APPENDIX C

STRUCTURAL INVESTIGATION & FULL-SCALE GIRDER TESTING

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(The specimens are listed in the order they were fabricated and tested)

C.1 TENNESSEE STATE SPECIMENS

Description and design calculations

The team has been working with Don Thomson of Construction Products, Inc. in Jackson, Tennessee. They fabricated two forty-two foot long I-girders, which were produced in the prestressing bed at the same time as the production of their projects. Therefore, our girders are of the same shape, and contain the same amount of prestressing as the production girders.

The specimens are Type III AASHTO Beams with a specified concrete strength, $f_c^{'}$, of 7,000 psi and release strength, $f_{ci}^{'}$, of 6,000 psi. Each specimen has thirty 0.5 in. diameter, 270 kip, low relaxation prestressing strands, stressed to 33.8 kips per strand. They also contained two partially stressed 0.5 in. diameter, strands in the top flange, stressed to 5 kips per strand. All strands were straight and contained no debonding. The bottom flange contained confinement reinforcement, as shown in Figure C.1-1. Shear reinforcement consisted of #5 C-shaped bars placed in pairs.

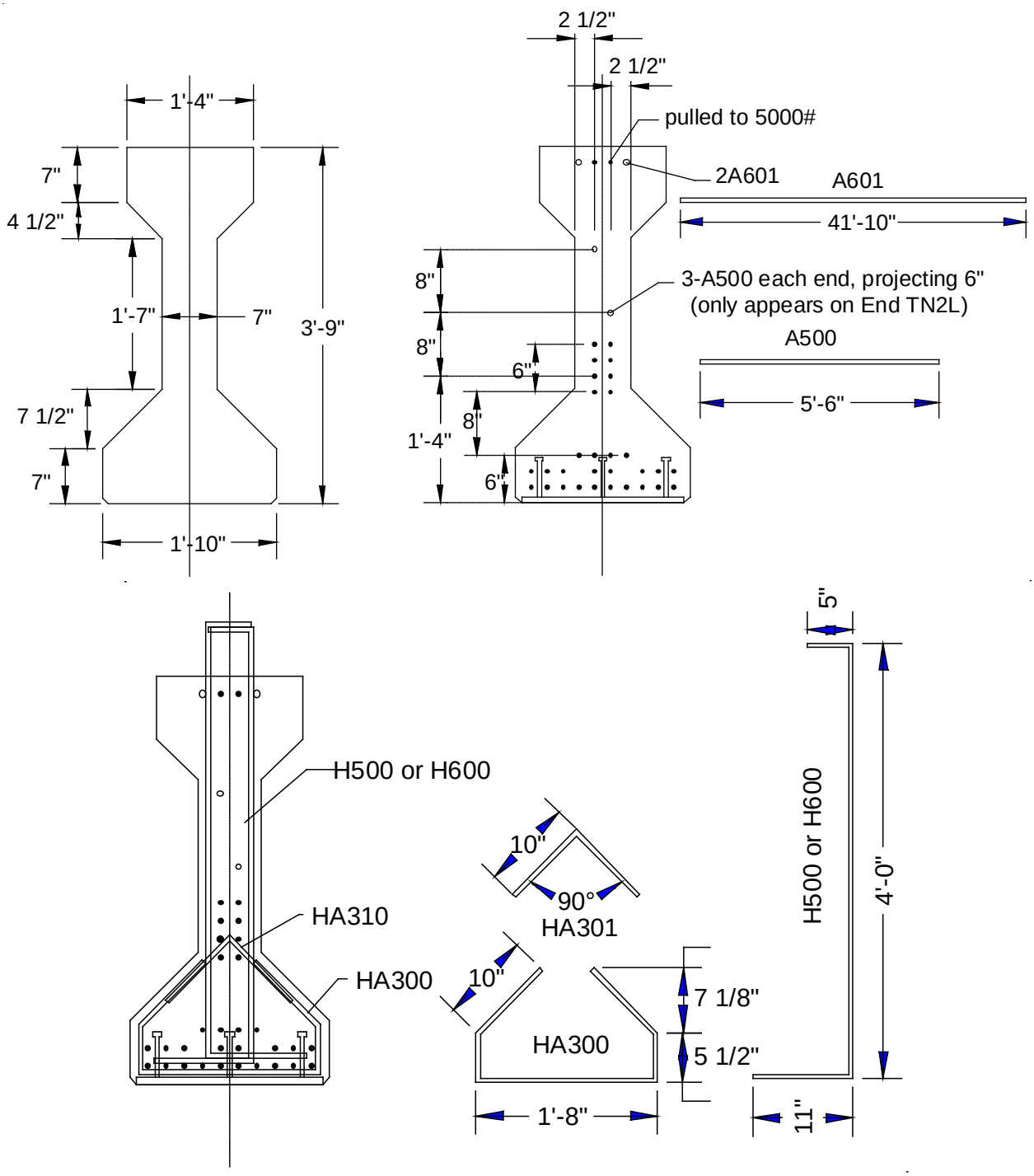


Figure C.1-1. Tennessee Specimens, TN1 and TN2

The Team designed these specimens such that they would fail in flexure, as opposed to shear. Each girder end has a different design for the end zone reinforcement. The details are shown below.

Flexural Capacity:

The specified $f_c^{'}$ = 7,000 psi. The girders were expected, however, to have an actual capacity close to $f_c^{'}$ = 9,000 psi, which was used in the capacity predictions.

A strain compatibility program was used to accurately determine the flexural capacity with a resistance factor of 1.0. Some of the important values of the analysis are given below:

Depth of the equivalent compression block = 9.66 in.

Tensile stress at the strand layer closest to the bottom fiber = 252 ksi

Nominal flexural capacity, M_n = 3120 ft-k

When testing the girders, they will be supported at 6 in. from the end, leaving an unsupported length of 41 ft. A point load will then be applied at 11.5 ft from one support (29.5 ft from the other support) until failure, see Figure C.1-2.

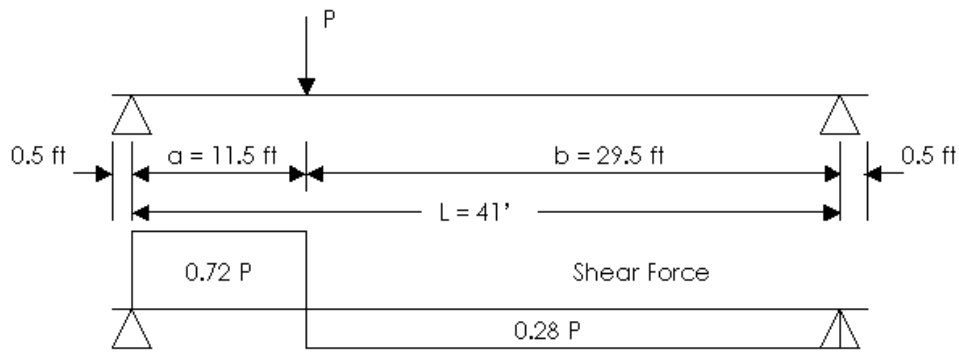


Figure C.1-2. Loading Position for Tennessee Specimens, TN1 and TN2

Therefore, the force point load, P, required for flexural failure is:

$$M_n = \frac{P.a.b}{L}, 3120 = \frac{P(11.5)(29.5)}{(41)}, P = 377 \text{ kips}$$

Shear Capacity:

The maximum load that the beam is expected to resist in flexure was rounded up to 400 kips. It was then used to design the shear reinforcement, in order to ensure the shear capacity exceeds the flexural capacity.

Effective shear depth, d_v, for the girder is the greater value of the following two equations:

$$d_v = \text{least } [\ 0.9*d = 0.9(37.9) = 34.1 \text{ in. and } 0.72*h = 0.72*(45) = 32.4 \text{ in. }]$$

Therefore, d_v = 34 in.

With a 400 kip load applied 11.5 ft from the end of the left support, maximum shear force is 288k. By iteration, θ is 23˚, and β is 3. Thus, A_v/s ratio can be estimates as follows:

$$V_n = 0.0316*\beta*\sqrt{(f'_c)}*b_v*d_v + (A_v/s)*f_y*d_v*cot(\theta)$$

$$288 = 0.0316*(3)*\sqrt{(7)}*(7)*(34) + (A_v/s)*(60)*(34)*\cot(23^\circ)$$

This gives an A_v/s = 0.0475 in^2/in or 0.57 in^2/ft.

The minimum spacing for the shear reinforcement according to Article 5.8.2.7 of the AASHTO LRFD Specifications = 12 in., therefore, required shear reinforcement, A_v = 0.57 in^2 = 2 #5 bars (with A_v = 0.62 in^2). To guarantee a flexural failure, the shear reinforcement was set = 2 #5 @ 6 in.

End Zone Reinforcement for End TN1L (Figure C.1-3)

The first "Left" end of Tennessee Specimen TN1 is labeled TN1L. It was designed by the research team to correspond to the end zone requirements of AASHTO LRFD 2007 Specifications. 4% of the prestressing force was assumed to be the splitting force. The splitting force was divided by an allowable steel stress of 20 ksi to obtain the splitting reinforcement. The corresponding area = 2.05 in^2. This amount is required to be placed within h/4 (one fourth of the total height) from the end of the beam, which in this case is 11.25 inches. Therefore, three pairs of #6 bars spaced at 3 in., center to center, were chosen. The first pair 3 in. away from the end of the girder. This places all of the steel within 9 inches from the end of the girder, well within the limits.

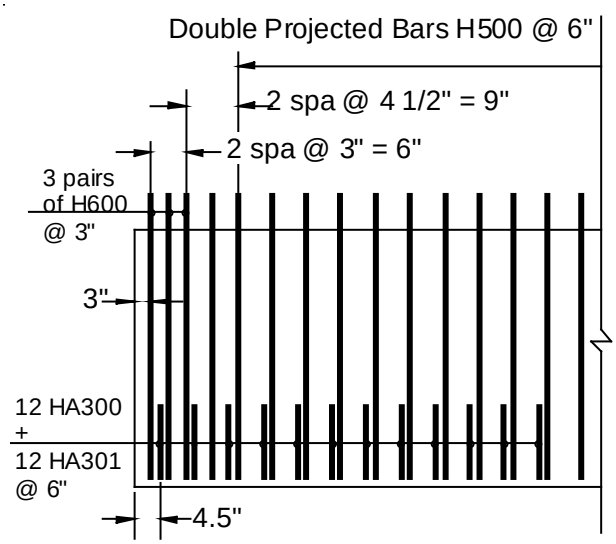


Figure C.1-3. End zone reinforcement of TN1L

End zone reinforcement for End TN2L (Figure C.1-4)

This end contained the same end zone details as the production girders. It consisted of a six pairs of #6 bars, spaced at 3 in. center-to-center, starting at 3 in. from the end of the girder. Beyond these six pairs, the shear reinforcement commenced, spaced at 6 in. This end also contains 3 horizontal #5 bars projecting 5 feet into the web of the girder, as given in the production girders.

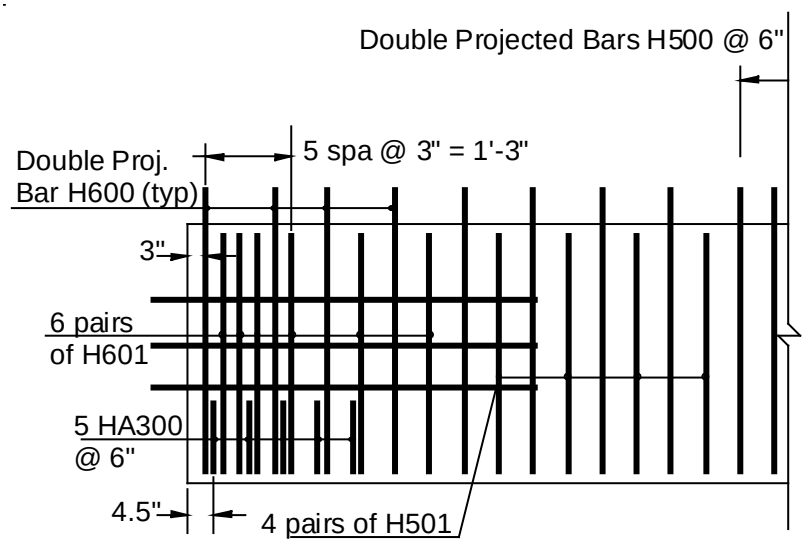


Figure C.1-4. End zone reinforcement of TN2L

End Zone Reinforcement for End TN1R and End TN2R (Figure C.1-5)

These are the right ends of the first and second specimens. They were both designed by using the improvements proposed for this project. Both contain the same end reinforcement details. The proposed method states that at least 2% of the prestressing force be taken at a stress of 20 ksi and the resulting reinforcement placed at h/8 from the end, with the first bar(s) placed as close to the girder end as possible. The remainder of the 4%, i.e. the steel corresponding to 4% minus the steel already used in the first h/8, should be placed in a zone not exceeding 3h/8 beyond the first zone. This, for these specimens, place 2% or more within 5.625 inches and the balance of the 4% in the next (22.5-5.625) = 16.875 inches. Thus, each of the Ends TN1R and TN2R contained three pairs of #6 bars, spaced 2 in. center to center, with the first pair located 1-1/2 in. from the end of the girder, in order to satisfy a minimum clear cover of 1 in. This corresponds to 3*2*0.44 = 2.64 in^2 placed within 1.5+2+2 = 5.5 in., which would satisfy the recommended total reinforcement requirements. However, a fourth pair of #6 bars was placed in the second zone (3h/8) followed by the shear reinforcement 2-#5 @ 6 in. It should be noted that #6 bars may be allowed in this zone if the girder ends are embedded in diaphragms wide enough to enclose the #6 bars, and thus not violate to concrete cover requirements. In some states, Washington for example, the girders are embedded only 3 in. into the diaphragms. In this situation, smaller size bars must be used to meet the cover requirements. Each of these two ends was also required to contain base plates that were the width of the girder, and extended 18 in. into the girder's length. Each had 6 studs, in two rows of three. The base plate is expected to help reduce cracking in the end zone and is one of the Team's recommendations.

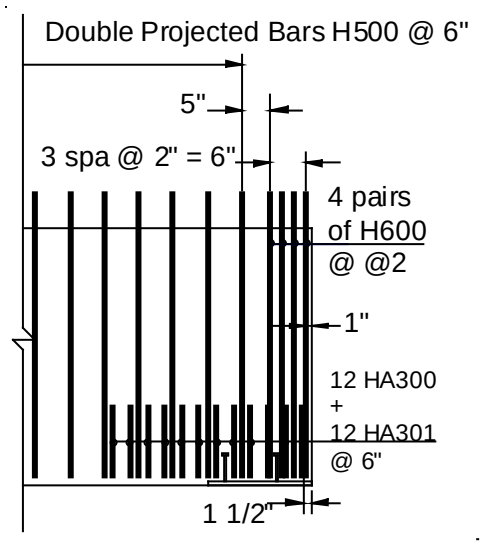


Figure C.1-5 End zone reinforcement of TN1R and TN2R

Fabrication and inspection

The two Tennessee girders were manufactured near the end of February. They were shipped to the University of Nebraska in the beginning of April 2008. Forty 4x8 in. concrete cylinders were received, along with samples of each size of bars, for materials testing.

Upon inspection, neither girder appeared to have experienced visible end zone cracking. The team suggests that the lack of end zone cracking is due to the limited amount of prestressing force, the presence of end zone reinforcement, and the size and shape of the girder. The girders contained thirty 0.5 in. diameter strands. This amount was the largest available to the producer, CPI, Inc., Jackson, Tennessee, at the time the request was made. The relatively small girder size and amount of prestressing, compared to the record depths, spans and levels of prestress in other states, has been a challenge to the research team. The Washington Specimens with 62-0.6" strands, and hopefully the Virginia specimens with 58-0.6" strands, will help create significant web end cracking conditions. The FL specimens have an intermediate value of 36-0.6" strands, as will be reported later. It will be a challenge to load these specimens to failure in the Structures Lab.

Structural testing and test results

(a) Girder TN2: (End TN2L)

On Friday May 16, 2008, the first event of the structural testing process took place. The girder was supported at each end and subjected to a point load, 12 feet from the girder end.

In order to simulate the decking system that would be placed on the production girders, a 7.5 in. deck was cast on top of the girder. The shear reinforcement extended above the girder surface throughout the entire length, and therefore the deck and girder would perform as a composite

system. This deck was designed to increase the amount of stress in the bottom steel strands at flexural failure.

The transverse reinforcement of the deck consisted of two #4 steel threaded rods provided at 4 ft spacing, one rod located 2 in. on center from the bottom of the deck and the other located 2 in. on center from the top of the deck, as shown in Figure C.1-6. The transverse reinforcement was used to support the side forms of the deck during construction, as shown in Figure C.1-7. The longitudinal reinforcement of the deck consisted of two #3 bars located 2.5 in. from the bottom of the deck and two #4 bars were located 2.5 in. from the top of the deck, as shown in Figure C.1-6.

In order to achieve high stress in the bottom prestressed strands at failure (260 ksi), the strength of the concrete deck was specified to be 9,000 psi. A self-consolidating concrete (SCC) was used. Two batches of same concrete mix were used to cast the deck. Batch A covered most of the deck, while Batch B was used for the last 8 feet of the end near TN2R. Figure C.1-7 shows the TN2 specimen during construction of the deck in the laboratory.

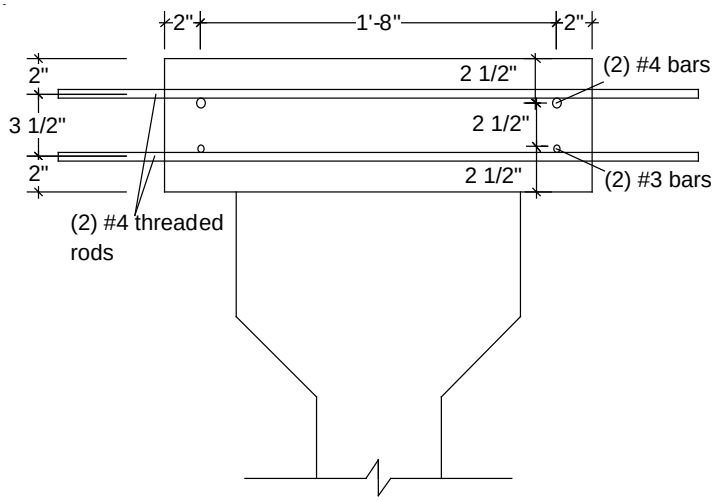


Figure C.1-6. Cross-section of TN2 showing the deck details

Figure C.1-7. Deck formwork

After the deck was cured, a clamping force mechanism was placed 30” in from the end of the girder in order to simulate the bridge weight acting on the supports. The clamping force was provided by using a hydraulic jack attached to a self-equilibrium frame built around the specimen as shown in Figures C.1-8, C.1-9 and C.1-10.

This 54-kip applied clamping force was calculated as the balance between the reaction generated by the 78 ft – 6 in. bridge that the original production girders were being manufactured for (taking into consideration weight of girder, deck and barriers) and the reaction generated by the 42 ft long specimen (taking into consideration weight of the girder and deck). This clamping mechanism was placed only at the end being tested, which was TN2L.

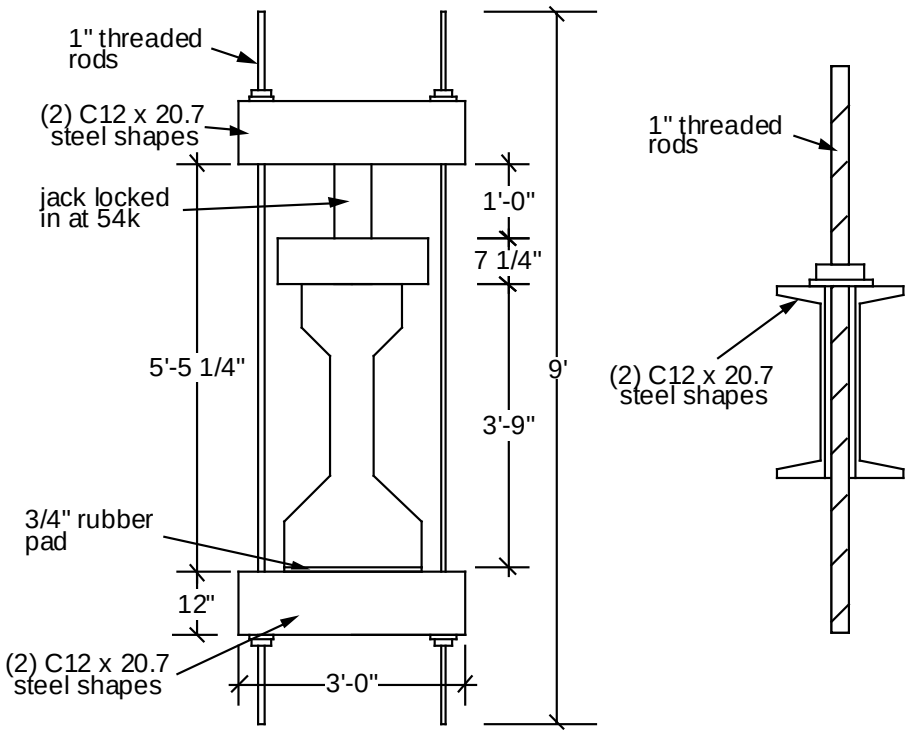


Figure C.1-8. End clamping detail

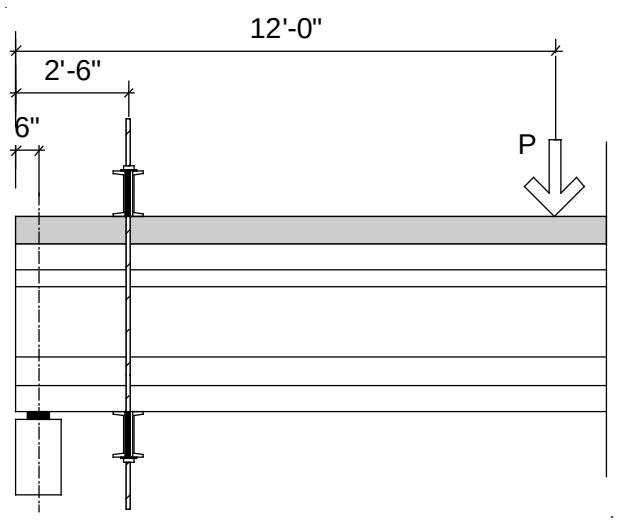


Figure C.1-9. Locations of point load, clamping mechanism

Figure C.1-10. Test setup before loading

Table C.1-1 shows the concrete strength of girder and the deck on the day of testing.

Table C.1-1. Concrete strength of the girder and deck of TN2

Compressive Strength (psi)	Girder	Deck (Batch A) TN2L	Deck (Batch B) TN2R
Minimum Specified	7,000	9,000	9,000
Actual	11,312	10,764	9,172

<u>End TN2L Test Results</u>

Table C.1-2 gives the Moment Capacity and Corresponding Point Load of the TN Specimens using the minimum specified and actual measured strength of the girder and deck.

Table C.1-2. Moment capacity and corresponding point load of the TN specimens

	Minimum Specified Strength	Actual Strength	Test Data
Moment Capacity (Mn)	4204 k-ft	4299 k-ft	4649 k-ft
Point Load (P)	492 k	504 k	546 k

Load was slowly applied to the system in intervals of 100k. After each additional 100k, the loading was paused so that the girder could be investigated and marked for cracks. Once the design load of 500k was reached the loading was stopped. At that level, the location directly below the load was experiencing a deflection of 1.38 in., as shown in Figure C.1-11. The load was then removed, leaving a permanent plastic deformation at the location of loading of 0.29 in.

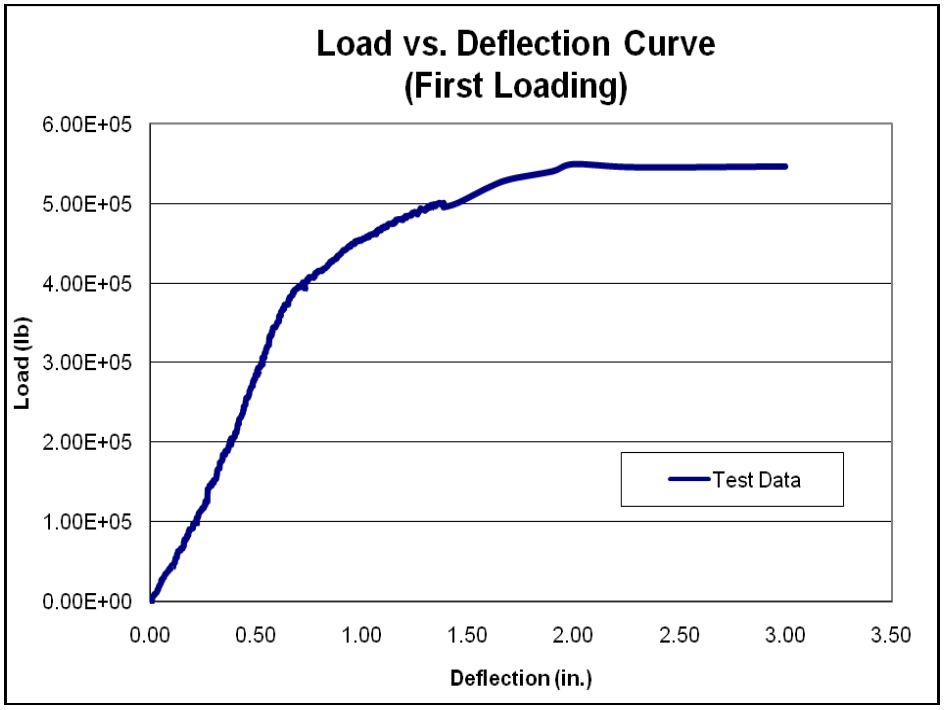


Figure C.1-11. Load Deflection Curve – First Loading Cycle

Force was then reapplied until the system began to no longer take any more load. At this point the applied load was 546 kips and the deflection at the location of loading reached 3 in. This was considered the point at which the girder reached failure. Cracking patterns that appeared were consistent with a classical flexural failure, Figure C.1-12. The beam was designed to fail in flexure, and this appeared to be the primary cause of failure. Some strand slippage, or bond failure was also apparent in the testing. Some of the strands sticking out of the ends rotated in the direction of the twisted wires as they slipped into the girder, Figure C.1-13.

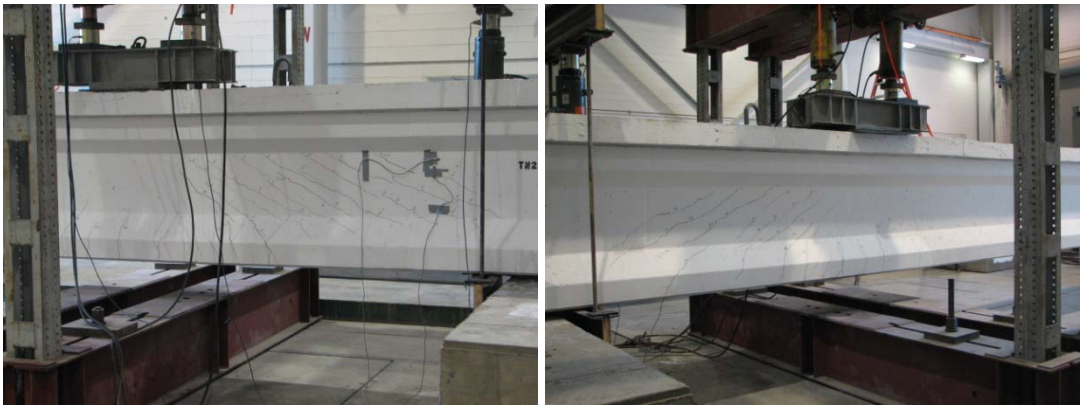


Figure C.1-12. Crack patterns at failure

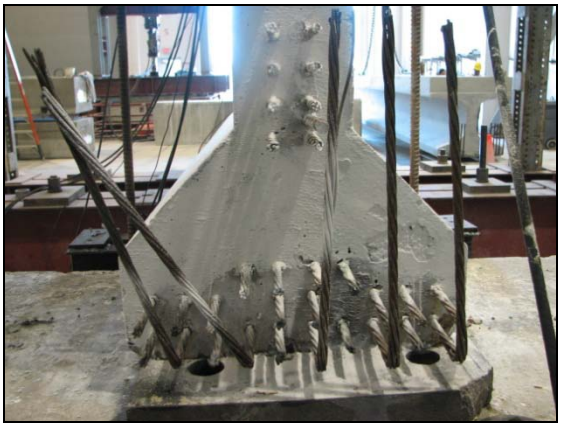


Figure C.1-13. Strand rotation at failure (see the counter clockwise rotation of the two bended strands on the left side of the picture)

Figure C.1-14 shows the load-deflection curve of the two cycles of loading. The girder with composite topping was calculated to resist a point load of up to 504 k, located 12 ft from the end. The actual maximum load reached before failure was 546 k, 8.3% higher than the expected load. There was no reduction in strength due to the conditions at the ends. As indicated earlier, there was no visible end zone cracking due to prestress release. Whatever microcracking existed, which was not possible to measure, did not appear to have any detrimental effect on the flexural capacity of the member. The development length was not harmed by end zone cracking. Shear capacity was designed to be larger than flexural capacity of that end. Again, no indicated of any reduction in shear capacity was observed as there was no diagonal compression crushing or stirrup breakage. As indicated before, this was not a severe case specimen. It would be premature to determine at this time if web end cracking has a detrimental effect until more specimens are tested.

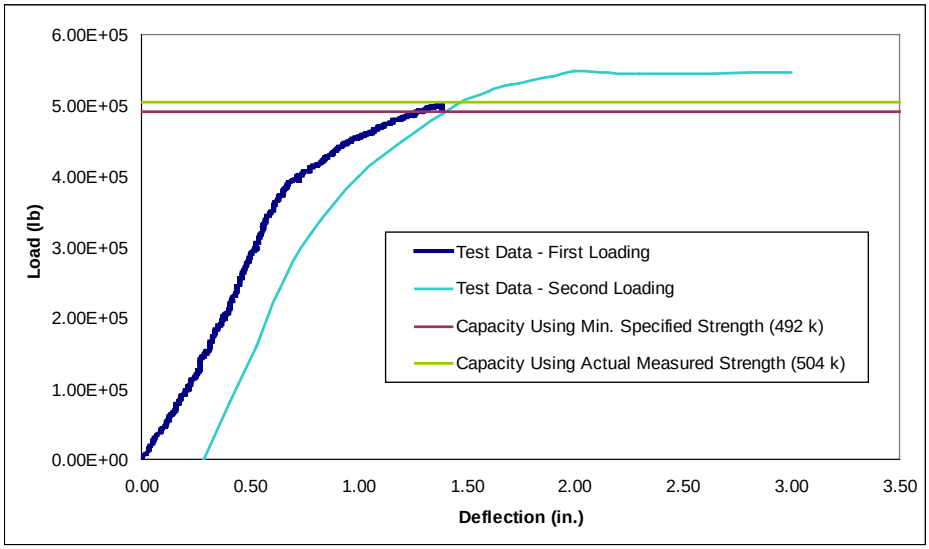


Figure C.1-14. Load deflection curve – first and second cycles of loading

Figure C.1-15 shows the clamping force applied to the end of the girder over time. Although the jack was locked into place at 54k, the force was observed to climb to as high as 56k. This increase in clamping force may have been caused by the rotation and shifting of the girder during testing. This increase is not expected to affect the results in any way.

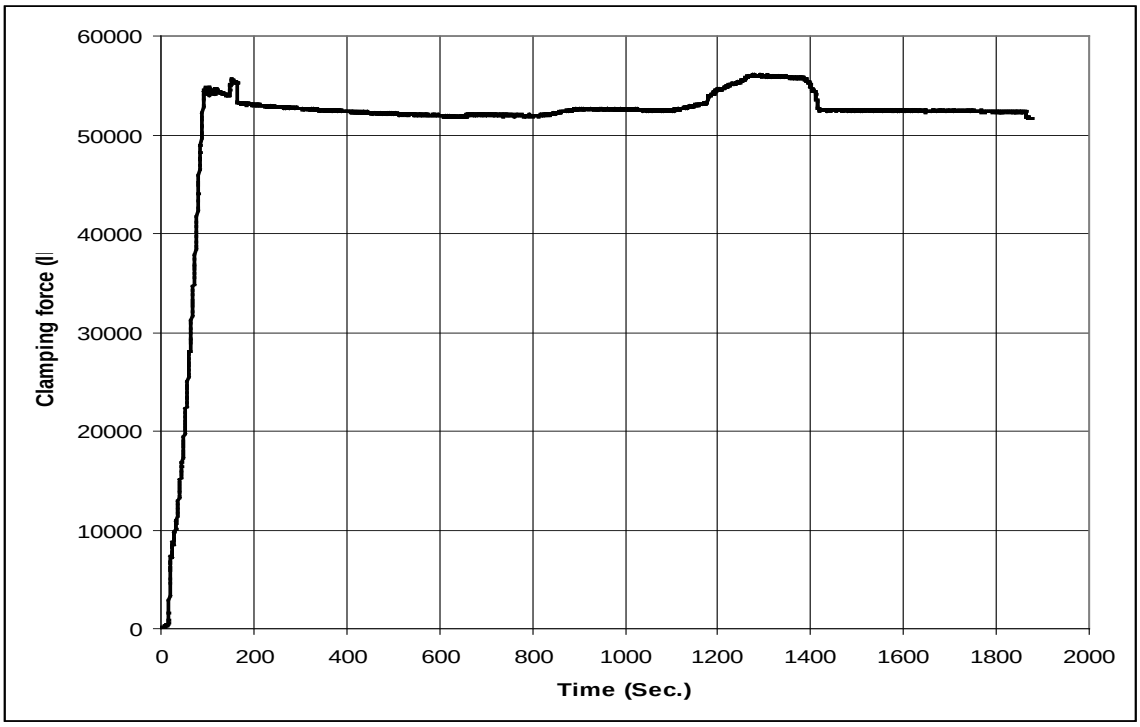


Figure C.1-15. Clamping force vs. time

End TN2R Test Results

End TN2R is located on the same girder as End TN2L. In order to test this end separate from the other, the support on end TN2L was moved in 12 ft so that it was directly under the previous point of loading. The support on End TN2R remained in the same location. The support changes ensured that the damaged end would not affect the next test. The development length for the strands is approximately 12 ft. With the proposed setup, there would be 12 ft or more of uncracked girder extending out from either side of the point load. The framework to apply the point load was also moved to the other side, so that the girder could be loaded 12 ft from end TN2R, as shown in Figures C.1-16 and C.1-17.

In the same way, the clamping apparatus was moved to the other end, 30 in. from end TN2R. Since there was some strand slippage reported in the first test, a string-pod was attached to a strand in the bottom row that extended beyond the end of the girder. This string-pod was attached to a system that would measure the displacement of the strand in order to determine if it slipped during the testing. A mark was made on all of the extended strands and a caliber was used to measure the distance from the girder end to this marking on each strand. The distance was measured both before and after testing to observe and difference, showing strand slippage.

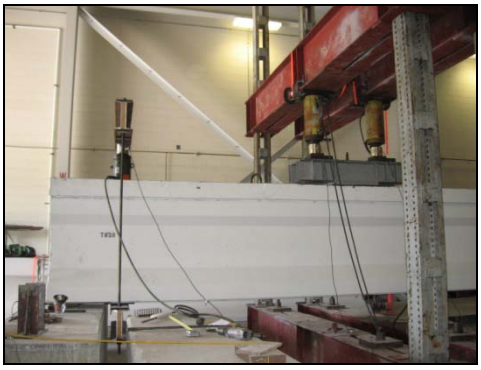


Figure C.1-16. Test setup before loading

Figure C.1-17. Strand slippage gauge

End TN2R was tested on July 18, 2008. Load was slowly applied to the system in intervals of 100 kips. After each additional 100 kips, the loading was paused so that the girder could be investigated and marked for cracks. After loading exceeded 500 kips, the pausing for inspection was stopped for safety concerns. The girder was loaded to 800 kips, which was the capacity of the jacks. However, at this point the specimen was almost to failure, and was taking on load very slowly. At the final load of the test, the point directly below the load experienced a deflection of 1.64 in. The load was then removed, leaving a permanent plastic deformation at the location of loading of 0.28 in. Slippage of the strands was measured from the string-pod device. It showed that the maximum strand slippage reached 0.038 in at a load of 400 kips. This value remained constant at around 0.038 to 0.032 in. until the load reached 750 kips. Cracking patterns that appeared were consistent with a classical flexural failure, as shown in Figure C.1-18. The beam was designed to fail in flexure, and this appeared to be the primary cause of failure. Table C.1-3 gives the moment capacity and corresponding point load based on the actual strength of the girder material and the test.

Table C.1-3. Moment capacity and corresponding point load of the TN2R specimen

	Minimum Specified Strength	Actual Strength	Test Data
Moment Capacity (Mn)	4204 k-ft	4299 k-ft	5675.9 k-ft
Point Load (P)	590 k	604 k	800 k

Figure C.1-18. Crack patterns at failure

Figure C.1-19. shows the load-deflection curve of End TN2R. The girder with composite topping was calculated to resist a point load of up to 604 kips, located 12 ft from the end. The actual maximum load reached before failure when the loading had to be stopped was 800 kips.

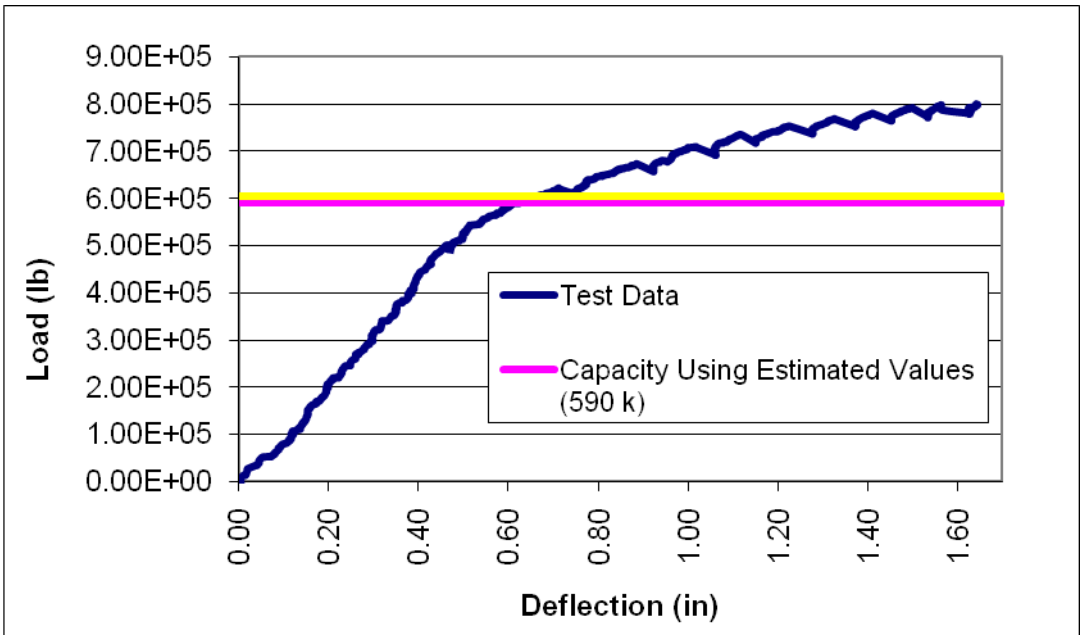


Figure C.1-19. Load deflection curve

The point of loading where the Load Deflection curve first attempted to level out was at 630 kips. This is still 4.3% higher than the expected load. This shows that there was no reduction in strength due to the conditions at the ends. As indicated earlier, there was no visible end zone

cracking due to prestress release. Whatever microcracking existed, which was not possible to measure, did not appear to have any detrimental effect on the flexural capacity of the member. No indicated of any reduction in shear capacity was observed as there was no diagonal compression crushing or stirrup breakage. Figure C.1-20 shows the strand slippage with time. The curve shows that no noticeable slippage occurred until the load reaches the 800-kip level. This shows that the strands were in full bondage with the concrete up to the 800-kip load level.

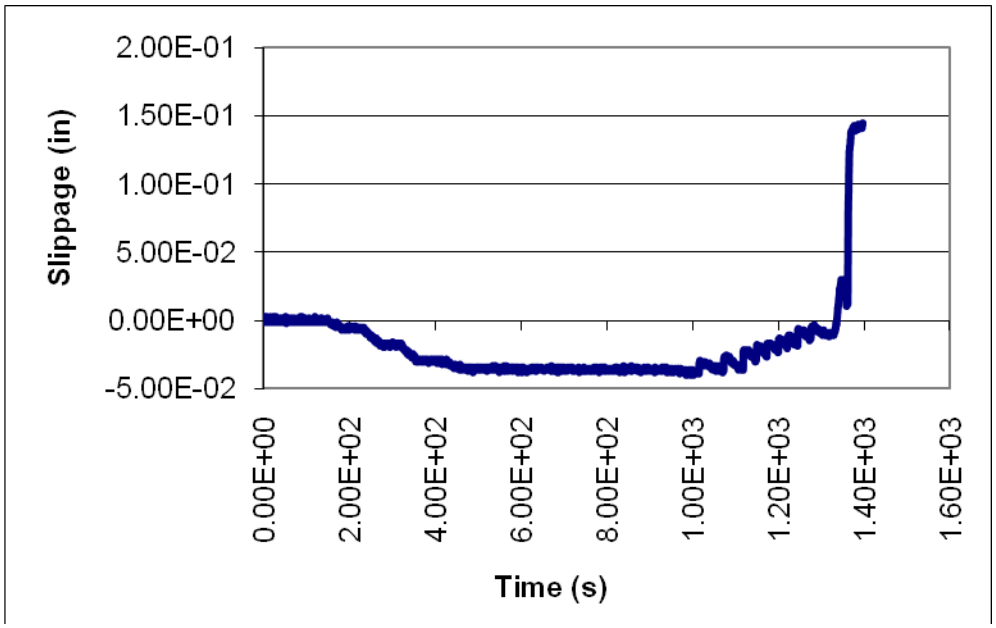


Figure C.1-20. Strand slippage vs. time curve

(b) Girder TN1

Girder TN1 was identical to Girder TN2 in every way except for the end zone reinforcement. It contained the same type and spacing of shear reinforcement, confinement reinforcement, and prestressing strands. As in the previous girder, a 7.5 x 24 in. deck was cast on top of the girder to simulate the decking system that would be placed on the production girders. This deck was formed around the horizontal shear reinforcement, causing the girder/deck system to work as a composite section. This deck was designed to increase the amount of stress in the bottom steel strands at flexural failure. For this deck, no transverse reinforcement was used. The only steel in the deck was the #4 threaded rods running perpendicular to the girder, as shown in Figure C.1-21. The SCC concrete mix that was used was the same mix as was used for Girder TN2. However, Type I/II concrete was used instead of Type III due to availability, so a rapid curing admixture was added to the mix. Table C.1-4 shows the specified and actual concrete strength of the girder and the deck.

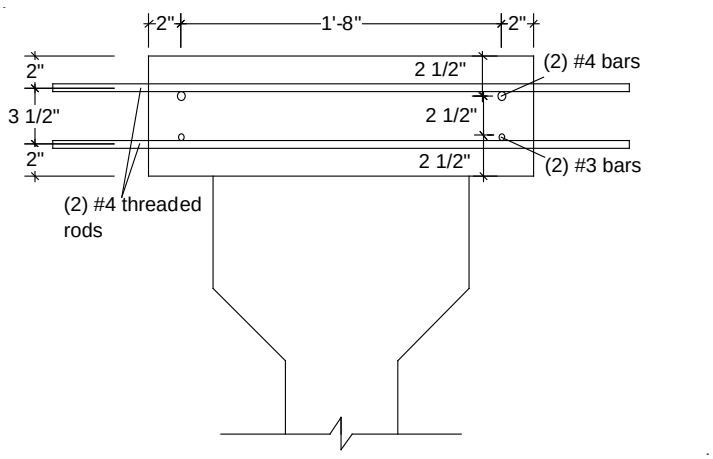


Figure C.1-21. Girder TN1 decking cross section

Table C.1-4. Concrete strength of the girder and deck of TN1

Compressive Strength (psi)	Girder	Deck
Minimum Specified	7,000	9,000
Actual	11,564	9,257

End TN1R Test Results

On Tuesday September 9, 2008, End TN1R was loaded until failure. The girder was supported at each end with 1 ft bearing and subjected to a point load, 12 feet from the girder end, Figure C.1-22. Since Girder TN2 showed signs of strand slippage, a string-pod with a distance gauge was attached to two of the strands to get an exact measurement of the amount of slippage. The two strands that were chosen were expected to be the most likely to slip in that they were both on the bottom row towards the exterior of the girder (second strands in), as shown in Figure C.1-23. A clamping force of 54 kip was applied 30 in. from the end of TN1R. This force was designed to simulate the bridge weight acting on the supports. The same hydraulic jack and frame from Girder TN2 was used.

Figure C.1-22. Test setup before loading

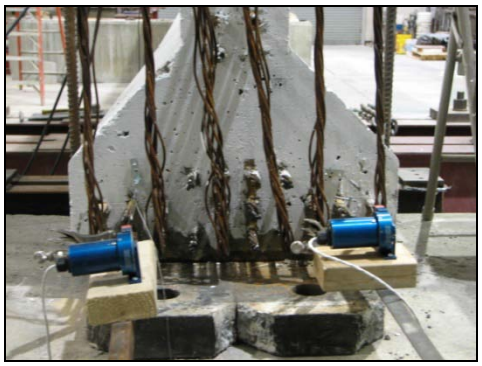


Figure C.1-23. Strand slippage gauges

Load was slowly applied to the system in intervals of 100 kips. After each additional 100 kip, the loading was paused so that the girder could be investigated and marked for cracks. This process continued until the girder was no longer able to carry any more load. This was considered the point at which the girder reached failure. This occurred when the girder was loaded with 527 kips. At this load at the location of loading, the girder had deflected 2.03 inches. When the load was removed, a permanent plastic deformation of 0.35 inches remained.

Cracking patterns that appeared were consistent with a classical flexural failure. The beam was designed to fail in flexure, and this appeared to be the primary cause of failure. Some strand slippage, or bond failure was also apparent in the testing. Some of the strands sticking out of the ends slightly rotated in the direction of the twisted wires as they slipped into the girder. Upon closer inspection, one could also see that the strands had recessed into the face of the girder. Figure C.1-24 and Figure C.1-25 show slippage of the strands as the girders were tested. The strand on the East side of the lab slipped 0.39 inches while the strand on the West side of the lab slipped 0.50 inches. Figure C.1-26 shows the load-deflection curve of End TN1R, and Figure C.1-27 shows the slippage of the East and West strands.

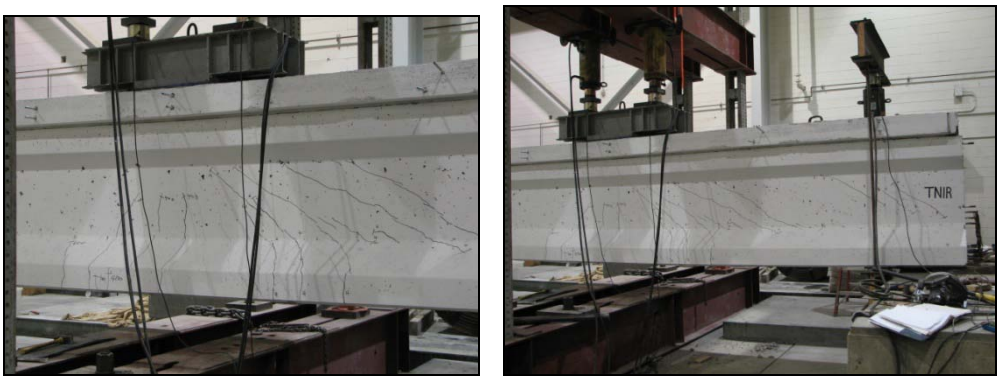


Figure C.1-24. Crack patterns at failure

Figure C.1-25. Evidence of strand slippage

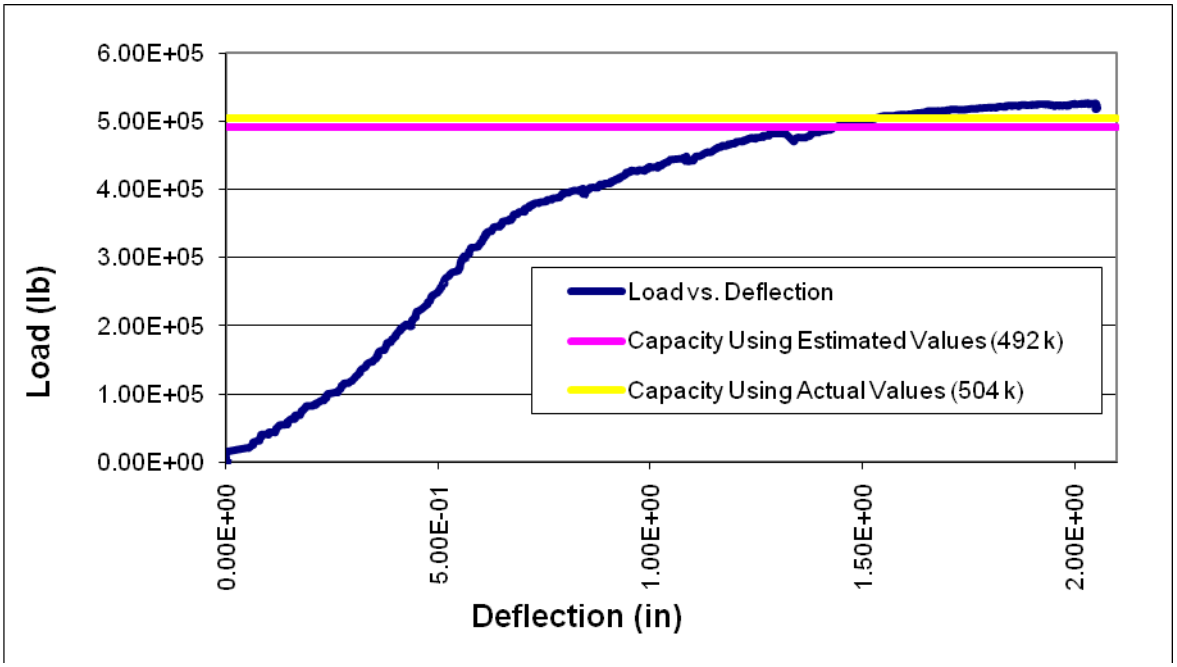


Figure C.1-26. Load deflection curve

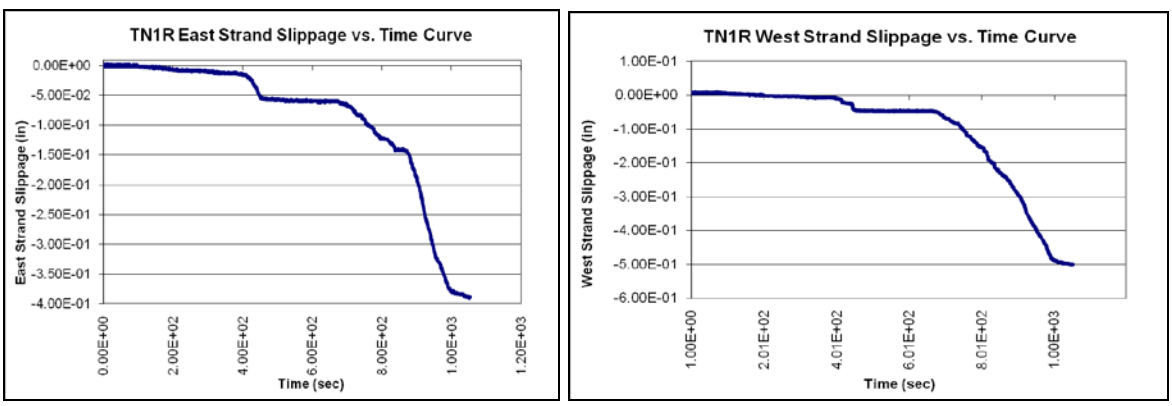


Figure C.1-27. Strand slippage vs. time curve for the East and West strands

The girder, along with composite topping was calculated to resist a point load of up to 504 k, located 12 ft from the end. The actual maximum load reached before failure was 527 k, 4.6% higher than the expected load, as shown in Table C.1-5.

Table C.1-5. Moment capacity and corresponding point load of the TN specimens

	Minimum Specified Strength	Actual Strength	Test Data
Moment Capacity (Mn)	4204 k-ft	4299 k-ft	4494 k-ft
Point Load (P)	492 k	504 k	527 k

This gives evidence that there was no reduction in strength due to the conditions at the ends. As indicated earlier, there was no visible end zone cracking due to prestress release. Whatever microcracking existed, which was not possible to measure, did not appear to have any detrimental effect on the flexural capacity of the member. Shear capacity was designed to be larger than flexural capacity of that end. Again, no indicated of any reduction in shear capacity was observed as there was no diagonal compression crushing or stirrup breakage. As indicated before, this was not a severe case specimen.

End TN1L Test Results

End TN1L is located on the same girder as End TN1R. In order to test this end separate from the other, the support on end TN1R was moved in 12 ft so that it was directly under the previous point of loading. The support on End TN1L remained in the same location. The support changes ensured that the damaged end would not affect the next test. The development length for the strands is approximately 12 ft. With the proposed setup, there would be 12 ft or more of uncracked girder extending out from either side of the point load. The framework to apply the point load was also moved to the other side, so that the girder could be loaded 12 ft from end TN1L, as shown in Figure C.128. In the same way, the clamping apparatus to apply 54 kips was moved to the other end, 30 in. from end TN1L. The same two bottom, exterior strands that were monitored on End TN1R had the distance gauges reapplied, this time to the part extending beyond End TN1L.

Figure C.1-28. Test setup before loading

Figure C.1-29. Strand slippage gauges

End TN1L was tested on September 9, 2008, the same day that End TN1R had been tested. Load was slowly applied to the system in intervals of 100k. After each additional 100 kips, the loading was paused so that the girder could be investigated and marked for cracks. Loading continued until the girder was holding a point load of 638 kips and was no longer taking on any further load. At this failure point, the girder was deflected 1.38 in at the location of loading. When the load was removed, the girder retained a permanent plastic deformation of 0.32 in. Strand slippage was very apparent in this test. Some of the strands sticking out of the ends rotated in the direction of the twisted wires as they slipped into the girder. The concrete around the presressing strands began to break away, as shown in Figure C.1-30. The gauge on the strand on the West side of the lab gave a maximum slippage of 0.327 in., as shown in Figure C.1-31 The gauge on the East side of the lab malfunctioned, and did not give useable results. Cracking patterns that

appeared were consistent with a classical flexural failure, as shown in Figure C.1-32. The beam was designed to fail in flexure, and this appeared to be the primary cause of failure.

Figure C.1-30. Evidence of strand slippage

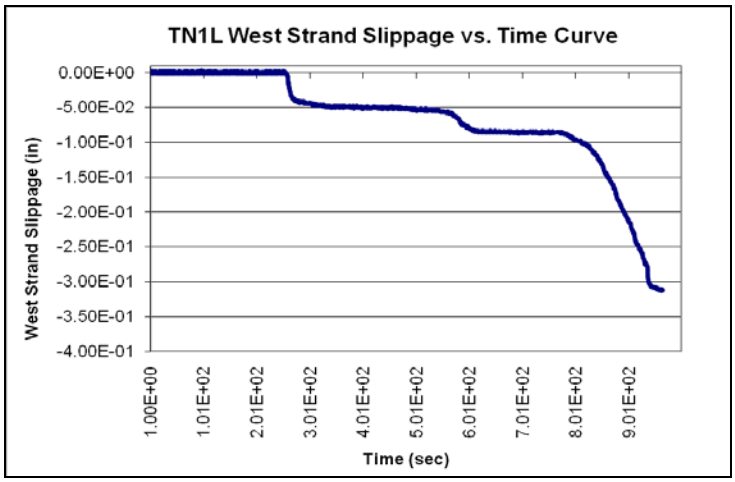


Figure C.1-31. Strand slippage vs. time curve for the West strand

Figure C.1-32. Crack patterns at failure

Figure C.1-33 shows the load-deflection curve of End TN1L. The girder with composite topping was calculated to resist a point load of up to 604 kips, located 12 ft from the end. The actual maximum load reached before failure was 604 kips, 2.4% higher than the expected load, as shown in Table C.1-6. There was no reduction in strength due to the conditions at the ends. As indicated earlier, there was no visible end zone cracking due to prestress release. Whatever microcracking existed, which was not possible to measure, did not appear to have any detrimental effect on the flexural capacity of the member. The development length was not harmed by end zone cracking. Shear capacity was designed to be larger than flexural capacity of that end. Again, no indicated of any reduction in shear capacity was observed as there was no diagonal compression crushing or stirrup breakage.

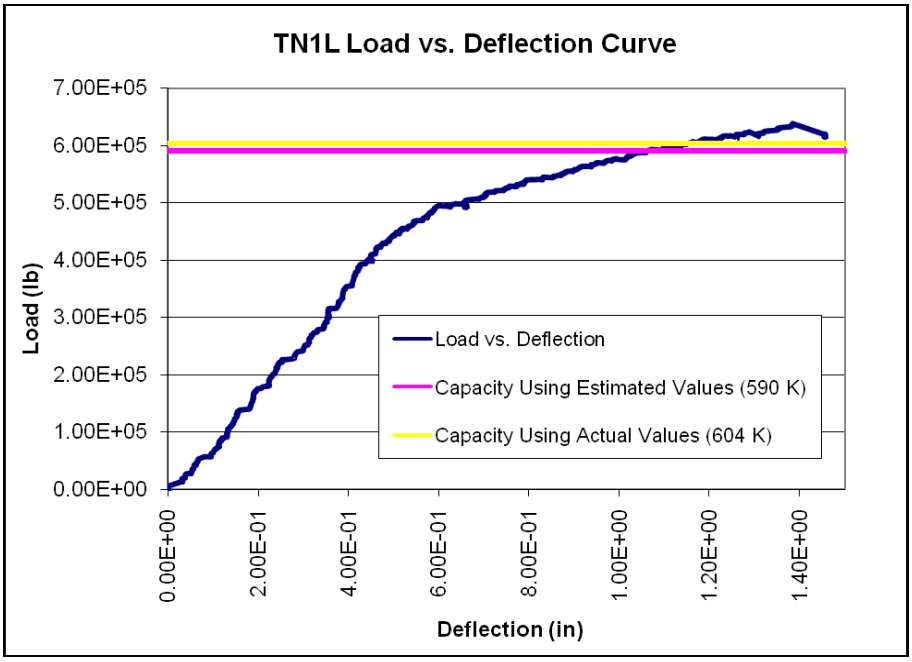


Figure C.1-33. Load deflection curve

Table C.1-6. Moment capacity and corresponding point load of the TN specimens

	Minimum Specified Strength	Actual Strength	Test Data
Moment Capacity (Mn)	4204 k-ft	4299 k-ft	4539 k-ft
Point Load (P)	590 k	604 k	638 k

C.2 Washington State Specimens

Description and Design Calculations

The team has been working with Concrete Technology Corporation (CTC), Tacoma, Washington. They have agreed to produce two 42-ft long I-girders for the project. The girder size is the standard 58 in. deep Washington Super Girder, more recently called Wide Flange Girders.

The test specimens will be the same shape and contain the same amount of prestressing as the production girders, as shown in Figure C.2-1. Both girders contain an identical configuration of prestressing strands and shear reinforcement. Only the vertical reinforcement in the ends differs. The specified concrete strength $f_c^{'}$ = 8,000 psi and release strength $f_{ci}^{'}$ = 6,000 psi. Each specimen contains 38 straight 0.6 in. diameter, 270 ksi low relaxation prestressing strands in the bottom portion of the girder, jacked to 43.9 kips per strand. At the top of the web, each specimen contains 20 additional straight strands, representing the draped strands in the production girders. All prestressing strands in the girder are straight, and experience no debonding. In addition, the top flange contains 4 "temporary" post-tension 0.6 in. diameter strands. They represent the strands used in the production girders to control camber and initial flexural stresses. These strands are used on the specimen as a way of attempting to maximize the prestressing on the short specimens and thus attempting to aggravate the end zone cracking as much as possible. They will be tensioned in the yard to 43.9 kips per strand. A series of #3 bars (302) bars are used for confinement of the bottom strands. In order to fail these specimens in shear, shear reinforcement consisting of pairs of vertical #4 bars (401), spaced at 12 in., were designed.

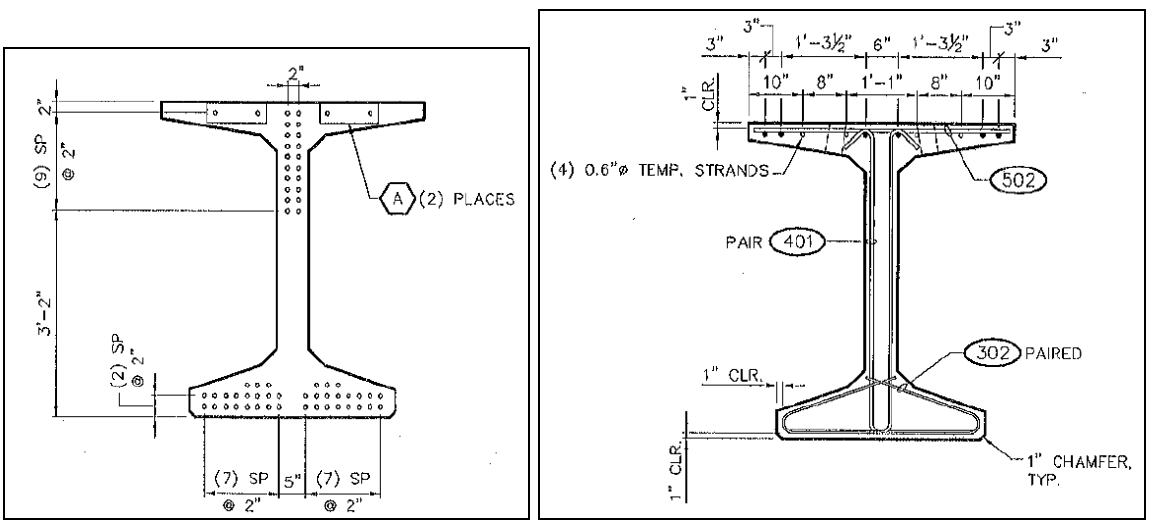


Figure C.2-1 Washington Specimens WA1 and WA2.

Estimating an actual strength $f_c^{'}$ = 10,000 psi, the nominal flexural capacity, M_n = 8,980 ft-k. Using the same support configuration as for TN1 and TN2, the corresponding load P = 1,085 kips. To force shear failure, the girders were designed in shear for a force of about 500 kips. The corresponding reinforcement is 2-#4 @12 in.

<u>End Zone Reinforcement for End WA1L (Figure C.2-2)</u>

End WA1L was designed according to the recommendation of the research team. The 4% area of end reinforcement steel was calculated as 5.09 in^2. At least half of this amount, 2.55 in^2, must be placed within h/8, or 7.25 in. from the end of the girder.

In order to get the largest amount of steel closest to the edge of the girder, a single #8 bar is placed at 1½ inches from the end. It is bent in a C-shape with the horizontal legs oriented in the

direction of the girder axis. This is a detail that has been used by Central Premix Company in Washington with apparent success. A different concept used on the Platte River East Bridge in Nebraska involves use of a welded wire reinforcement (WWR) sheet in the center of the section for a length of three feet. However, due to time constraints in ordering the WWR, the team decided to experiment with the #8 bar. In addition, 2 pairs of #6 vertical bars are placed at the end on the two web faces. The first pair is 1-1/2 inches away from the end of the girder (in the same plane as the #8 bar), and the second pair spaced 3 inches away from the first pair. This way the three bars at the end stay above the minimum 1 in. clear cover away from the concrete surface. The total area of reinforcement in the first zone is thus 0.79+ 4*.44 = 2.55 in^2, which is equal the 2% reinforcement, in a distance = 1.5 +3 = 4.5 in., which is less than the h/8 requirement. If cover is an issue, 3 pairs of #5 at 3" would still have worked. Beyond these #6 bars, seven pairs of #4 bars at 3 in. spacing are placed. The total splitting reinforcement = 2.55 + 7*2*.2 = 5.35 in^2, which exceeds 5.09 in^2, placed in 1.5+7*3 = 22.5 in., within the h/2 = 29 in. maximum distance for zones 1 and 2. Beyond the splitting steel, shear reinforcement in pairs of #4 bars, spaced at 12 in. is placed.

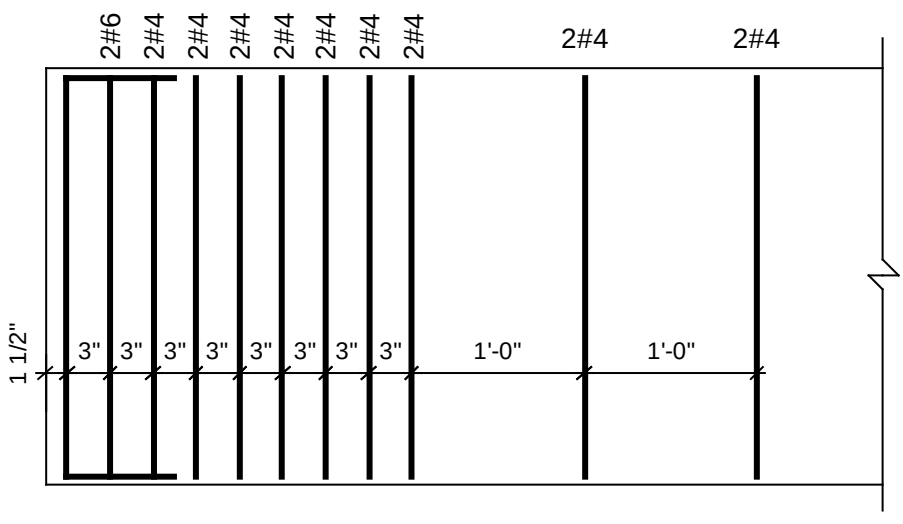


Figure C.2-2 WA1L end reinforcement details using proposed details

End Zone Reinforcement for End WA1R (Figure C.2-3)

This end was designed in accordance with the LRFD Specifications. The 4% area of end reinforcement steel was found to be 5.09 in^2. This needs to be placed within h/4 (14.5 inches) from the end of the girder. This worked out to be 6 pairs of #6 bars, spaced at 2-1/2 in. center-to-center, and bent in the typical fashion for vertical reinforcement of the production girders. The first pair of #6's was placed 1-1/2 in. from the end of the girder. If #6 bars are objectionable due to side cover requirements, LRFD would pose a dilemma. Using a center mesh or a center #8 bar might help. The team elected to use the #6 bars for the testing in order to have a direct comparison with WA1L.

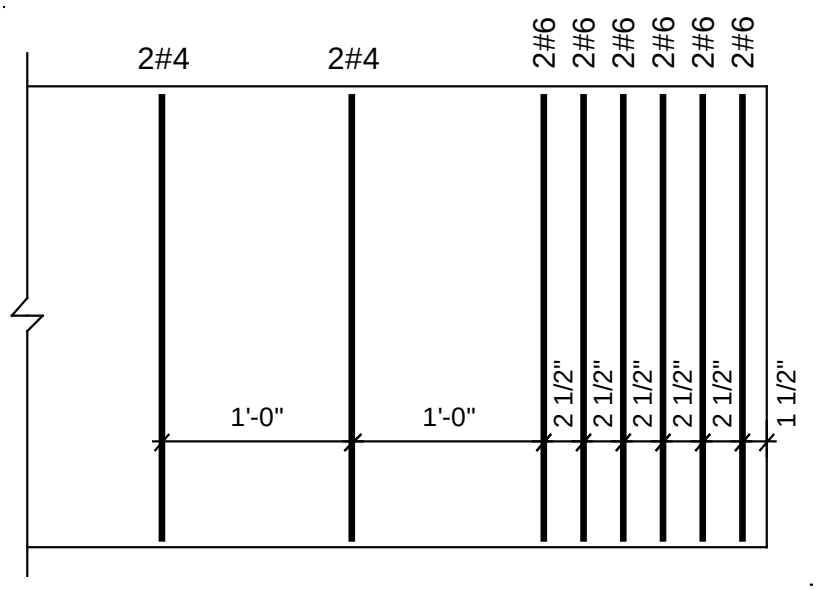


Figure C.2-3 End zone reinforcement of WA1R using LRFD provisions

End Zone Reinforcement for End WA2L and End WA2R (Figure C.2-4)

Both of these two ends contained no additional end zone reinforcement. The shear reinforcement of pairs of #4 bars, spaced every 12 in. was simply extended to the ends. These two ends are expected to experience more end cracking than in WA1. End WA2L will be sent to the University of Nebraska Structures Lab without any repair, while End WA2R will have end zone cracks repaired with epoxy injection before the specimen is shipped to Nebraska. This epoxy injection was performed by Pete Barlow of Contech, a company that specialized in structural epoxy repairs.

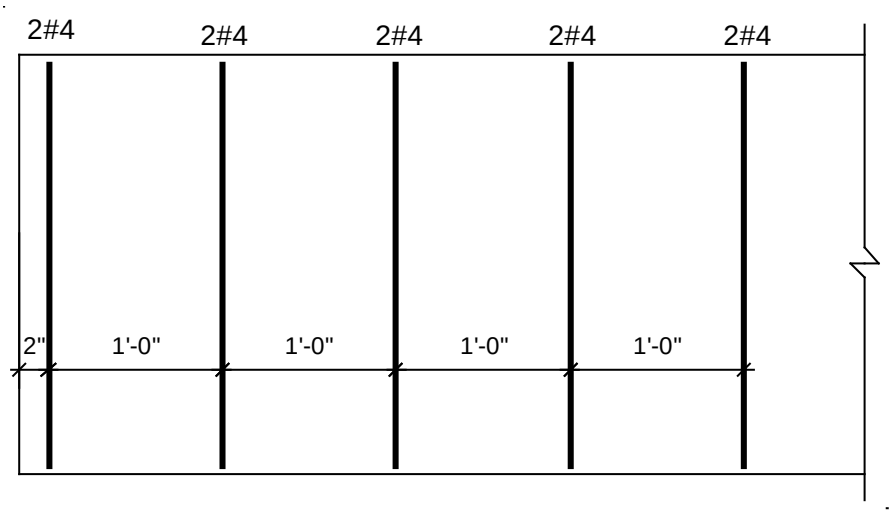


Figure C.2-4 End zone reinforcement of WA2L and WA2R. No additional steel to the shear reinforcement was placed.

Flexural Capacity:

The specified $f_c^{'}$ = 8,000 psi (as given by the precast producer). Since the produced concrete always has a higher strength than required, a value of $f_c^{'}$ = 10,000 psi was used in the flexural calculations. A strain compatibility program was used to determine the flexural capacity as follows:

Height of the equivalent compression block = 13.3 in.

Neutral axis depth (measured from the top fiber) = 20.45 in.

Tensile stress at the strand layer close to the bottom fiber = 248 ksi

Nominal flexural capacity, M_n = 8,980 ft-k

When testing the girders, they will be supported at 6 in. from either end, leaving an unsupported length of 41 ft. A point load will then be applied at 11.5 ft from one support (29.5 ft from the other support) until failure, as shown in Figure C.2-5.

Therefore, the force point load, P, required for flexural failure is:

$$M_n = \frac{P.a.b}{L}, \quad 8980 = \frac{P(11.5)(29.5)}{(41)}, \quad P = 1{,}085 \text{ kips}$$

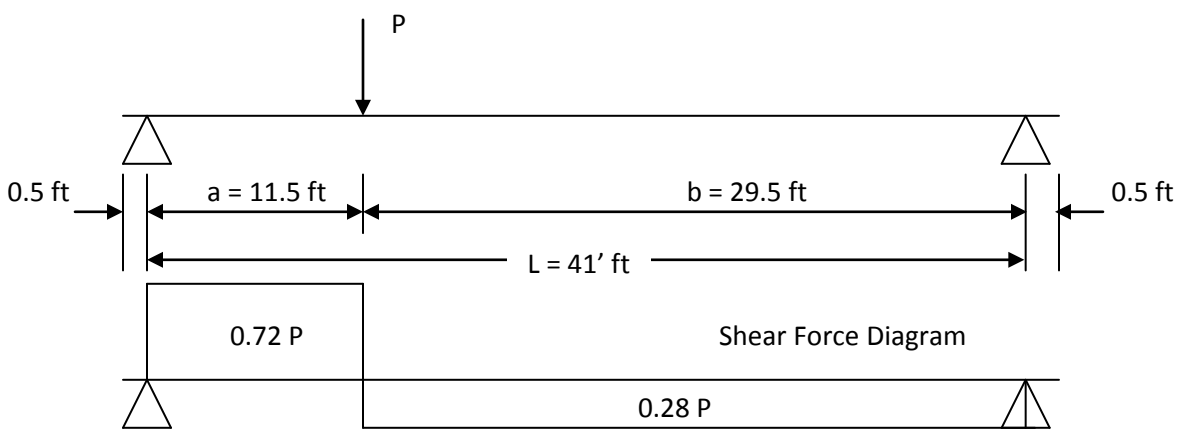


Figure C.2-5 Test Setup

Shear Failure:

For the Washington girder, the team chose to design it to fail in shear. Therefore, the point load required to fail the bean in shear, was set to 50% of the point load needed to fail the beam in flexure, i.e. about 500 kips.

Effective shear depth, d_v, is the greater value of the following two equations:

$$d_v = 0.9*d = 0.9(54.53) = 49 \text{ in.}$$

$$= 0.72*h = 0.72*(58) = 41.8 \text{ in.}$$

Therefore, d_v will be taken as 42 in.

With a 500 kip point load applied at 11.5 ft from the left end support, the reaction at the left support was 360 kips and the reaction at the right end support was 140 kips. Therefore, the space between the left support and the point load will experience the highest amount of shear force of 360 kips. Assuming that θ is 23°, and that β is 3, the A_v/s ratio was calculated for shear reinforcement.

$$V_u = 0.0316*\beta*\sqrt{(f'_c)}*b_v*d_v + (A_v/s)*f_y*d_v*\cot(\theta)$$

$$360 = 0.0316*(3)*\sqrt{(8)}*(6.125)*(49) + (A_v/s)*(60)*(49)*\cot(23^\circ)$$

$$A_v/s = 0.04 \text{ in}^2/\text{in or } 0.48 \text{ in}^2/\text{ft}$$

The spacing s is set equal to the minimum spacing for the shear reinforcement = 12 in. according to Article 5.8.2.7 of the AASHTO LRFD Specifications.

Therefore, the vertical shear reinforcement could not be placed farther apart than 12 in.

A_v = 0.48 in^2 of steel that must be placed every 12 in.

= 2 #4 bars (with A_v = 0.40 in^2)

2 #4 bars @ 12 in. were used. It should be noted that this amount of reinforcement delivers A_v = 0.40 in^2, which is less than the calculated 0.48 in^2. This was done because the nominal shear capacity might be overestimated as it depends significantly on the assumed values of θ and β.

Fabrication and Inspection

The production girders, according which the NCHRP 18-14 specimens were developed, had already been manufactured and were in storage in the yard at the time the specimens were made. They contained the same amount of prestressing strands in the same configuration, but were 147'-2" long. The prestressing strands in the full girders were harped at two tie-down points. They experienced end zone cracking. Most cracks extended approximately 10 to 20 in. into the girder and had widths of 0.002 in. The largest cracks viewed extended 45 in. into the girder and had widths of 0.004 in. The cracks usually began at the same point where the upper strands entered the side of the girder. They then propagated downward, at a slope greater than that of the harped strands. The cracks were not required to be epoxy repaired. The girders were finished with a sack rub procedure that fills in any imperfections on the girder with a concrete mixture, and then rubbed smooth.

CTC personnel have expressed concern about attempting to satisfy LRFD requirements for girders containing large amounts of prestress. In particular, they are not allowed to use bars larger than #5 by the WADOT rules, while they are not allowed to place the steel beyond h/4 by LRFD rules. Therefore the smaller bars become crowded in the end, and the concrete is unable to flow smoothly to all sections. Pockets of air form, creating voids in the concrete near the girder ends on the vertical reinforcement when the forms are removed.

CTC used accelerated curing with heated forms. The forms for the two test girders had heated wires running along them, which were heated up to 160°F. The concrete cylinders were also cured in a special form that heated at the same rate as the girders. There was a problem with this heating system on the night that the test girders were curing. Therefore, the forms did not reach the intended temperature and the girders were not at the expected strength in the morning. The minimum concrete strength for release was f'_c = 5,800 psi, and it took 8 hours longer than expected to reach this strength.

Once the concrete strength surpassed 5,800 psi, the forms were removed and the detensioning process began. While removing the forms, one form caught the upper flange on a corner of End WA2L (WA2 UME), causing a crack at the base of the flange. This crack was present before the detensioning process began, and grew in width as the strands were released. At CTC, the prestressed strands were completely jacked down, and were only cut once the tension was completely released. The detensioning process occurred in stages. First the strands in the top of the girder were partially jacked down, then the strands in the bottom half of the girder had half of the tension released. The process was repeated, fully detensioning the top strands, then fully detensioning the bottom strands. As soon as the forms were removed, cracks were apparent on all ends.

The intent of the research team was to place as much strand as possible to force significant cracking. It is well known from theory and observation that top strands and harped strands contribute to web end cracking. On April 10, 2008, CTC post-tensioned the top four 0.6 in. diameter strands to $0.75f_{pu}$, the same as performed on the production girders. There was no measurable crack growth or spreading after this procedure.

End WA2L was epoxy injected on the morning of Wednesday, April 16th. This was the marked end of the girder without any additional end zone reinforcement. The end experienced cracking to a lesser degree than the unmarked end of WA2R. The epoxy injection was done by Pete Barlow of Contech without any noted complications.

The girders are to be shipped to Coreslab Structures, Inc. in Omaha, NE for storage until space is available for them at the University of Nebraska Laboratory in Omaha, NE. Concrete cylinders test in compression produced strength values of about 9,500 psi at 28 days, as shown in Figure C.2-6.

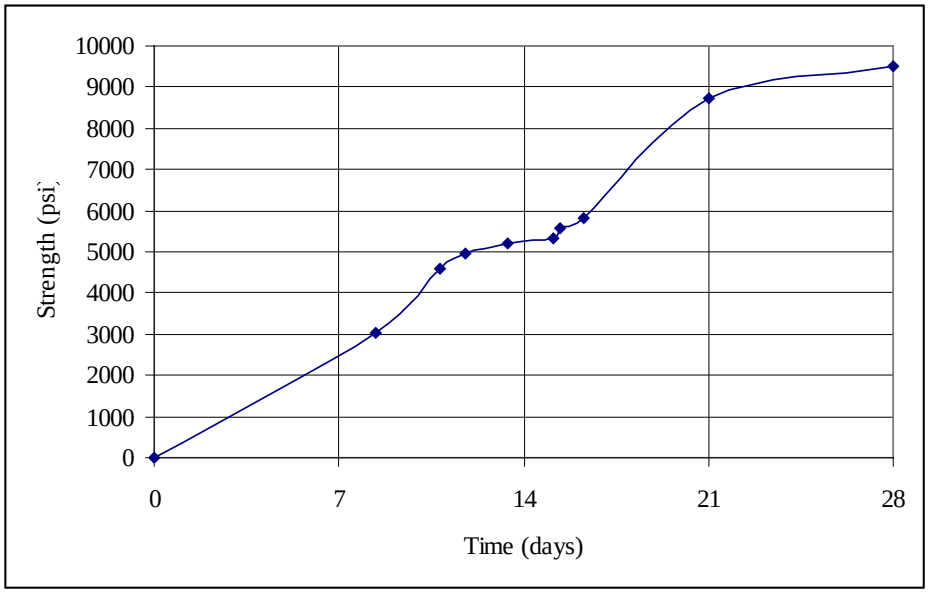


Figure C.2-6. Strength versus Time for the Concrete Girder

Observed End Zone Crack Patterns

All ends experienced some end zone cracking. The cracking in Girder WA2, without extra end zone reinforcement, was much more prevalent than the cracking in Girder WA1, with extra end zone reinforcement. Girder WA2 experienced cracking in the range of 0.004 inches to 0.014 inches, with the longest crack extending 83 inches into the girder. A more comprehensive summary of the cracks is shown in the following figures.

As the prestressing was slowly released, the cracks were noted to grow in both width and length, and new cracks appeared. As the tension in the strands was relieved, the girder closest to the detensioning side (WA2) began moving towards the dead end of the bed. As this occurred, the bottom flange on the side in which the girder was traveling began to rise up slightly. This may be a sign of the friction acting on the bottom of the girder against the bottom of the prestressing bed. This is why it is important to have an end bearing plate to protect the ends from further cracking at the time of detensioning.

On the girders with special end zone reinforcement, cracking still occurred, but much of it was only visible beyond the end reinforcement zone. Girder WA1 on the side designed with LRFD experienced cracks in the range of 0.001 to 0.005 inches, and extended a maximum of 38 inches into the girder. Girder WA1 on the side designed with the details proposed by the project, experienced cracks in the range of 0.001 to 0.003 inches, and extended a maximum of 40 inches into the girder. On both ends of Girder WA1, the cracking did not start until a distance of a few inches into the girder. This may indicate a concentration of end reinforcement is effective in controlling the end cracks, but the reinforcement should be gradually diminished as it is distributed into the beam. The 38-40 in. cracking length is about ¾ of the total depth of 58 in. Figures C.2-7 to C.2-10 shows the end zone cracking of the WA State specimens just after release of the strands.

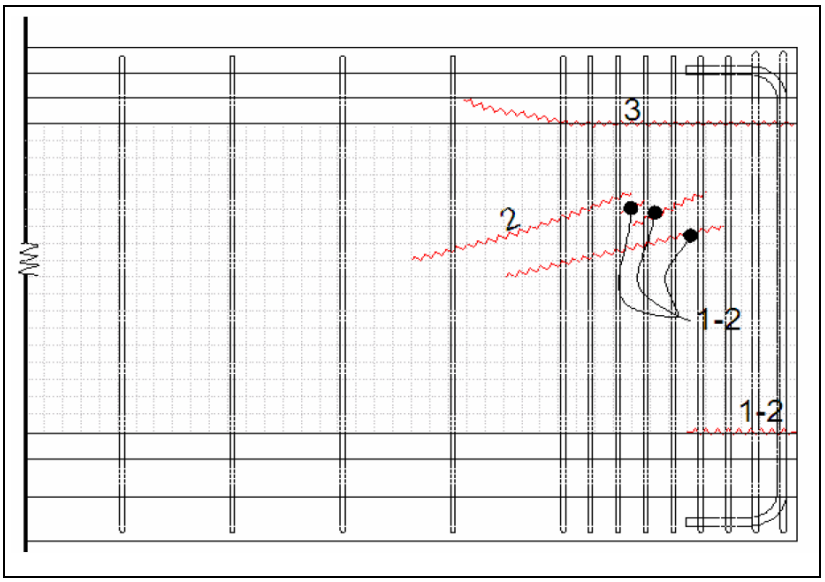


Figure C.2-7. End Zone Cracking of WA1L (WA1 M.E.) (Note: the cracks are shown highlighted in red. The value next to each crack is the crack width in thousands of an inch)

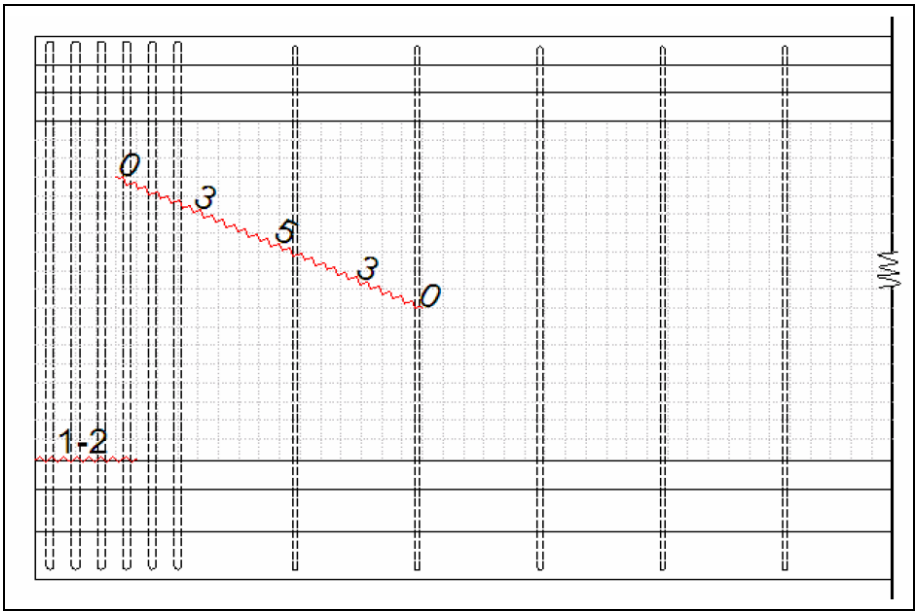


Figure C.2-8. End Zone Cracking of WA1R (WA1 U.M.E.) - LRFD Design (Note: the cracks are shown highlighted in red. The value next to each crack is the crack width in thousands of an inch)

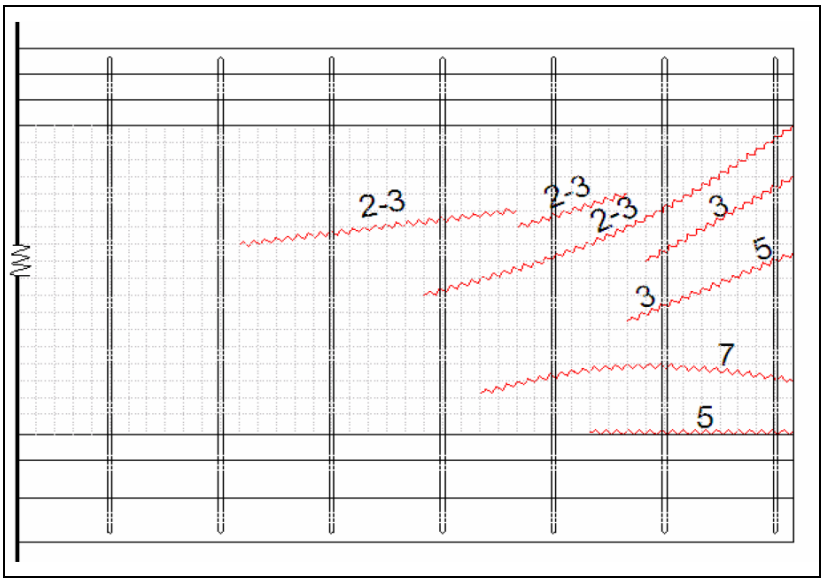


Figure C.2-9. End Zone Cracking of WA2L (WA2 M.E.) - no end reinforcement (Note: the cracks are shown highlighted in red. The value next to each crack is the crack width in thousands of an inch)

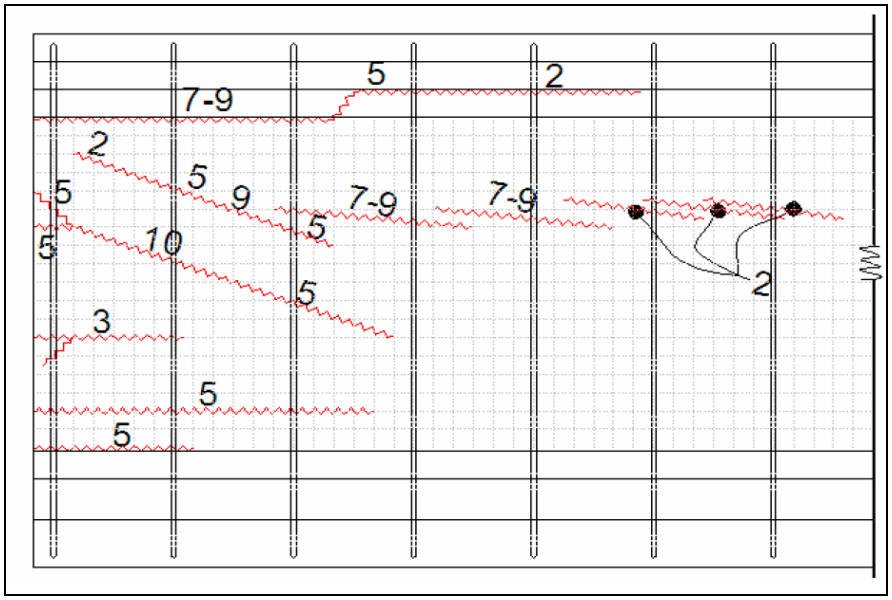


Figure C.2-10. End Zone Cracking of WA2R (WA2 U.M.E.) - no end reinforcement (Note: the cracks are shown highlighted in red. The value next to each crack is the crack width in thousands of an inch)

Structural Testing and Test Results

(a) Girder WA2: End WA2L

On Tuesday, September 23, 2008 the first end of the Washington girders was tested. It was end WA2L, which contained no additional end zone reinforcement besides the typical shear reinforcement of #4 bars placed every 12 in. This vertical reinforcement extended to the very ends of the girder, and were designed to resist shear cracking. No deck nor any other additions were made to the girder before testing.

The girder was supported on each end with a 1 ft. bearing. A point load was applied to the girder 12 ft. from the end WA2L.

A clamping force mechanism was placed 30 inches in from the end of the girder in order to simulate the bridge weight acting on the supports. The clamping force was provided by using a hydraulic jack attached to a self-equilibrium frame built around the specimen as shown in Figure C.2-11 and C.2-12.

This 80-kip applied clamping force was calculated as the balance between the reaction generated by the 147 ft – 2 in. girder span that the original production girders were being manufactured for (taking into consideration weight of girder, deck and barriers) and the reaction generated by the 42 ft long specimen (taking into consideration weight of the girder and deck). This clamping mechanism was placed only at the end being tested, which was WA2L.

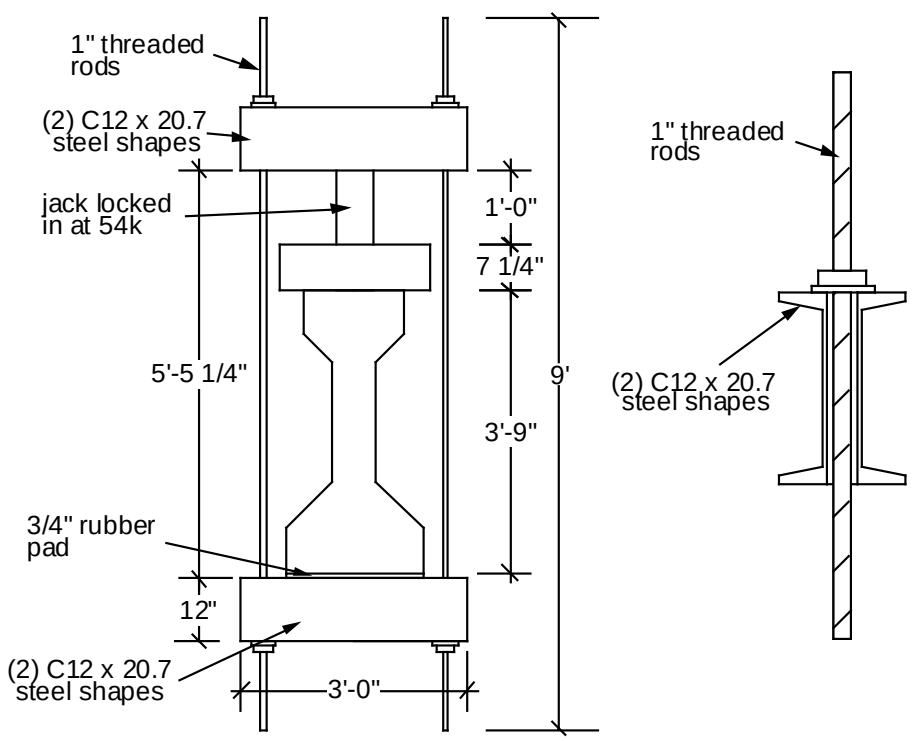


Figure C.2-11. End clamping detail

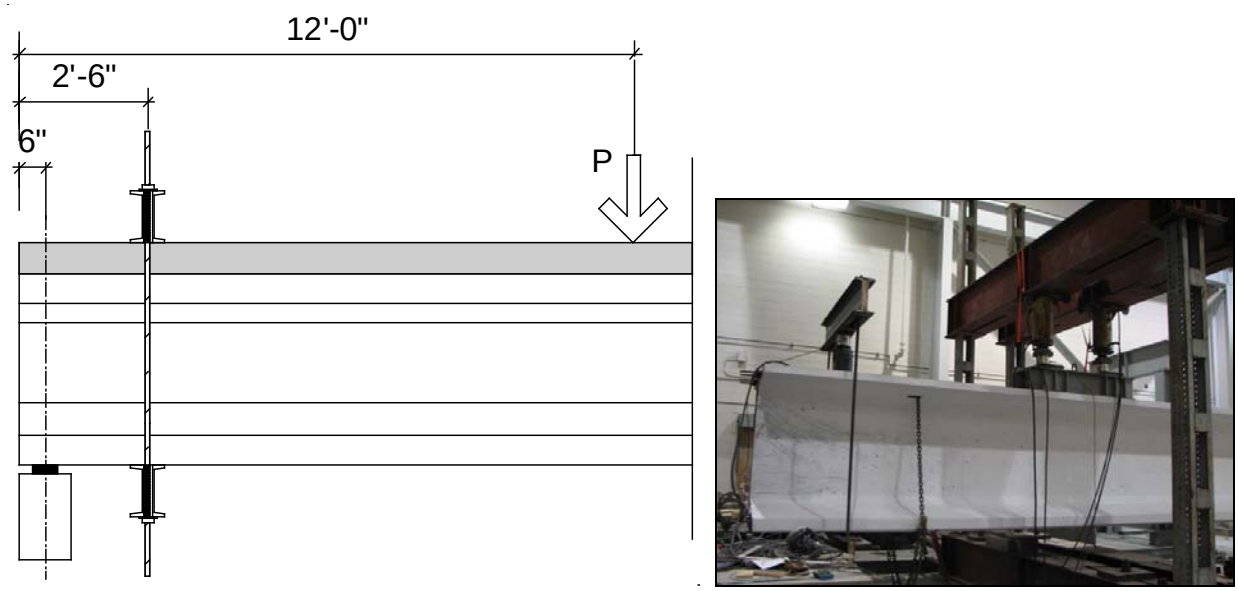


Figure C.2-12. Locations of point load, clamping mechanism

The end zone crack on End WA2L of the Washington girder were repaired with epoxy injection, shown in Figures C.2-13 and C.2-14. The process used was the proceure typically used by Washington and outlined in the PCI Concrete Repair Manual. Injection ports were affixed along the lengths of the cracks that were repaired, and the exterior of the cracks were closed using a sealant. Epoxy was then pumped into the injection ports, moving from the bottom of the girder to the top until all of the cracks were completely sealed.

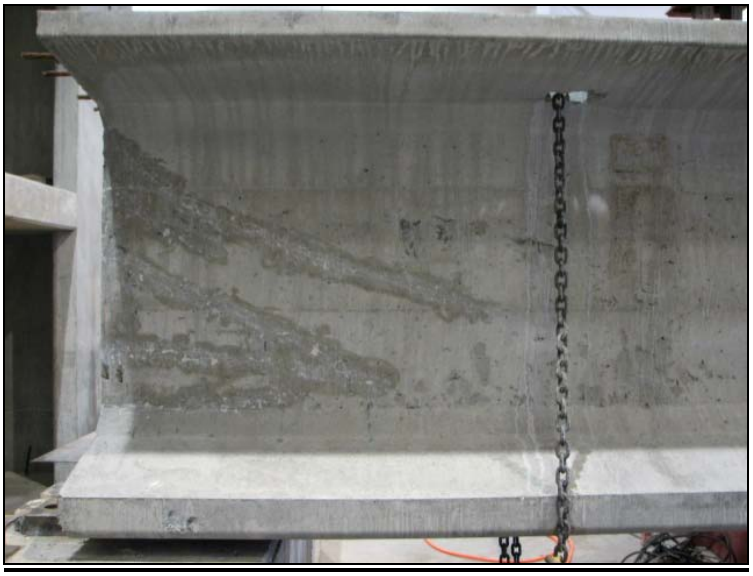


Figure C.2-13. End WA2L showing repaired cracks

Figure C.2-14. Cracks showing epoxy injection

Since the previous Tennesse showed signs of slipping prestressing strands, the team set up a method of recording the strand slippage for the Washiongton girders. A string-pod deflection gauge was attached to each of two chosen prestressing strands. The East gauge was affixed to a strand on the outter edge of the flange (left side in Figure C.2-15), while the West gauge was affixed to a strand towards the inner part of the girder (right side in Figure C.2-15).

Figure C.2-16 shows the girders after they had been painted white. The girders were painted white so that it would be easier to detect very small cracks formed during testing. The end zone cracks were also drawn in with marker so that they could be easily seen from a distance.

Figure C.2-15. Gauges used to determine strand slippage

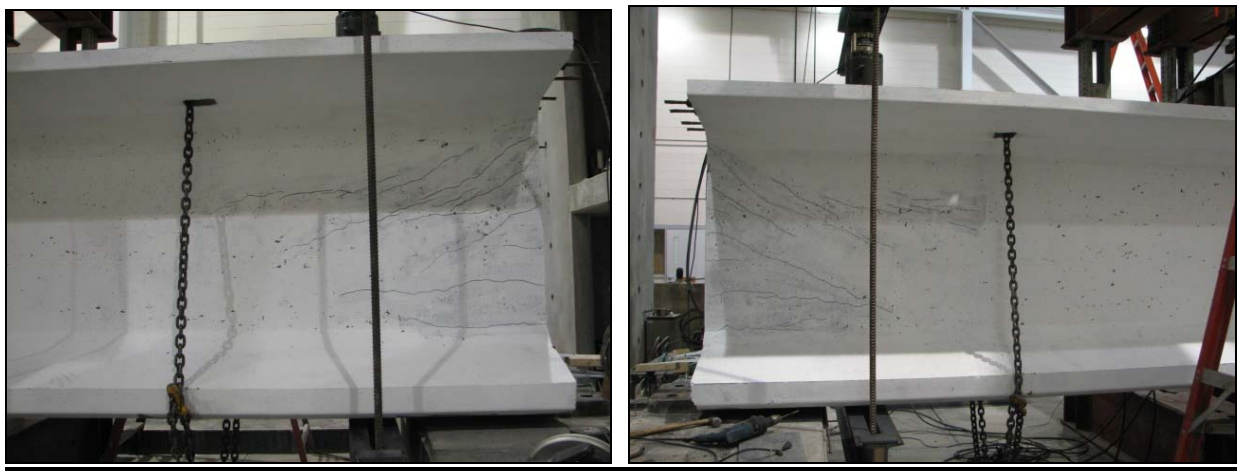


Figure C.2-16. End zone cracks before testing

Table C.2-1: gives the moment capacity and corresponding point load of the WA Specimens using the minimum specified and actual measured strength of the girder and deck.

Table C.2-1. Moment capacity and corresponding point load of the WA2L specimen

	Minimum Specified Strength	Actual Strength	Test Data
Shear Capacity (Vn)	311.3 k	319.8 k	434.2 k
Point Load (P)	432.7 k	444.5 k	603.4 k

Load was slowly applied to the system in intervals of 100k. After each additional 100k, the loading was paused so that the girder could be investigated and marked for cracks. This continued until it began to approach the design load and it became not longer safe to approach the beam. At this point, loading continued until failure, as shown in Figure C.2-17. The highest load reached was 603.4 k. At this point there was a deflection of 0.977 in. After failure, the beam was unloaded, leaving a permanent plastic deformation of 0.224 in. at the location of loading.

Figure C.2-17. End WA2L after shear failure

Figure C.2-18 shows the load-deflection curve of the test. The girder was calculated to resist a point load of up to 444 k, located 12 ft from the end. The actual maximum load reached before failure was 603 k, 35.7% higher than the expected load. There was no reduction in strength due to the conditions at the ends, in fact, the actual maximum load was much higher than that calculated. No indication of any reduction in shear capacity was observed as there was no diagonal compression crushing or stirrup breakage. The epoxy injection did not have any detrimental effects on the capacity of the beam, however further investigation must be done to determine if the repair procedure produced any positive effects on the girder strength.

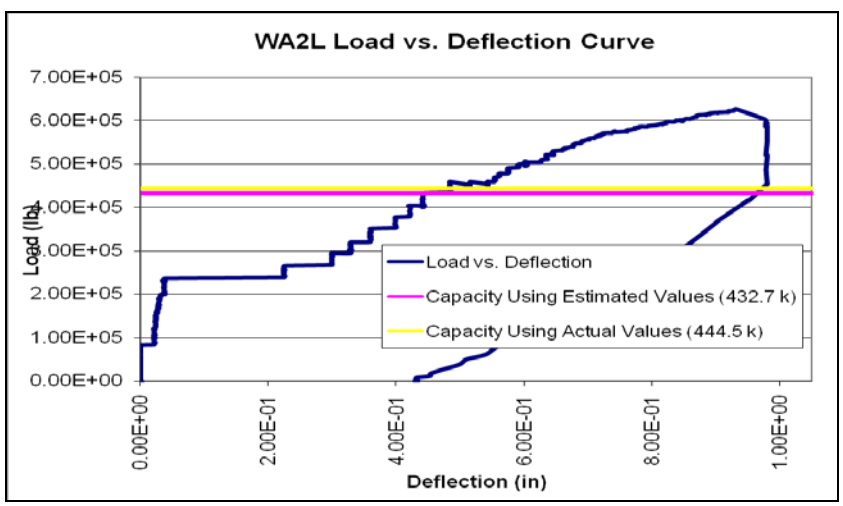


Figure C.2-18. Load Deflection Curve for End WA2L

There was evidence of strand slippage in the prestressing strands of the girder. Simply by looking at the end of the girder one could tell that the strands had retracted in, visible in Figure C.2-19. Originally, the prestressing strands had been cut flush with the end of the girder. The lab

in Nebraska accentuated this flush surface by grinding down the jagged edges of the strands so that they were flat and even with the girder edge. Manual measurement of the slippage with a ruler showed that the strands slipped somewhere between 1/8 in. and 1/4 in. (0.125-0.25 in.) According to the slippage gauges, the East strand on the outter edge of the flange had a slippage of 0.340 in. at failure.(Figure C.2-20) The West strand located towards the centerline of the girder had a slippage of 0.207 in. at failure.(Figure C.2-21) This strand slippage would have affected the capacity of the girder. The failure load would have been higher if no strand slippage had occurred. Therefore, it can be deduced that in an ideal situation with improved end anchorage conditions, the girder would have performed even better than the test showed with a higher load capacity.

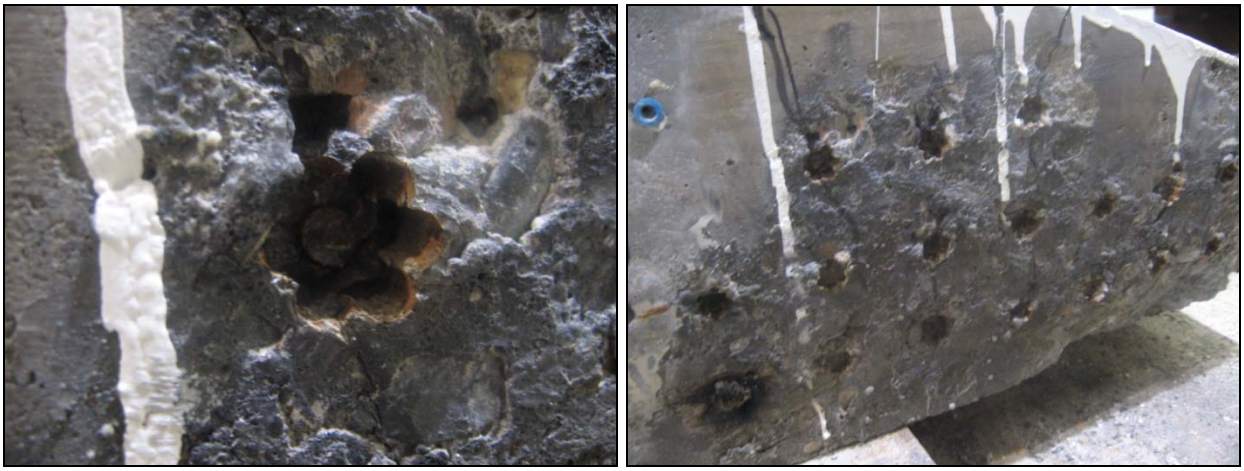


Figure C.2-19. Evidence of Strand Slippage

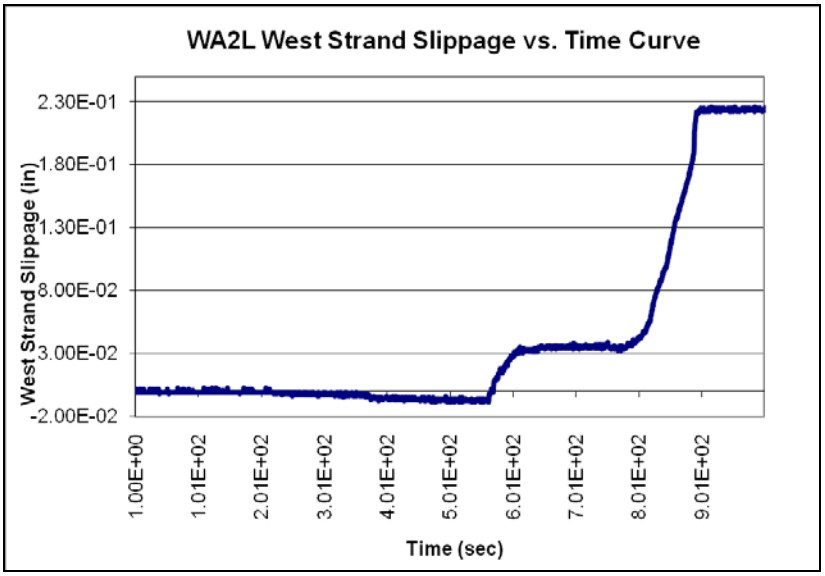


Figure C.2-20. Strand Slippage on West Strand (interior strand)

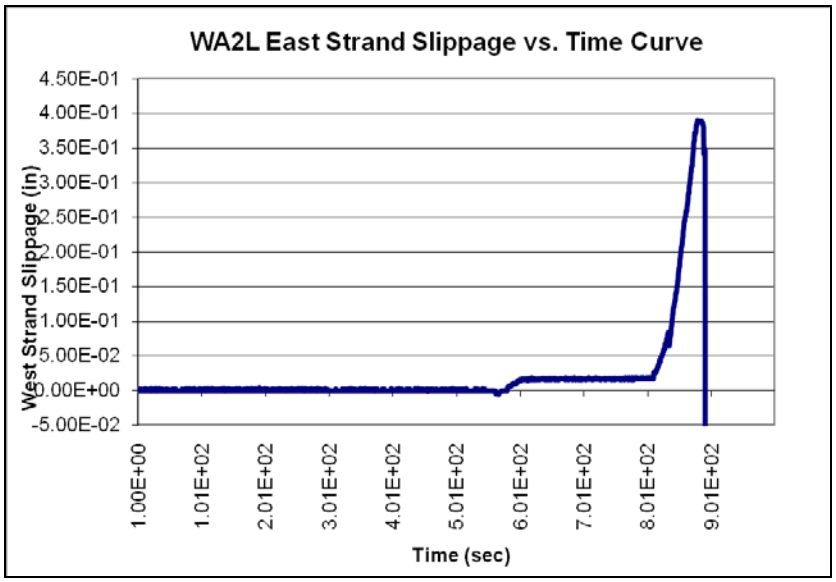


Figure C.2-21. Strand Slippage on East Strand (exterior strand)

(b) Girder WA2: End WA2R

End WA2R was tested on Tuesday, September 23, 2008 as well. It also contained no additional end reinforcement besides the typical shear reinforcement of pairs of #4 bars spaced every 12 in. However, in this case, the horizontal and diagonal end zone cracks were not repaired in any way. They remained in the same condition as they were at the precasting plant. It was located on the same girder, but opposite end of WA2L shown above. In order to avoid the effects of having one end already reached failure, the support on end WA2L was moved in 12 ft. to keep the damaged portion of the girder out of the new span. This created a 29.5 ft. span from center to center of the 1 ft. long bearing supports, with a 11.5 ft. of end WA2L cantilevering past the support. This support change ensured that the damage would not affect the second test. However, since the support length was decreased, the capacity of the girder increased.

The framework and clamping apparatus was moved to End WA2R so that a point load could be applied 12 ft. from the end. The deflection gauges and slippage gauges were applied to this second end as well, in the same relative locations. The slippage gauges are shown in Figure C.2-22.

Figure C.2-22. Gauges used to determine strand slippage

Figure C.2-23 and C.2-24 shows the girder after it had been painted white. The girders were painted white so that it would be easier to detect very small cracks formed during testing. The end zone cracks were also drawn in with marker so that they could be easily seen from a distance.

Figure C.2-23. End zone cracks before testing

Figure C.2-24. End zone cracks before testing

Table C.2-2 gives the moment capacity and corresponding point load of the WA Specimens using the minimum specified and actual measured strength of the girder and deck.

Table C.2-2. Moment capacity and corresponding point load of the WA2R specimen

	Minimum Specified Strength	Actual Strength	Test Data
Shear Capacity (Vn)	311.4 k	319.8 k	457.5 k-ft
Point Load (P)	510.3 k	524.1 k	749.8 k

Load was slowly applied to the system in intervals of 100k. After each additional 100k, the loading was paused so that the girder could be investigated and marked for cracks. This continued until it began to approach the design load and it became not longer safe to approach the beam. After this time, loading continued until failure. The highest load reached was 749.8 k. This load produced a deflection of 0.783 in. After failure, the beam was unloaded, leaving a permanent plastic deformation of 0.426 in. at the location of loading. Figures D.2.1-25 and C.2-26 show the typical shear crack patterns after failure.

Figure C.2-25. End WA2R after shear failure

Figure C.2-26. End WA2R after shear failure

Figure C.2-27 shows the load-deflection curve of the test. The girder was calculated to resist a point load of up to 524 k, located 12 ft from the end. The actual maximum load reached before failure was 750 k, 43.17% higher than the expected load. The diagonal cracking pattern observed appears consistent with what would be expected for shear failure. As shown in the figures, this cracking pattern is perpendicular to the cracks originated from end zone cracking. This opposing force from loading would put pressure on the end zone cracks to come together and close.

There was no reduction in strength due to the conditions at the ends, in fact, the actual maximum load was much higher than that calculated. No indication of any reduction in shear capacity was observed as there was no diagonal compression crushing or stirrup breakage. The fact that this end was not repaired with epoxy injection or by any other means does not appear to diminish the structural capacity of the girder. The beam still allowed a much higher load than expected, in spite of the large amount of end zone cracking. It does not appear that end WA2L, which was epoxy injection repaired, performed any better than end WA2R, which received no kind of repair at all.

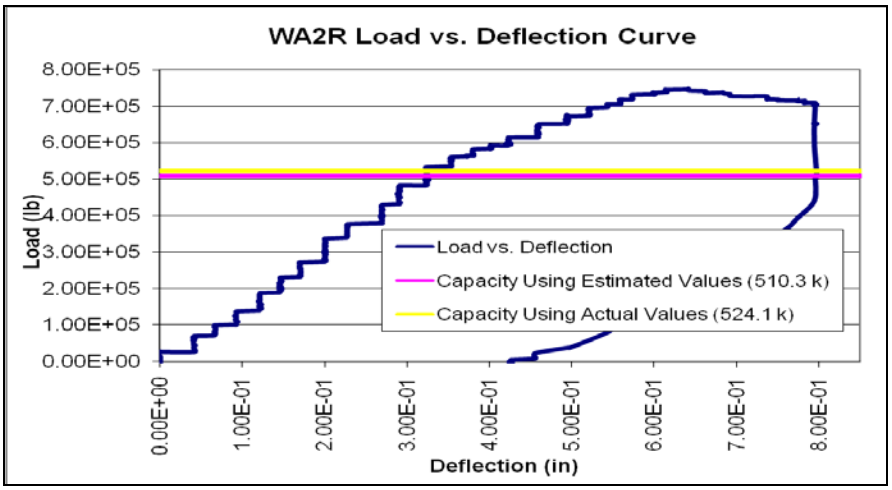


Figure C.2-27. Load Deflection Curve for End WA2R

There was evidence of strand slippage in the prestressing strands of the girder. Simply by looking at the end of the girder one could tell that the strands had retracted in, visible in Figure C.2-28. Originally, the prestressing strands had been cut flush with the end of the girder. The lab in Nebraska accentuated this flush surface by grinding down the jagged edges of the strands so that they were flat and even with the girder edge. Manual measurement of the slippage with a ruler showed that the strands slipped somewhere around 1/16 in. (.0625) According to the slippage gauges, the West strand on the outter edge of the flange had a slippage of 0.075 in. at failure. (Figure C.2-29) The East strand located towards the centerline of the girder had a slippage of 0.23 in. at failure.(Figure C.2-30) This strand slippage would have affected the capacity of the girder. The failure load would have been higher if no strand slippage had occurred. Therefore, it can be deduced that in an ideal situation with improved end anchorage conditions, the girder would have performed even better than the test showed with a higher load capacity.

Figure C.2-28. Evidence of Strand Slippage

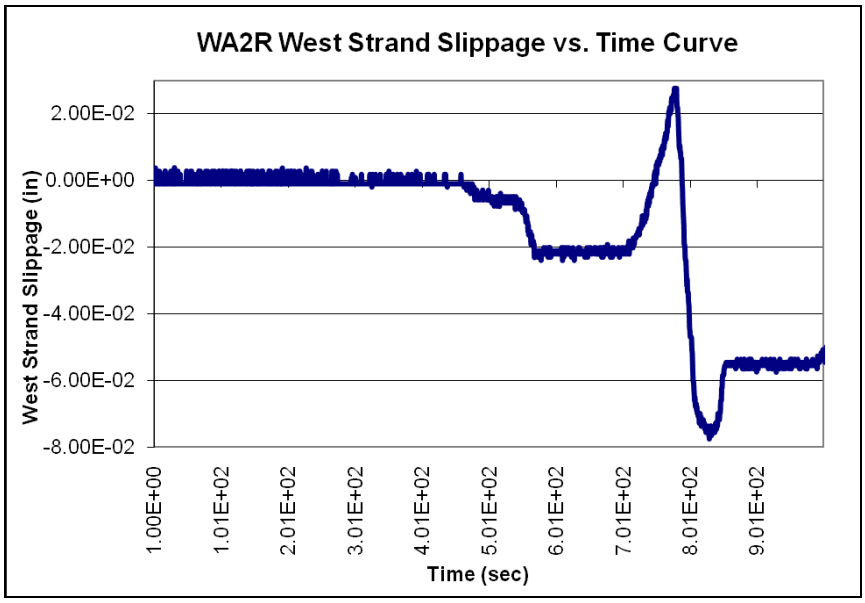


Figure C.2-29. Strand Slippage on West Strand (exterior strand)

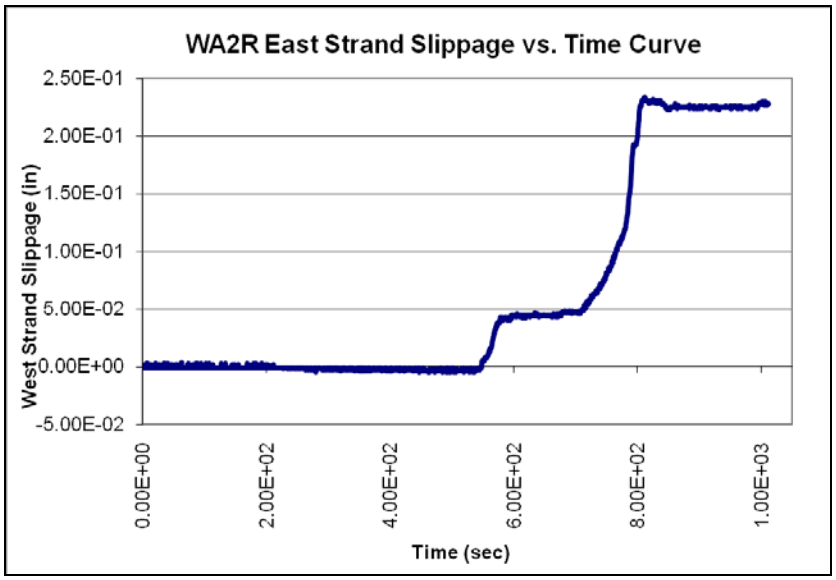


Figure C.2-30. Strand Slippage on East Strand (interior strand)

(a) Girder WA1: End WA1L

End WA1L was the first end tested on the second Washington girder. It was loaded on Tuesday, September 20, 2008. The end reinforcement for this section was designed using the proposed plans outlined in the PCI Journal article. A much larger amount of steel was placed closer to the end than the common LRFD design guide instructs. At 1.5 in. away from the end of the girder, a #8 bent C-shaped bar as well as two #6 vertical bars were placed. A second pair of #6 bars as well as pairs of #4 bars were placed after this. The end cracks that formed were not epoxy injected nor repaired in any other way.

The girder was supported on each end with 1 ft. bearing on each side, leaving a center of support to center of support span of 41 ft. The same clamping device used earlier was also affixed to the girder, 30 in. away from the end. As described above, the clamping device applied 80 k to the end of the girder to simulate the weight of the bridge acting on the supports.

In this case, only one strand slippage gauge was used. It was affixed to one of the strands on the bottom row, farthest away from the girder centerline on the outter edge of the flange. Placement is shown in Figure C.2-31. From the previous tests, the team determined that the most critical strands vulnerable to slipping are those on the edges of the strand bundle, hence why the gauge was attached to one in the bottom row and one farthest away from other strands. Figure C.2-32 shows the end zone cracks before testing.

Figure C.2-31. Gauge used to determine strand slippage

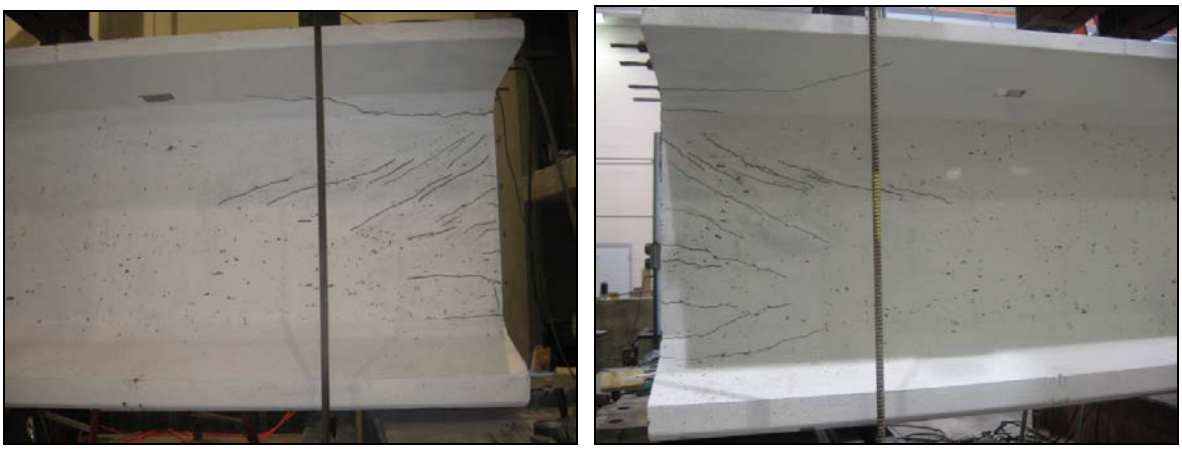


Figure C.2-32. End zone cracks before testing

Table C.2-3 gives the moment capacity and corresponding point load of the WA Specimens using the minimum specified and actual measured strength of the girder and deck.

Table C.2-3. Moment capacity and corresponding point load of the WA1L specimen

	Minimum Specified Strength	Actual Strength	Test Data
Shear Capacity (Vn)	311.3 k	319.8 k	508.5 k
Point Load (P)	432.7 k	444.5 k	706.7 k

Load was slowly applied to the system in intervals of 100k. After each additional 100k, the loading was paused so that the girder could be investigated and marked for cracks. This continued until it began to approach the design load. At this point, loading continued uninterrupted until failure. The highest load reached was 706.7 k. At this point there was a deflection of 0.947 in. Figure C.2-33 shows the end zone cracks after testing. After failure, the

beam was unloaded, leaving a permanent plastic deformation of 0.41 in. at the location of loading.

Figure C.2-33. End WA1L after shear failure

Figure C.2-34 shows the load-deflection curve of the WA1L test. The girder was calculated to resist a point load of up to 444 k, located 12 ft from the end. The actual maximum load reached before failure was 706.7 k, 59% higher than the expected load. There was no reduction in strength due to the conditions at the ends, in fact, the actual maximum load was much higher than that calculated. Even without any kind of repair at all, the girder was still able to perform above capacity when loaded in this manner.

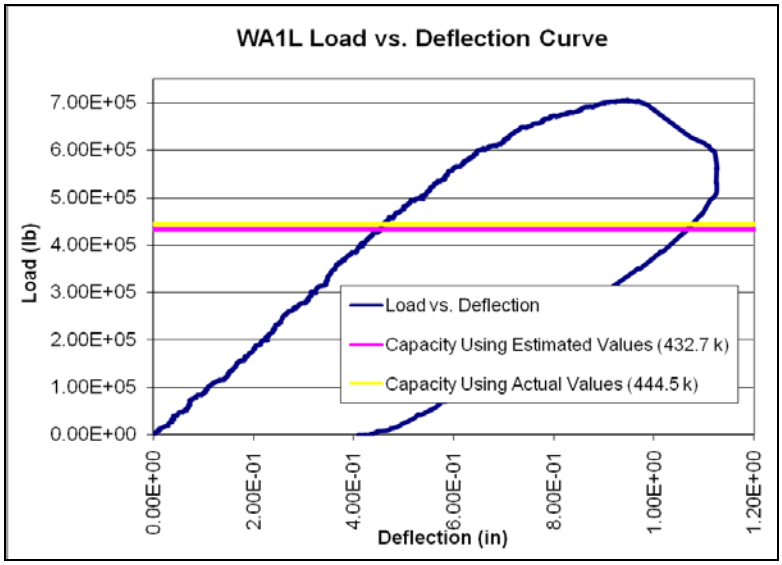


Figure C.2-34. Load Deflection Curve for End WA1L

Strand slippage in the prestressing strands was visible, as shown in Figure C.2-35. Originally, the prestressing strands had been cut flush with the end of the girder. The lab in Nebraska accentuated this flush surface by grinding down the jagged edges of the strands so that they were

flat and even with the girder edge. After testing it was apparent that the strands had pulled back into the girder, leaving a space between the end of the girder and where the end of the strands ended up. Manual measurement produced slippage results between 1/8 in. and 1/4 in. Unfortunately, the slippage gauge did not give reasonable readings.

This strand slippage would have affected the capacity of the girder. The failure load would have been higher if no strand slippage had occurred. Therefore, it can be deduced that in an ideal situation with improved end anchorage conditions, the girder would have performed even better than the test showed with a higher load capacity.

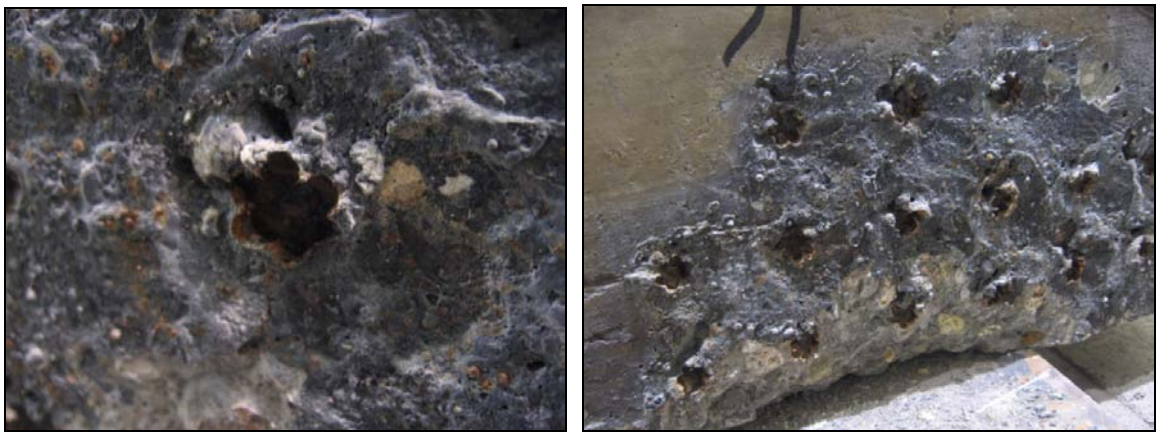


Figure C.2-35. Evidence of Strand Slippage

(b) Girder WA1: End WA1R

End WA1R was tested in the lab at the University of Nebraska on Thursday, October 2, 2008. This was the side opposite End WA1L described above. In order to test this end separate from the other, the support on end WA1L was moved in 12 ft so that it was directly under the previous point of loading. The support on end WA1R remained in the same location. These support changes ensured that the damaged end would not impact the next test. The framework was moved to the opposite side, so that a point load could be applied 12 ft. from the end WA1R. The clamping device, deflection gauge, and slippage gauge were also moved to this new end. The development length for the strands is approximately 12 ft. With the proposed setup, there would be 12 ft or more of uncracked girder extending out from either side of the point load. The strand slippage gauge is shown in Figure C.2-36. Figure C.2-37 shows the end zone cracks before testing

Figure C.2-36. Gauge used to determine strand slippage

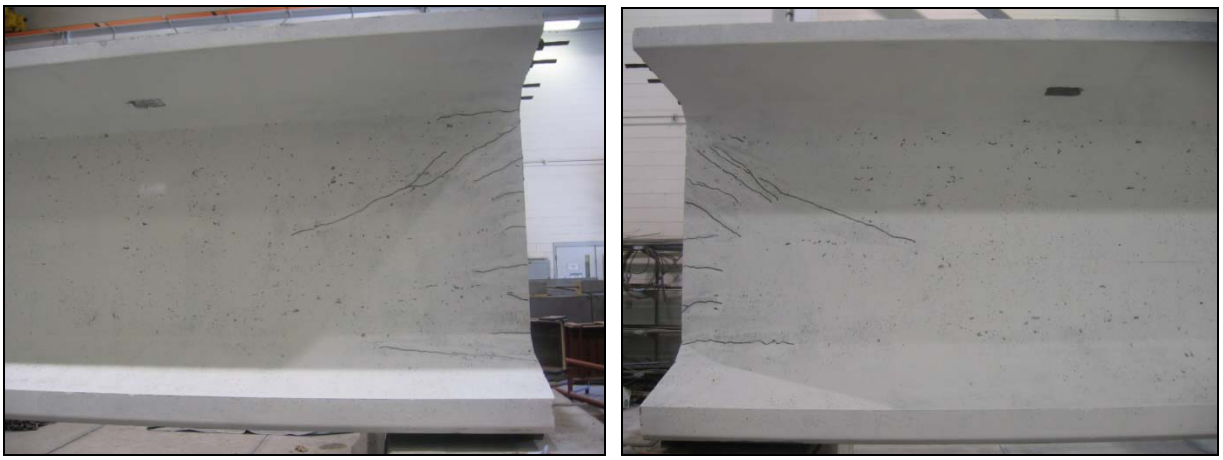


Figure C.2-37. End zone cracks before testing

Table C.2-4 gives the moment capacity and corresponding point load of the WA1R Specimens using the minimum specified and actual measured strength of the girder and deck.

Table C.2-4. Moment capacity and corresponding point load of the WA1R specimen

	Minimum Specified Strength	Actual Strength	Test Data
Shear Capacity (V_n)	311.4 k	319.8 k	488.1 k
Point Load (P)	510.3 k	524.1 k	800 k

Load was slowly applied to the system in intervals of 100k. After each additional 100k, the loading was paused so that the girder could be investigated and marked for cracks. This continued until it began to approach the design load. After this time, loading was applied continuously, expecting failure to occur. However in this case, the strength of the girder exceeded that of the capacity of the hydraulic jacks to apply load. Two hydraulic jacks were used for this test setup. Each had a capacity of 400k. Therefore, once the loading of end WA1R reached the limit of 800 k, no more load could be applied. The test had to stop even though the girder had not yet reached capacity and shown failure.

Figure C.2-38 shows the girder end after loading was stopped. From the data recorded up to that point, a load vs. deflection graph was still able to be produced, as shown in Figure C.2-39. Regardless of the actual capacity of the girder, since it reached 800 k it was still far over the calculated capacity. At 800 k when the test was stopped, the deflection read 0.629 in. After the girder was unloaded, a permanent plastic deformation of 0.19 in. remained at the location of loading.

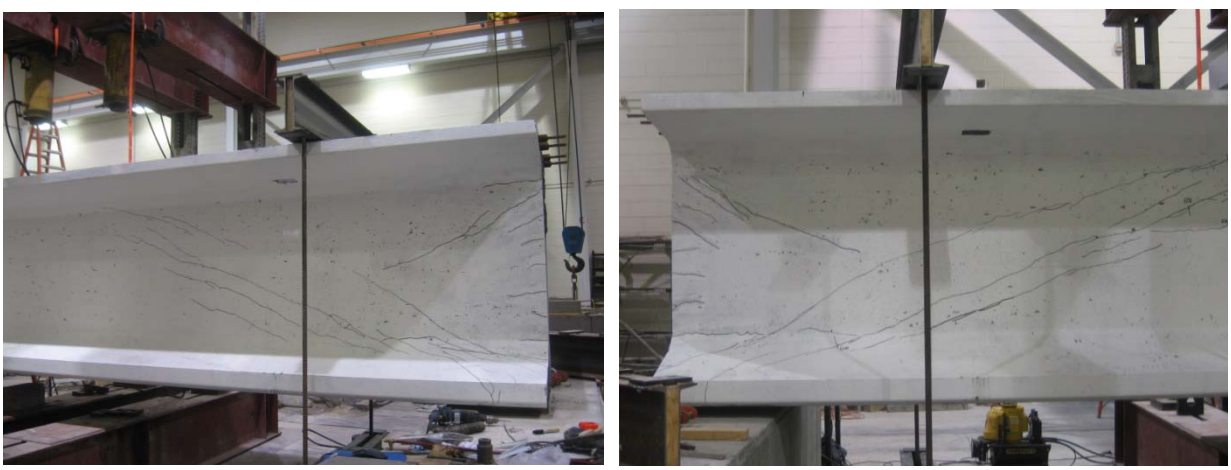


Figure C.2-38. End WA2R after loading was stopped

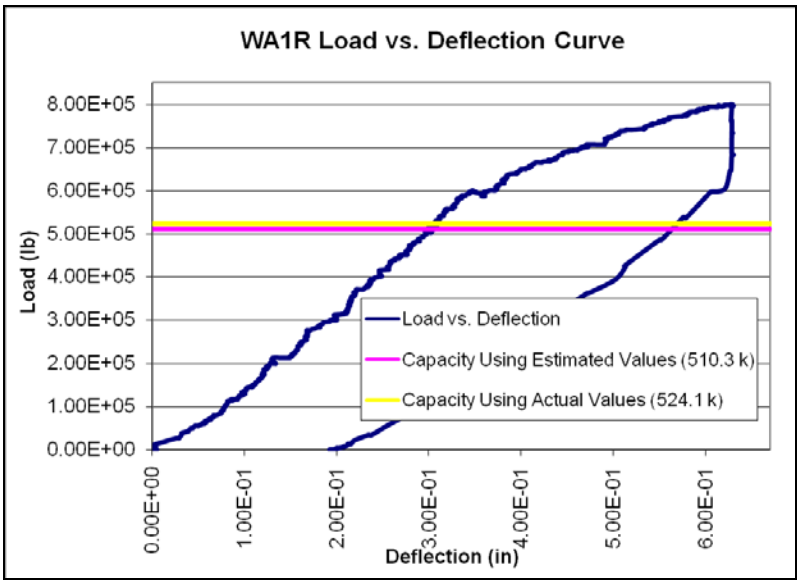


Figure C.2-39. Load Deflection Curve for End WA1R

There was evidence of strand slippage in the prestressing strands of the girder for this end as well. Visual investigation of the strands at the end of the girder showed that they had moved into the girder a bit, but not as far as the previous end. This is shown in Figure C.2-40. This would be because the end didn't actually reach failure. If it had, the strands would have traveled farther into the girder as in the other cases. Originally, the prestressing strands had been cut flush with the end of the girder. The lab in Nebraska accentuated this flush surface by grinding down the jagged edges of the strands so that they were flat and even with the girder edge. The slippage gauge was applied to a strand in the bottom row, towards the outter edge of the flange. This was estimated to be one of the strands most likely to slip.

Figure C.2-40. Evidence of Strand Slippage

At the end of testing,the slippage gauges showed a reading that the strand had slipped 0.033 in. (Figure C.2-41) This strand slippage would have affected the capacity of the girder. The failure load would have been higher if no strand slippage had occurred. Therefore, it can be deduced that in an ideal situation with improved end anchorage conditions, the girder would have performed even better than the test showed with a higher load capacity.

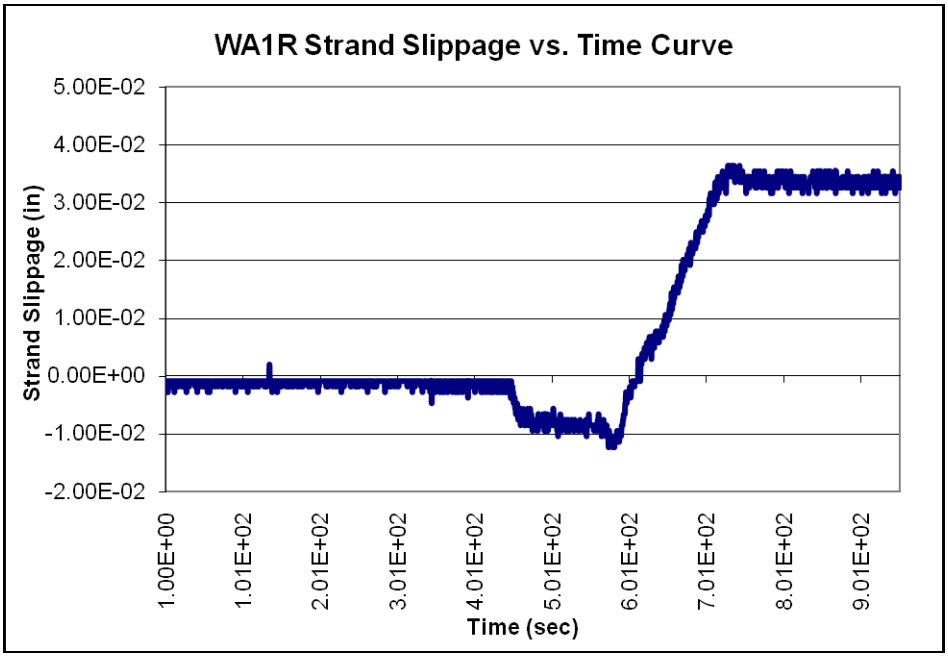


Figure C.2-41. Strand Slippage on the Exterior Strand

C.3 VIRGINIA SPECIMENS

Description and Design Calculations

Bayshore Concrete Products of Cape Charles, Virginia agreed to donate two girders for the project. They are providing two forty-two foot long girders, fabricated next to existing girders that are currently in production. These girders are 45-in. Bulb-T girders. The research girders will be the same shape, and contain the same amount of prestressing as the production girders. There are 38 - 0.6 in. diameter, 270 ksi low relaxation prestressing strands in the bottom flange of the girders, each tensioned to 44k, as shown in Figure C.3-1. The top of the girder contains 14 - 0.6 in. diameter, 270 ksi low relaxation prestressing strands that are also tensioned to 44k each. All strands are straight and have no debonding. The bottom flange contains #4 bar (402A) confinement reinforcement around the bottom group of strands. The top flange also contains a #4 bar (401A) reinforcement loop. The girders were designed to fail in shear, so the shear reinforcement consists of #5 bars (501A), in pairs, spaced every 4 in.

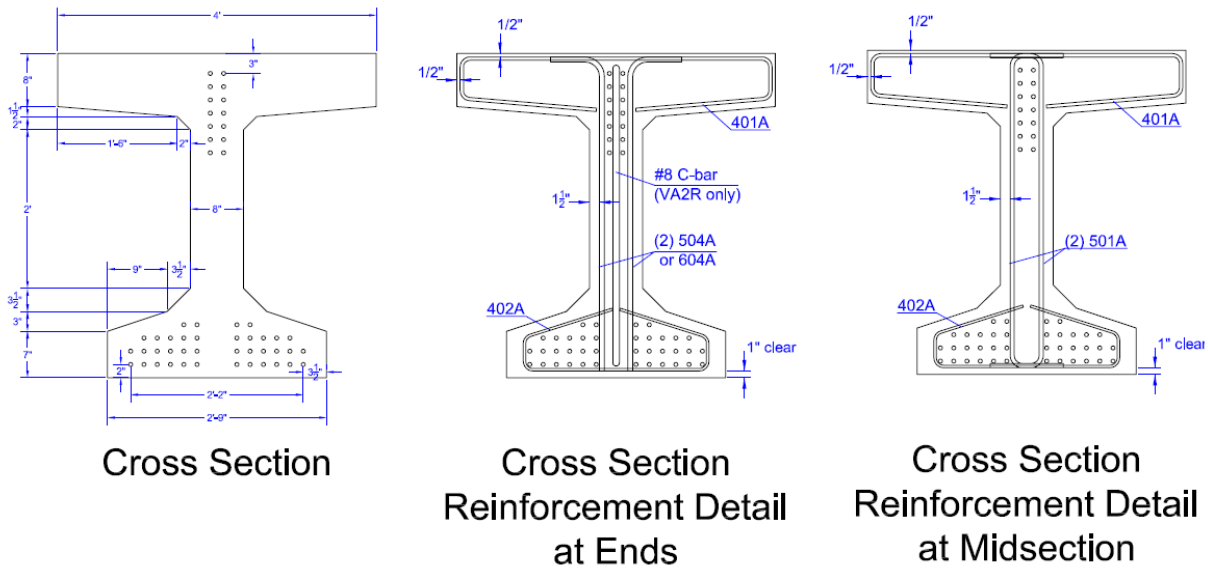


Figure C.3-1. Cross Section Details of the VA Specimens

The two ends of the first specimen contain no additional end zone reinforcement; only shear reinforcement extended to the ends, as shown in Figures C.3-2.

The two ends of the second specimen have end zone reinforcement. One end is designed according to LRFD specifications with reinforcing steel representing 4% of the prestressing force placed within h/4 from the end of the girder. The final end uses the proposed improved end zone reinforcement method to place 2% of the prestressing force within h/8 from the end of the girder, and the remainder of the original 4% before h/2 from the girder end, as shown in Figures C.3-3 and C.3-4.

All ends are designed to fail in flexure, and those ends contain the shear reinforcement of pairs of #5 bars every 4 in. All mild reinforcement bars are bent in a fashion typical of the Virginia DOT.

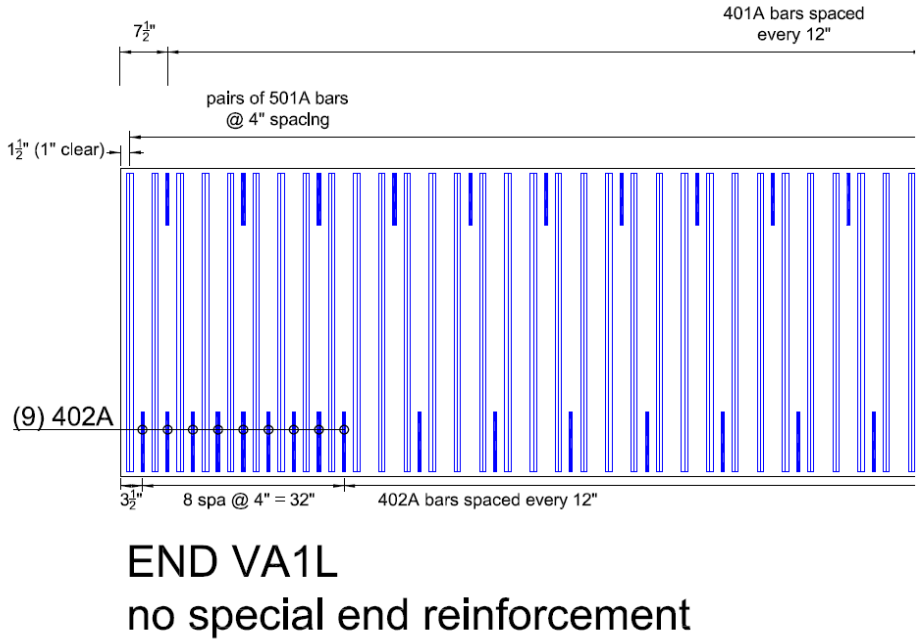


Figure C.3-2. End Reinforcement Details of VA1L and VA1R (with no end zone reinforcement)

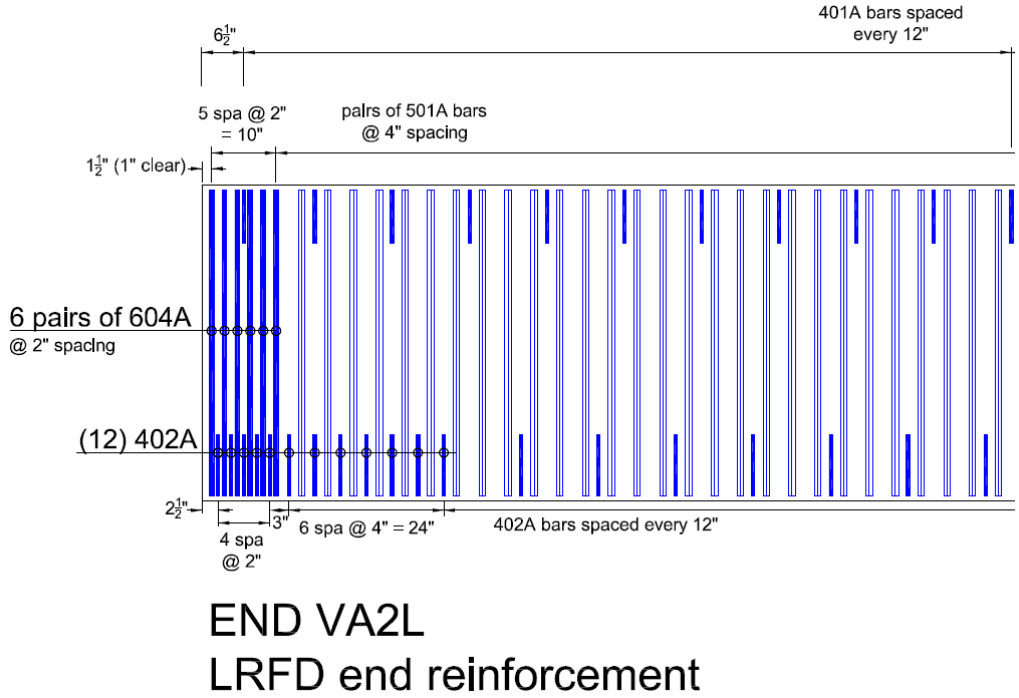


Figure C.3-3 End Zone Reinforcement of VA2L (AASHTO LRFD Detail)

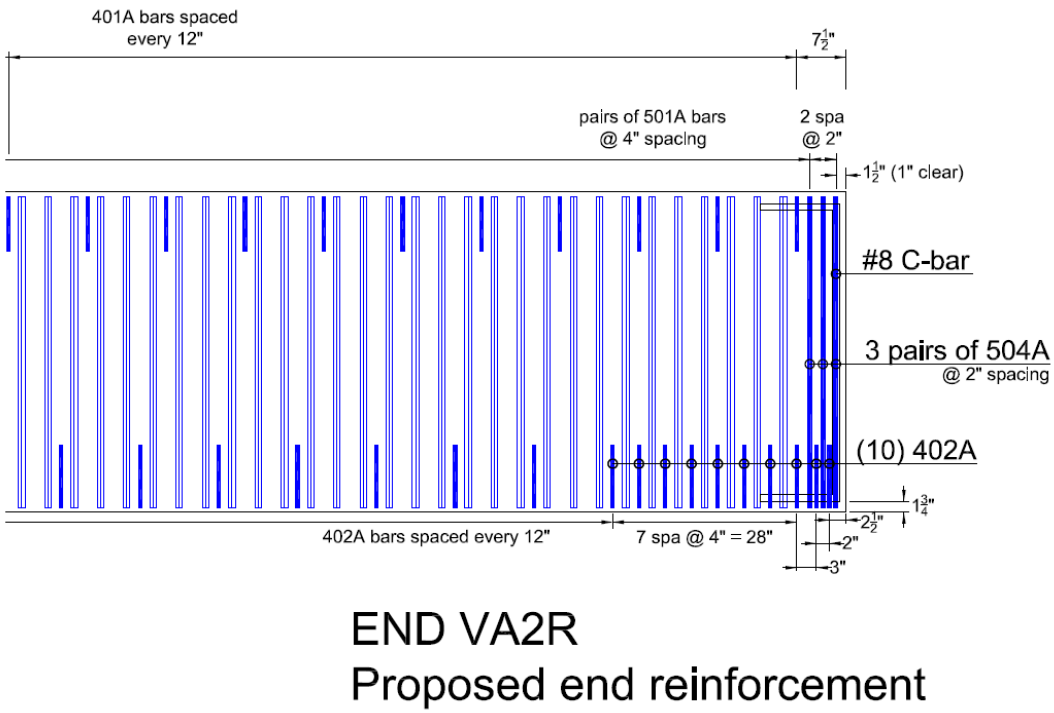


Figure C.3-4. End Zone Reinforcement of VA2R (proposed detail)

Flexural Capacity:

The girders produced by Virginia have a specified concrete strength of f'_c = 8,500 psi. A flexural strength strain compatibility program was used to find the nominal flexural moment capacity of the girder to be 7471 ft-k. The program gave the depth of the compression block as 7.39 in. and the tensile stress in the strands closest to the bottom fiber to be 258 ksi. When loading the girder with a point load 11.5 ft from the support center as shown in the image below, it can carry up to 900k.

$$Mu \leq \phi Mn,\ \phi = 1.0,\ \phi Mn = \frac{P*a*b}{L}$$

$$7471ftk = \frac{P(11.5')(29.5')}{41'} \qquad P = 902k$$

Flexural Failure:

Once the flexural capacity of the girder was determined, the specimen could be designed to fail in either flexure or shear. In order to design the beam to fail in flexure, the shear reinforcement must exceed the amount that is required for a 902 kip point load. The shear reinforcement must also follow all requirements laid out in AASHTO 2007.

With the test setup shown in the figure above, the location of highest shear would be between the left support and the point load, *P*. For the 902 kip point load applied 11.5 ft from the support center, V_u was determined to be 649k while M_u was determined to be 89,363 in-k.

$E_c = (57{,}000 * \sqrt{8{,}500})/1000 = 5255$ ksi

dv = d - a/2 = 35.08” - (7.386”/2) = 31.39”

= 0.9*d = 0.9(35.08”) = 31.57”

= 0.72*h = 0.72(49”) = <u>38.71”</u>

AASHTO LRFD gives

$\phi = 1.0$

$V_n \leq V_c + V_s \leq 0.25 * f'_c * b_v * d_v$

$$Vc = 0.0316 * \beta * \sqrt{f'c} * bv * dv$$

$$Vs = \frac{Av}{s} * fy * dv * \cot(\theta)$$

To solve for $_{x,}$ first estimate a Θ value of 25°.

$$\epsilon x = \frac{\frac{Mu}{dv} + 0.5|Vu + Vp| * \cot(\theta) - Aps * fpo}{2(Es * As + Ep * Aps)}$$

$$\varepsilon x = \frac{\frac{89{,}363}{38.71} + 0.5|647.56 + 0| * \cot(25°) - (52 * 0.217) * 202.5}{2(28{,}500 * (52 * 0.217))} = 0.001$$

(positive value, OK)

$$vu = \frac{Vu}{\emptyset * bv * dv} = \frac{649}{0.9 * 8'' * 38.71''} = 2.32 \qquad \frac{vu}{f'c} = \frac{2.32}{8.5} = 0.27 \nleq 0.25$$

Table 5.8.3.4.2-1 (LRFD Specs.) → Θ = 30.6° , = 2.12

Solving for $_x$ with the new Θ value of 30.6°

$$\varepsilon x = \frac{\frac{89{,}363}{38.71} + 0.5|647.56 + 0| * \cot(30.6°) - (52 * 0.217) * 202.5}{2(28{,}500 * (52 * 0.217))} = 0.00888$$

Table 5.8.3.4.2-1 → Θ = 30.6° , = 2.12

$V_n \leq V_c + V_s$ (= 1.0)

$$\phi V_n \leq 0.0316 * \beta * \sqrt{f'c} * bv * dv + \frac{Av}{s} * fy * dv * \cot(\theta)$$

$$649k = 0.0316 * 2.12 * \sqrt{8.5} * 8*38.71" + \frac{Av}{s} *60ksi*38.71 * \cot(30.6)$$

$$\rightarrow \frac{Av}{s} = 0.15$$

$$s = \frac{Av}{0.15} = \frac{2*0.31}{0.15} = 4.13"$$

<u>Two #5 bars every 4 in.</u>: $\frac{2*0.31}{4"} = 0.155 > 0.15 \quad OK$

For the equation shown below, AASHTO 2007 states that a factor of 0.25 may be used if the girder in question has strands that are adequately anchored beyond the girder end, such as in an end block. Otherwise, a factor of 0.18 must be used.

$\phi V_n \leq 0.25*f'_c*b_v*d_v \quad (\phi = 1.0)$

$649k \leq 0.25 * 8.5 * 8 * 38.71 = 658\ k$ OK, anchorage device needed

Check Min Reinforcement

$$Av = 0.0316 * \sqrt{f'c} * \frac{bv * s}{fy} = 0.0316 * \sqrt{8.5} * \frac{8" * 4"}{60ksi} = 0.049 < 0.62 \quad OK$$

Check Min Spacing

If $v_u \geq 0.125*f'_c$ then $s_{max} = 0.4*d_v \leq 12"$

If $v_u < 0.125*f'_c$ then $s_{max} = 0.8*d_v \leq 24"$

$2.32 \geq 0.125*8.5 = 1.0625$, $s_{max} = 0.4*d_v = 0.4*38.71" = 15.5" \leq$ <u>12"</u>

4" < 12" OK

Fabrication and Inspection

The two 42-ft long girders were fabricated by *Bayshore Concrete Products*, Cape Charles, Virginia, on July 1, 2008. A member of the research team visited the precast plant two times: (1) before the concrete was cast to check the reinforcement layout, and (2) immediately after the strands were released to document the end zone cracking. Figures C.3-5 to C.3-8 show the end zone cracks detected after the prestress release.

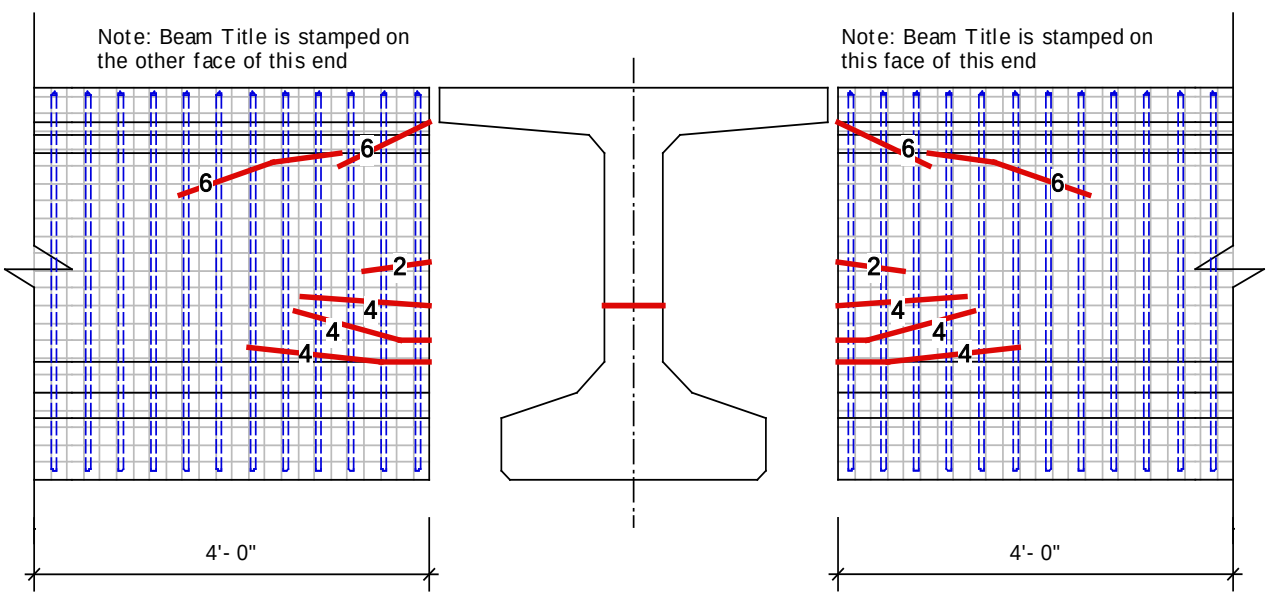


Figure C.3-5. VA1R crack pattern (crack size in thousands of an inch)

Figure C.3-6. VA1L crack pattern (crack size in thousands of an inch)

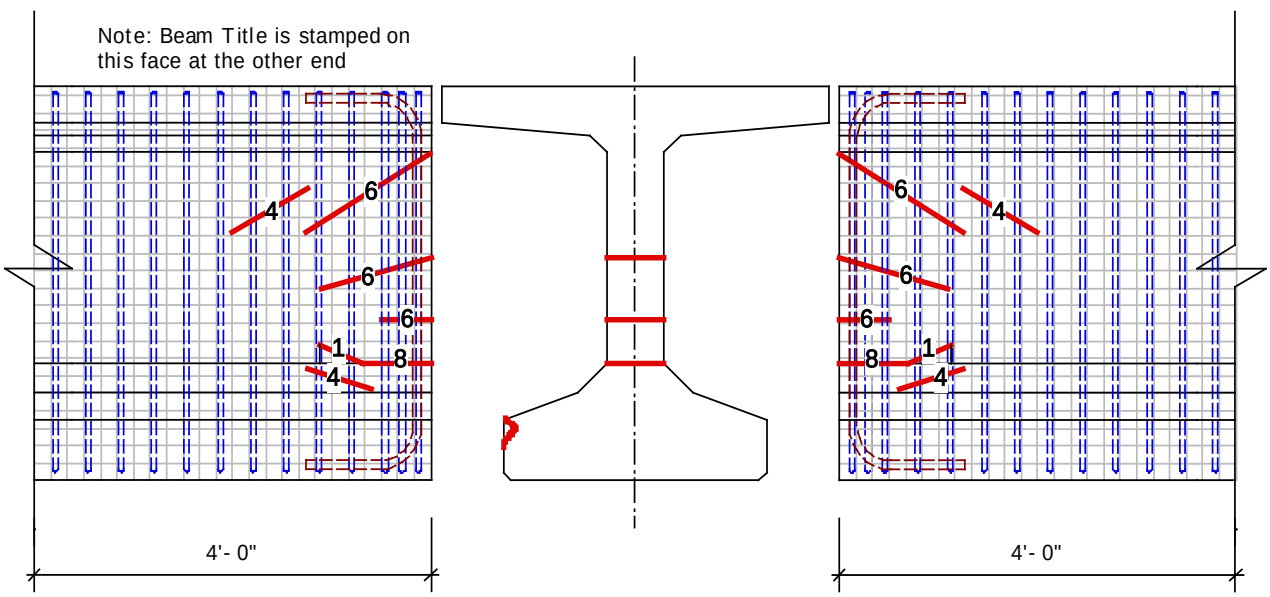


Figure C.3-7. VA2R crack pattern (crack size in thousands of an inch)

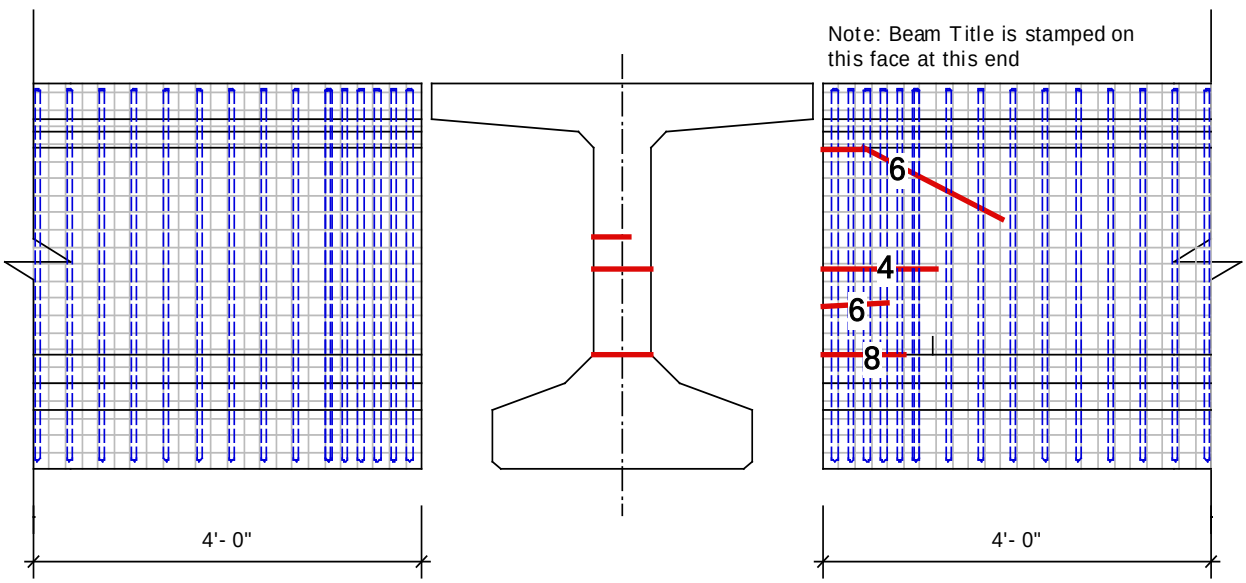


Figure C.3-8. VA2L crack pattern (crack size in thousands of an inch)

The girders arrived at the University of Nebraska, Omaha Laboratory, after spending some time in storage at Coreslab Ind., also in Omaha, NE. All four ends of the girders contained end zone cracks. It appeared that these cracks had even grown slightly from the cracks documented at the prestressing plant in Virginia.

As the girders were being tested in Omaha, NE, test cylinders from the same concrete mix were tested at the prestressing plant in Virginia. The average strength of these six cylinders gave an actual concrete strength of f'_c = 12,215 psi. This concrete strength was used to calculate the actual strength of the girders.

Structural Testing and Test Results

(a) Girder VA1: End VA1R

On Friday, November 7, 2008 End 1R of the first Virginia girder was tested. It was an end that contained no additional end reinforcement besides the typical shear reinforcement. Therefore, it had pairs of #5 bars spaced every 4 inches all the way to the end of the girder. The girder was tested as it had come from the precasting plant, without a deck or any other additions.

The girder was supported on each end with a 1 ft. bearing on steel plates. A point load was applied to the girder 12 ft. from the end VA1R. Setup is shown in Figure C.3-10. With the previous Tennessee and Washington girders, the required maximum load was usually within 800k. Therefore, the test setup utilized two 400k capacity jacks. For the Virginia and Florida girders, a larger jacking force will be required to reach failure. Another 400k capacity jack was added to the setup, making the maximum attainable point load 1,200 k.

A clamping force mechanism was placed 30 inches in from the end of the girder in order to simulate the bridge weight acting on the supports. The clamping force was provided by using a hydraulic jack attached to a self-equilibrium frame built around the specimen as shown in Figures C.3-9 and C.3-10.

This 60 k applied clamping force was calculated as the balance between the reaction generated by the original production girders that were being manufactured (taking into consideration weight of girder, deck and barriers) and the reaction generated by the 42 ft long specimen (taking into consideration weight of the girder and deck). This clamping mechanism was placed only at the end being tested, which was VA1R.

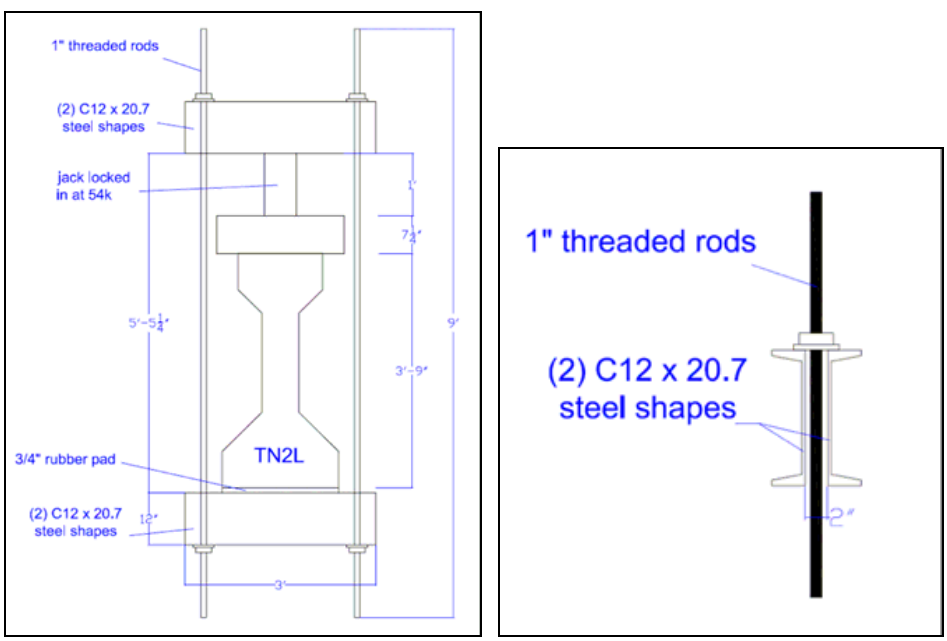


Figure C.3-9. End clamping detail

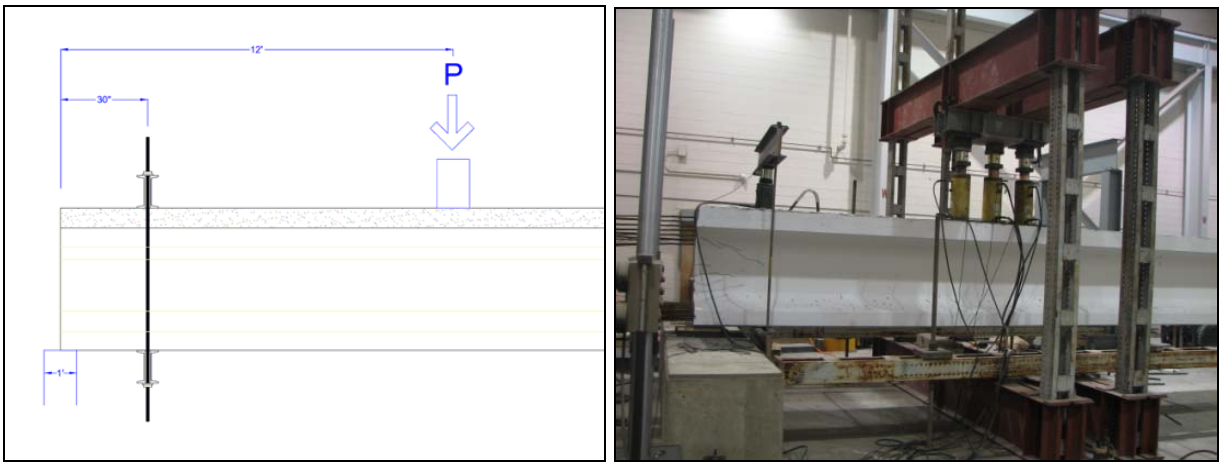


Figure C.3-10. Locations of point load and clamping mechanism

For the Virginia girders, the ends of the prestressing strands were left extended beyond the girder ends. In most cases, these strands were beginning to unravel, and had the appearance of being frayed at the ends. In order to measure strand slippage, permanent marker was used to mark each individual strand a distance away from the girder surface. This way, after the test the distance between the girder surface and the mark could be measured and compared to the original distance to determine how far the strand slipped into the girder. The team did not have access to extra string-pod distance gauges at this time and it was decided that a manual measurement of the strand slippage was adequate in this case.

Figure C.3-11 shows the girders after they had been painted white. The girders were painted white so that it would be easier to detect very small cracks formed during testing. The cracked paint is easier to see than cracked concrete alone. The end zone cracks were also drawn in with marker so that they could be easily seen from a distance.

Figure C.3-11. End zone cracks before testing

Figure C.3-11(cont.) End zone cracks before testing

Load was slowly applied to the system in intervals of 100k. After each additional 100k, the loading was paused so that the girder could be investigated and marked for cracks. This continued until it began to approach the design load. At this point, loading continued until failure. This end of the girder was designed for flexural failure. However, in this case, the failure that occurred was bearing failure at 897 k. The girder was supported on a steel base that was 12 inches in the direction of the girder length, and 16 inches in the direction of the girder width. This was not enough bearing to support the girder under this loading. Therefore, at failure the concrete around the support crumbled and burst outwards. Images of the girder after failure are shown in Figure C.3-12.

Figure C.3-12. End VA1R after bearing failure

Figure C.3-12(cont.) End VA1R after bearing failure

One half of the bottom prestressing strands shifted upwards. The bottom confinement reinforcement bars were of the same shape and design of those used in the Virginia production girders. The #4 bars were bent around the bottom prestressing strands in a closed shape; however the ends did not overlap. Therefore, at failure, the closed shape did not hold. If the ends had overlapped, then the bars would have more of a chance to develop within the concrete and would have taken more force before bursting.

The Virginia girders were designed without internal base plates. Instead, the girder had to be placed on a metal base, designed to be large enough to distribute the load to the girder without causing failure. For End VA1R, the metal base used was not large enough. For the next test, End VA1L, a larger metal base was used to prevent the same premature failure. Also, the metal base on each end was placed on metal rollers. The rollers would allow the supports to pivot, making flexural failure more likely.

Due to the nature of failure and the crumbling of concrete at the girder end, readings on strand slippage were not deduced. Table C.3-1 gives the Moment Capacity and Corresponding Point Load of the VA Specimen using the minimum specified and actual measured strength of the girder and deck.

Table C.3-1. Moment capacity and corresponding point load of the VA1R specimen

	Minimum Specified Strength	Actual Strength	Test Data
Moment Capacity (Mn)	7,471 ft-k	7,809 ft-k	7,593 ft-k
Point Load (P)	882 k	923 k	896.6 k

Figure C.3-13 shows the load-deflection curve of the test. The highest load reached was 896.6k. At this point the girder failed prematurely due to bearing failure. At the location of loading, the girder had deflected 1.86 in. With the actual concrete strength of f'_c = 12,215 psi, the girder was calculated to resist a point load of 923 k located 12 ft. away from the girder end. The maximum load from the test was only 26 k below the expected capacity. Since the girder failed

prematurely due to inadequate bearing, it is expected that with improved bearing support, the end would have resisted a load much greater than the expected capacity. The crack pattern on the girder other than the bearing failure was consistent with flexural-shear failure cracks.

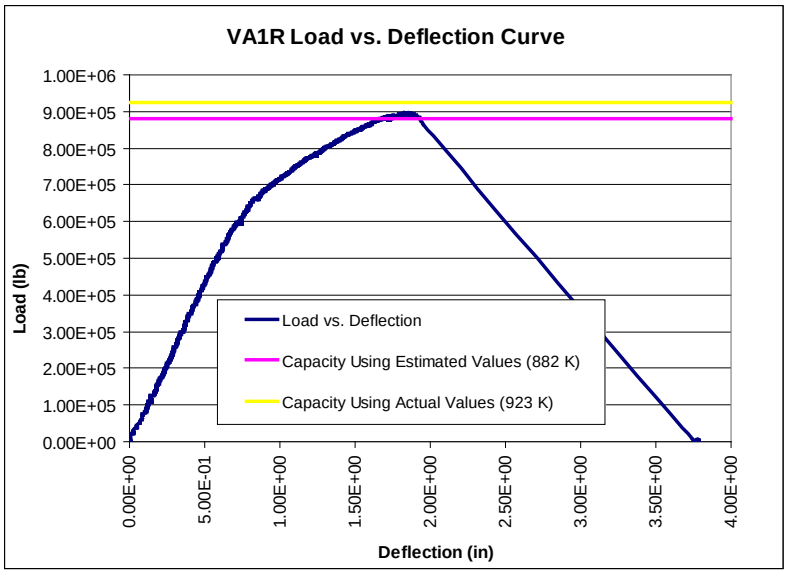


Figure C.3-14. Load Deflection Curve for End VA1R

(b) Girder VA1: End VA1L

End VA1L was tested on Wednesday November 12, 2008. It also contained no additional end reinforcement besides the typical shear reinforcement of pairs of #5 bars spaced every 4 in. It was located on the same girder, but on the opposite end of VA1R described earlier. In order to avoid the effects of having one end already reached failure, the support on end VA1R was moved in 12 ft. to keep the damaged portion of the girder out of the new span. This created a 29.5 ft. span from center to center of the 1 ft. long bearing supports, with a 11.5 ft. of end VA1R cantilevering past the support. This support change ensured that the damage would not affect the second test. However, since the support length was decreased, the capacity of the girder increased.

The framework and clamping apparatus was moved to End VA1L so that a point load could be applied 12 ft. from the end. The deflection gauges was applied to this second end as well, directly underneath the point load.

In order to measure strand slippage, permanent marker was used to mark each individual strand a distance away from the girder surface. This way, after the test the distance between the girder surface and the mark could be measured and compared to the original distance to determine how far the strand slipped into the girder

Due to the bearing failure of the previous girder, the base plate detail was improved for End VA1L. A 26”x24”x2” metal plate was placed under each support. At least 18 in. of the girder end had to be placed upon the base plate. Metal rollers were midway under each metal plate. The

team tried to get the metal roller to be directly under the beam at 6 in. from the girder end to stay consistent with the previous tests. This roller allowed the end of the girder to rotate during testing, decreasing the possibility of another bearing failure. Figure C.3-15 shows the roller and baseplate support.

Figure C.3-15. Support with roller

Figure C.3-16 shows the girders after they had been painted white. The girders were painted white so that it would be easier to detect very small cracks formed during testing. The cracked paint is easier to see than cracked concrete alone. The end zone cracks were also drawn in with marker so that they could be easily seen from a distance.

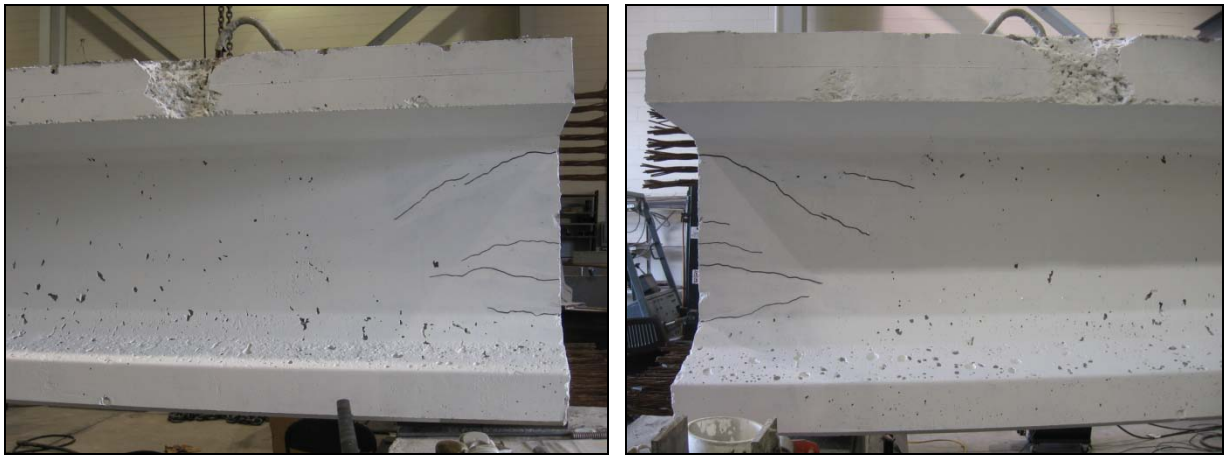


Figure C.3-16. End zone cracks before testing

Figure C.3-16(cont.) End zone cracks before testing

Load was slowly applied to the system in intervals of 100k. After each additional 100k, the loading was paused so that the girder could be investigated and marked for cracks. This continued until it began to approach the design load. After this time, loading continued until failure. The main source of failure for this girder was shear. Flexural cracks appeared as well, but they were not as predominant as the shear cracks.

The highest load reached was 1,108 k. This load produced a deflection of 1.18 in. After failure, the beam was unloaded, leaving a permanent plastic deformation of 2.42 in. at the location of loading. It was visibly damaged and drooped down at that location. Figures C.3-17 and C.3-18 show End VA1L after failure.

Figure C.3-19 shows the load-deflection curve of the test. The highest load reached was 1,108 k. At this point the girder failed in shear. Flexural cracks were also appearing near the girder's failure, however, the shear cracks appeared first and were much wider. At failure at the location of loading, the girder deflected 1.18 in. With the actual concrete strength of f'_c = 12,215 psi, the girder was calculated to resist a point load of 1,102 k located 12 ft. away from the girder end. The failure for this case would be flexural, not shear. The capacity for this end of the girder exceeded the calculated capacity, but just by a few kips. However, the girder failed in shear when it was expected to fail in flexure. It seems that the point of flexural failure for that beam was higher than the point of shear failure. Regardless, the girder still outperformed the estimated capacity, showing that the end zone cracks were unable to lower the capacity of the girder below the expected value.

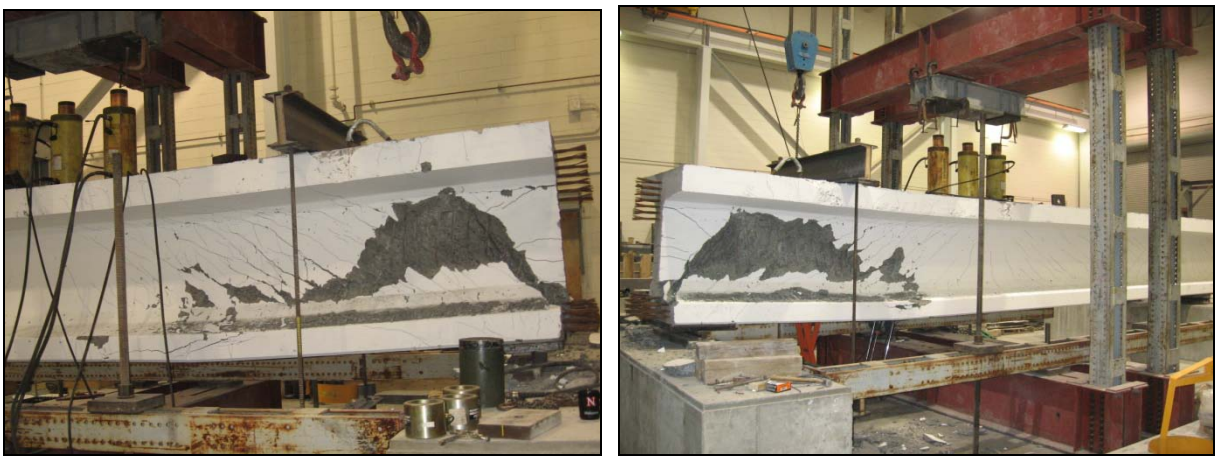


Figure C.3-17. End VA1R after shear failure

Figure C.3-18. Center of VA1 where the shear cracks for each end overlapped

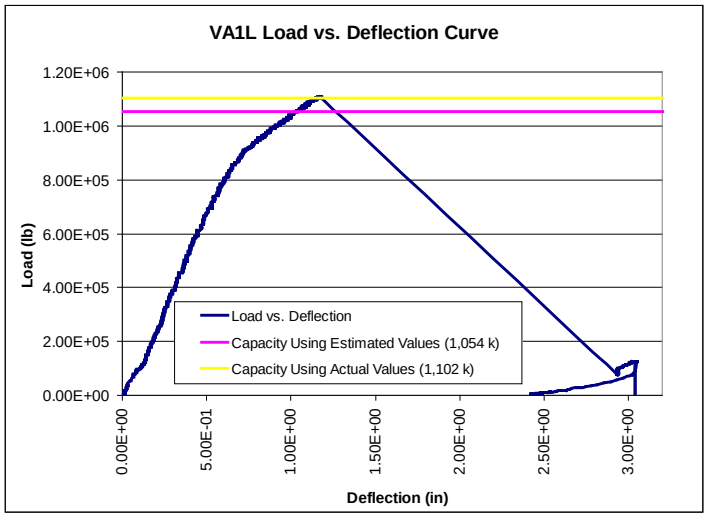


Figure C.3-19. Load Deflection Curve for End VA1L

Table C.3-2 gives the moment capacity and corresponding point load of the VA Specimen using the minimum specified and actual measured strength of the girder and deck.

Table C.3-2. Moment capacity and corresponding point load of the VA1L specimen

	Minimum Specified Strength	Actual Strength	Test Data
Moment Capacity (Mn)	7,471 ft-k	7,809 ft-k	7,852 ft-k
Point Load (P)	1,054 k	1,102 k	1,108 k

There was evidence of strand slippage in the prestressing strands of the girder. By looking at the marks made on the extended prestressing strands, one could tell that the strands had retracted. It also appeared that the strand ends had unwound further as they slipped into the girder, making the frayed ends of the prestressing steel more prominent. By measuring the movement of the mark on the strands, it showed that they slipped less than 1/8 in. This strand slippage would have affected the capacity of the girder. The failure load would have been higher if no strand slippage had occurred. Therefore, it can be deduced that in an ideal situation with improved end anchorage conditions, the girder would have performed even better than the test showed with a higher load capacity.

For all of the remaining three Virginia girders, it was found that during the testing the whole group of prestressing strands moved inwards toward the girder. It was not only individual strands that slipped, but the entire section of prestressing strands. This phenomenon is shown in Figure C.3-20. The midsection part of the girder remained where it originally was, while the two prestressing strand filled flanges pulled in towards the girder, bringing concrete with them. The sections moved in so far such that a space was created behind the middle section.

Figure C.3-20. Girder end showing strand bundle moved inwards

(a) Girder VA2: End VA2L

On November 17, 2008 End VA2L was tested in the Omaha laboratory at the University of Nebraska. The end reinforcement for this section was designed using the standard AASHTO LRFD specifications. The end zone cracks for this section were not repaired in any way.

The girder was supported on both sides by the same 26”x24”x2” metal plate and roller system as the previous girder. The rollers were placed approximately 6 in. from the end of the girder, leaving a 41 ft. unsupported span. The same clamping device used earlier was also affixed to the girder, 30 in. away from the end. The clamping force was provided by using a hydraulic jack attached to a self-equilibrium frame built around the specimen. As described before, the clamping device applied 60 k to the end of the girder in order to simulate the weight of the bridge acting on the supports.

In order to measure strand slippage, permanent marker was used to mark each individual strand a distance away from the girder surface. This way, after the test the distance between the girder surface and the mark could be measured and compared to the original distance to determine how far the strand slipped into the girder

When the girder arrived at the lab in Omaha, NE, there was some damage to End VA2L, shown in Figure C.3-21. A few inches of concrete was chipped off of the bottom corner. This exposed some of the bottom confinement reinforcement bars, as well as exposing a few of the prestressing strands a few inches too soon. This would move the development length for these few strands in a few inches.

Figure C.3-21. Damaged ends upon arrival

Figure C.3-22 shows the girders after they had been painted white. The girders were painted white so that it would be easier to detect very small cracks formed during testing. The cracked paint is easier to see than cracked concrete alone. The end zone cracks were also drawn in with black marker so that they could be easily seen from a distance.

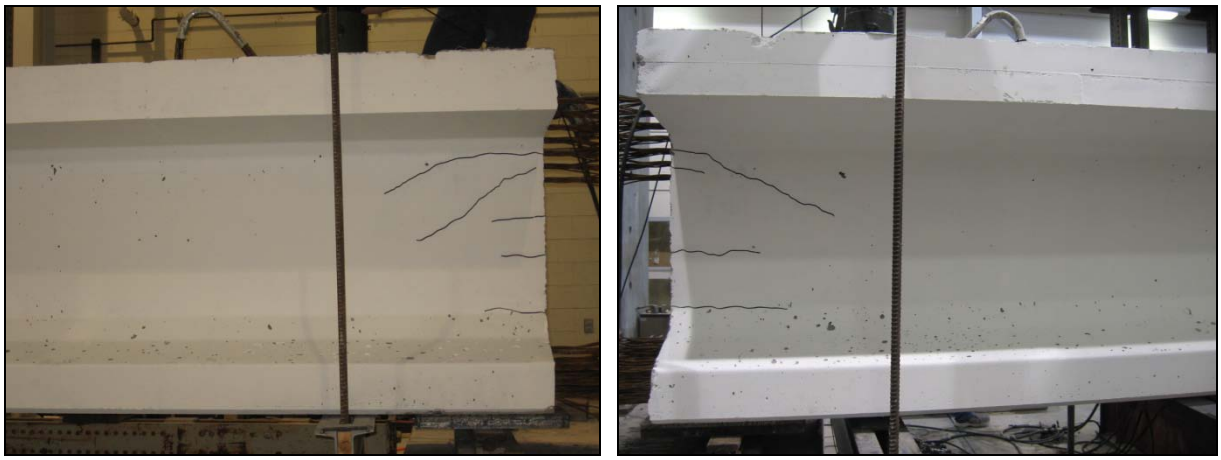


Figure C.3-22. End zone cracks before testing

Load was slowly applied to the system in intervals of 100k. After each additional 100k, the loading was paused so that the girder could be investigated and marked for cracks. This continued until it began to approach the design load. At this point, loading continued uninterrupted until failure. The highest load reached was 971.8 k. At this point there was a deflection of 2.41 in. The girder was visibly deformed. After failure, the beam was unloaded, leaving a permanent plastic deformation of 2.58 in. at the location of loading.

Table C.3-3 gives the moment capacity and corresponding point load of the VA Specimen using the minimum specified and actual measured strength of the girder and deck.

Table C.3-3. Moment capacity and corresponding point load of the VA2L specimen

	Minimum Specified Strength	Actual Strength	Test Data
Moment Capacity (Mn)	7,471 ft-k	7,809 ft-k	8,215 ft-k
Point Load (P)	882 k	923 k	971.8 k

Figure C.3-23 shows the load-deflection curve of the VA2L test. The girder was calculated to resist a point load of up to 923 k, located 12 ft from the end. The actual maximum load reached before failure was 971.8 k, 5.3% higher than the expected load. Failure for the girder was both flexural and shear. Cracks consistent with shear failure appeared first and were more wide, but there was still an abundance of flexural cracks. Therefore, shear was the primary failure mode. This may skew the results in that the estimated capacity was calculated assuming flexural failure. There did not appear to be a reduction in strength due to the conditions at the ends. Even without any kind of repair at all, the girder was still able to perform above capacity when loaded in this manner.

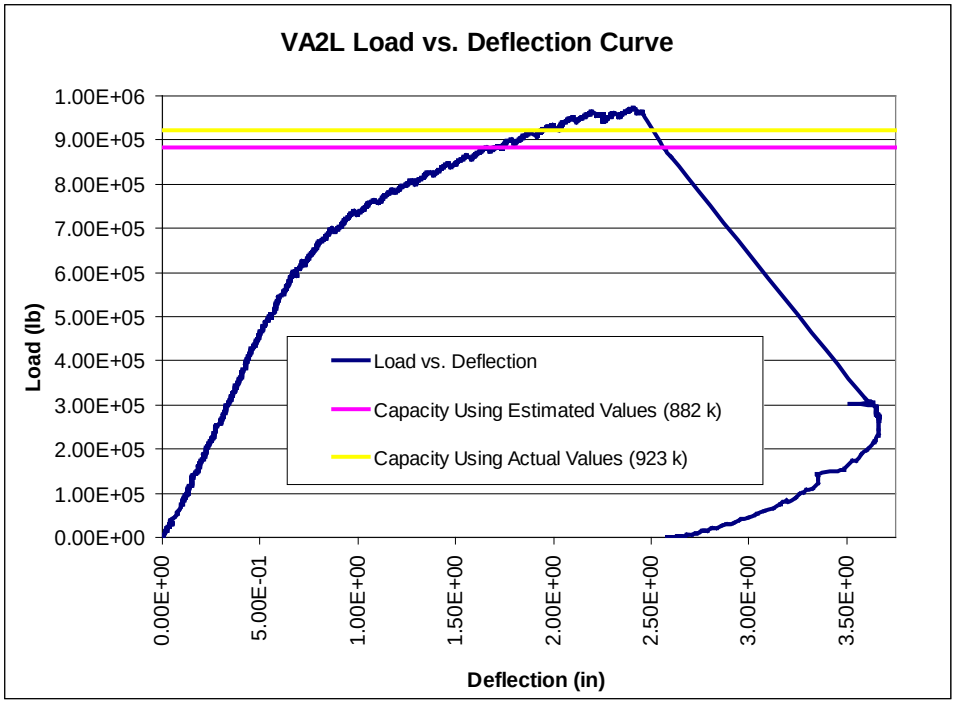


Figure C.3-23. Load Deflection Curve for End VA2L

There was some evidence of strand slippage from the marked external prestressing strands. However the slip was minimal at less than ¼ in. This strand slippage would have affected the capacity of the girder. The failure load would have been higher if no strand slippage had occurred. Therefore, it can be deduced that in an ideal situation with improved end anchorage conditions, the girder would have performed even better than the test showed with a higher load capacity. Figure C.3-24 shows end VA1R after shear failure.

Figure C.3-24. End VA1R after shear failure

Figure C.3-24(cont.) End VA1R after shear failure

(b) Girder VA2: End VA2R

End VA2R of the second Virginia girder was tested on November 18, 2008. This was the side opposite End VA2L described above. In order to test this end separate from the other, the support on end VA2L was moved in 12 ft. so that it was directly under the previous point of loading. The support on end VA2R remained in the same location. These support changes ensured that the damaged end would not impact the next test. The framework was moved to the opposite side, so that a point load could be applied 12 ft. from the end VA2R. The clamping device and deflection gauge was moved to this new end. The clamping force was provided by using a hydraulic jack attached to a self-equilibrium frame built around the specimen. As described above, the clamping device applied 60 k to the end of the girder in order to simulate the weight of the bridge acting on the supports. The development length for the strands is approximately 12 ft. With the proposed setup, there would be 12 ft. or more of uncracked girder extending out from either side of the point load.

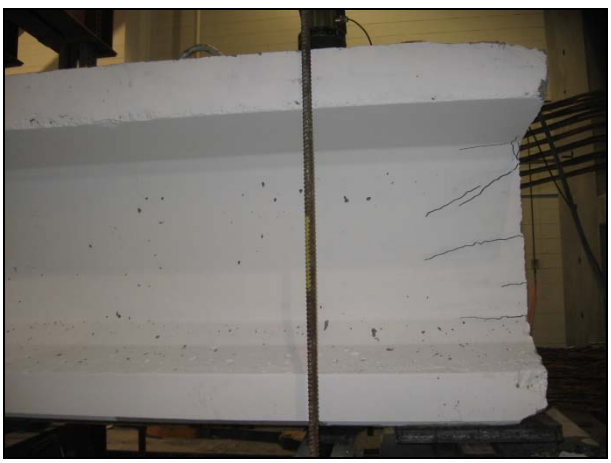
The girder was supported on both sides by the same 26”x24”x2” metal plate and roller system as the previous girder. The rollers were placed approximately 6 in. from the end of the girder, leaving a 41 ft. unsupported span. The same clamping device used earlier was also affixed to the girder, 30 in. away from the end.

In order to measure strand slippage, permanent marker was used to mark each individual strand a distance away from the girder surface. This way, after the test the distance between the girder surface and the mark could be measured and compared to the original distance to determine how far the strand slipped into the girder

When the girder arrived at the lab in Omaha, NE, there was some damage to End VA2R, shown in Figure C.3-25. A few inches of concrete was chipped off of the bottom corner. This exposed some of the bottom confinement reinforcement bars, as well as exposing a few of the prestressing strands a few inches too soon. This would move the development length for these few strands in a few inches.

Figure C.3-25. Damaged ends upon arrival to the laboratory

Figure C.3-25 shows the girders after they had been painted white. The girders were painted white so that it would be easier to detect very small cracks formed during testing. The cracked paint is easier to see than cracked concrete alone. The end zone cracks were also drawn in with black marker so that they could be easily seen from a distance.



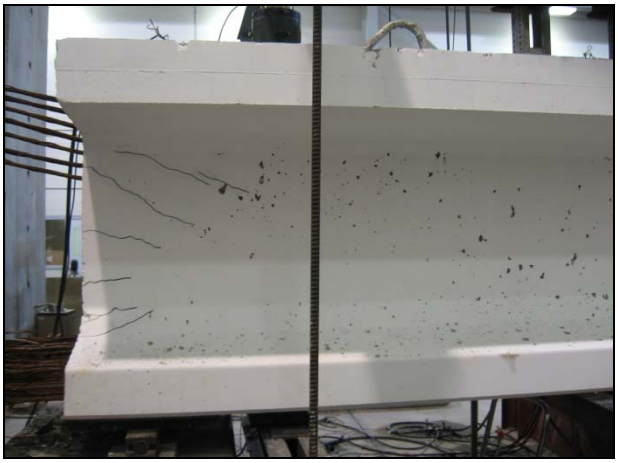


Figure C.3-25. End zone cracks before testing

Load was slowly applied to the system in intervals of 100k. After each additional 100k, the loading was paused so that the girder could be investigated and marked for cracks. This continued until it began to approach the design load. At this point, loading continued uninterrupted until failure. The highest load reached was 1,199.2 k. At this point there was a deflection of 0.95 in. Soon after the sudden failure, the girder deflected to a maximum of 2.63 in. After failure, the beam was unloaded, leaving a permanent plastic deformation of 2.06 in. at the location of loading. The capacity of the hydraulic jack system was 1,200 k. So the test would have been stopped at this loading. However, the girder had a sudden failure just right before the cutoff load.

Figures C.3-26 and C.3-27 show End VA1L after failure. Figure C.3-28 shows the load-deflection curve of the test. The highest load reached was 1,199.2 k, 8.8% higher than the expected load. At this point the girder failed in shear. There were very few, very small flexural cracks. With the actual concrete strength of 12,215 psi, the girder was calculated to resist a point load of 1,102 k located 12 ft. away from the girder end. The failure for this calculation was assumed to be flexural, not shear. The capacity for this end of the girder exceeded the calculated capacity. However, the girder failed in shear when it was expected to fail in flexure, making it hard to compare the theoretical capacity with the experimental capacity. Regardless, the girder still outperformed the estimated capacity, showing that the end zone cracks were unable to lower the capacity of the girder below the calculated value.

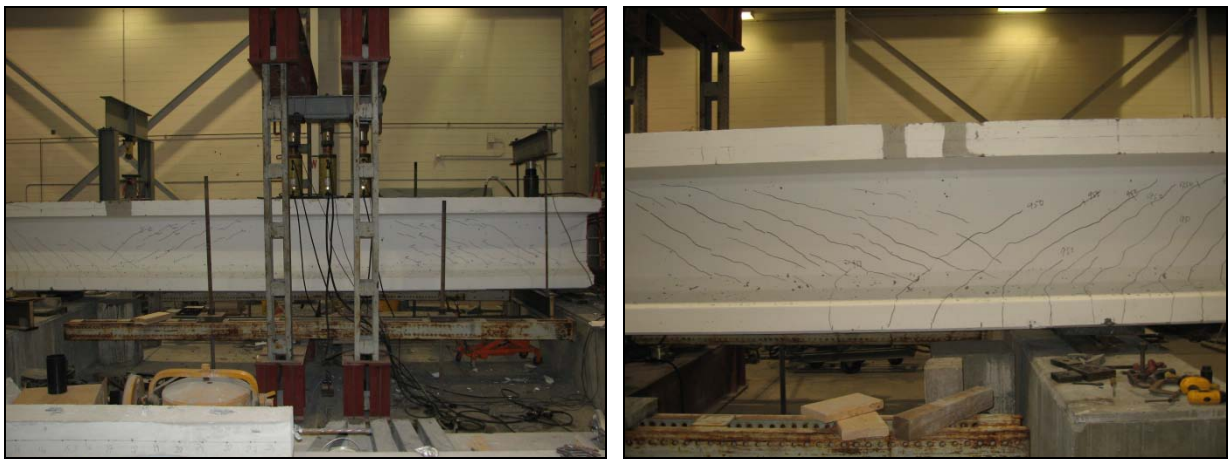


Figure C.3-26. End VA1R with signs of shear cracking

Figure C.3-27. End VA1R after shear failure

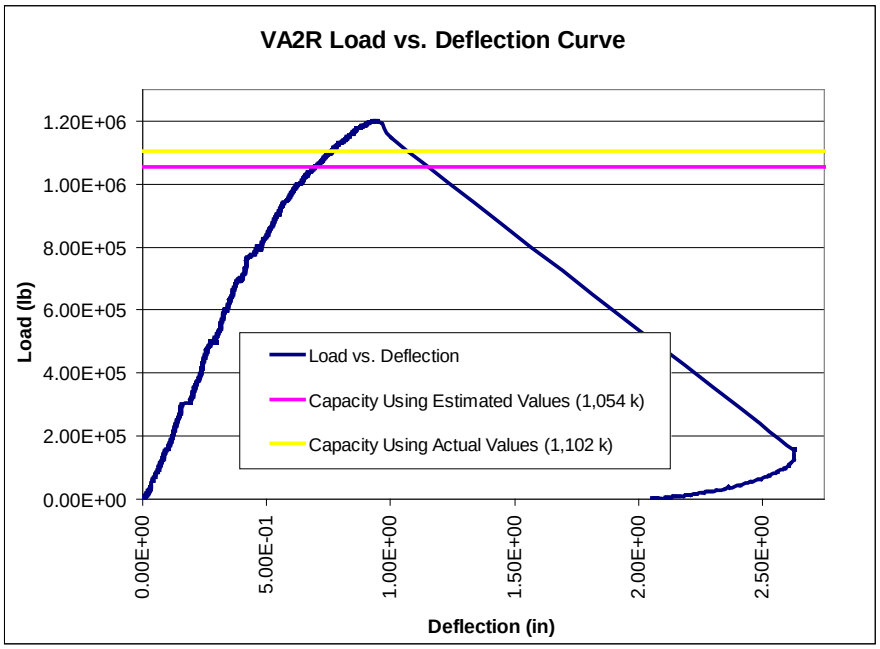


Figure C.3-28. Load Deflection Curve for End VA2R

Table C.3-4 gives the moment capacity and corresponding point load of the VA Specimen using the minimum specified and actual measured strength of the girder and deck.

Table C.3-4. Moment capacity and corresponding point load of the VA2R specimen

	Minimum Specified Strength	Actual Strength	Test Data
Moment Capacity (Mn)	7,471 ft-k	7,809 ft-k	8,492 ft-k
Point Load (P)	1,054 k	1,102 k	1,199.2 k

There was some evidence of strand slippage from the marked external prestressing strands. However the slip was minimal at less than 1/8 in. This strand slippage would have affected the capacity of the girder. The failure load would have been higher if no strand slippage had occurred. Therefore, it can be deduced that in an ideal situation with improved end anchorage conditions, the girder would have performed even better than the test showed with a higher load capacity.

As with the Virginia girder VA2, the sections of concrete that contained the prestressing strands attempted to shift in towards the center of the girder at failure. In this case the top flanges stayed in place while the bottom flange and web slipped inwards. Figure C.3-29 shows how the section containing the top prestressing strands has separated from the sections of top flange, which contains no prestressing force.

Figure C.3-29. End VA1R after shear failure, showing movement

C.4 FLORIDA SPECIMENS

Description and Design Calculations

Standard Concrete Products of Tampa, Florida produced two forty-two foot long girders for the NCHRP project. They are 60 in. deep inverted T beams with 36 – 0.6 in. diameter, 270 ksi low relaxation prestressing strands in the bottom flange, tensioned to 43.94 k each. (Figure C.4-1) The top of the girder has six #6 bars of mild steel reinforcement traveling the full length of the girder. These six bars were placed to combat the top tensile forces produced by the prestressing strands. The result was over, yet very close to the tensile limits. It was decided that the design was close enough to perform as needed. The bottom flange contains #3 bar confinement reinforcement around the prestressing strands. The girders were designed to fail in flexure, so the shear reinforcement is pairs of #5 bars, spaced every 6 in. Each end also has a 2'2" x 1' x ½" base plate with four ½" diameter studs that extend 4 in. into the base of the girder. The concrete has a specified 28-day strength of 8,500 psi, and a specified strength at release of 6,000 psi.

The other girders on the production line at Standard Concrete were modified 72" Bulb Tee Beams. The sample girders for this project were further modified from the original Florida production girders in that the top flange was not included. To compensate for the loss of top concrete, the six #6 mild reinforcement bars mentioned earlier were added to the two sample girders only. In order to achieve high stress in the bottom prestressed strands at failure, a deck was placed on the girders once they reached Nebraska. This deck was designed to simulate the decking system that would be placed on the bridge girders in the field. The deck was 10" thick and designed to be at least 10,000 psi in strength. Further details are given below.

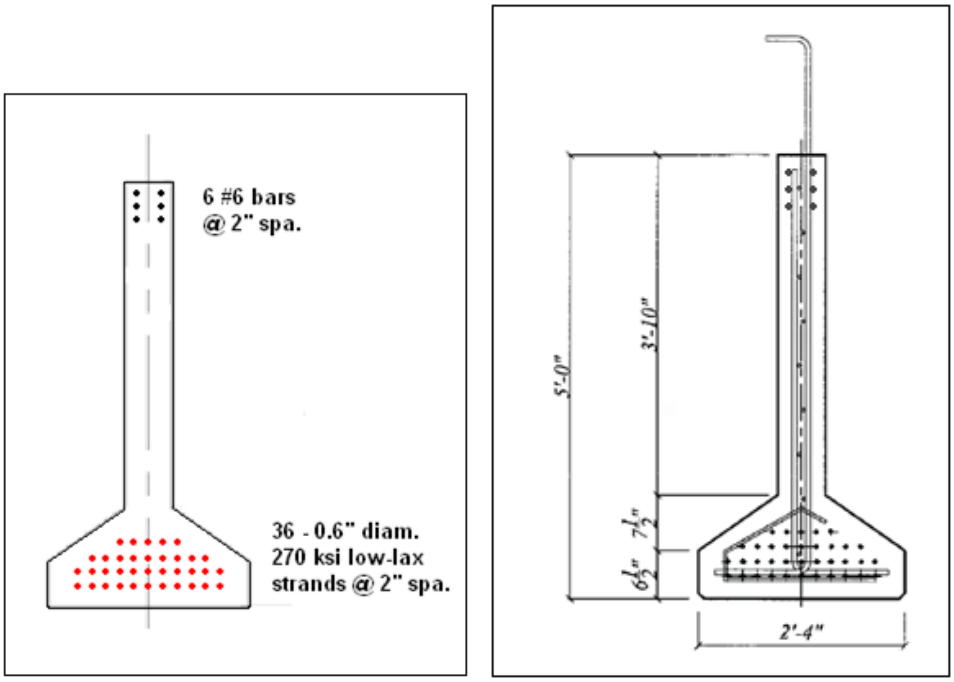


Figure C.4-1. Florida Girder Cross Section

End Zone Reinforcement for End FL1L (see Figure C.4-2)

End FL1L contains end zone reinforcement details consistent with typical Florida designs. They are similar to the end details of the production girders that our test girders were modeled after. There are 23 vertical C-shaped #5 bars, spaced 3 in. center to center at the end. Five smaller, L-shaped #5 bars are paired with the first five of the larger C-shaped bars, as shown in Figure C.4-1 and Figure C.4-2. Also, there are two pairs of unbent #4 bars paired with the first two groups of C-shaped and L-shaped bars. Following the end reinforcement are the normal shear reinforcement details of pairs of #5 bars at 6 in. spacing.

End Zone Reinforcement for End FL1R (see Figure C.4-3)

End FL1R contains end zone reinforcement details consistent with typical Florida designs, but with some slight alterations. It is identical to End FL1L, except that it does not contain the five L-shaped #5 bars nor the four unbent vertical bars. Therefore, the end contains 4 of the #5 C-shaped bars closest to the end at 2 in. spacing center to center, followed by 2 of the #5 C-shaped bars at 3 in. spacing. After this end reinforcement, the shear reinforcement of pairs of #5 bars every 6 in. begins.

End Zone Reinforcement for End FL2L (see Figure C.4-4)

End FL2L was designed according to LRFD specifications. Therefore 4% of the prestressing force is placed within h/4 (60”/4 = 15”) from the end of the girder. This area of steel was calculated at 3.16 in^2. To satisfy this, there are six pairs of #5 vertical bars placed at the end of the girder, spaced at 3” center to center. In order to leave enough cover at the end, the first pair of #5 bars is located 1-1/2” from the girder end. Following is the typical shear reinforcement of pairs of #5 bars spaced every 6”.

End Zone Reinforcement for End FL2R (see Figure C.4-5)

End FL2R was designed using the proposed improved design method. It states that 4% of the prestressing force must be within h/2 (30”) from the end of the girder. Half of that (2%) must be placed within h/8 of the end of the girder (7.5”). In order to achieve this, five pairs of #5 bars were placed at the end, spaced 2” center to center. The shear reinforcement of pairs of #5 bars spaced every 6” follows this special end reinforcement, and continues throughout the remainder of the girder. In order to leave enough cover at the end, the first pair of #5 bars is located 1-1/2” from the girder end. The area of steel provided by the first three pairs of #5 bars (1.86 in^2) is enough to satisfy the requirement for 2% of the prestressing force (1.80 in^2). The following few rows of #5 bars satisfy the other requirement of placing 4% of the prestressing force within 30” from the end of the girder. This method of design places a larger percentage of the steel closer to the girder end than the typical LRFD design.

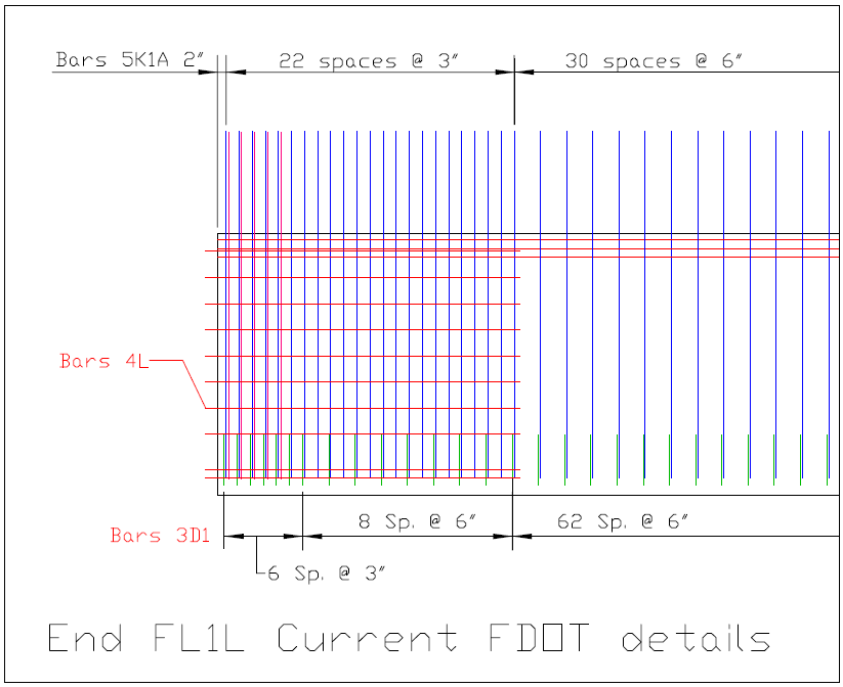


Figure C.4-2. End Zone reinforcement of FL1L

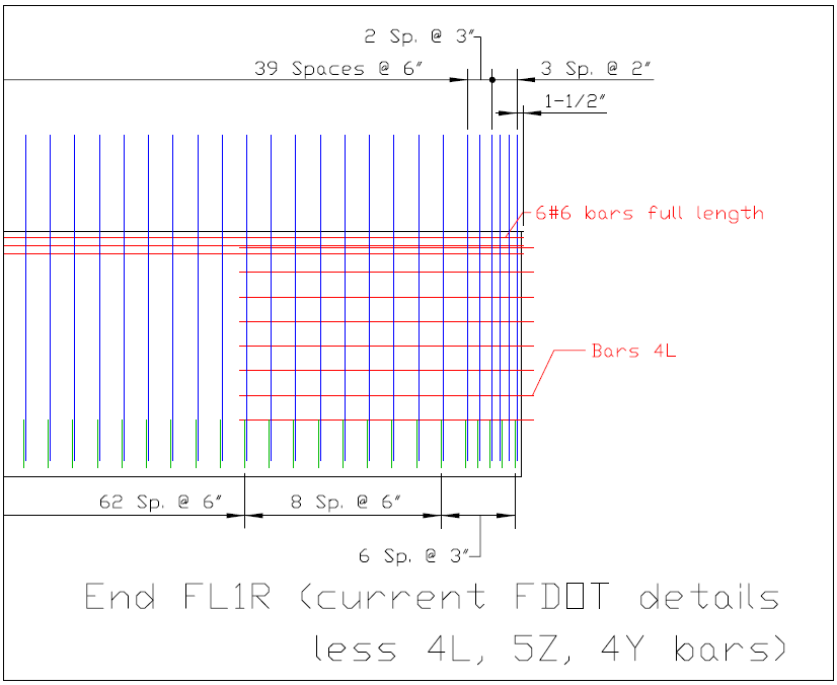


Figure C.4-3. End Zone reinforcement of FL1R

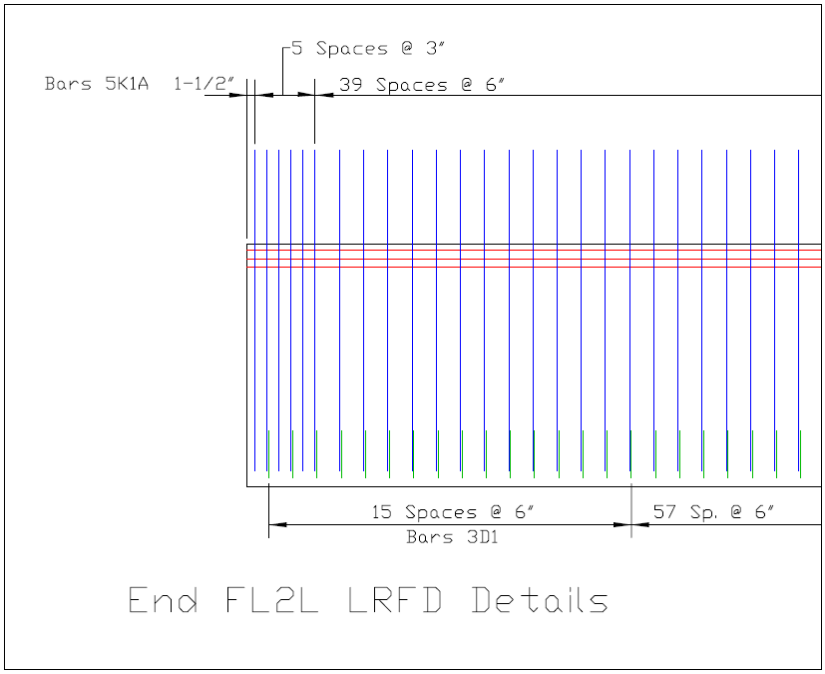


Figure C.4-4. End Zone reinforcement of FL2L

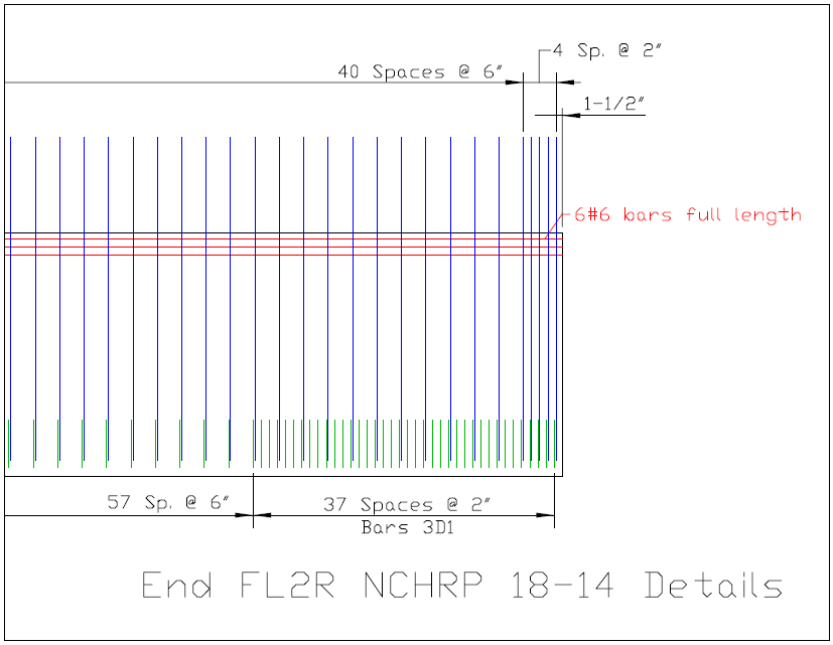


Figure C.4-5. End Zone reinforcement of FL2R

<u>Flexural Capacity:</u>

The girders produced by Florida have a specified concrete strength of f'_c = 8,500 psi. A flexural strength strain compatibility program was used to find the nominal flexural moment capacity of the girder to be 10,039 ft-k. The program gave the depth of the compression block as 14.9 in. and

the tensile stress in the strands closest to the bottom fiber to be 259.4 ksi. When testing the girders, the center of the supports were 6 in. from either end, leaving an unsupported length of 41 ft. When loading the girder with a point load 11.5 ft. from the support center as shown in Figure C.4-6, it can carry up to 1,195 k.

$$Mu \leq \phi Mn$$

$$\phi = 1.0$$

$$\phi Mn = \frac{P * a * b}{L}$$

The force point load, P, required for flexural failure with an unbraced length of 41 ft is:

$$M = \frac{P * a * b}{L} + (Moment\ from\ own\ weight)$$

$$10{,}039\ ftk = \frac{P(11.5')(29.5')}{41'} + 151k$$

$$P = 1{,}195\ kips$$

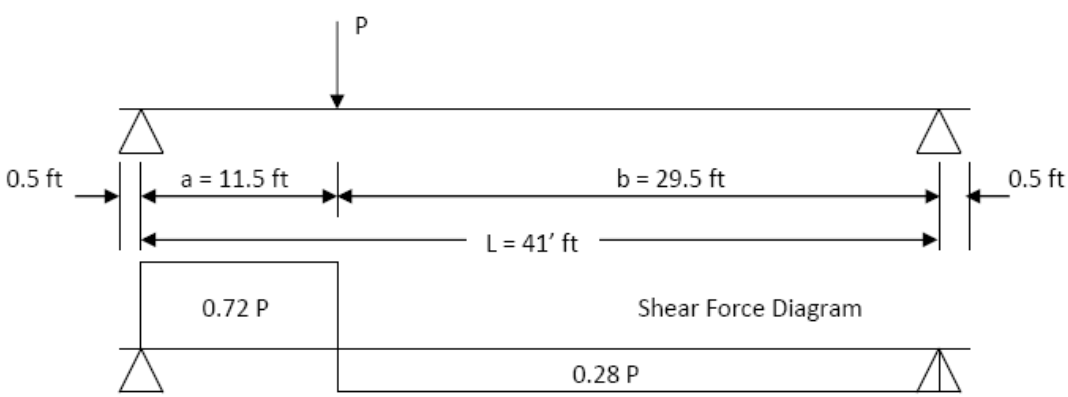


Figure C.4-6. Test Setup

When testing the second end of each girder, the support on the already tested end must be moved in past the damaged area. This way the second girder end can perform separate from the first end as if it had not been yet loaded. Before this test, the center of support was moved in to a location 12 ft from the end of the girder. This left a shorter 29.5 ft of unbraced length, therefore the point

load required to fail the girder must be larger. The point load was applied to the same location, 12 ft from the end of the girder being tested (11.5 ft from the center of the support).

The force point load, P, required for flexural failure with an unbraced length of 41 ft is:

$$M = \frac{P * a * b}{L} + (Moment\ from\ own\ weight)$$

$$10{,}039\ ftk = \frac{P(18')(11.5')}{29.5'} + 67\ k$$

$$P = 1{,}421\ kips$$

<u>Flexural Failure:</u>

Once the flexural capacity of the girder was determined, the specimen could be designed to fail in either flexure or shear. The first setup was considered, with the unbraced length of 41 ft. In order to design the beam to fail in flexure, the shear reinforcement must exceed the amount that is required for a 1,195 k point load. The shear reinforcement must also follow all requirements laid out in AASHTO 2007.

With the test setup shown in Figure C.4-6, the location of highest shear would be between the left support and the point load, P. For the 1,195 k point load applied 11.5 ft from the support center, V_u was determined to be 744 k.

$$Vs = (1.8)\frac{Av * fy * d}{s}$$

$$744k = \frac{Av}{s}(1.8)(60ksi)(70")$$

$$\frac{Av}{s} = 0.098\ sqin/\text{in}$$

Using pairs of #5 bars at 6” gives an A_v/s of 0.103 in^2/in which is greater than the required 0.098 in^2/in. Therefore for shear reinforcement pairs of #5 bars with 6” spacing was used.

Fabrication and Inspection

The FL girders were manufactured in June of 2008 in Tampa, Florida at Standard Precast Products. The sample girders were made along with the production bulb tee beams that they were modeled after. A member of the research team was at the precast plant when the reinforcement was being assembled. An error in the shear reinforcement placement was found at this time. According to the design calculations and drawings, the typical shear reinforcement was

determined to be pairs of #5 bars at 6” spacing. The precaster had placed #5 bars singly spaced at 6” for the shear reinforcement. This would be half the amount of shear reinforcement intended, and would greatly reduce the shear capacity of the girder. The member of the team that was present addressed the error and was told that it would be fixed. However, a miscommunication may have occurred because the shear reinforcement was not fixed in at least one of the girders. This was not known until after testing. Details are shown below in the section on End FL1L.

The girders were shipped to Coreslab Industries in Omaha, NE for storage until space became available at the University of Nebraska structures lab.

After arriving at the lab, the girders were painted white. The girders were painted so that it would be easier to detect very small cracks formed during testing. The end zone cracks were also traced with black marker so that they could be easily seen from a distance.

In order to simulate the decking system that would be placed on the production girders, a 10” deck was cast on top of the girder. Florida left the #5 C-shaped shear reinforcement bars extended above the girder surface. At Coreslab Structures in Omaha they were cut to size and bent to fit in the deck as shown in Figure C.4-7. The deck was then poured around these bars making the deck and girder a composite system.

Figure C.4-7. Forms for pouring the deck

#4 threaded rods were placed 2 in deep in the deck, approximately every 4 ft in alternating singles and pairs. The primary purpose of these rods was to hold the formwork together, but they were left in the concrete after the forms were removed. No additional transverse reinforcement was used in the deck.

In order to achieve high stress in the bottom prestressed strands at failure, the strength of the concrete deck was specified to be at least 10,000 psi. A self-consolidating concrete (SCC) was used, and the actual concrete strength was measured at approximately 12,510 psi.

After the deck was cured, a clamping force mechanism was placed 30” in from the end of the girder in order to simulate the bridge weight acting on the supports. This 60-kip applied clamping force was calculated as the balance between the reaction generated by the 125 ft bridge that the original production girders were being manufactured for (taking into consideration weight of girder, deck and barriers) and the reaction generated by the 42 ft long specimen (taking into consideration weight of the girder and deck). This clamping mechanism was placed only at the end being tested, which was TN2L. It was located 30” from the end of the girder. The clamping force was provided by a system of steel beams located below and above the girder connected by threaded rods, as shown in Figure C.4-8 and Figure C.4-9. The steel beams were tightened with a hydraulic jack. Table C.4-1 shows the concrete strength of the girders and deck slab at time of testing.

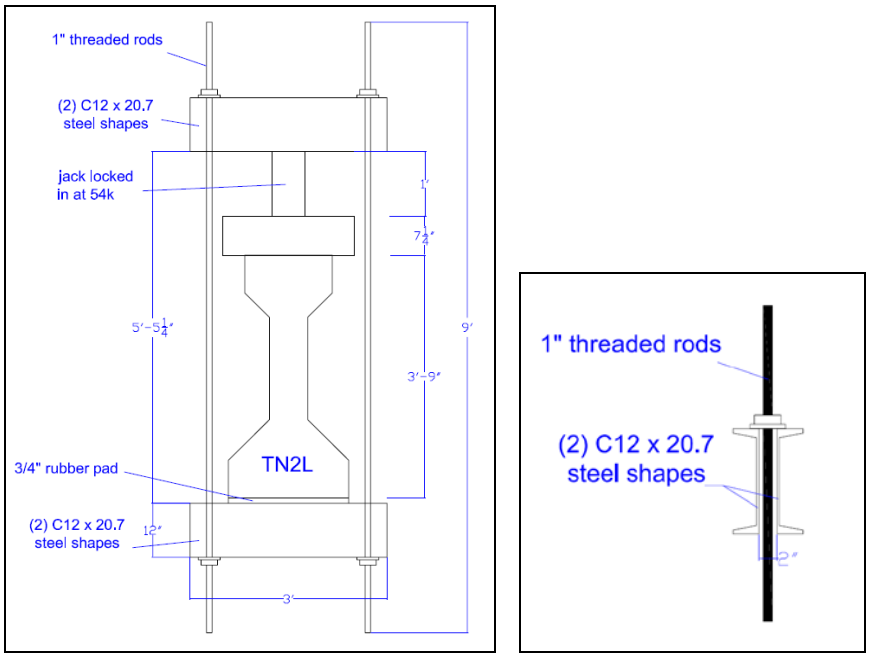


Figure C.4-8. End Clamping Detail

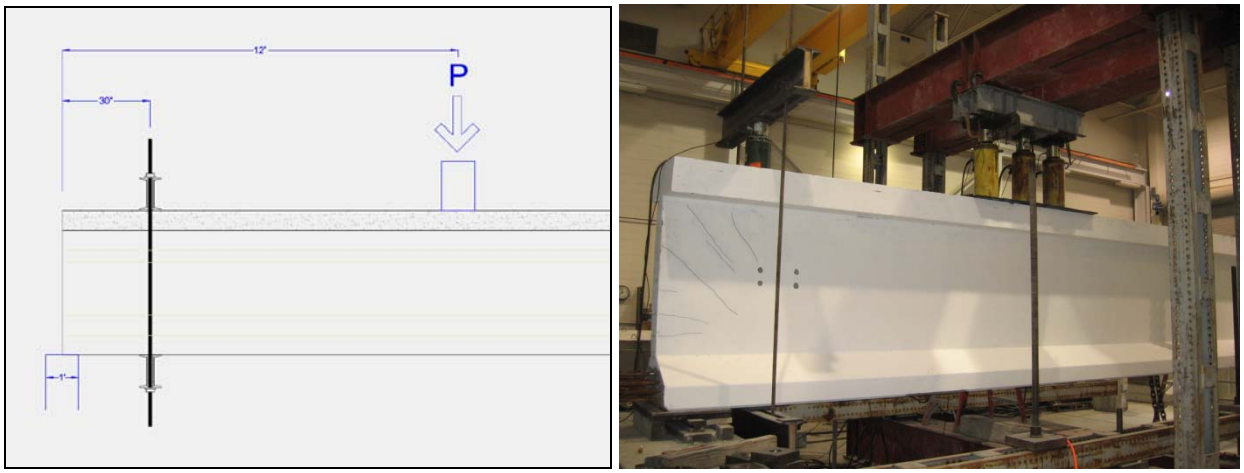


Figure C.4-9. Locations of Point Load and Clamping Mechanism

Table C.4-1. Concrete Strength

	FL Girders	FL Deck
Concrete strength at time of testing, f'_c	10,870 psi	12,510 psi

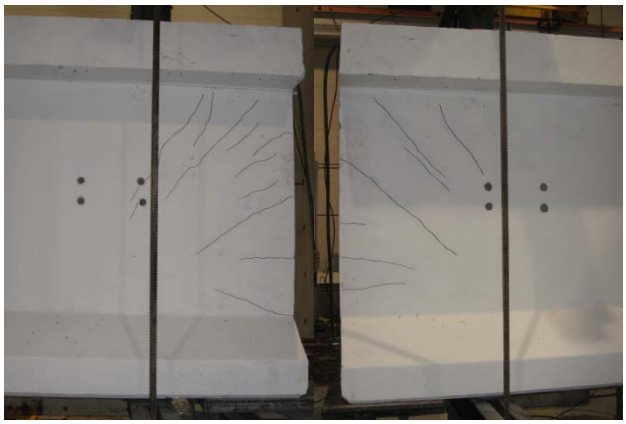
Observed End Zone Crack Patterns

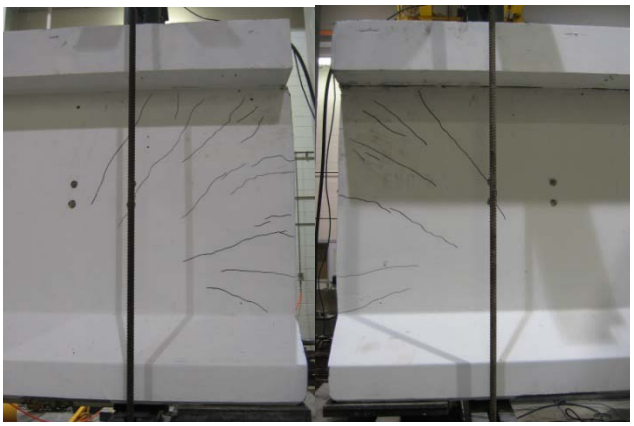
All four ends experienced end zone cracking. The longest cracks did not extend further than 3 ft in from the girder end. All cracks were around 0.004" to 0.006" in width. These crack lengths and widths are common for end zone cracks and are comparable to the other girders that have been brought into the lab. At this width, the cracks are usually not expected to be much of a problem by the precaster.

The cracks appeared as the prestressing was released at the prestressing plant in Tampa, FL. There did not appear to be much of a change in the cracks from when they were in Florida to when they arrived in Nebraska; possibly a slight lengthening of the cracks.

Since the girder was an inverted-T, there was no top flange at the time of detensioning to counteract the tensile forces at the top of the girder. Six #6 bars were placed at the top of the web to offset these tensile forces, but it was not enough. As it was produced, the beam was in violation of the AASHTO limits on top tension. However, it was decided that the forces were in control enough for this experiment. Because of this tensile force at the top of the member, there were vertical cracks running downward from the top of the web. These cracks appeared approximately every 6" or so, were 1-2 feet in length, and ran the entire length of the girder. These cracks were not a hindrance to the experiment, so they were ignored.

End FL2L appears to have a larger number of end zone cracks than End FL2R (Figure C.4-10). Both ends look similar, but there are more cracks on End FL2L and they extend further into the girder than those on End FL2R. End FL2L was designed using typical LRFD specifications while End FL2R was designed using a method that alters LRFD to place a larger percentage of the reinforcement steel closer to the girder end. In this case it appears to have done a better job at preventing cracks than the typical LRFD design.





FL2L FL2R

Figure C.4-10. Comparison of Ends FL2R and FL2L

End FL1L and FL1R experienced very similar cracking patterns and amount of cracking (Figure C.4-11). The crack widths for each were also similar. End FL1L had end reinforcement typical of Florida beams while End FL1R contained that same design of end reinforcement, but with a large number of bars removed. The cracks on End FL1L did not extend as far into the girder as the cracks on End FL1R. This would be consistent with the reinforcement details. However, there is not a big enough improvement between the two to justify using that much more steel for reinforcement.

FL1L FL1R

Figure C.4-11. Comparison of Ends FL1L and FL1R

Structural Testing and Test Results

(a) Girder FL2: End FL2R (Figure C.4-12)

The right end of the second Florida girder was designed using the method proposed by the research team. This method expands on the procedure indicated in the LRFD manual and places a greater percentage of the vertical bursting reinforcement closer to the end of the girder. As explained in the previous section 4% of the prestressing force must be within h/2 from the end of the girder while half of that (2%) must be placed within h/8 of the end of the girder. The team designed the end zone reinforcement to be five pairs of #5 bars spaced 2” apart, center to center, at the girder end. Beyond these five pairs, the typical shear reinforcement took over with pairs of #5 bars spaced every 6”. It is assumed that the precaster went back and placed the required amount of shear reinforcement in the girder, but since the girder did not fail and therefore did not break open, it cannot be certain.

End FL2R was tested in the structures laboratory in Omaha, NE on November 18, 2008. Three jacks were used to provide the maximum amount of force on the girder. Each jack had a capacity of 400k, therefore the total capacity of the setup was a point load of 1,200k. According to

calculations, the girder should have been able to withstand at least a 1,229 k point load before failing in flexure. Therefore it was expected that the loading setup would not be enough to fail the girder, but enough to cause cracking and get useful data.

Figure C.4-12. End zone cracks before testing

Load was applied to the girder in intervals of 100 k. After each additional 100 k, loading was paused so that the girder could be investigated for cracks and photographed. The cracks were traced with black marker as they appeared and denoted at which load they appeared. This process of loading and pausing continued until it was no longer safe to come near the girder. Then loading resumed at a constant rate until the point load reached 1,200 k. At this point the equipment could not apply any further load so the force was released back to zero.

Table C.4-2 gives the moment capacity and corresponding point load of the FL Specimens using the minimum specified and actual measured strength of the girder and deck.

Table C.4-2. Moment capacity and corresponding point load of the FL2R specimen

	Minimum Specified Strength	Actual Strength	Test Data (reached capacity)
Moment Capacity (M_n)	10,039 ft-k	10,317 ft-k	10,072 ft-k
Point Load (P)	1,195 k	1,229 k	1,200 k

The load that was reached was very close to, but still under, the anticipated capacity of 1,229 k. Under this much load the girder was starting to crack and show signs of approaching failure. The crack patterns after the full load was applied are shown in Figure C.4-13 and Figure C.4-14. Most of the cracks that appeared were shear in nature. Flexural cracks were small and very few, and did not start to appear until the load reached 900 k. Therefore it is assumed that the girder

would have failed in shear first before the flexural failure point was reached. This may be caused by a insufficient amount of shear reinforcement due to placement error, or it may be that the flexural failure capacity exceeds that which pairs of #5 bars at 6” spacing could provide. At the maximum loading the girder experienced a deflection of 0.946” at the location of loading. Further load upon the system would cause this deflection to increase until failure.

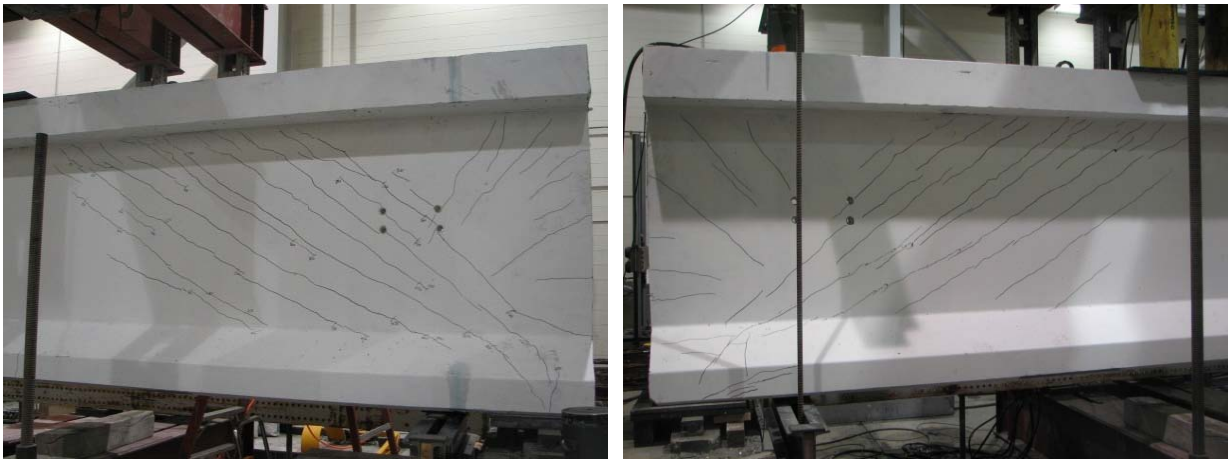


Figure C.4-13. End FL2R after loading was stopped (shear cracks)

Figure C.4-14. End FL2R after loading was stopped (flexure cracks)

The test setup did not have the capacity to cause the girder to fail, but the load cells from the test were able to record enough data to get a fine load vs. deflection curve, shown in Figure C.4-15. From the curve one can see that the girder was able to withstand the design load of 1,195 k, and would have been able to exceed this load as well if it had been subjected to a greater force. Therefore it can be stated that even though the girder experienced end zone cracks that were not repaired in any way, it still was able to exceed the required capacity. The prestressed strands were extended beyond the end of the girder and were marked before the test to pinpoint their location with respect to the girder end surface. After the test was complete these marks were inspected to see if any strand slippage occurred. This showed that there was no significant visible

change in the placement of the strands, and therefore strand slippage was not a major concern for this test, nor a reason for reduced structural capacity.

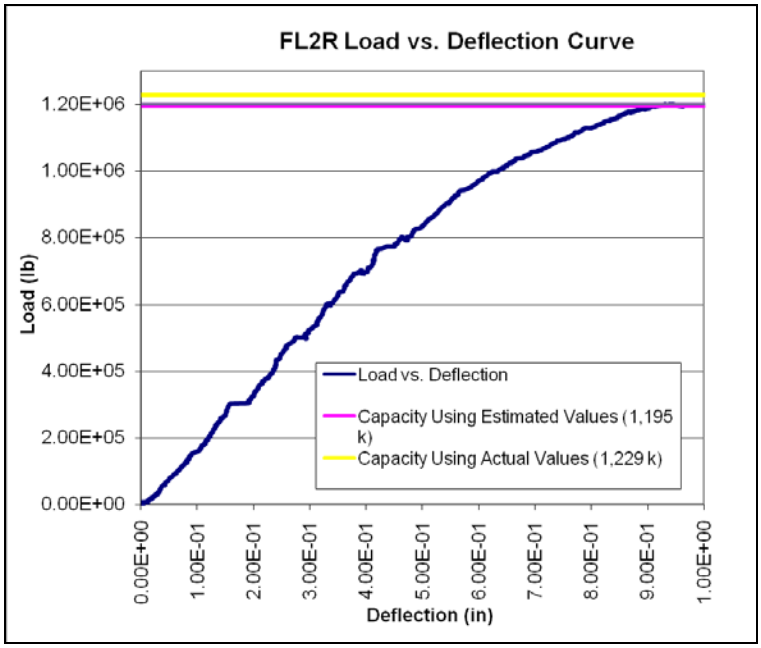


Figure C.4-15. Load Deflection Curve for End FL2R

(b) Girder FL2: End FL2L (Figure C.4-16)

End FL2L was the other end of the second Florida girder, shared with the first end tested FL2R. This end had its end zone reinforcement designed using the typical LRFD specifications. 4% of the prestressing force was placed within h/4 from the end of the girder. This translated into six pairs of #5 bars placed at the end of the girder, spaced 3” apart, center to center. After these six pairs the typical shear reinforcement resumed with pairs of #5 bars spaced every 6”. As stated above, it is not absolutely certain on this second Florida girder whether the shear reinforcement was placed as requested in pairs, or if it was left as singles placed every 6”. This variation would make a big difference on the final capacity of the girder.

End FL2L was tested on November 26, 2009 in the University of Nebraska Omaha structures lab. The girder had to be flipped around from the previous test so that the point load could be applied 12’ from End FL2L. In order to eliminate damaged portions of the girder from the test, the support on End FL2R was moved in 12 ft. The support was situated in the same location as the point load from the previous test. This left an unbraced length of 29.5’ for the second test.

Like the previous test the girder was not expected to completely fail because the designed capacity was much greater than the capacity that the hydraulic jacks provided. The three jacks were only able to exert a 1,200 k point load while the expected capacity of the girder in this setup was a point load of 1,461 k. Therefore, the load application was only expected to cause cracking and be enough to acquire load vs. deflection data.

Load was applied to the girder in intervals of 100 k. After each additional 100 k, loading was paused so that the girder could be investigated for cracks and photographed. The cracks were traced with black marker as they appeared and denoted at which load they appeared. This process of loading and pausing continued until it was no longer safe to come near the girder. Then loading resumed at a constant rate until the point load reached 1,200 k. At this point the equipment could not apply any further load so the force was released back to zero.

Figure C.4-16. End zone cracks before testing

Table C.4-3 gives the moment capacity and corresponding point load of the FL Specimens using the minimum specified and actual measured strength of the girder and deck.

Table C.4-3. Moment capacity and corresponding point load of the FL2L specimen

	Minimum Specified Strength	Actual Strength	Test Data (reached capacity)
Moment Capacity (M_n)	10,039 ft-k	10,317 ft-k	8,466 ft-k
Point Load (P)	1,421 k	1,461 k	1,200 k

As expected, the girder did not fail before reaching the maximum load of 1,200 k. It is anticipated however, that the girder would have failed in shear before reaching the calculated flexural failure load. Under the heavy point load, the girder was experiencing cracks as it approached failure. The majority of the cracks were due to shear stress, while only a few small cracks in the bottom flange were from flexural stresses. Images of both types of cracking are shown in Figure C.4-17 and Figure C.4-18. Just like in the previous test, flexural cracks in the bottom flange right below the point of loading did not appear until the load reached 900 k. Therefore, from looking at the girder and from looking at how the load vs. deflection graph tapers off, it is expected that the girder would have failed in shear before the anticipated flexural failure point of 1,461 k was reached. Once again, this may be caused by there not being enough shear reinforcement placed in the girder at the precasting plant. However, since the girder was

not broken open at this point, the team did not yet consider this as a problem. At the maximum loading, the girder had deflected 1.247” at the location of loading. When the load was released a permanent deflection of 0.278” remained.

Figure C.4-17. End FL2L after loading was stopped (shear cracks)

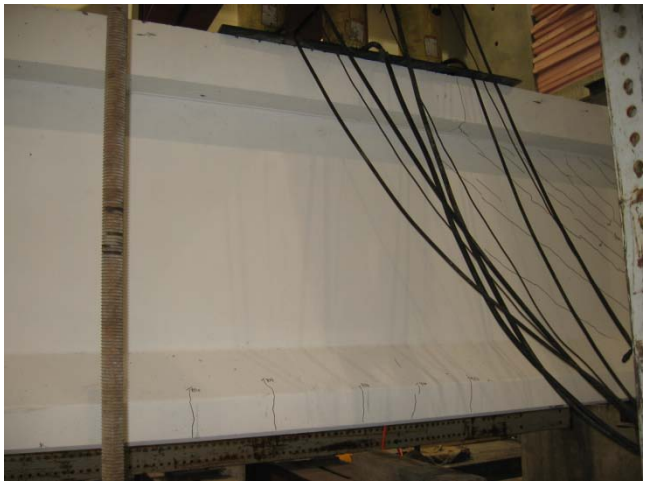


Figure C.4-18. End FL2L after loading was stopped (flexure cracks)

The load vs. deflection curve from testing End FL2L is shown in Figure C.4-19. From the curve, it can be deducted that the failure of the girder would have occurred not long after the maximum load of 1,200 k. It is assumed that this failure would have been due to shear forces. This would have caused the girder to fail before the required design load. However, it is expected that if the shear reinforcement had been increased to what had originally been designed, the girder would have withheld more load and flexural failure would have been the controlling factor once again. Even though the girder may have failed before its capacity had been reached, this failure was not due to end zone cracking. Even though the cracks were unrepaired, there was no sign that these cracks did anything to accelerate the failure of the girder. Upon close investigation, the end zone cracks did not open any during the testing, nor was there any other type of movement around them. In fact, since the primary motive for cracking during loading was shear, the forces acting

on the girder at this time would have been pushing the end zone cracks together. Looking at Figure C.4-17, it can be observed that the shear cracks caused by loading run perpendicular to the diagonal end zone cracks. From this, it can be deducted that the presence of end zone cracking on this girder did not affect the final structural capacity in any way.

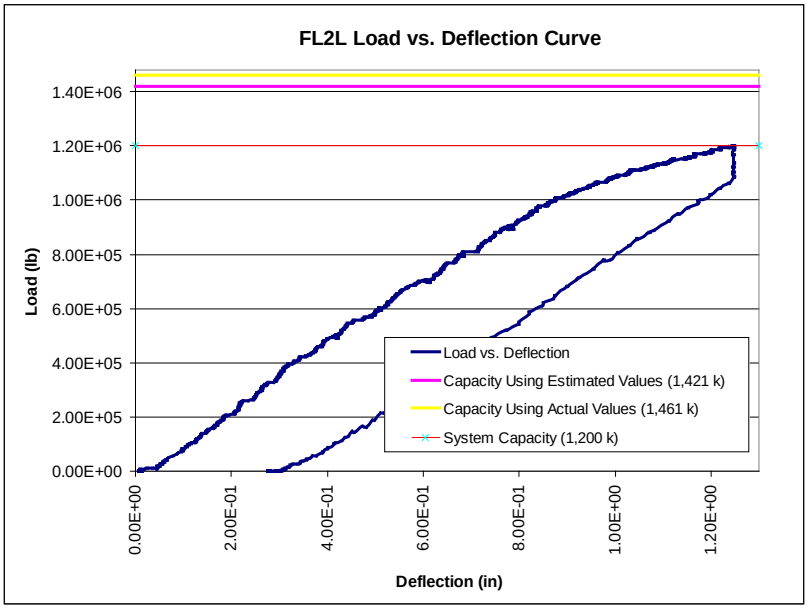


Figure C.4-19. Load Deflection Curve for End FL2L

The prestressed strands were extended beyond the end of the girder and were marked before the test to pinpoint their location with respect to the girder end surface. (Figure C.4-20) After the test was complete these marks were inspected to see if any strand slippage occurred. This showed that there was no significant visible change in the placement of the strands, and therefore strand slippage was not a major concern for this test, nor a reason for reduced structural capacity.

Figure C.4-20. Markings for strand slippage

(c) Girder FL1: End FL1L (Figure C.4-21)

The left end of the Florida girder FL1 contained end zone reinforcement consistent with that typically used in the state of Florida. It was the same end reinforcement as the production girders that these samples were modeled after. The full description of the end reinforcement is given in the section above and shown in Figure C.4-1 and Figure C.4-2. It contains #5 bars in pairs spaced every 3" and paired with #4 bars for a distance at the girder end. Typical shear reinforcement picked up where the special details ended. The design called for pairs of #5 bars spaced every 6", and when the girders arrived it was believed that this is what they contained. In the testing of End FL1L there was a sudden unexpected shear failure of the girder that blew out concrete in sections of the web, exposing the reinforced bars underneath. With the bars exposed, it was visible that the #5 bars were still spaced at 6" as requested, but were placed singly instead of in pairs. This effectively reduces the amount of shear reinforcement in half, causing the unexpected failure. This error in shear reinforcement placement was caught at the precasting company by a member of the research team. The problem was addressed and the team was assured that it would be fixed. However, an error in communication may have caused the bars to be left as they were, leading up to the premature failure of the girder.

Figure C.4-21. End zone cracks before testing

End FL1L was tested in the Omaha structures lab at the University of Nebraska on January 6, 2009. The setup show in Figure C.4-6 was used with an unbraced length of 41 ft between support centers. The clamping device was attached 30 in from the girder end. The normal three jacks were used to apply a point load to the girder, 12 ft from the end. These jacks could only apply a combined force of 1,200 k. According to the calculations of the intentional girder design, it should have been able to resist a point load of 1,229 k before failing in flexure. The girder was not expected to fail under these laboratory conditions. As in the similar testing of End Fl2R it was expected to support the 1,200 k without problem, crack, and then be unloaded.

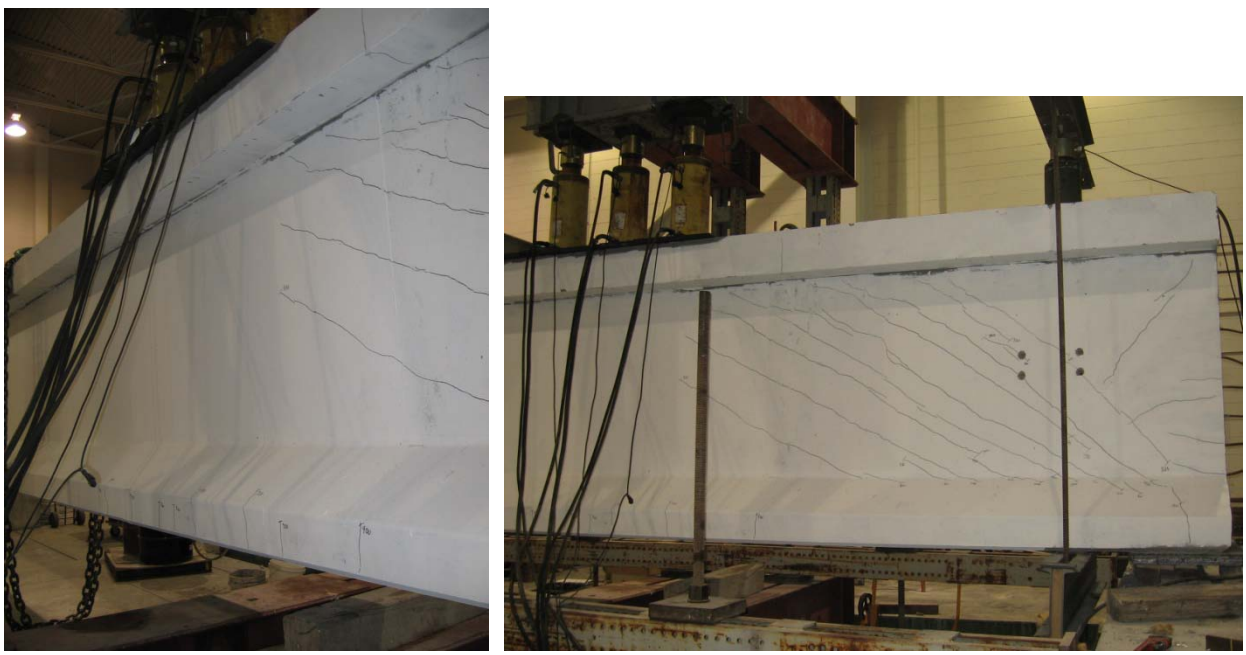
Load was applied to the system in intervals of 100 k. After each additional 100 k, loading was paused so that the girder could be investigated for cracks and the cracks could be traced with marker and photographed. This process of alternating loading and pausing continued until around 900 k when it was deemed unsafe to approach the girder any longer. Loading subsequently continues until the girder failed in shear at 1,177 k.

Table C.4-4 gives the moment capacity and corresponding point load of the FL Specimens using the minimum specified and actual measured strength of the girder and deck.

Table C.4-4. Moment capacity and corresponding point load of the FL1L specimen

	Minimum Specified Strength	Actual Strength	Test Data
Moment Capacity (M_n)	10,039 ft-k	10,317 ft-k	9,890 ft-k
Point Load (P)	1,195 k	1,229 k	1,177 k

The failure was sudden and violent. Before failure, shear cracks were appearing on both sides of the point load. A few were on the end towards FL1R, but the majority of the cracks were right after the support on the end FL1L. Figure C.4-22 shows this pattern, just before failure.



Flexural cracks Shear cracks

Figure C.4-22. End FL1L Crack Patterns just before failure

When the girder failed, pieces of concrete in the web burst from the girder, leaving a large gaping hole in the middle of the beam. Images of the girder after failure are shown in Figure C.4-23 and Figure C.4-24.

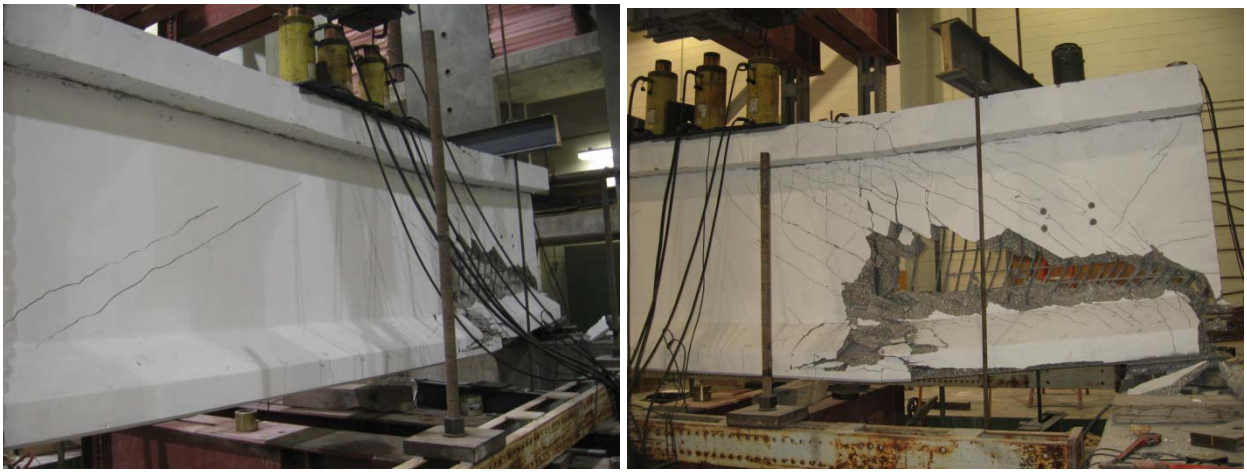


Figure C.4-23. End FL1L after failure, East Side

Figure C.4-24. End FL1L after failure, West Side

The edges of the hole corresponded with the cracks that were created during testing. This opening exposed the shear reinforcement and end zone reinforcement bars that were concealed inside the web. This was how it was discovered that there was not enough vertical shear reinforcement bars placed. When the concrete in the web was removed, there was nothing holding the reinforcing strands in the bottom flange in place. They attempted to retract, pulling the entire bottom flange with them. This phenomenon is shown in Figure C.4-24. The red lines indicate the edge surface of the web and of the bottom flange and how far they have been moved apart from one another. Upon viewing the failed girder from the side, one can see that the bottom girder has a much sharper angle of bend than the top girder. As the section of the bottom flange moved inward, the end of it that was closest to the girder end moved upward, tilting that segment of bottom flange. All of this movement was strong enough to bend and warp the vertical shear reinforcement bars. This bending is shown in Figure C.4-25 where the bars are clearly bent and pulled diagonal.

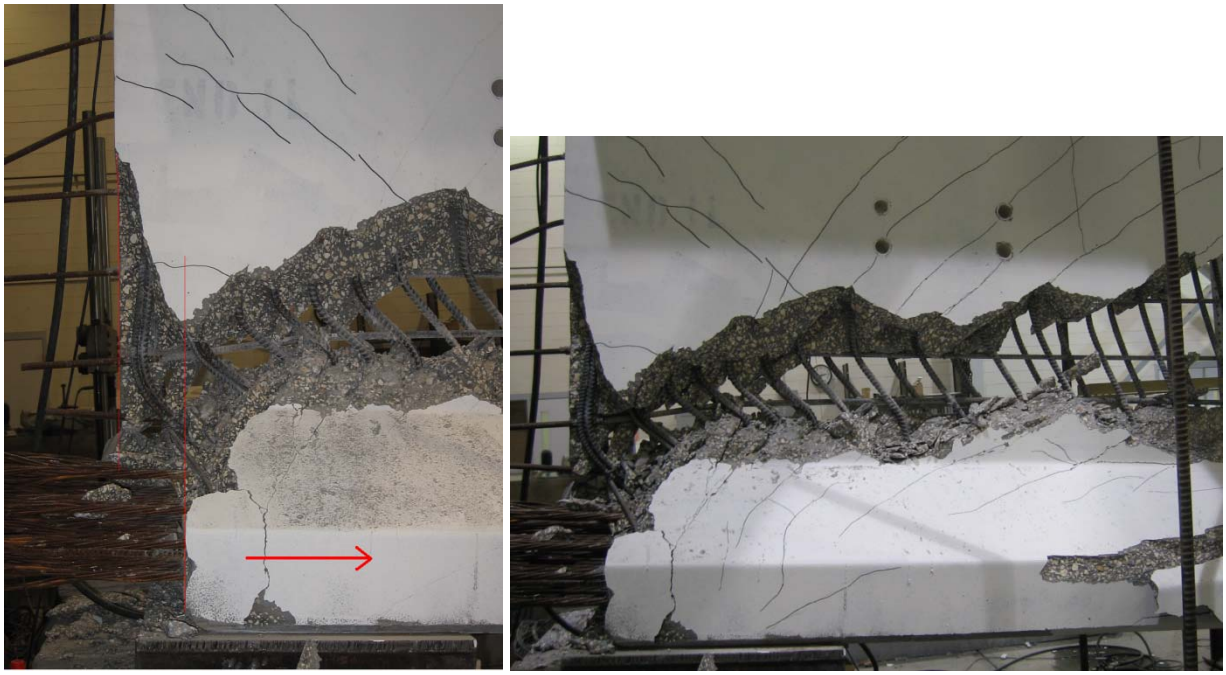


Figure C.4-25. Retraction of bottom flange and bending of shear reinforcement bars

The point of failure of this end was slightly close to the anticipated capacity of 1,229 k. However the capacity was not reached, and the method of failure was different than what was expected. All of the cracks that appeared before failure were shear in nature; there were no flexural cracks. It is now known that this was caused by the lack of adequate shear reinforcement. It is estimated that if there had been sufficient shear reinforcement then the girder would have behaved more predictably and flexural cracks would have been present.

At the highest load that the girder carried, right before failure, the beam was deflected 1.10” at the point of loading. The maximum deflection that was recorded was 5.33”, right after failure. When the load was removed the girder lifted slightly leaving what was left of the girder a permanent deflection of 4.47”

The load vs. deflection curve from testing End FL1L is shown in Figure C.4-26. The curve shows the gradual tapering off of the load just before failure. The girder failed to reach the required design capacity, but as explained earlier, this was due to insufficient shear reinforcement. If the correct amount of vertical bars had been implemented it is assumed that the girder would have been able to exceed capacity. The failure of the girder was not affected by the end zone cracks that were present. The cracks were unchanged throughout the testing, so there was no width growth nor slipping. The method of failure was shear, and the shear cracks that appeared were perpendicular to the already present diagonal end zone cracks. This shows that the forces experienced by the girder during testing were working in opposition to the forces that caused the end zone cracks. This force would have been working to close these end cracks. Therefore, the presence of end zone cracks in the Florida beam did not advert affect the structural capacity of the girder.

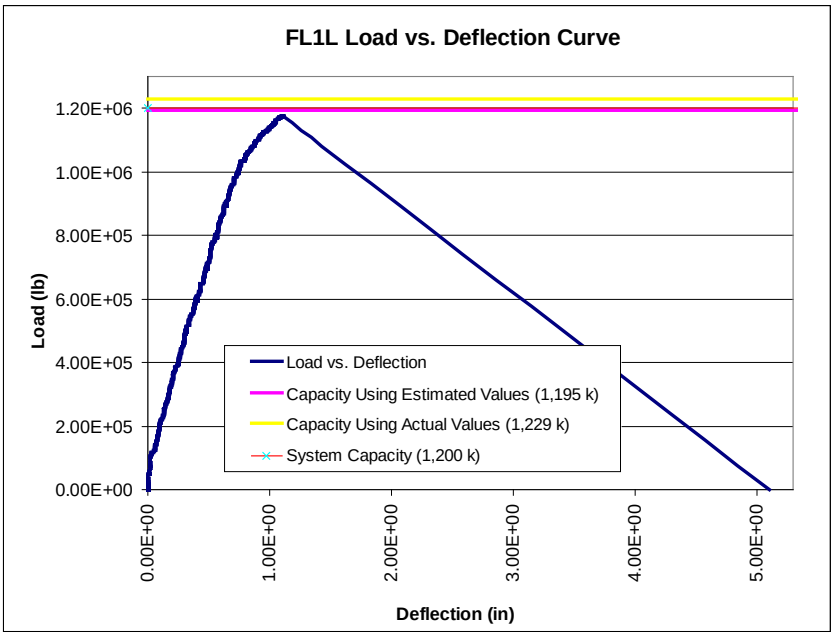


Figure C.4-26. Load Deflection Curve for End FL1L

(d) Girder FL1: End FL1R (Figure C.4-27)

End FL1R was on the end opposite to FL1L on the first Florida girder. Like FL1L, the end reinforcement for this section was similar to that typically used by Florida. Design began with the end reinforcement used on the production girders that Standard Concrete Products were originally manufacturing. The details were then altered to provide less steel than End FL1L contained. The L-shaped #5 bars and the unbent #4 bars were removed, leaving only the #5 C-shaped bars. As discovered in the previous test, girder FL1 was shown to only have half the amount of shear reinforcement as designed. There were single #5 bars placed every 6" instead of pairs of #5 bars every 6". Since this was true for the left end of the girder, it would be true for the right end as well. This alteration will reduce the expected capacity of the girder.

End FL2R was the last girder tested in the structures lab in Omaha, NE. It was tested on January 12, 2009. The girder had to be flipped around after the previous test. This way the jacks on the frame were positioned 12 ft from the end of FL2R. The supports under End FL2L were also moved in 12 ft so that there were little to no damaged sections in the unbraced length of 29.5 ft. The clamping device was moved to this end as well and placed 30 in from the end of FL2R.

Since the same setup was used, the maximum load that could be achieved was still 1,200 k. Caution was used with this loading since the test on the opposite end caused failure. Calculations were performed on the girder with the actual reduced shear reinforcement, and it was concluded that with the reduced length, the girder would still most likely not fail before 1,200 k. The testing continued, and load was applied to the system in intervals of 100 k. After each 100 k, the loading was stopped so the girder could be viewed closely and marked for cracks. Photographs were taken at each interval so that the progression of cracks and crack lengths could be monitored throughout the test. This process of loading and pausing continued until it was likely not safe to

approach the beam. Then the loading continued at a constant speed until the maximum load of 1,200 k was reached. At this point the girder had not yet reached failure, but no further load could be applied, so the jacks were released and the girder was unloaded (Figure C.4-28).

Figure C.4-27. End zone cracks before testing

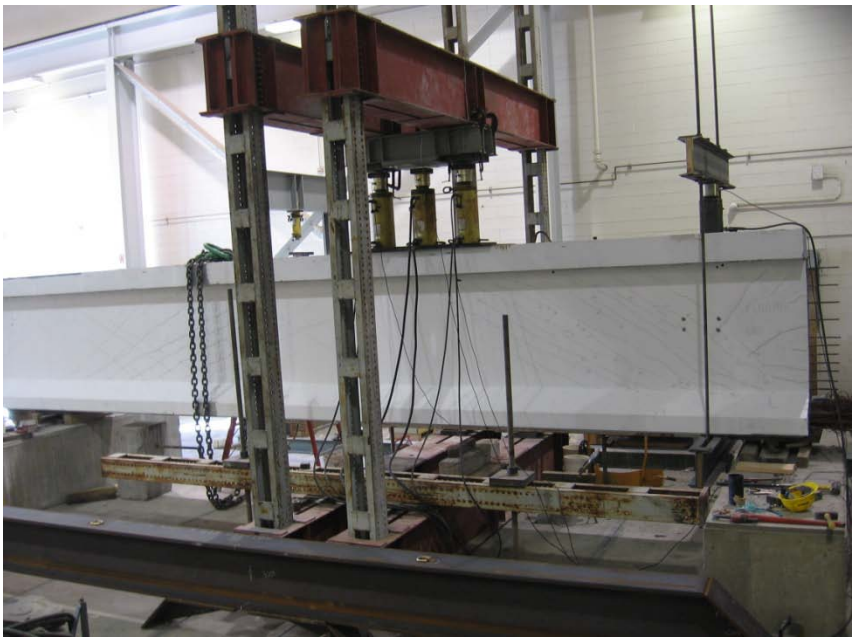


Figure C.4-28. Full girder after testing was complete; showing shear cracks

Table C.4-5 gives the moment capacity and corresponding point load of the FL Specimens using the minimum specified and actual measured strength of the girder and deck.

Table C.4-5. Moment capacity and corresponding point load of the FL1R specimen

	Minimum Specified Strength	Actual Strength	Test Data (reached capacity)
Moment Capacity (M_n)	10,039 ft-k	10,317 ft-k	8,494 ft-k
Point Load (P)	1,421 k	1,461 k	1,201 k

Consistent with calculations, the girder did not fail before reaching the load of 1,200 k. The beam would have been able to take on more load if it had been available. The loading did cause the girder to crack as it approached failure. All of the cracks that appeared were due to shear forces. The shear cracks appeared on both sides of the loading near each support. The crack patterns of each side are shown in Figure C.4-28, Figure C.4-29, and Figure C.4-30.

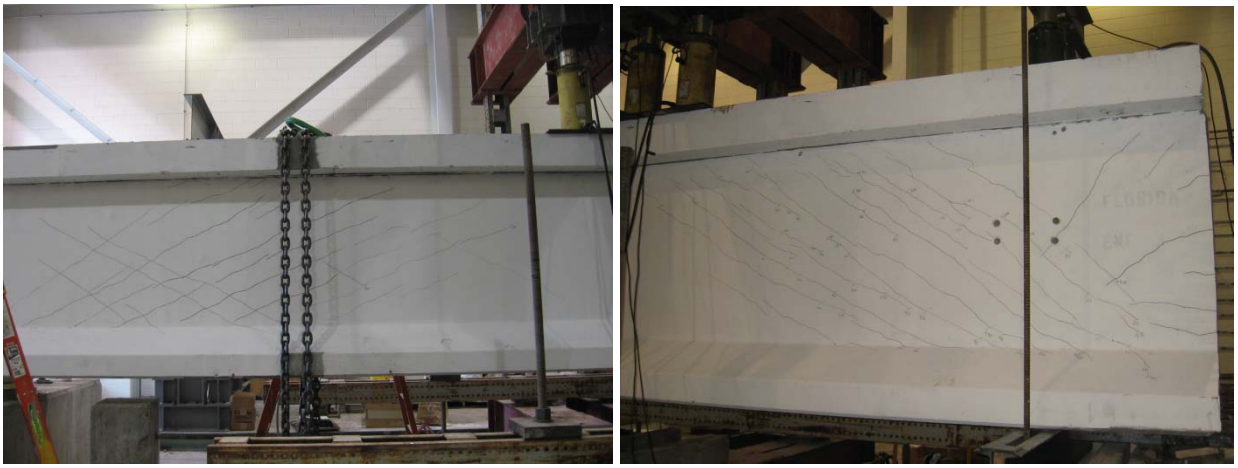


Figure C.4-29. East Side of End FL1R after loading was stopped (shear cracks)

Figure C.4-30. West Side of End FL1R after loading was stopped (shear cracks)

The shear cracks that appeared by the support that was moved in towards the center of the beam crossed with the shear cracks from the previous test of End Fl1L. The cracks were perpendicular

to one another, so it was assumed that they did not noticeably influence the test results. The presence of shear cracks was consistent with what was expected from insufficient shear reinforcement. It is expected, that when the girder would fail, it would be in shear, similar to the previous test of End FL1L. At the maximum load, the girder had deflected 0.758" at the location of loading. When the load was released, there was still 0.235" of permanent deflection remaining.

The load vs. deflection graph from testing End FL2L is shown in Figure C.4-31. It is clearly shown that the curve had not yet started to taper off, and that there was quite a bit of capacity left in the girder before failure. It is not shown whether the girder would reach the intended capacity of 1,461 k or not, but with the reduced shear reinforcement it is not likely. However, this possible failure to perform to specifications is not the fault of the end zone reinforcement, but of the shear reinforcement. The end zone cracks were not altered or opened in any way during testing. In fact, the diagonal shear cracks that appeared were perpendicular to the diagonal end zone cracks. This shows that the forces on the member were acting in a direction that would push the end zone cracks together, not apart (Figure C.4-29 and Figure C.4-30). Therefore, the unrepaired end zone cracks on the Florida specimen did not negatively affect the structural capacity of the girder.

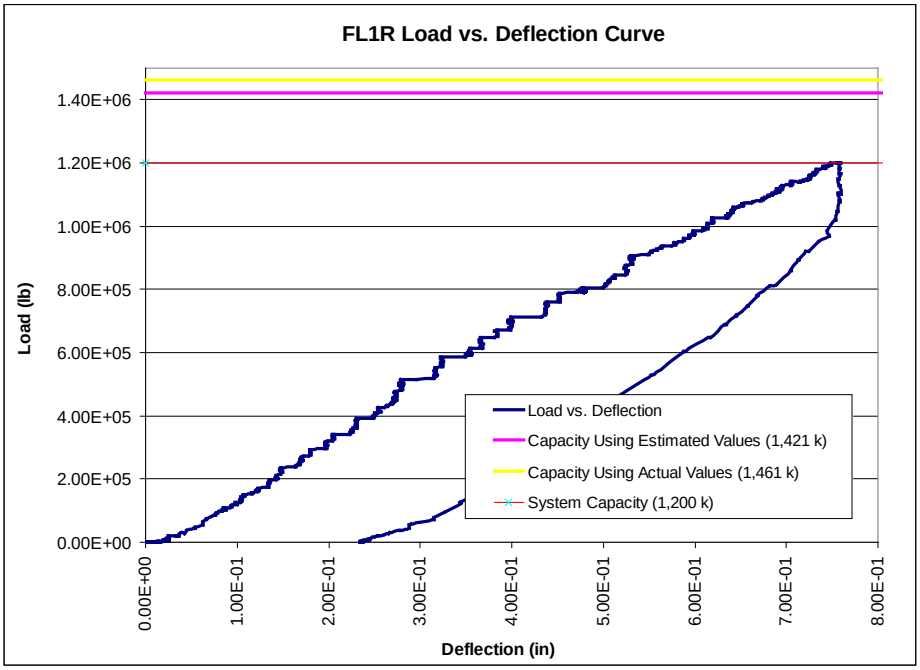


Figure C.4-31. Load Deflection Curve for End FL1R

The prestressing strands on the girder extended out each end of the bottom flange where they were torched off. Before testing, black marker was used to mark the location of each strand with respect to the girder surface. After testing, investigation of these marks showed that the strands did not visibly slip into the girder. Therefore strand slippage was not a major component of the test.