# Shear Behavior of Texas U-Beams

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## ABSTRACT

*A thorough review of the literature revealed there are no reported shear tests on U-shaped prestressed beams. Given the extensive use of U-Beams throughout the state of Texas since the introduction of the standard in the 1990s, and the increased use of the same or similar designs in other states in the United States, a testing program on this beam design was justified.*

*Two Texas U54 beams fabricated and tested are described in this paper. The first beam was designed following the current standard with regard to reinforcement and geometry. The failure of this test region brought to light a deficiency in the design: the beam failed at the bottom flange-to-web interface. The failure occurred at an applied shear 44% below the calculated vertical shear capacity.*

*The second beam was designed with modified reinforcement and geometric details, specified for the purpose of strengthening the bottom flange-to-web interface. With regard to calculated vertical shear capacity, the strength of this second test specimen was similar to the first. The modified test specimen failed in flexure-shear at an applied shear 27% in excess of the calculated vertical shear capacity.*

*The performance of these two U-Beam test specimens – one with the existing standard reinforcement and one with the modified reinforcement – are presented in this paper.*

**Keywords:** Prestressed concrete, shear, horizontal shear, U-Beam

**INTRODUCTION**

A comprehensive research study on the shear behavior of prestressed concrete U-Beams was performed at the University of Texas at Austin. Shear tests were performed on full-scale bridge beams as part of Texas Department of Transportation (TxDOT) Research Project 0-5831[1]. This paper focuses on the results from shear tests conducted on two test specimens.

The first test specimen was designed following the current TxDOT U-Beam standard drawings. The second specimen was fabricated with confining reinforcement, a lengthened end block, and supplementary reinforcement crossing the bottom flange-to-web interface.

This paper begins with a description of the Texas U-Beam and the motivation behind this research. The details of the two test specimens and the test results are the focus of the remainder of the paper.

TEXAS U-BEAM

The cross-sectional shape and dimensions of the Texas U54 U-Beam can be seen in Figure 1. The beam design was introduced to the TxDOT bridge standards[2] in 1998 as an alternative to I-Beams in high-visibility intersections[3]. The use of U-Beams in an overpass is considered more aesthetically pleasing than a comparable bridge of I-Beams as fewer beam lines are needed, improving the appearance as viewed from below.

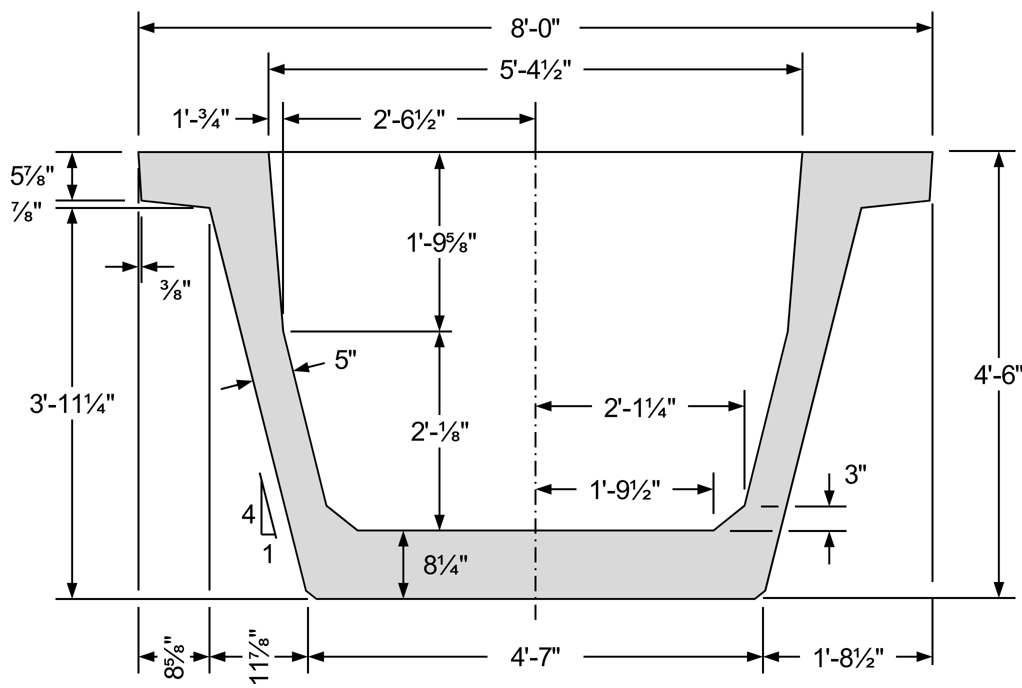


Figure 1: Cross-section of the Texas U54.

The cross-section of the U-Beam was optimized with regards to flexural capacity[3]. The cross-section has a large bottom flange that can hold a maximum of 81 prestressing strands, two 5-in. web walls, and an open top. The general dimensions came about through modifications to Houston Trapezoidal Girders, which were widely used in the state at the time. The major difference between the two designs is the lack of a monolithic top slab in the U-Beam; the open top of the U-Beam design allows for the use of reusable steel void forms that are removed after casting.

While a handful of studies performed in the 1990's investigated some aspects of the Texas U-Beam behavior[4,5,6], the shear strength of the beam was never studied experimentally.

RESEARCH SIGNIFICANCE

Codified equations exist to estimate the vertical shear capacity of the beam. These equations have been shown to be conservative when compared to results from the literature[7]. The majority of tests in the literature, however, were conducted on small rectangular and I-shaped beams. The equations were generally calibrated using these same small specimens with simple geometry. The Texas U-Beam, massive in size, typically heavily prestressed, and with unusual geometry, does not resemble these beams. Prior to performing a full investigation of the behavior of the beam, it was not clear whether the behavior of the Texas U-Beam would resemble that seen in the simpler beams present in the literature.

To highlight the difference between the Texas U-Beam and previously-tested beams, consider shear area ($b_w d$). The University of Texas Prestressed Concrete Shear Database[6] contains 1688 shear tests from the literature, from 99 sources, reported between 1954 and 2010. The largest shear area reported was 543 in.$^2$. The standard Texas U-Beam has a shear area of 605 in.$^2$. While the measured shear capacity of specimens reported in the literature generally exceeded the capacity calculated following existing design equations, the appropriateness of the equations have never been confirmed for a beam the size of the Texas U-Beam. As more U-Beams are constructed in Texas and the design is used in exact or modified form in other states, the need to verify the design equations becomes more critical.

## TEST SETUP

Prior to shear testing, an 8 in. cast-in-place deck was cast on the U-Beams. The deck was eight feet wide; there was no overhang (Figure 2). The beams were simply supported and loaded with two rams positioned over the webs an equal distance from the bearing. The shear span was 12'-10" (154 in.), resulting in an $a/d$ of 2.6. The clear span for specimen B2N was 25'-3" (303 in); the clear span for specimen B6S was 29'-0" (348 in.). The studied end of a beam in the load frame is illustrated in Figure 2.

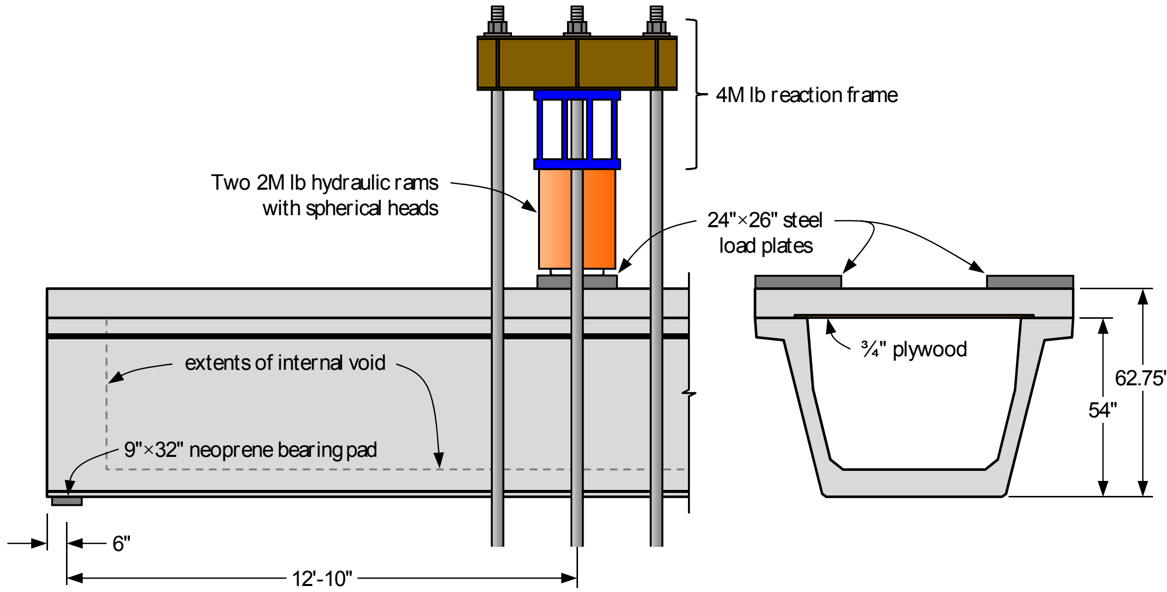


Figure 2: Shear test frame and beam cross-section.

The test regions rested on a single bearing pad, 32 in. wide by 9 in. long. The distance from centerline of bearing pad to beam end was 6 in. Shear in the test region was measured through two load cells placed beneath the bearing pad at beam end, pictured in Figure 3. The dead load shear for the point halfway between beam support and load point was included in the failure shear values.

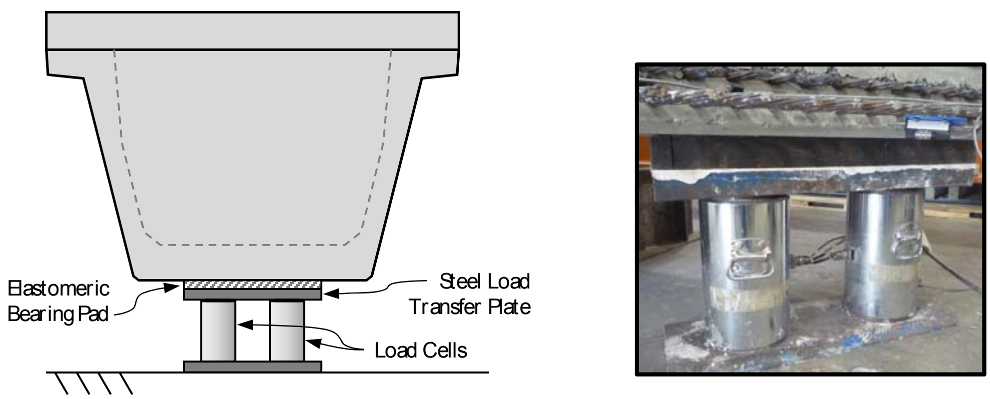


Figure 3: Bearing condition used during testing, with load cells pictured.

Load was applied through two hydraulic rams positioned over the beam webs. The rams were fitted with spherical heads, and rested on steel plates measuring 24 in. long by 26 in. wide.

The exact vertical reinforcement layout used in each test region will be provided later in this paper. Each contained bars spaced at 4 in. near the beam end and spaced at 6 in. closer to the load point. Given the nearly constant shear in the section (varying longitudinally only due to dead load), the failure was expected to occur in the region of the beam with reinforcement spaced at 6 in.

## SPECIMEN WITH CURRENT TXDOT DESIGN

The first test region discussed in this paper, B2N, was designed following the existing Texas Department of Transportation (TxDOT) Prestressed Concrete U-Beam Standard Drawings[2], issued in 2006. Ignoring end block reinforcement, vertical reinforcement consisted of a D20 welded wire reinforcing mesh stirrup spaced at 4 in. from beam end to 6'-3" (Figure 4). At that point, the reinforcement spacing increased to 6 in. The beam end block measured 18 in. long, the minimum allowed by the TxDOT Standard.

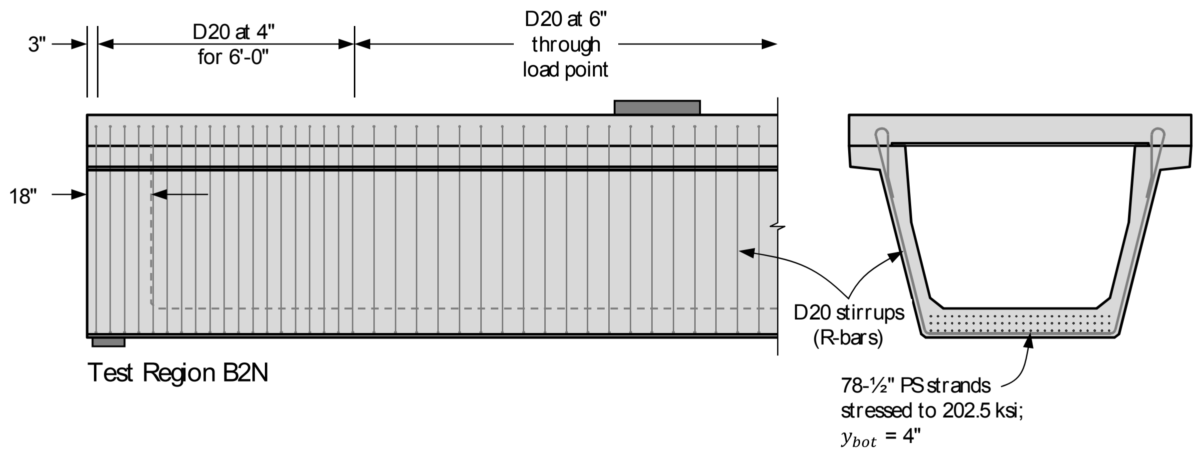


Figure 4: Test region B2N reinforcement layout.

Test specimen B2N contained 78-½ in. prestressed strands, stressed to 202.5 ksi. It was calculated that after accounting for all losses, the stress in the strands was 157 ksi. The welded wire reinforcing mesh used for vertical reinforcement had a measured yield stress of 85.2 ksi. The compressive strength of the beam concrete on the day of testing was 11.5 ksi; the compressive strength of the deck concrete was 8.6 ksi.

## RESULTS

Test specimen B2N failed at a shear of 610 kip, equal to 56% of the calculated vertical shear capacity, found following the AASHTO LRFD Bridge Design Specifications[9] General Procedure (Section 5.8.3.4.2). The test region is shown after failure in Figure 5. It should be noted that the damage that occurred at failure was concentrated in the very end of the beam, near the bottom flange-to-web interface, not at mid-height of the web near the load point, as had been expected.

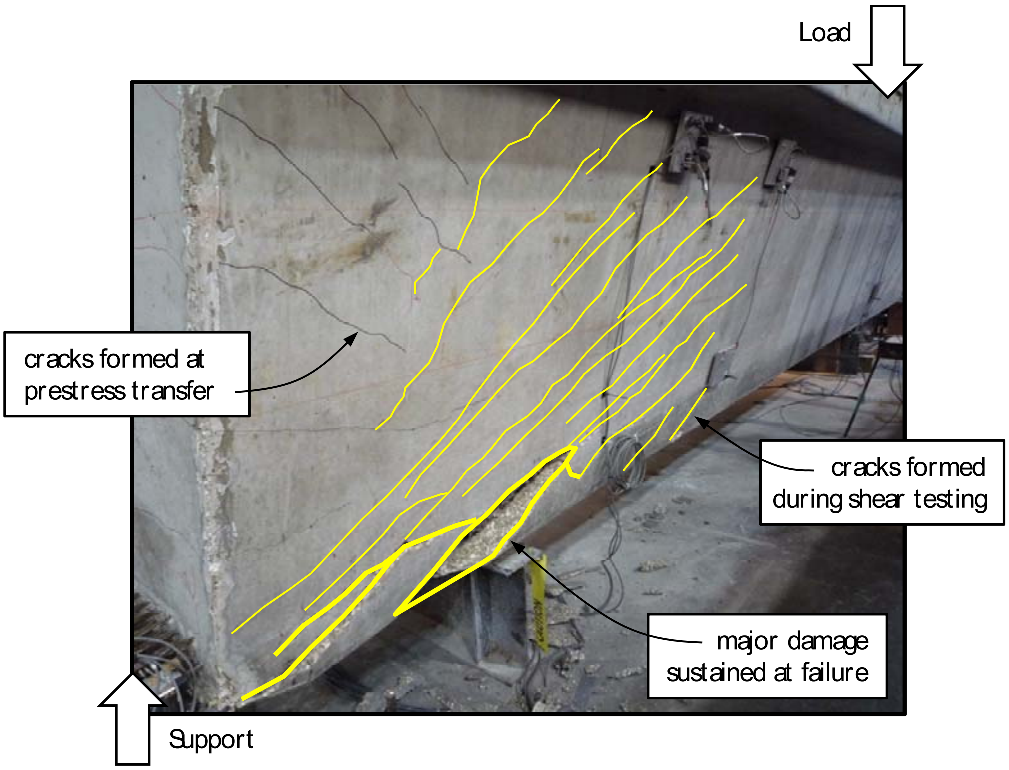


Figure 5: Test specimen B2N after failure.

Prior to removing the beam from the laboratory, it was cut into two pieces. Once the cut was made, the damage sustained on the interior of the beam could be examined. A horizontal shear crack could be seen along the interface between the bottom flange and the web of the beam (Figure 6).

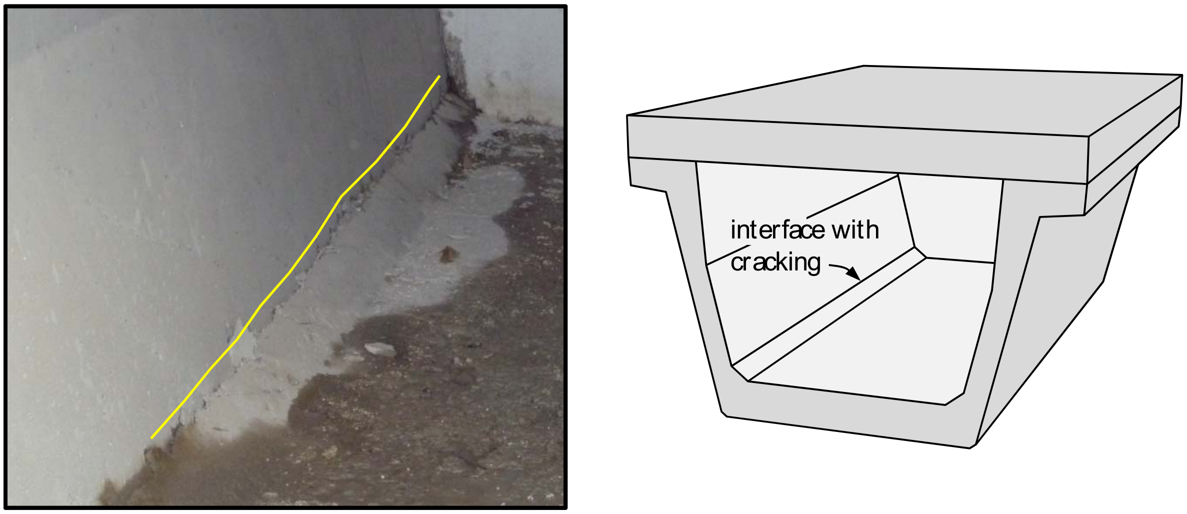


Figure 6: Horizontal cracking visible at the bottom flange-to-web interface on the inside of test specimen B2N.

HORIZONTAL SHEAR

The damage seen at failure of specimen B2N revealed that the strength of the beam was governed by horizontal shear: a sliding of the web relative to the bottom flange. The failure was precipitated by vertical loads causing horizontal stresses along the bottom flange-to-web interface that exceeded the capacity of that interface. This type of failure has previously been reported in the literature[10,11], but has generally been seen at loads that exceed calculated vertical shear capacities. This U-Beam specimen failed in horizontal shear at a load 54% below the calculated vertical shear capacity. No significant distress was seen in the webs at failure.

In an effort to prevent this failure mode, modifications to the standard design were suggested by the research team and tested using three test regions not discussed in this paper. The amount of steel crossing the bottom flange-to-web interface was varied in these three test regions. Based on the observed failure modes of the three test regions, a final steel distribution was recommended for use in the standard; the failure behavior of this design was confirmed using test specimen B6S.

## SPECIMEN WITH MODIFIED DESIGN

A cross-section of test specimen B6S showing the reinforcement used is given in Figure 7. The D20 stirrups (R-bars) were spaced at 4 in. from beam end to 8'-3". At that point, the reinforcement spacing increased to 6 in. The beam end block measured 30 in. longitudinally. Supplementary reinforcing bars (L-bars) were used, with two #5 L-bars paired with each R-bar in each web from beam end until the spacing change at 8'-3". Confining reinforcement (C-bars), not currently used in Texas U-Beams, was included, also paired with the R-bars through the spacing change.

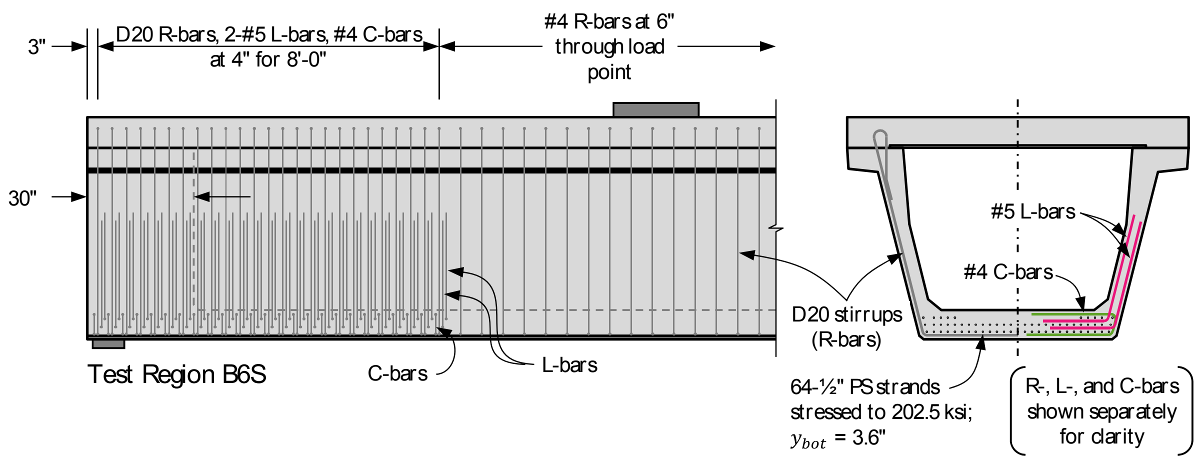


Figure 7: Test region B6S reinforcement layout

Test specimen B6S contained 64-½ in. prestressed strands, stressed to 202.5 ksi. It was calculated that after accounting for all losses, the stress in the strands was 157 ksi. The primary vertical reinforcement was made of welded wire reinforcing mesh with a measured yield stress of 85.0 ksi. The compressive strength of the beam concrete on the day of testing was 12.0 ksi; the compressive strength of the deck concrete was 10.7 ksi.

## DETAILS OF ADDITIONAL REINFORCEMENT

The L- and C-bars used in test specimen B6S do not currently exist in the Texas U-Beam standard. Justification of the addition of these bars and their intended purpose is provided here.

### Bottom Flange-to-Web Interface Steel (L-bars)

Additional reinforcing bars were added across the bottom flange-to-web interface in order to strengthen that interface against horizontal shear forces. The available resistance to sliding of the interface of a wall to a beam or a corbel to a column is calculated following codified equations for shear-friction (e.g., AASHTO LRFD Equation 5.8.4.1-3). Adding steel across the critical interface will increase the available capacity of that interface to resist sliding forces.

The amount of steel crossing the interface was chosen after studying the amount of steel used in the end regions of other prestressed concrete bridge girders for which horizontal shear has not been a concern. Ignoring end block reinforcement, the existing standard U-Beam design contains 1% reinforcement crossing the bottom flange-to-web interface at beam end; a Texas I-Girder contains 6% reinforcement at this interface. The recommended design and the design used in test specimen B6S contained 4.1% reinforcement at the bottom flange-to-web interface.

Serviceability and constructability considerations guided decisions regarding the detailing of supplementary reinforcement. The added bars were dimensioned so that they would not contribute to the vertical shear capacity. Increasing the amount of steel contributing to vertical shear capacity without increasing the concrete contribution would increase the likelihood of diagonal cracks forming at service-level shears (as $V_{cw}/V_n$ is decreased)[10]. The L-bars were detailed to stop halfway up the beam web; with that geometry the bars can be developed at the bottom flange-to-web interface.

Following good detailing practices, two #5 bars were used in each web rather than a single larger bar. The narrow web walls of the U-Beam and the tight spacing of the strands through which the bar must pass make installation of a large diameter bar (with an associated large bend radius) difficult. The L-bars recommended for use can be placed without significant congestion or interference.

### Confinement (C-bars)

Confining reinforcement was included around the prestressing strands to satisfy the AASHTO LRFD Specification[9] §5.10.10.2 that states:

> *For the distance of 1.5d from the end of the beams other than box beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall not be less than No. 3 deformed bars, with spacing not exceeding 6.0 in. and shaped to enclose the strands (pg. 5-158).*

The existing standard U-Beam design does not include any confinement to the prestressing strands other than what is provided by the bend of the stirrup. This design was implemented following the research of Barrios[4], who studied the response of Texas U54 beams with and without confinement steel at prestress transfer. As no cracks were found in the lower region of the beam at prestress transfer, the recommendation at the time was to use the design without additional confining reinforcement. Those beams were not load tested.

A study of AASHTO Type II girders tested with and without confining reinforcement[8] found that beams containing confining reinforcement were able to carry 10 to 20% more load prior to failure.

RESULTS

Test specimen B6S failed in flexure-shear at a shear of 1054 kip, 27% in excess of the calculated vertical shear capacity for the section. A picture of the test region after failure is given in Figure 8. It can be seen that the damage is concentrated towards the load point, where reinforcing bars were spaced at 6 in. Diagonal cracks in the webs measuring greater than 0.02 in. were seen prior to failure.

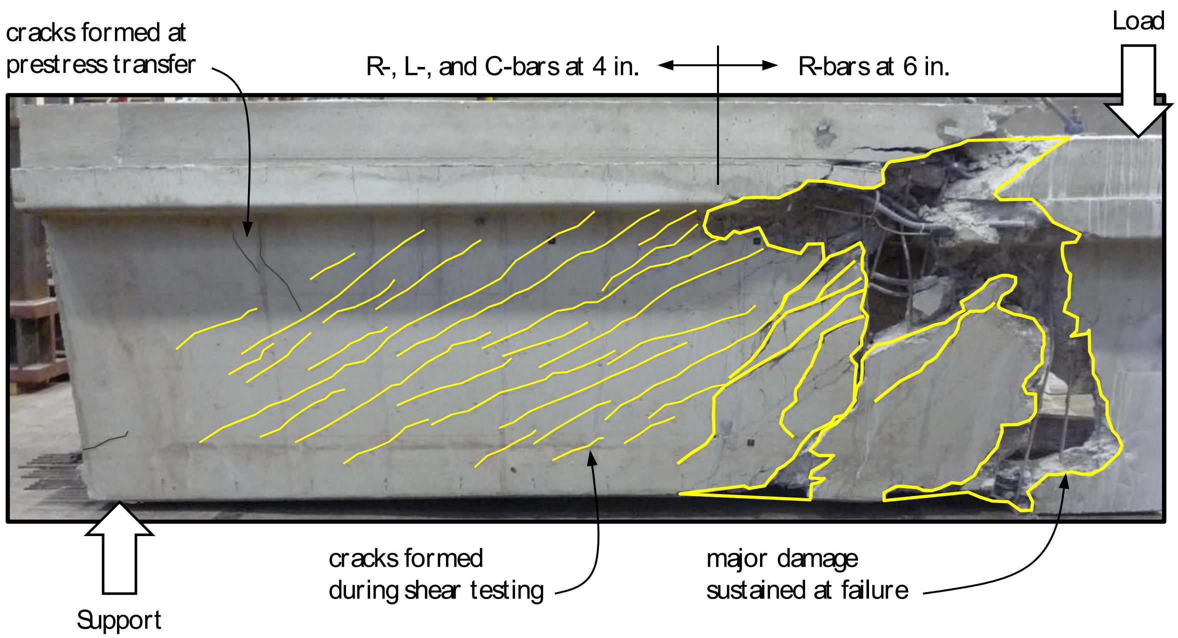


Figure 8: Test specimen B6S after flexure-shear failure.

The supplementary bars (L-bars) added in the beam end to strengthen the bottom flange-to-web interface did so sufficiently: there was no evidence of distress along the critical interface on the outside or inside of the beam.

## DISCUSSION OF RESULTS

Test regions B2N and B6S contained very similar vertical shear reinforcing bars, consisting of D20 welded wire reinforcing mesh spaced at 4 in. near beam end and 6 in. towards the load point. The capacity calculations for each region of each test specimen, found following the AASHTO LRFD General Procedure[9], are given in Table 1. Given the similar properties of the beams, the calculated capacities in the 4 in. spacing regions and 6 in. spacing regions are similar between the two beams.

Table 1: Vertical shear capacity calculation details.

| | | $f_c'$ [ksi] | $b_v$ [in.] | $d_v$ [in.] | $M_u$ [kip-in.] | $V_u$ [kip] | $A_{ps}$ [in.$^2$] | $f_{po}$ [ksi] | $E_c$ [ksi] | $A_{ct}$ [in.$^2$] | $\varepsilon_s$ $\times 10^3$ | $\beta$ | $V_c$ [kip] | $A_v$ [in.$^2$] | $f_y$ [ksi] | $\cot\theta$ | $s$ [in.] | $V_s$ [kip] | $V_n$ [kip] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| B2N [std] | End | 11.5 | 10.0 | 52.9 | 74,330 | 978 | 11.9 | 203 | 6107 | 717.5 | 0.0 | 4.8 | 273 | 0.40 | 85.2 | 1.8 | 4.0 | 814 | 1087 |
| | Mid | 11.5 | 10.0 | 52.9 | 58,075 | 764 | 11.9 | 203 | 6107 | 717.5 | -0.1 | 5.3 | 298 | 0.40 | 85.2 | 1.8 | 6.0 | 551 | 849 |
| B6S [rec'd] | End | 12.0 | 10.0 | 53.2 | 66,794 | 867 | 9.8 | 203 | 6249 | 717.5 | 0.5 | 3.5 | 204 | 0.40 | 85.0 | 1.7 | 4.0 | 760 | 964 |
| | Mid | 12.0 | 10.0 | 53.2 | 57,732 | 750 | 9.8 | 203 | 6249 | 717.5 | 0.0 | 4.9 | 287 | 0.40 | 85.0 | 1.8 | 6.0 | 547 | 833 |

The measured failure shear, calculated shear capacities, and failure crack patterns for the two test specimens are summarized side-by-side in Figure 9. While the calculated capacities of the two beams were similar, the ability of the beam to carry load was increased significantly in specimen B6S, due to the modifications to the reinforcing bars and end block length. The failure mode was also different: while specimen B2N failed at the bottom flange-to-web interface near beam end, with little damage to the web near the load point, specimen B6S failed in flexure-shear in the weaker section of the beam where the bars were wider spaced.

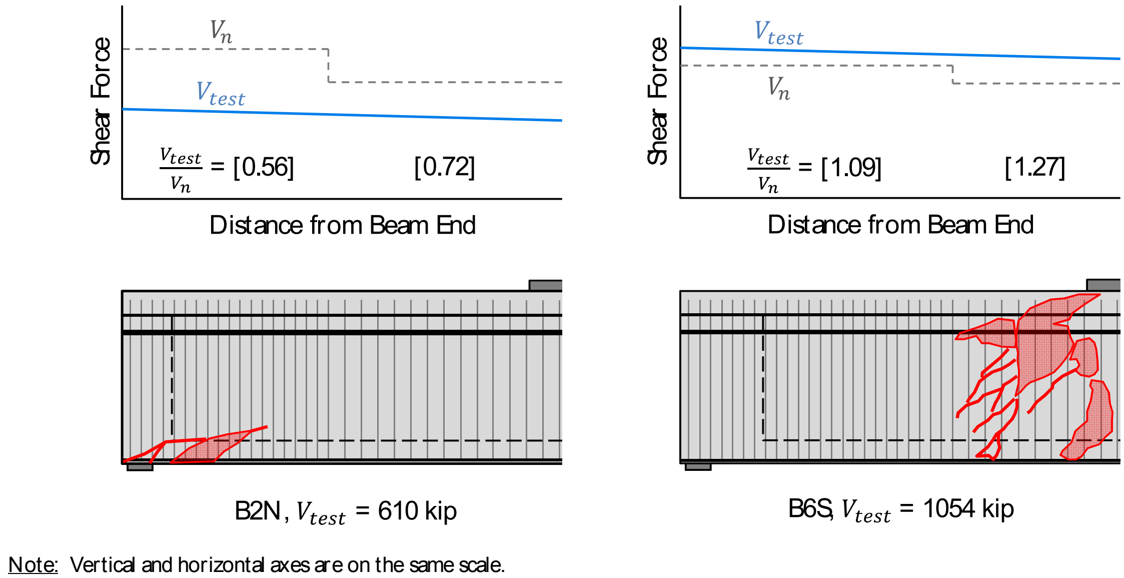


Figure 9: Summary of shear performance of test specimens B2N and B6S.

The reinforcement crossing the bottom flange-to-web boundary in B2N was not sufficient to prevent horizontal shear from occurring. Unlike previously-reported cases of horizontal shear failure in prestressed beams, the failure of the interface occurred at a load much below the calculated vertical shear capacity. The detailing changes made in specimen B6S increased the capacity of the bottom flange-to-web interface. The test specimen failed in a typical web-shear failure mode, with no signs of horizontal shear distress. The failure shear was well in excess of the calculated vertical shear capacity.

## CONCLUSIONS AND RECOMMENDATIONS

Two U-Beam test specimens are discussed in this paper. The first was designed following existing standard TxDOT U-Beam drawings. The second contained modified end-region details recommended by the research team. The end-region details of the second specimen differed from the first in four respects: (i) the amount of steel crossing the bottom flange-to-web interface at beam end was increased from 1% to 4%; (ii) the 4 in. bar spacing used at beam end extended two additional feet into the span; (iii) the end block was lengthened from 18" to 30"; and (iv) confinement steel was added.

The influence of these changes was significant. While the test specimen reinforced following the current standard (B2N) failed in horizontal shear at an applied shear 44% below the calculated vertical shear capacity, the test specimen with the recommended amount of steel (B6S) failed in flexure-shear 27% above the calculated vertical shear capacity. It is recommended that future U-Beams be fabricated with inclusion of these design changes.

## ACKNOWLEDGEMENTS

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The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Texas Department of Transportation.

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