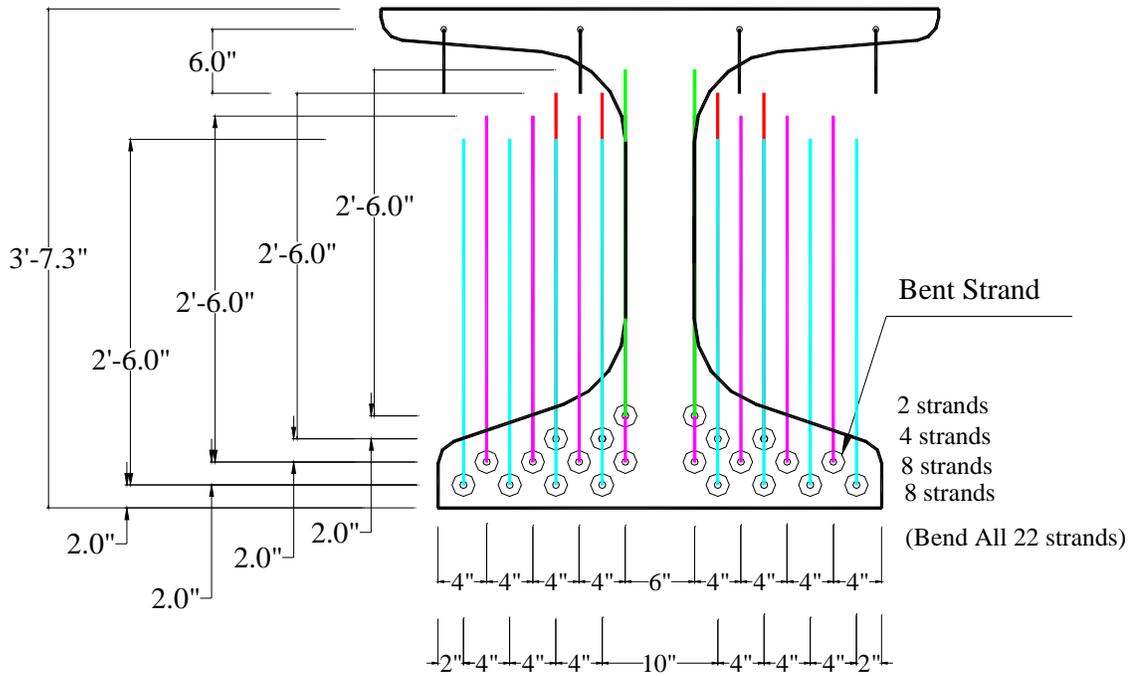


Figure 4.4 a) End Section of NU1100 I-girder (B1E1)



Figure 4.4 b) Bent Strands for Specimen B1E1

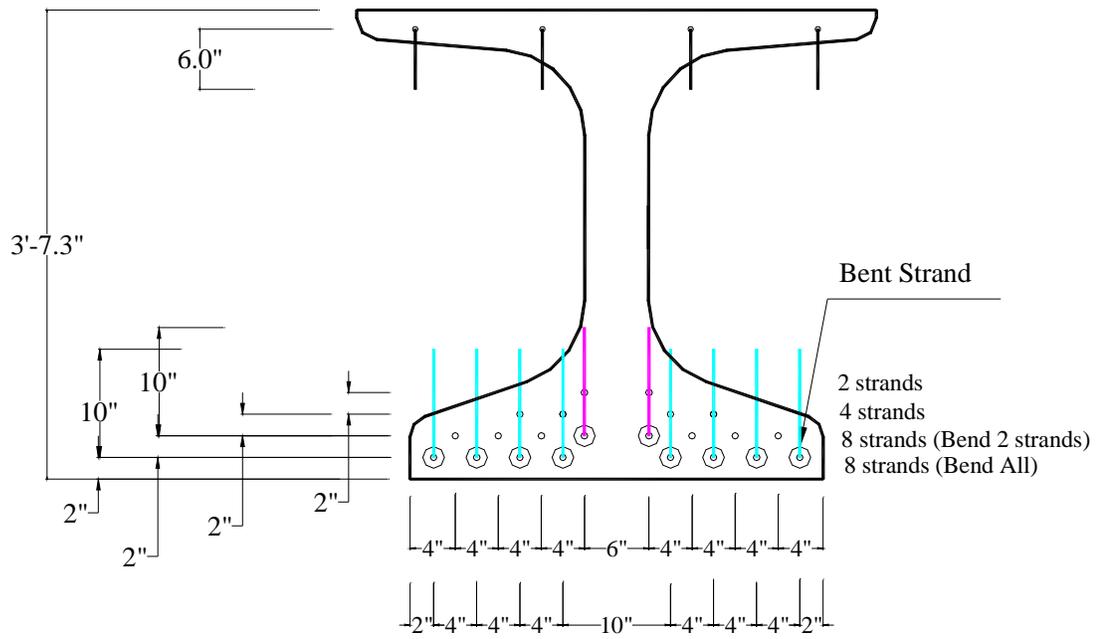


Figure 4.5 a) End Section of NU1100 I-girder (B3E2)



Figure 4.5 b) Bent Strands for Specimen B3E2

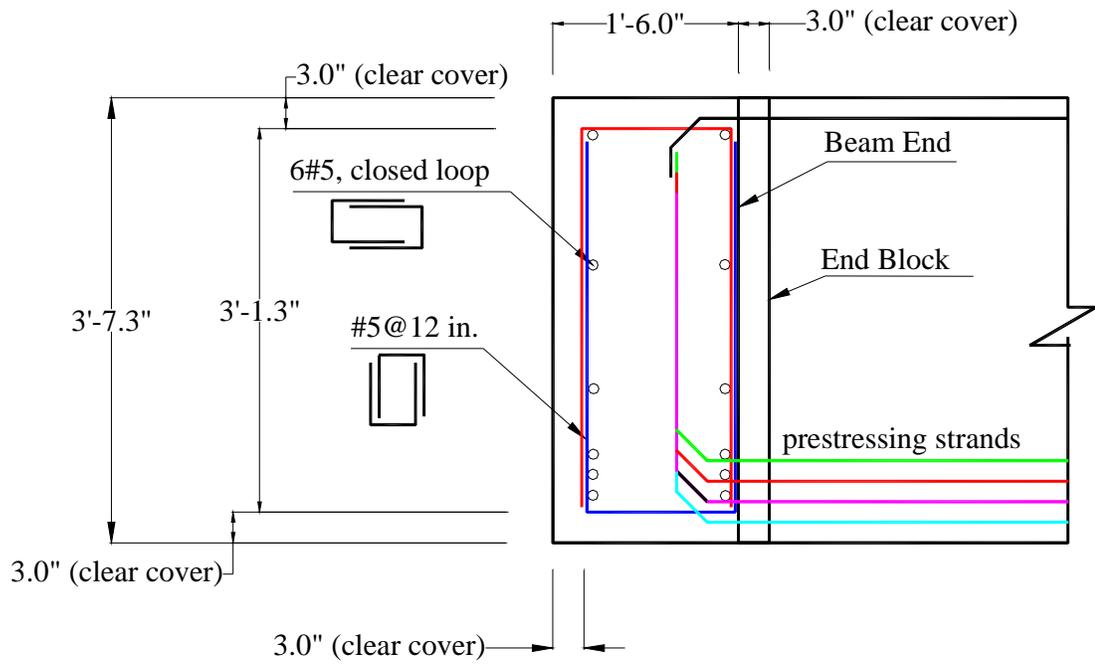


Figure 4.6 Elevation of Typical Reinforcement in the End Block



Figure 4.7 Elevation of Reinforcement in the End Block (B3E2)

4.2.2 Testing Procedure

The load test was performed twice on each beam near both ends but with different span lengths. One end of the beam was tested with a span length of 30 ft. Due to damage from the first end testing; the second end of each beam was tested with a shorter span length. A total of eight tests were performed. Table 4.2 shows information on the test set-up and prediction of failure loads using the AASHTO LRFD method. Figure 4.8 shows the test set-up for one girder. During testing, deflection was measured at the one-quarter point and the midspan of the beam using a position transducer.

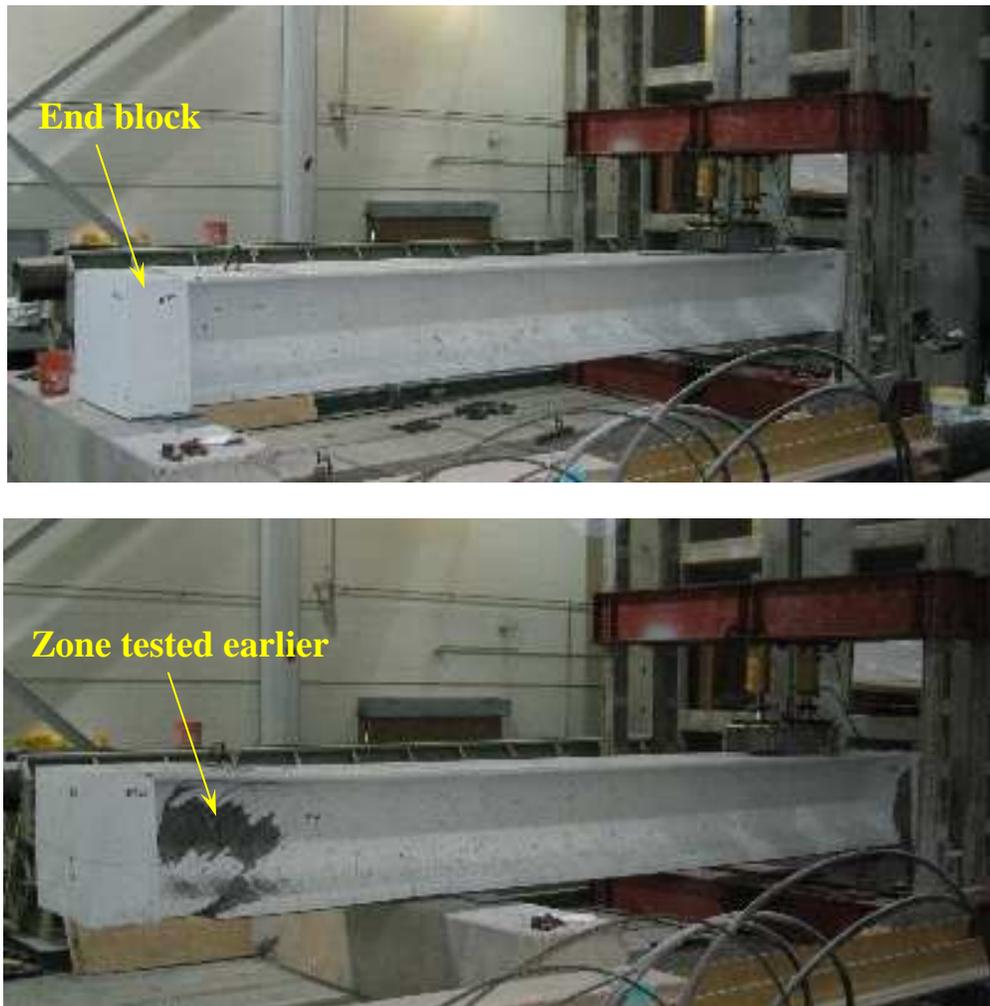


Figure 4.8 Test Set-Up

Table 4.2 Test Specimens

| Specimen | r (%) | f_y (ksi) | b_v (in.) | d (in.) | d_v (in.) | span (in.) | a (in.) | a/d | % of bent strands | $0.25f'_c b_v d_v$ (kips) |
|----------|----------|----------------|----------------|------------|----------------|---------------|------------|------|----------------------|------------------------------|
| B1E1 | 1.695 | 72 | 5.9 | 39.3 | 38.2 | 360 | 60 | 1.53 | 100.0 | 528.5 |
| B1E2 | 1.695 | 72 | 5.9 | 39.3 | 38.2 | 240 | 60 | 1.53 | 100.0 | 528.5 |
| B2E1 | 1.695 | 72 | 5.9 | 39.3 | 38.2 | 360 | 96 | 2.44 | 72.7 | 545.0 |
| B2E2 | 1.695 | 72 | 5.9 | 39.3 | 38.2 | 252 | 96 | 2.44 | 72.7 | 545.0 |
| B3E1 | 1.695 | 72 | 5.9 | 39.3 | 38.2 | 240 | 60 | 1.53 | 45.5 | 541.2 |
| B3E2 | 1.695 | 72 | 5.9 | 39.3 | 38.2 | 360 | 60 | 1.53 | 45.5 | 541.2 |
| B4E1 | 1.695 | 72 | 5.9 | 39.3 | 38.2 | 252 | 96 | 2.44 | 27.3 | 561.4 |
| B4E2 | 1.695 | 72 | 5.9 | 39.3 | 38.2 | 360 | 60 | 1.53 | 0 | 561.4 |

Note: 1 in. = 25.4 mm; 1 ksi = 6.9 MPa; 1 kip = 4.45 kN. r = vertical shear reinforcement percentage = $A_v/b_v s_v\%$, and f_y = actual stirrup strength.

4.3 Test Results and Discussion

4.3.1 Ultimate Loads and Failure Mode

The ultimate shear strengths (V_{u-test}) and their modes of failure are summarized in Table 4.3. Based on this experiment, the beams can be separated into three groups of failure modes. Specimens B1E1, B1E2, B3E1, and B3E2 behaved in a similar manner. The beams started cracking in the web between the support and the applied load. The cracking angle relative to the longitudinal axis was approximately 45 degrees. While the applied load increased, the cracks extended from both crack tips toward the support and the applied load. During the development of the cracks, flexure cracks began at the bottom flange and connected to the diagonal cracks. Near the failure loads, the cracking angle became flatter, especially near the support. Spalling of the concrete surface of the web was first visible in the middle region of the web-shear crack. Finally, the beams

failed in diagonal compression, crushing the concrete in the beam webs as shown in Figure 4.9.

Table 4.3 Summary of Test Results

| Specimen | V_{u-test} (kips) | $\frac{V_{u-test}}{f_c' b_v d_v}$ | $V_{Predicted, LRFD}$ (kips) | $\frac{V_{u-test}}{V_{Predicted, LRFD}}$ | $\frac{V_{u-Test}}{V_{B4E2}}$ | $\frac{V_{Predicted LRFD}}{V_{B4E2}}$ | Mode of failure |
|----------|------------------------|-----------------------------------|---------------------------------|--|-------------------------------|---------------------------------------|-----------------|
| B1E1 | 530.2 | 0.251 | 302.5 | 1.75 | 1.37 | 0.78 | web crushing |
| B1E2 | 542.6 | 0.257 | 302.5 | 1.79 | 1.40 | 0.78 | web crushing |
| B2E1 | 472.8 | 0.217 | 311.7 | 1.52 | 1.22 | 0.80 | pure shear |
| B2E2 | 477.0 | 0.219 | 311.7 | 1.53 | 1.23 | 0.80 | pure shear |
| B3E1 | 494.2 | 0.228 | 309.4 | 1.59 | 1.27 | 0.80 | web crushing |
| B3E2 | 483.0 | 0.223 | 309.4 | 1.56 | 1.25 | 0.80 | web crushing |
| B4E1 | 434.9 | 0.194 | 321.0 | 1.36 | 1.12 | 0.83 | pure shear |
| B4E2 | 387.7 | 0.173 | 321.0 | 1.21 | 1.00 | 0.83 | shear/bond |

Note: 1 in. = 25.4 mm; 1 ksi = 6.9 MPa; 1 kip = 4.45 kN.

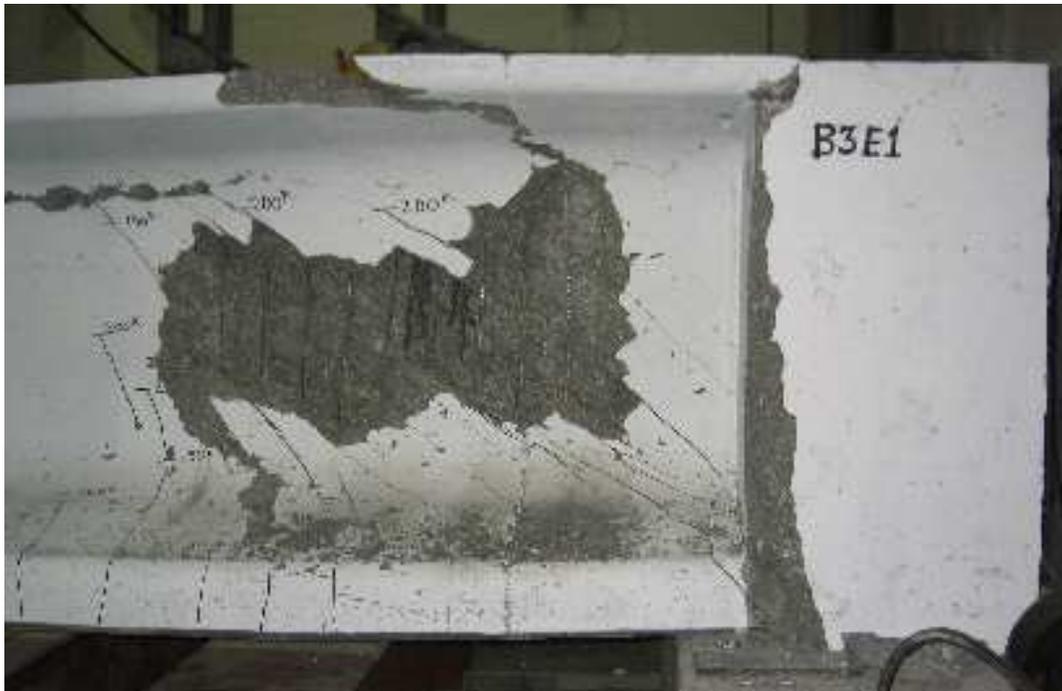


Figure 4.9 Web Crushing Failure of Specimen B3E1

A small crack was found at the connection between the girder end and the block around the perimeter of the NU section. At very high load levels, the covering concrete at the corner of the block spalled out in some girders as shown in Figure 4.9. This was caused by the high bearing stress at the corner of the end block when the beams were bent with the large deflection. Two patterns of cracks at the end block back were observed. For the end blocks with the total embedment length of 36 in., a few vertical cracks were found at the end block as shown in Figure 4.10. The first crack at the mid-block started from the bottom face when the applied load reached about 420 -500 kips, corresponding to a shear force of 350-375 kips. When the applied load was increased, the next crack appeared as numbered in Figure 4.10. At beam failure stage, the end blocks were still in good shape. The cracks were hairline cracks.



Figure 4.10 Crack of End Block at Failure of Specimen B1E2

For the end blocks with a total embedment length of 16 in., the vertical cracks occurred at the mid-block, and started from the bottom face in manner similar to that of the block with a total embedment length of 36 in. However, at an applied load higher than 500 kips, a horizontal crack appeared. The horizontal crack continued from the vertical crack and horizontally moved toward both sides of the end block. At failure, the horizontal cracks across the blocks were observed as shown in Figure 4.11. Similar behavior occurred in all blocks with an embedment length of 16 in.

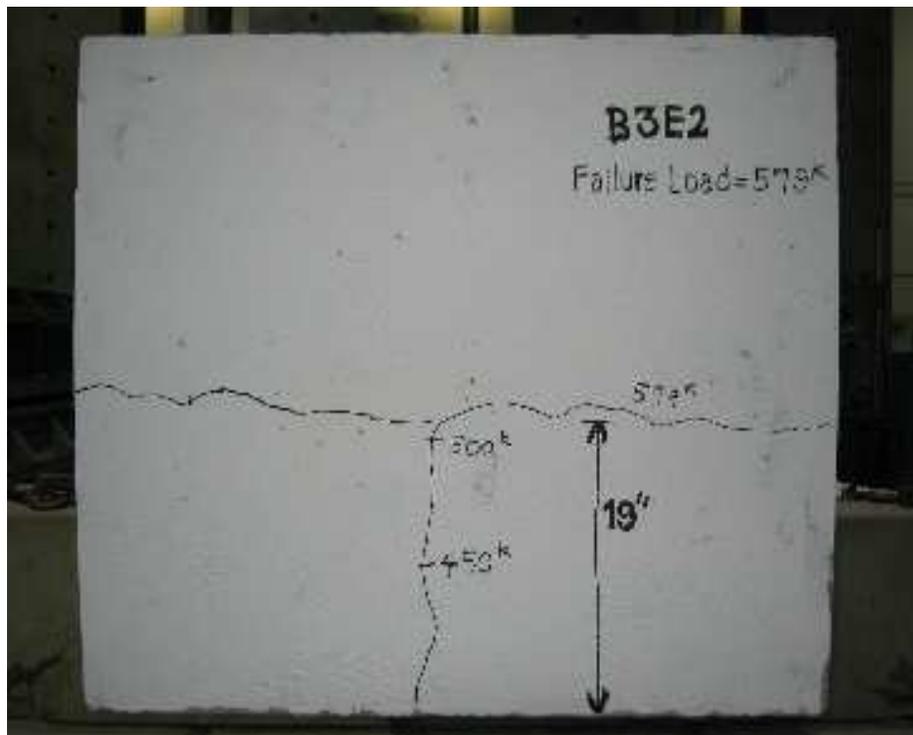


Figure 4.11 Crack of End Block at Failure of Specimen B3E2

Specimens B2E1, B2E2 and B4E1, which failed in pure shear failure mode, started cracking from the bottom flanges. When the loads increased, the vertical flexure crack changed direction to form diagonal cracks and moved toward the applied load. At failure, the beams were sheared through, starting from beneath of the applied load and

moving toward the bottom flange at a distance of 40 in. away from the applied load. Six bars of shear reinforcement with 90-degree hooks were pulled out of the top flanges. This behavior is exactly the same for both specimens. Figure 4.12 represents shear failure of Specimens B2E1, B2E2, and B4E1.



Figure 4.12 Pure Shear Failure of Specimen B2E1

Specimen B4E2 is the only specimen that did not have an end block and failed in shear-bond failure mode. At an early stage, the cracks appeared in a web-shear cracking form. As the load increased, however, the web shear crack extended into the beam end and tended to cut the bottom flange off. At failure, the concrete around the bottom flange at the beam end broke out. It was observed that significant strand slippage occurred as shown in Figure 4.13.



Figure 4.13 Shear/Bond Failure of Specimen B4E2

4.3.2 Discussion of Test Results

In this research project, all girders have the same properties as shown in Table 4.1, except for concrete strength which usually varies. The specimens were designed to study the influence of the total embedment length and the number of bent strands embedded in the blocks on shear capacity. All end blocks produced in this experiment have the same material properties, detailing, and configurations as shown in Table 4.1 and Figures 4.6 and 4.7. The block was reinforced with #5 bars with a detail that will be proposed for practical use. In a real bridge, an end block with limited width does not exist, but a continuous end diaphragm does exist. Thus, the U-shape reinforcement was provided in the end blocks of this test to represent the confinement of the continuous diaphragm.

As shown in Tables 4.2 and 4.3 and Figure 4.14, the test shear capacities are higher than those predicted by the AASHTO LRFD method for all specimens. This test shows that anchoring the longitudinal reinforcement in the end block appears to result in higher shear strength. The testing specimens with the longitudinal reinforcement anchored in the end diaphragm experienced web-crushing failure instead of shear bond failure, as previously discussed. It is also observed from the full-scale testing that the bulky bottom flange of the NU I-beams is effective in developing the tension tie function as long as the strand is anchored into the end diaphragm.

Based on the test results, the beam without the end block gives the minimum shear capacity of 387.8 kips, which is higher than the predicted value by about 20 percent. The beam with the end block and 22 bent strands gives the maximum shear capacity of 542.6 kips, which is higher than the predicted value by about 80 percent. The embedment

lengths of 16 in. and 36 in. anchored in the end blocks do not show significantly different results on shear capacity as shown in Figure 4.16. The number of the embedded strands is a significant factor that increases shear capacity.

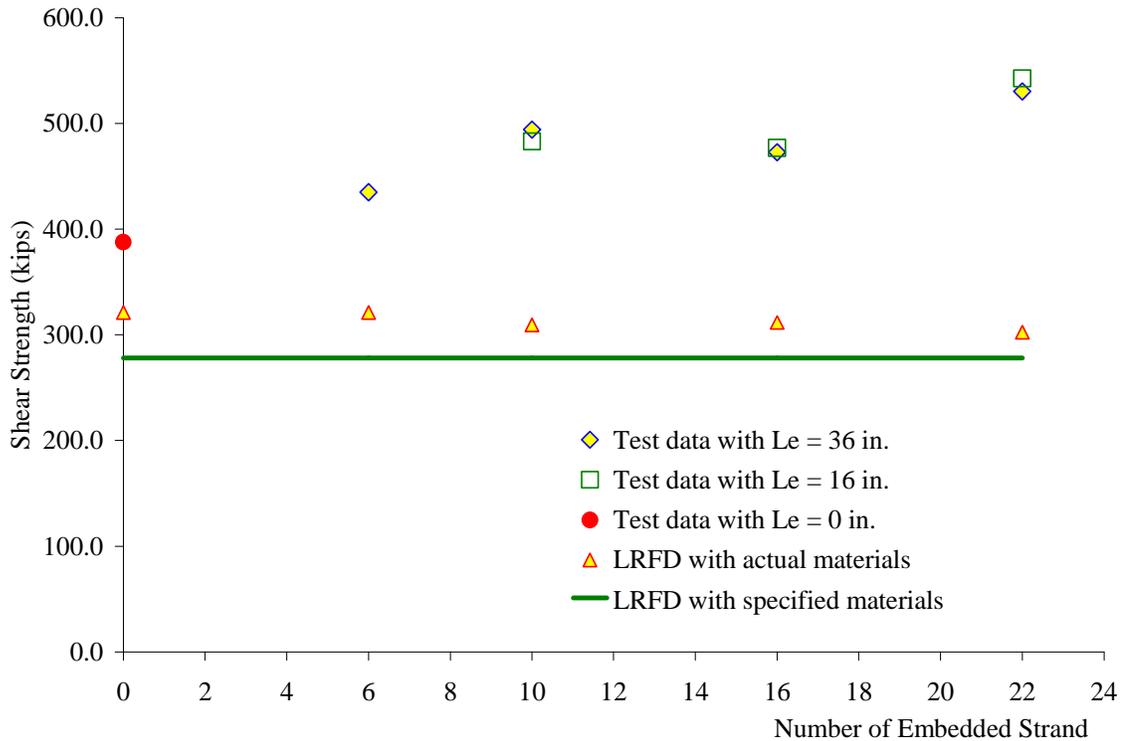


Figure 4.14 Shear strength versus number of non-tensioned embedded strands

As shown in Table 4.3, Specimens B1E1 and B1E2 with 22 bent strands embedded into the end block can reach the maximum shear limit of $0.25f_c b_v d_v$ introduced in the AASHTO LRFD Specifications⁽¹³⁾. The beams with 10 and 16 bent strands do not exhibit a significant difference of shear capacity as shown in Figure 4.14. This situation may arise due to the following reasons: 1) the shear span to depth ratio, a/d , of the beam with 16 embedded strands is larger than that of the beam with 10 embedded strands, and 2) the beams with 16 embedded strands lost their capacities due to insufficient anchorage

of the shear reinforcement in the top flanges. As previously mentioned, six bars of shear reinforcement were pulled out of the top flanges of Specimen B2E1 and B2E2.

To study the efficiency of the anchorage end block, all test and predicted shear strengths were normalized with the shear capacity of Specimen B4E2. The results are shown in Table 4.3 and Figure 4.15. Obviously the AASHTO LRFD method is rather conservative in predicting shear capacity, especially if the anchorages are provided at the beam ends. The beam with 22 embedded strands can provide adequate shear capacity through more than 30 percent of the beam without the block, and as high as 80 percent of the shear strength predicted by the LRFD method.

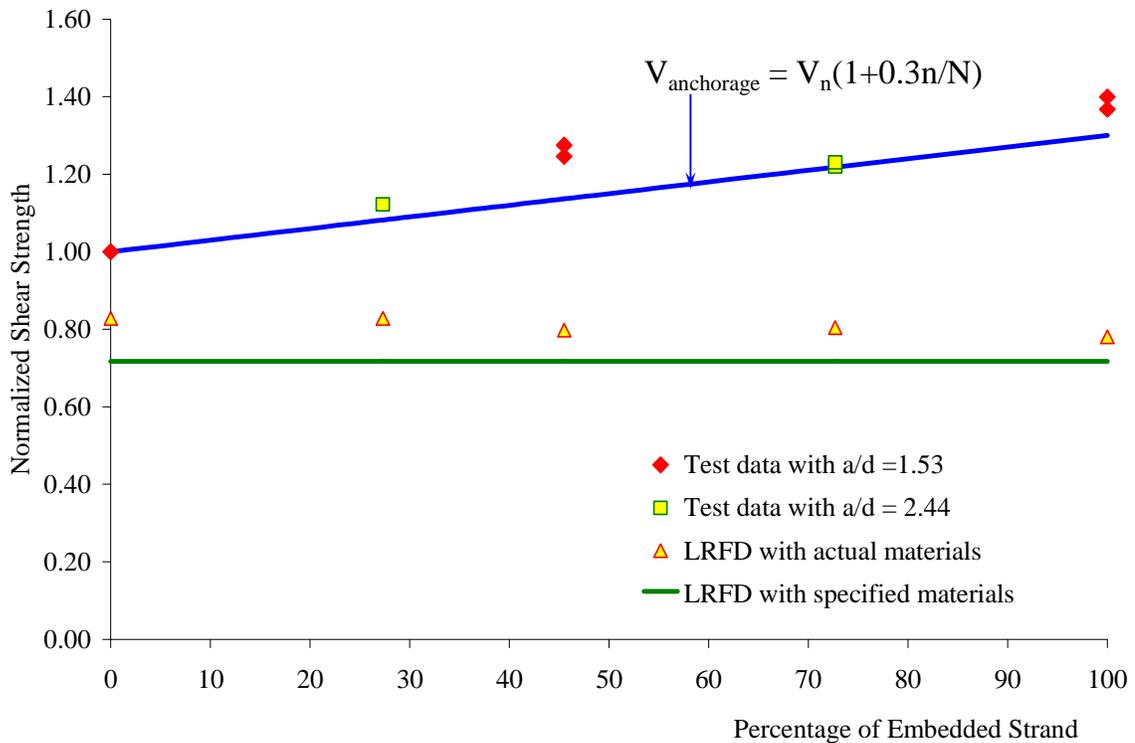


Figure 4.15 Normalized shear strength versus anchorage of non-tensioned strands

4.4 Analyses of the Strand Stress in the End Blocks

The distribution of the tension force in the strands can be calculated using a truss model and is shown in Figure 4.16 and Table 4.4. Because the test results show that the embedment length does not increase shear capacity, only the beams with different numbers of bent strands are presented. Figure 4.16 shows that the small shear span of 5 ft affects the increase in tension force more near the support than the large shear span of 8 ft. In other words, the anchorage will be more effective if the load is applied near the support. This is because when the shear span is longer than the development length, the yield strength of the strands can be reached without requiring additional anchorage.

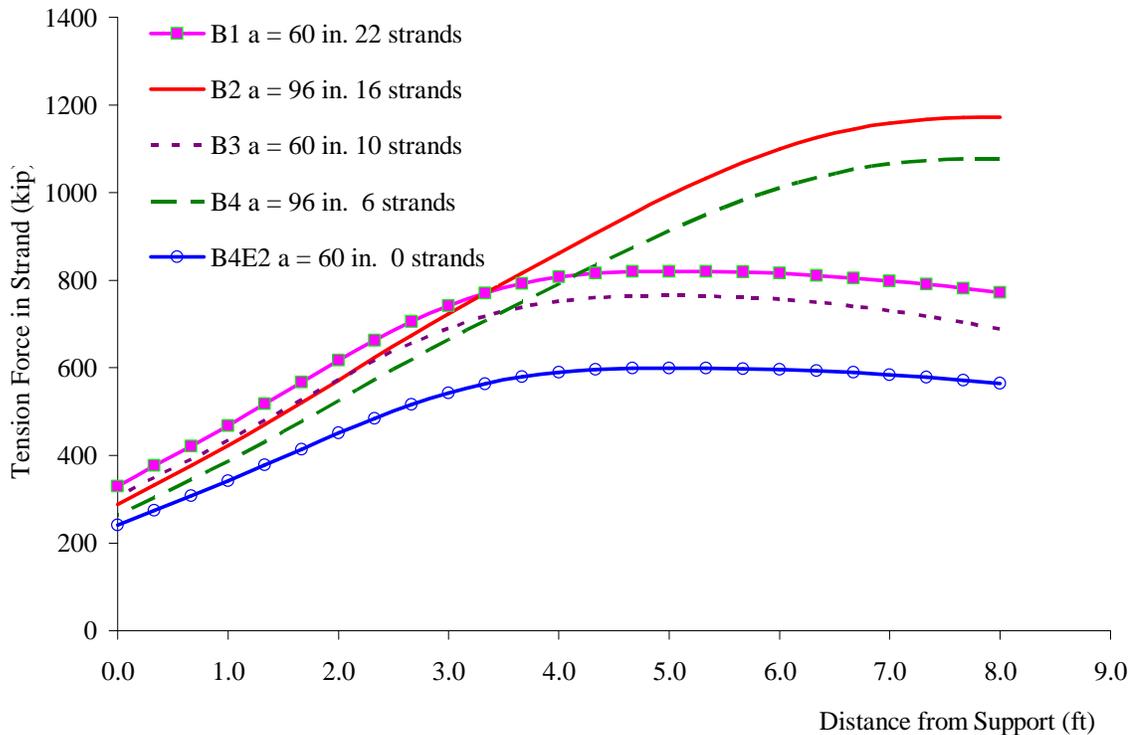


Figure 4.16 Tension force in longitudinal reinforcement along shear span

Table 4.4 Parameters to calculate strand stress

| Specimen | Load (kips) | Tension Force at Support Face (kips) | Support Width (in) | Measured Crack Angle, θ_{AV} (degree) | Strand Stress at Support Face (ksi) |
|----------|----------------|--|--------------------------|--|---|
| B1E1 | 636.2 | 330.3 | 6 | 42.9 | 69.2 |
| B1E2 | 723.5 | 338.0 | 6 | 42.1 | 70.8 |
| B2E1 | 644.7 | 288.4 | 6 | 39.3 | 64.1 |
| B2E2 | 770.6 | 291.0 | 6 | 39.8 | 64.9 |
| B3E1 | 658.9 | 307.8 | 6 | 41.8 | 81.2 |
| B3E2 | 579.6 | 300.9 | 6 | 41.9 | 80.0 |
| B4E1 | 702.6 | 265.4 | 6 | 39.4 | 136.4 |
| B4E2 | 465.2 | 241.5 | 12 | 40.9 | 50.6 |

Note: Support width is considered only a part of the beam on the 12-in wide bearing pad.

B4E2 is used as a reference beam to calculate the strand stresses at the section of the support face. In Chapter 3 it is shown that the proposed equation of the strand stresses results from the direct pullout tests of individual strands. In this chapter, the strand stress of Specimen B4E2 was calculated from the tension force due to the load applied from the beam top. As shown in Table 4.4 and Figure 4.17, the maximum strand stress at the support face section is 50.6 ksi. Assuming that only the length of the bearing area is effective in developing the tension tie force, the embedment length would be the bearing width of the support, 12 in. For this beam the embedment length is considered to be the straight embedded strands. Compared to the results in Chapter 3, it is found that for a total embedment length of 10 in., the minimum strand stress is 81.0 ksi. However, the strand stresses in Chapter 3 are based on bent strands. In a pretensioned concrete beam, the strand stress at the considered section can be calculated from, for example, $m(f_{pe})/36$ for a 0.6-in. diameter strand, where m is the distance from the beam end to the section

considered. If the bearing width is 12 in. and $f_{pe} = 145$ ksi, the strand stress at the support face is $12(145)/36 = 48.3$ ksi. This value is close to the stress of 50.6 ksi calculated based on the test data. Thus, it is proposed that the strand stress of non-tensioned and tensioned straight strands can be calculated as:

$$f_s = 50.6m/12 = 4.22m \simeq 4.0m \quad (4.1)$$

where f_s = strand stress of prestressing strand at the section considered (ksi)

m = a distance from the beam end to the section considered (in.)

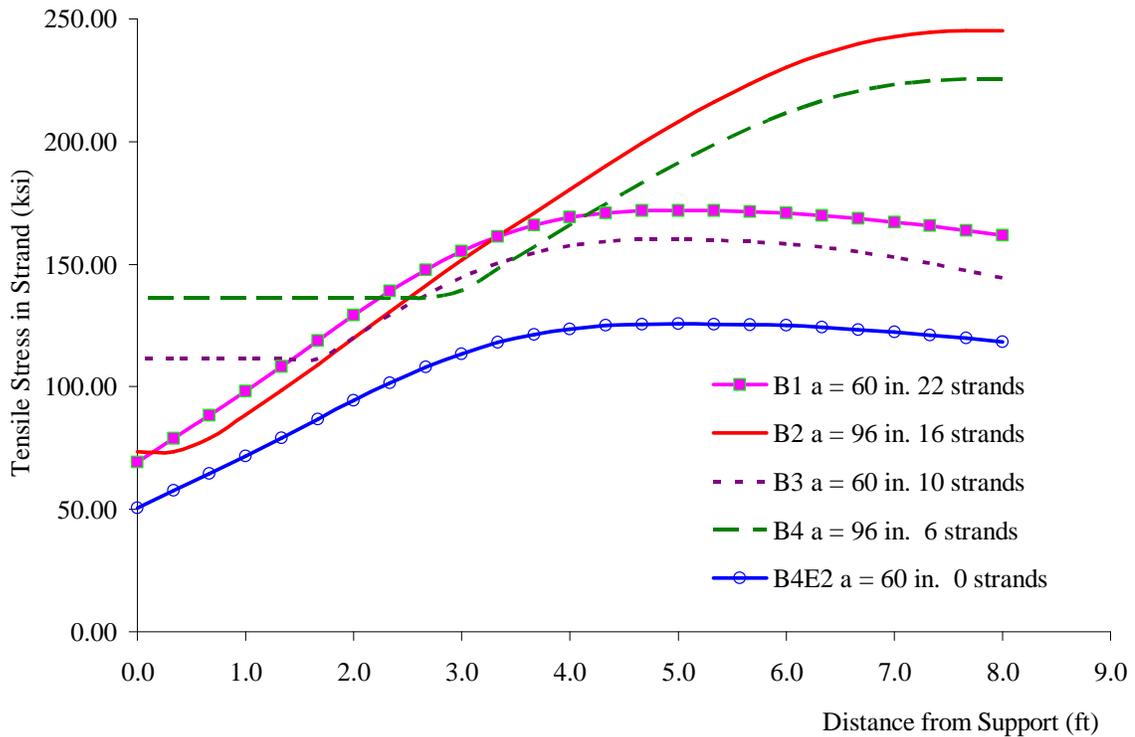


Figure 4.17 Tensile stress in longitudinal reinforcement along shear span

Using Equation 4.1 for other specimens with a support width of 6 in., the strand stress at the support section for an embedded straight strand is 25.3 ksi. Therefore, the strand stress of the bent strands at the support section in each beam can be calculated as

shown in Table 4.4 and Figure 4.17. The horizontal lines of strand stresses are considered as a lower bound value. However, in calculating the number of embedded bent strands, only the stress at the support face is needed.

Specimen B4E1 gave the most critical strand stress of 136.4 ksi. This specimen had 6 bent strands and 16 straight strands, and the total embedded length was 36 in. From Chapter 3, the strand stresses of the total embedded length of 36 in. ranged from 104 – 150 ksi. However, the test in Chapter 3 showed that at this level of strand stress, the blocks broke completely. Because the end blocks in this test did not break out, and only a few hair-line cracks were observed as explained above, this test confirmed that the stress equation in Chapter 3 can be used as a lower bound of the end block with confinement of reinforcement. Thus, Equation (3.1) may be used to design the number of bent strands to enhance shear capacity.

4.5 Conclusions of Beam End Anchorage Experiment

Based on the presented experiments in this chapter, the following conclusions can be drawn:

1. The number of the embedded strands is a significant factor that increases shear capacity if the strands are anchored in the end blocks with confinement reinforcement.
2. The test beams reach the maximum shear capacity of $0.25f'c b_w d_v$ with all bent strands embedded into the end block.
3. The strand stresses in all beams do not exceed the strand stress design limit of $0.8f_{pu}$ in Chapter 3.

4. The embedment lengths of 16 in. and 36 in. anchored in the end blocks with confinement reinforcement do not show significantly different results on the shear capacity.
5. If proper reinforcement detail of the concrete diaphragm at the concrete bridge I-beam is used, one can reach the maximum shear of $0.25f_c b_v d_v$ without adding more mild steel in bottom flange to meet the requirement of AASHTO LRFD Specifications Section 5.8.3.5.

CHAPTER 5

APPLICATION OF ANCHORAGE AND DESIGN EXAMPLES

5.1 Introduction

This chapter covers the application of the proposed formula and design recommendations of anchorage length for non-prestressed 90-degree bent strands in the end diaphragm. Based on the test results in Chapter 3 and 4, the authors propose a unified embedment length equation for 0.5 and 0.6 in (13 and 15 mm) diameter strands. The equation was fitted at the fifth percentile value from lower limit. In addition, two numerical design examples are presented.

5.2 Development Length and Application to Shear Design

The maximum possible shear capacity can be achieved if the bond strength between the longitudinal flexural reinforcement and the surrounding concrete does not control the failure. According to a shear test of pretensioned NU I-girders ^(1,2), adequate strand anchorage is necessary to attain the maximum shear capacity and avoid bond failure.

Bridge girders are subject to a number of loading patterns corresponding to several failure modes. For example, the load can act near the beam end and, if the strands are cut off at the beam end faces, shear/bond failure might occur because of the lack of development length. In addition, strand slip can occur in the transfer zone. This prevents the beam from attaining its nominal moment capacity ⁽¹¹⁾. Recently, researchers ^(10, 11) have tried to propose appropriate equations for the development length of the tensioned strand.

A proposed approach for solving this problem is presented here. The pullout test results, discussed in Chapter 3, indicate that a 30-in. (760 mm) total embedment length of a 90-degree hooked 0.5 in. (13 mm) diameter strand is required. The corresponding length for a 0.6 in. (15 mm) diameter strand is 36 in. (914 mm). These lengths can develop strand stresses of at least $0.9f_{pu}$. However, the maximum capacity was obtained when the strands were pulled one at a time. In actual structures, all strands are pulled at the same time. This action might cause a reduction in the maximum capacity of the section. The upper limit for Eq (3.1) is thus reduced to $0.8 f_{pu}$. Eq. (3.1) may be used to estimate the strand stress if the minimum lengths specified above are not available in shallow members.

Eq. (3.1) can be applied to satisfy Section 5.8.3.5 of the requirement of the AASHTO LRFD Bridge Design Specifications ⁽¹³⁾. According to these specification, the tension force at each section shall not be greater than the tensile capacity of the reinforcement on the flexural tension side of the member. However, due to lack of full development near the supports (if the strands are cut at the beam ends), mild steel reinforcement may have to be provided to meet this requirement. In practice, this arrangement is difficult because the bottom flange typically has many prestressing strands, and any additional steel would create congestion and possible stress concentration. The proposed solution is to anchor the strands in the end diaphragm. When the strands are embedded in the diaphragm, the developed stresses in the strands are

higher at a given section; hence, they can be designed to meet the tensile force requirement.

There is no additional cost if the existing strands at the end of girders are bent up rather than cut off flush with the member end. The number of strands that need to be bent to provide the required anchorage can be determined as follows.

Figures 5.1a and 5.1b show a simply supported beam subjected to a concentrated load at midspan and a variation of tension force in the longitudinal reinforcement. This example is used to determine the number of strands that would be bent into the end diaphragm.

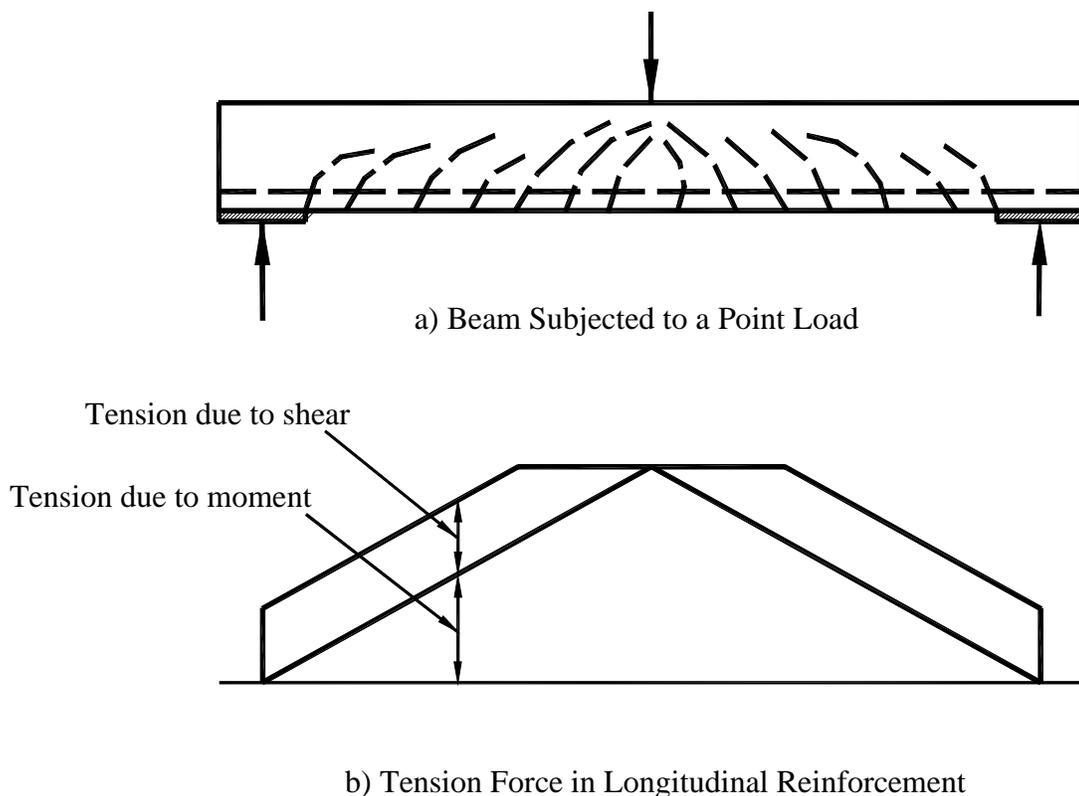


Figure 5.1 Force Variations in Flexural Reinforcement over Beam Span

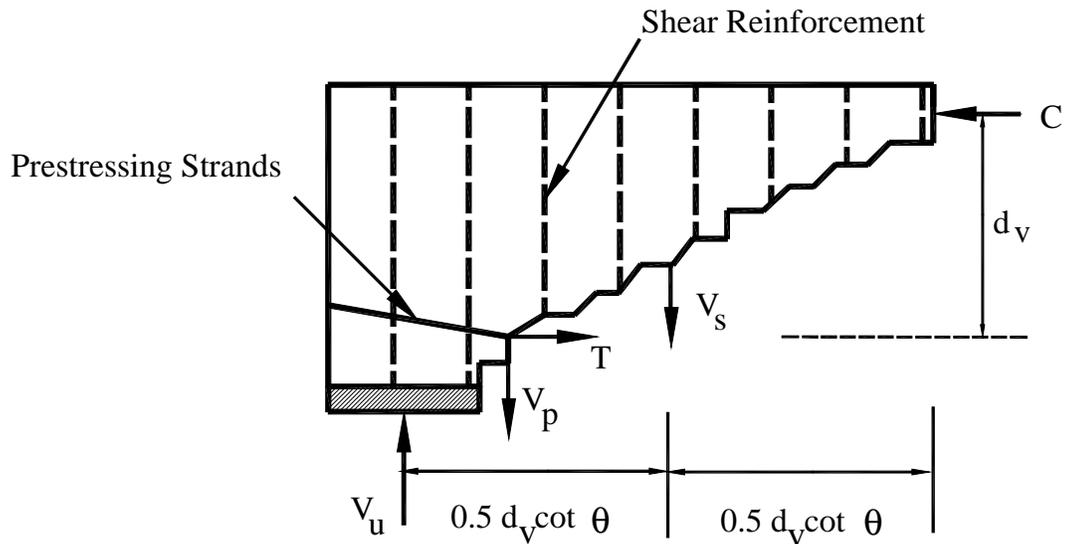


Figure 5.2 Shear and Moment Resistance Forces near Support

From the equilibrium conditions of the free body diagram, shown in Figure 5.2, and assuming that the moment at the support is zero and neither axial force nor torsion is present in the beam, the following equation is obtained:

$$T \geq \left(\frac{V_u}{\phi} - 0.5V_s - V_p \right) \cot\theta \quad (5.1)$$

where

T = tension force in longitudinal reinforcement, kip

ϕ = resistant factor for shear

V_u = factored shear force at critical section, kip

V_s = shear resistance provided by shear reinforcement at given section, kips

V_p = component of the effective prestressing force in the direction of the applied shear, kip

θ = angle of inclination of diagonal compressive stress

Eq. (5.1) is the reduced form of Eq. (5.8.3.5-1) in the 2000 Interim AASHTO LRFD Bridge Design Specifications ⁽¹³⁾, when the factored moment and factored axial force are zero. To calculate the number of bent strands, the tensile force in the reinforcement is set to equal the tensile force developed by the strands as:

$$T = n A_{ps1} f_{ps} \quad (5.2)$$

where

n = the number of bent strand(s)

A_{ps1} = cross-sectional area of one strand, sq in.

f_{ps} = the developed stress in each strand, ksi

5.3 Recommendations of Reinforcement Detail in the End Diaphragm

In current pretensioned concrete bridge girder design, only the length of the bearing area is considered effective in developing the tension tie force. The extra length of bent strand embedded in the end diaphragm can improve strand development in this critical zone of the member. Moreover, appropriate reinforcement details of the concrete diaphragm at beam ends can increase strand stress capacity and reduce demand for embedment length. Tests on I-beam ends completed in this study, utilizing 0.6 in. (15 mm) strands, have demonstrated no drop-off in shear capacity if the embedment length is reduced from 36 in. (914 mm) to 16 in. (406 mm). A number of factors may contribute to this enhanced strand anchorage capacity. These factors include the confinement of the diaphragm concrete, the presence of diaphragm steel, and the presence of an anchor bar at the strand bend. However, it is suggested that for all NU I-beams, a total length of 36 in.

strand extension be embedded in the diaphragm. This conservative recommendation does not take the above-mentioned enhancements into account.

For inverted tee beams, with 0.5 in. (13 mm) diameter strands, a 30 in. total extension, based on the pullout tests is recommended. A shorter extension may be justifiable based on additional inverted tee beam end testing. For inverted tees that are too shallow to accommodate this length, a partial anchorage stress should be calculated using Equation (3.1).

The detailing of the reinforcement in the end diaphragm is proposed in Figure 5.3.

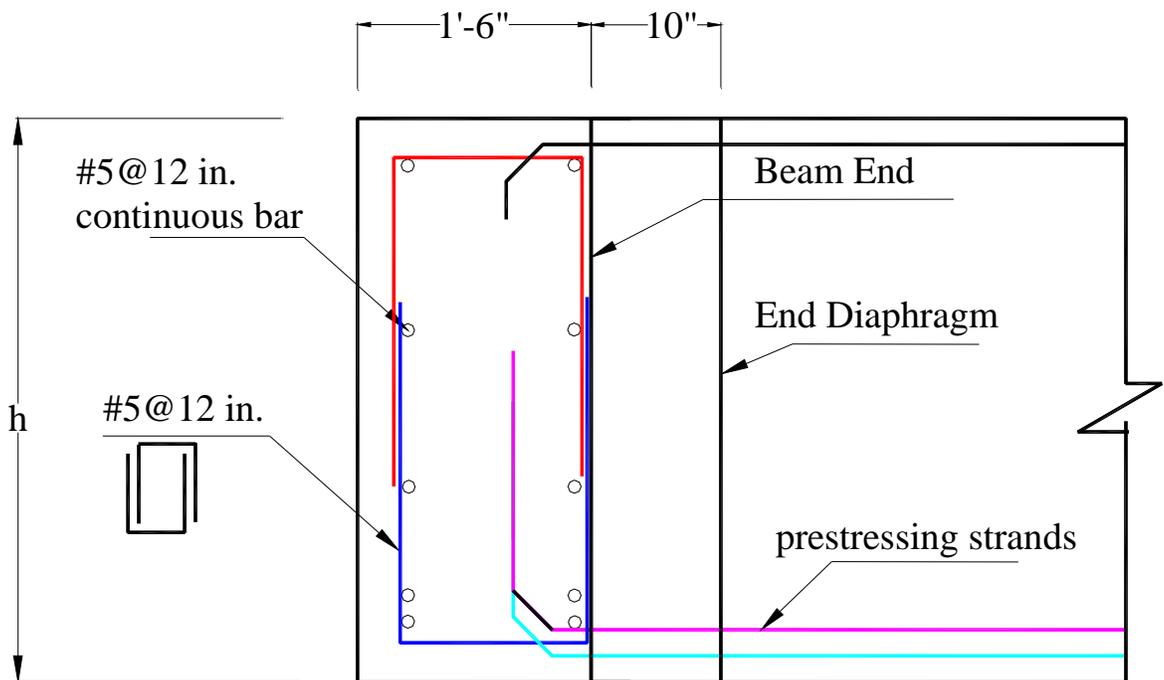


Figure 5.3 Proposed Detail Reinforcement in the diaphragm

The test results clearly indicate that the shear design procedures given in the AASHTO LRFD Specifications are rather conservative for the anchorage details used in Chapter 4. It is believed that the outstanding performance of these specimens is the result

of many factors. Most significantly, the strands were fully anchored into an end diaphragm, forming a strong tie of the flexural reinforcement. This detail resulted in pure shear or web crushing, rather than bond or flexural failure combined with shear failure, as reported in most shear testing programs. It has been verified by testing that the specimen without the end block results in the lowest shear capacity in this research project.

5.4 Design Examples

Two numerical design examples are presented below. The first example shows the procedure for obtaining the number of bent strands corresponding to the tensile forces at the support sections calculated by Eq. (5.1) or the reduced form of Eq. (5.8.3.5-1) in the 2000 Interim AASHTO LRFD Bridge Design Specifications. In addition, the tension capacity provided by the bent strand anchorage was checked according to section 5.8.3.5-1 of the AASHTO LRFD Bridge Design Specifications.

The second example demonstrates an application of the recommended embedment length to design for crack control at the bottom fiber due to member creep at the interior support of two-span continuous beams.

Example 1

This example provides the calculations for the required number of bent strands at the ends of a 120-ft (36.6 m) single-span PCI bulb-tee beam. Information in this example was obtained from section 9.4 of the PCI Bridge Design Manual ⁽¹⁴⁾.

Table 5.1 shows the values of the data needed for this example. It was either directly extracted or obtained by linear interpolation from the solution in that example.

Table 5.1: Data from Example 9.4 of the PCI Bridge Design Manual¹⁴

| Distance from the Support centerline (in.) | V_u (kips) | M_u (k-ft) | V_s (kips) | V_p (kips) | Centroid of tension reinforcement (in.) | d_v |
|--|--------------|--------------|--------------|--------------|---|-------|
| 0.00 | 350.5 | 0.0 | 282.6 | 23.4 | 4.22 | 57.95 |
| 3.00 | 349.3 | 75.1 | 282.6 | 23.4 | 4.22 | 57.95 |
| 13.29 | 345.2 | 332.9 | 282.6 | 23.4 | 4.22 | 57.95 |
| 72.00 | 321.8 | 1803.4 | 282.6 | 23.4 | 4.22 | 57.95 |

Figure 5.4 shows the proposed bent strand arrangement.

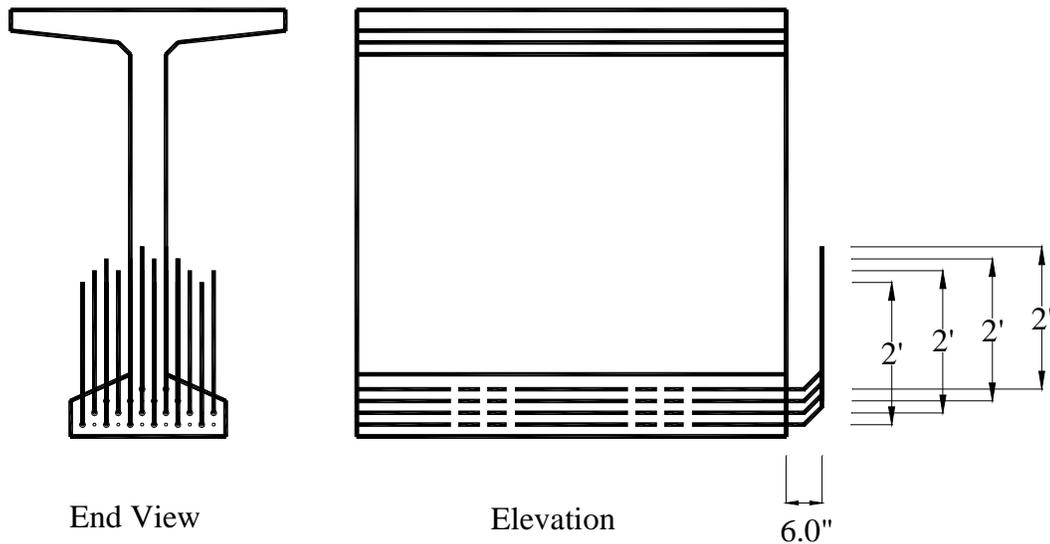


Figure 5.4 Bent Strand at Girder End

Detailed Method: Analysis At the first diagonal crack section near the support

In this method the strand embedment at the first diagonal crack, assumed to be initiated at the inside face of the support must be adequate to satisfy the anchorage requirements of the LRFD Bridge Design Specifications⁽¹³⁾ for longitudinal tension reinforcement.

According to section 5.8.3.5 of AASHTO LRFD, the tensile capacity of the reinforcement of the flexural tension side of the member has to be greater than or equal to the tensile force, T, at the considered section calculated as:

$$T = \frac{M_u}{\phi d_v} + 0.5 \frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p \right) \cot\theta \quad (\text{LRFD Eq. 5.8.3.5-1})$$

Where M_u = factored moment at section corresponding to maximum factored shear force

N_u = applied factored axial force

d_v = effective shear depth

V_u = factored shear force at given section

V_s = shear resistance provided by shear reinforcement at given section

V_p = component in the direction of the applied shear of the effective prestress force

θ = angle of inclination of diagonal compressive stress

The bearing width of the support is assumed = 6 in.. The slope of the diagonal crack was found from Example 9.4 to be 22.3°. As shown in Figure 5.5, the assumed crack plane crosses the centroid of the 36-straight strands at a distance of (6/2 + 4.22cot 22.3 = 13.29 in.) from the support. Substituting for the values of the various parameters from Table

5.1, and recognizing that $N_u = 0$ throughout and that the strength resistance factor $\phi = 1.0$ for flexure and 0.9 for shear:

$$\begin{aligned}
 T &= \frac{M_u}{\phi d_v} + 0.5 \frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p \right) \cot\theta \\
 &= \frac{332.9 \times 12}{57.95(1.0)} + 0 + \left(\frac{345.2}{0.9} - 0.5 \times 282.6 - 23.4 \right) \cot 22.3^\circ \\
 &= 602.6 \text{ kips (2680 kN)}
 \end{aligned}$$

The transfer length for transfer of prestress in bonded strands that are terminated at the end face of the member is specified by the LRFD Specifications to be 60 times the strand diameter, or 30 in. Since the embedment distance of these terminated strands is only 13.29 in., the stress at this critical point is estimated proportionately as $(16.29/30)$ $f_{pe} = (16.29/30) (149) = 80.9$ ksi.

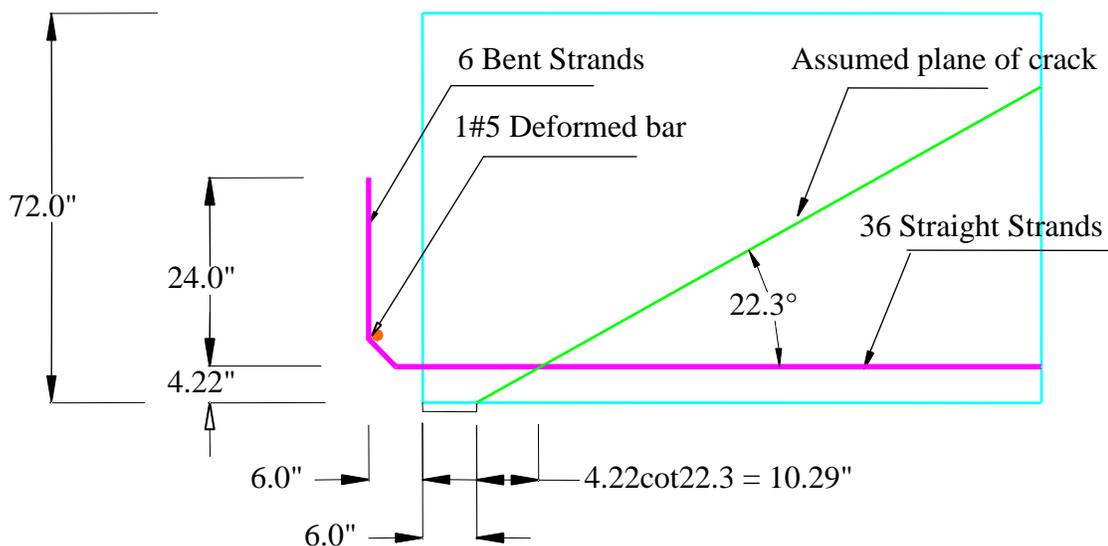


Figure 5.5 Bent Strand Details

Based on the bent strand pullout test study' if the length bent strand that is embedded into an end diaphragm, L_e , is 30 in. (a horizontal segment of 6 in. and a vertical segment, L_v , of 24 in.), the strand can develop a stress given by the equation

$$f_{ps} = (0.017x f_{pu} \times L_v / d_b), \text{ but not greater than } 0.8f_{pu},$$

Substituting for $f_{pu} = 270$ ksi, $L_v = 24$ in. and $d_b = 0.5$ in., the steel stress that can be developed in the bent strands is 216 ksi.

Let n = number of bent strands

$T \leq$ the total tension capacity of embedment of the strands beyond the critical location of 16.29 in. from member end.

Thus, $602.6 \text{ kips} \leq n (0.153) (216) + (36-n) (0.153) (80.9)$

$$n \geq 7.6 \text{ strands}$$

Therefore, it is required to bend 8 strands into the end diaphragm.

Approximate Method: At the inside support face section

Simplifying LRFD Equation 5.8.3.5-1 by neglecting the bending moment at support face and dropping the normal force which is generally taken as zero for this type of member, the equation can be written as:

$$T = \left(\frac{V_u}{\phi} - 0.5V_s - V_p \right) \cot\theta \quad (5.1)$$

Substituting into Eq. 5.1 with the values for $x = 3$ in. from the Table 5.1, $T = 544.7$ kips (2423 kN).

The straight strands are embedded only 6 inches at this section. Thus, their stress is estimated to be $(6/30) f_{pe} = (6/30) (149) = 29.8$ ksi.

The bent strands with a 24 in. vertical embedment would still have a capacity of 216 ksi. The number of required bent strands can then be calculated from the relationship:

$$\text{Thus, } 544.7 \text{ kips} \leq n (0.153) (216) + (36-n) (0.153) (29.8)$$

$$n \geq 13.36 \text{ strands}$$

Therefore, it is required to bend 14 strands into the diaphragm to satisfy the longitudinal reinforcement anchorage in the LRFD shear design specifications.

Example 2

An interior girder of a bridge with four equal 130 ft (40 m) spans was chosen from an example in Reference 6 to illustrate the application of bending strands into the diaphragms to resist the positive moment over the pier. Figure 5.6 shows the cross section of the bridge. The prestressing steel consists of 48- ½ in. (13 mm) diameter low-relaxation strands in each girder.

Assume the following design criteria:

Compressive strength of prestressed beam at release: $f'_{ci} = 4000$ psi (28 MPa)

Compressive strength of prestressed beam at 28 days: $f'_c = 5000$ psi (34 MPa)

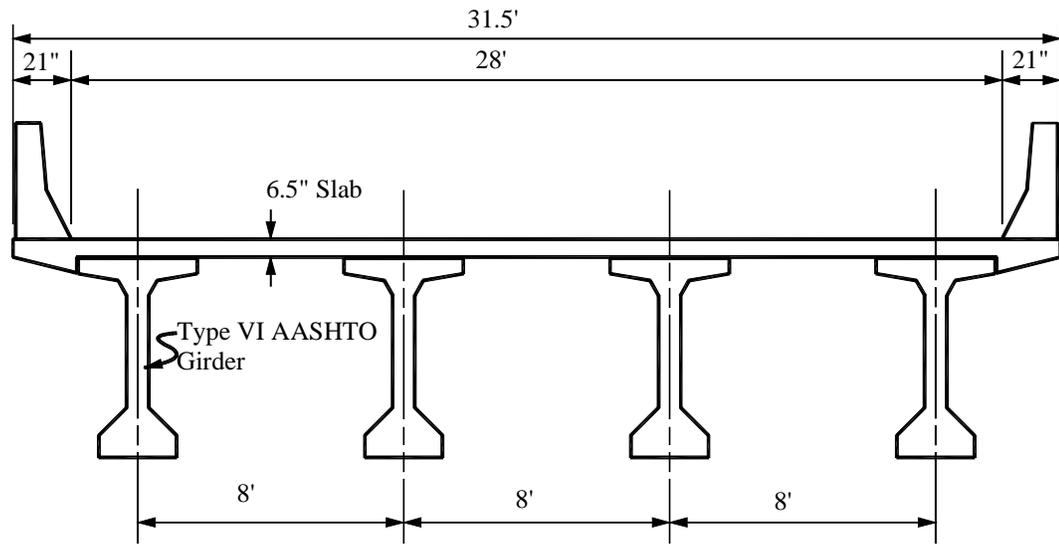
Compressive strength of deck slab and diaphragm: $f'_c = 4500$ psi (31 MPa)

Loading: AASHO HS20-44

Prestressing strand: ½ in. (12.7 mm) diameter, seven wire, low relaxation steel

-Area of one strand = 0.153 sq in. (99 mm²)

-Prestressing force (after losses) = 23.6 kips per strand (105 kN/strand)



note: 1 in. = 25.4 mm; 1 ft = 0.305 m

Figure 5.6 Cross Section of Bridge

Figure 5.7 shows the pier location of the bridge. According to the analysis given in Reference 6, positive moments develop over the piers due to girder creep and the effect of live loads in remote spans.

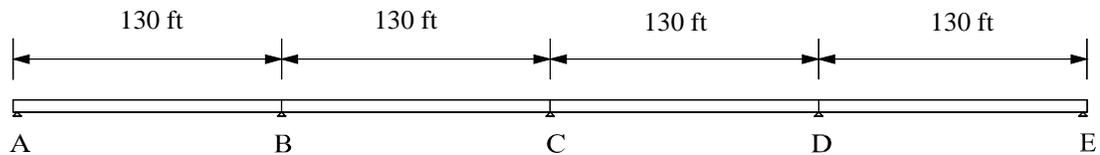


Figure 5.7 Bridge Elevation Showing Abutment and Pier Locations

Note that the word “creep” in this report represents all time-dependent effects of creep and shrinkage of the girder and deck concrete as well as of relaxation of the prestressing steel. It can be seen that the positive restraint moment, $M = 883$ ft-kips (1197 kN-m), at Pier C is the most critical.

To design for the 883 ft-kips (1197 kN-m) moment at Pier C, the strands at the girder end are extended into the diaphragm, as shown in Fig. 5.8. An approximate area of untensioned strands may be estimated using the tension-compression lever arm $jd = 0.9d$, where d is the effective depth from the top of the slab to the centroid of the strands being bent.

Assuming that the centroid of the bent strands is 4 in. (100 mm) from the bottom fiber, $d = 74.5$ in. (1892 mm). Using $f_s = 30$ ksi (210 MPa) for crack control,

$$\begin{aligned}A_{ps} &= M / (0.9d)(f_s) \\A_{ps} &= 883,000 \times 12 / (0.9 \times 74.5 \times 30,000) \\&= 5.27 \text{ sq in. (3400 mm}^2\text{)}\end{aligned}$$

The number of strands required is $= 5.27 / 0.153 = 34.4$, use 36 strands.

Such a large number of strands is needed because of the very high value of the creep coefficient used in the analysis in Reference 6, and because Reference 6 does not consider that allowing controlled cracking could significantly reduce the magnitude of the restraint moment. Normally the required number of bent strands is in the range of 20 to 40 percent of the total number of available bottom strands.

The Nebraska Department of Roads has in the past used the rule that 30 percent of the available bottom strands, but not less than eight strands, should be bent into the abutment diaphragm. As discussed above, the total embedment length should not be less than 16 in. (406 mm). The detail of the bent strands is shown in Figure 5.8.

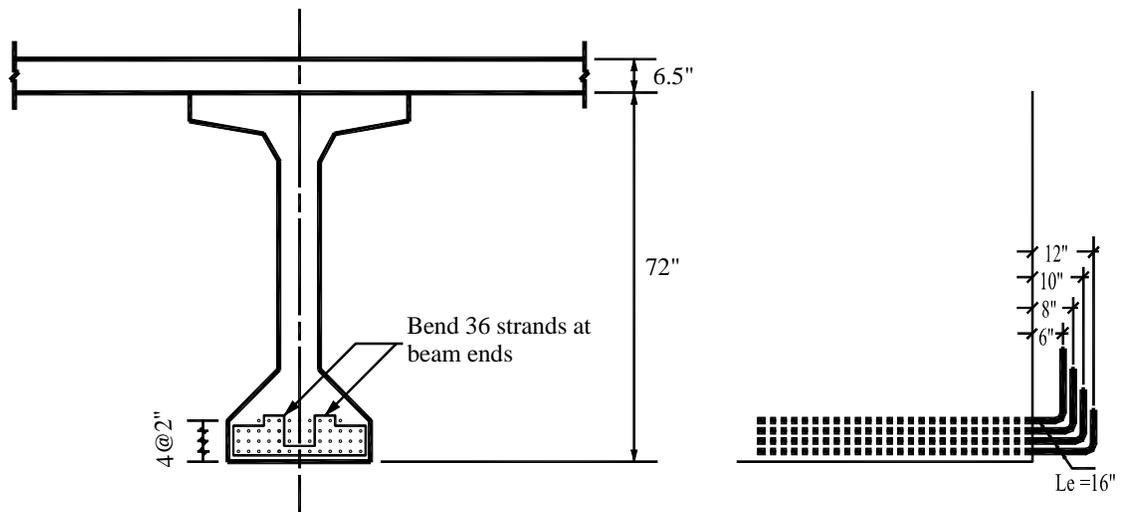


Figure 5.8 Required Strand Embedment at Beam End

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of non-prestressed bent strand pullout tests of 0.5 and 0.6 in. (13 and 15 mm) diameter strands embedded in cast-in-place concrete diaphragms with a concrete compressive strength of 4000 psi (28 MPa) or greater, the following conclusions and recommendations can be made:

1. The amount of additional cost in labor and materials caused by extending strands and embedding them into end diaphragms is negligible.

2. The pullout capacity of the bent strands is proportional to the total embedment length.

3. The pullout stress of a non-prestressed bent strand for a given embedment length can be predicted by Eq. (3.1). This equation can be used to determine the embedment length required at bridge abutments in applying Section 5.8.3.5 of the AASHTO LRFD Bridge Design Specifications, which requires that a “tension tie” be provided at beam end for shear capacity calculations.

4. It is conservative to assume that strands can attain 80 percent of the specified strand strength, $0.8f_{pu}$, when the embedment lengths are at least 30 and 36 in. (760 and 914 mm) for 0.5 and 0.6 in. (13 and 15 mm) diameter strands, respectively.

5. A minimum embedment length of 16 in. (406 mm.) is recommended for crack control (at service load level) due to time-dependent restraint positive moments at piers.

The full-scale tests of NU I-beam end anchorages have demonstrated that the pullout capacity formula can be conservatively utilized to establish anchorage of strands into end diaphragms. This anchorage can be provided at abutments to enhance the shear capacity, and at the piers to control time-dependent restraint member cracking. The maximum shear capacity of $0.25f_c' b_v d_v$ given in the AASHTO LRFD Bridge Design Specifications is achievable with adequate strand anchorage as recommended in this study. Utilization of this rather high shear capacity should result in significant economy of I-beam bridge systems, which was not recognized when the AASHTO Standard Specifications were used for design.

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