DEPARTMENT OF TRANSPORTATION

BR27568 – Experimental Shear Capacity Comparison Between Repaired and Unrepaired Girder Ends

Carol Shield, Principal Investigator
Department of Civil, Environmental, and Geo- Engineering
University of Minnesota

February 2018

Research Project
Final Report 2018-07

To request this document in an alternative format, such as braille or large print, call 651-366-4718 or 1-800-657-3774 (Greater Minnesota) or email your request to ADArequest.dot@state.mn.us. Please request at least one week in advance.

Technical Report Documentation Page

1. Report No.	2.	3. Recipients Accession No.
MN/RC 2018-07		

4. Title and Subtitle	5. Report Date
BR27568 – Experimental Shear Capacity Comparison Between Repaired and Unrepaired Girder Ends	February 2018
	6.
7. Author(s) Carol Shield, Paul Bergson	8. Performing Organization Report No.
9. Performing Organization Name and Address Civil, Environmental and Geo- Engineering University of Minnesota 500 Pillsbury Drive SE Minneapolis, MN 55455	10. Project/Task/Work Unit No. CTS #2017054 11. Contract (C) or Grant (G) No. (c) 1003325 (wo) 19
12. Sponsoring Organization Name and Address Minnesota Department of Transportation Research Services & Library 395 John Ireland Boulevard, MS 330 St. Paul, Minnesota 55155-1899	13. Type of Report and Period Covered Final Report 14. Sponsoring Agency Code

15. Supplementary Notes

http://mndot.gov/research/reports/2018/201807.pdf

16. Abstract (Limit: 250 words)

Over time, the southbound exterior girder ends on each side of Pier 4 and Pier 26 of Bridge 27568 suffered significant corrosion damage that exposed transverse reinforcement, prestressing strands in the exterior side of the bottom flange and the sole plate anchorages. The girder ends were repaired in 2013 by encasing supplementary steel reinforcement in shotcrete over a 4 ft. length of the girder. The two repaired girders and two companion girders, removed when the bridge was replaced in 2017, were brought to the University of Minnesota and tested to failure in shear to determine the effectiveness of the repair. The laboratory testing showed that the repair was able to return the girders with significant corrosion damage to the strength of the companion girders, indicating that the repair was effective.

17. Document Analysis/Descriptors	18. Availability Statement
Prestressed concrete bridges, Girders, Repairing, Shear strength, Shotcrete	No restrictions. Document available from: National Technical Information Services, Alexandria, Virginia 22312

19. Security Class (this report)	20. Security Class (this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	144	

BR27568 – Experimental Shear Capacity Comparison Between Repaired and Unrepaired Girder Ends

FINAL REPORT

Prepared by:

Carol Shield
Paul Bergson
Department of Civil, Environmental, and Geo- Engineering
University of Minnesota

February 2018

Published by:

Minnesota Department of Transportation
Research Services & Library
395 John Ireland Boulevard, MS 330
St. Paul, Minnesota 55155-1899

ACKNOWLEDGMENTS

Funding for this research was provided by the Minnesota Department of Transportation (MnDOT) and it is gratefully acknowledged. This research would not have been possible without the expertise of members of the MnDOT technical advisory panel (TAP); their knowledge and assistance were appreciated.

Materials, advice, and support were also provided by the following groups: Advance Shoring and the Cement Masons, Plasterers & Shophands Local No. 633 union.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

Minnesota Department of Transportation (MnDOT) Bridge 27568 over the Nine-Mile Creek was removed from service in early 2017. The bridge, constructed in 1975 using Type 45 prestressed concrete girders, consisted of forty-nine 60 ft. long spans, eight-girder lines, and expansion joints every 120 ft. Over time, the southbound exterior girder ends on each side of Pier 4 and Pier 26 of Bridge 27568 suffered significant corrosion damage that exposed the transverse reinforcement, prestressing strands on the exterior side of the bottom flange and the sole plate anchorages at the end of the girder. The corrosion damage was due to failed seals in the expansion joints. Girder ends on Pier 4 and Pier 26 were repaired in the Fall of 2013 by encasing supplemental steel reinforcement in shotcrete over a 4 ft. length of the girder. This research project was executed to determine if the repair was sufficient to restore the shear strength of the damaged girder to that of undamaged companion girders in the same bridge. Load testing in shear to failure was conducted on four girder ends that were removed from the bridge, two with the repair, and two undamaged ends without the repair. The purpose of the testing was to compare the failure load between the repaired ends and the unrepaired ends and to observe the failure mode.

The two companion girders were chosen so that each repaired girder had a companion girder that had been cast on the same prestressing bed at the same time using concrete batched with the same materials. Choosing companion girders in this way minimized the effect that different concrete strengths might have on the girder strengths. The two repaired girder ends and two companion girder ends were removed during the demolition of BR27568 and cut down to a length of approximately 37.5 ft. for the purpose of testing the girder ends to failure in shear. During the removal process, the existing deck located away from the repair was gently removed to the surface of the top flange to expose the top loops of the transverse reinforcement. After the girders were delivered to the University of Minnesota Galambos Structural Engineering Structures Laboratory, a concrete deck was cast on the girders to restore their flexural capacity. The girders were tested to failure in shear after the concrete deck achieved sufficient strength.

The repaired and companion unrepaired girder in each set failed at similar loads, with the repaired girder of each pair failing at load at least 1% larger than the unrepaired companion girder. Upon completion of the tests, minimal new cracking was observed in the repaired sections. The repaired section did not separate in any way from the girder during the testing.

From the close comparison in failure loads between the repaired and unrepaired girder in each set, it was concluded that the shotcrete repair investigated in this study returned the girder strength to that of unrepaired girders made at the same time on the same bed using the same concrete.

The ability to effectively repair corrosion damaged girder ends extends the useful life of prestressed concrete bridges. These repairs are significantly less expensive than replacing isolated beams and associated bridge deck or replacing the bridge altogether. Repairing damaged beams without substantial deck removal also minimizes traffic interruptions associated with more extensive repair or replacement options. Experimentally demonstrating that the repair restores the girders up to the design strength

enhances the safety of the bridge and provides MnDOT with a documented substantiated repair method that can be applied to other damaged prestressed concrete girder ends.

CHAPTER 1: INTRODUCTION

Over time the southbound exterior girder ends on each side of Pier 4 and Pier 26 of Bridge 27568 suffered significant corrosion damage that exposed transverse reinforcement, exterior flange prestressing strands and the sole plate anchorages. A history of the bridge and condition is provided by MnDOT in Appendix A. Engineers from the MnDOT Bridge Office designed a reinforced shotcrete based repair for the most heavily damaged ends, which was applied in the fall of 2013. An opportunity to test the efficacy of the repair presented itself when the bridge was scheduled for replacement in 2017. Four girders, two with repaired ends and two that had not needed to be repaired were salvaged from the bridge to be tested at the Galambos Structural Engineering Laboratory at the University of Minnesota.

Figure 1.1 shows the detail in elevation and Figure 1.2 shows the detail in section for the MnDOT designed repair that was applied to the girders that were tested as a part of this report. The repair consisted of the following four steps:

1. Sounding the concrete surface with hammers to locate the hollow sounding areas
2. Removing the delaminated concrete
3. Supplementing the corroded reinforcement over a 4 ft. length of the beam
4. Encasing the supplemental reinforcement inside a block of shotcrete

Figures 1.3 through 1.5 show the fascia girder end at Pier 26 after all loose concrete had been removed, but prior to the placement of the shotcrete. Figures 1.6 through 1.10 show the repair process and final repair on the fascia girder at Pier 26.

Four intact girders were removed from Bridge 27568 during demolition in February 2017. Each girder was cut to a length of approximately 37.5 ft. and had the demolition contractor carefully remove the deck while keeping the transverse reinforcement intact. The two girders removed from the north end of the bridge were labeled P24 and P26. These two girders were cast on the same bed at the same time, so it was anticipated that the concrete strengths for the two girders would be similar. Girder P26 had a repaired end, but Girder P24 did not. Two other girders were removed from the south end of the bridge, labeled P2 and P4. Similar to Girders P24 and P26, Girders P2 and P4 were cast at the same time on the same casting bed. Girder P26 had a repaired end, but Girder P24 did not. The labeling scheme for the girders was such that the number represented the pier on which the girder end to be tested was resting (i.e., Piers 2, 4, 24, and 26).

The four girder ends were brought to the University of Minnesota to be load tested to failure in shear. The remainder of this report documents the as-received condition of the four girder ends and the testing of those girder ends.

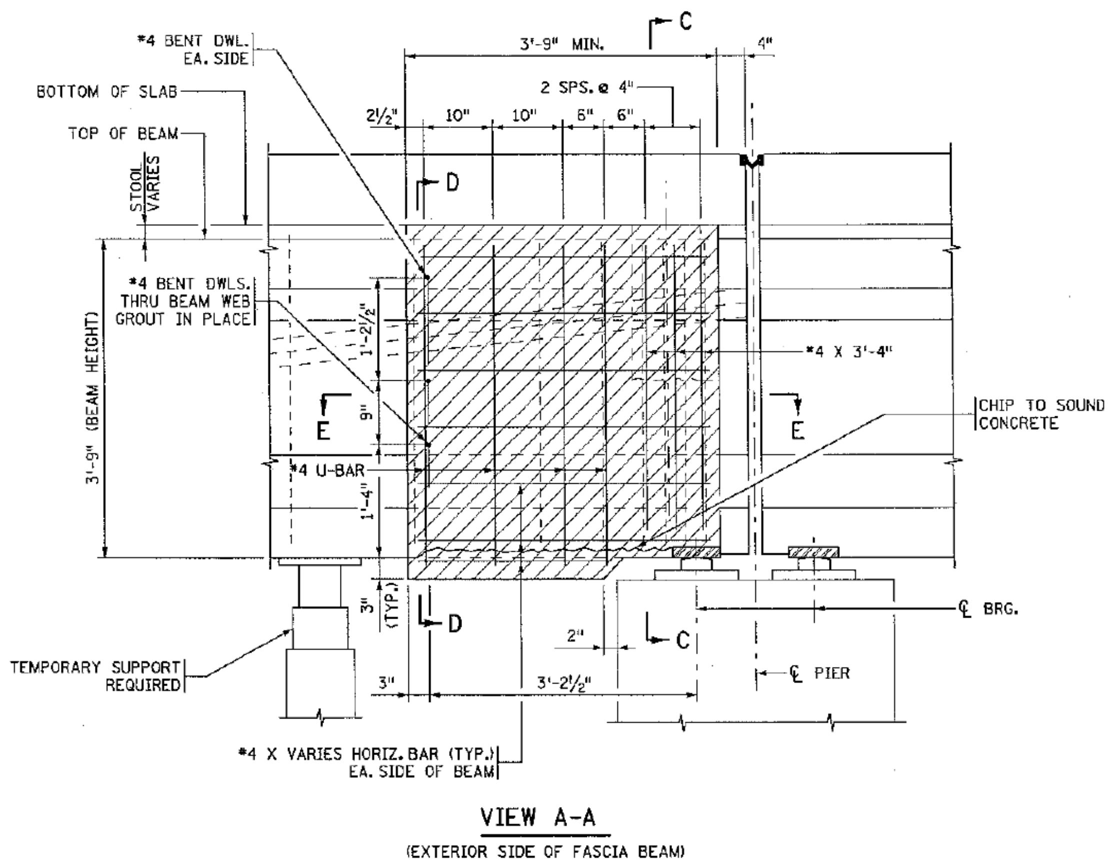


Figure 1.1 Repair – Elevation

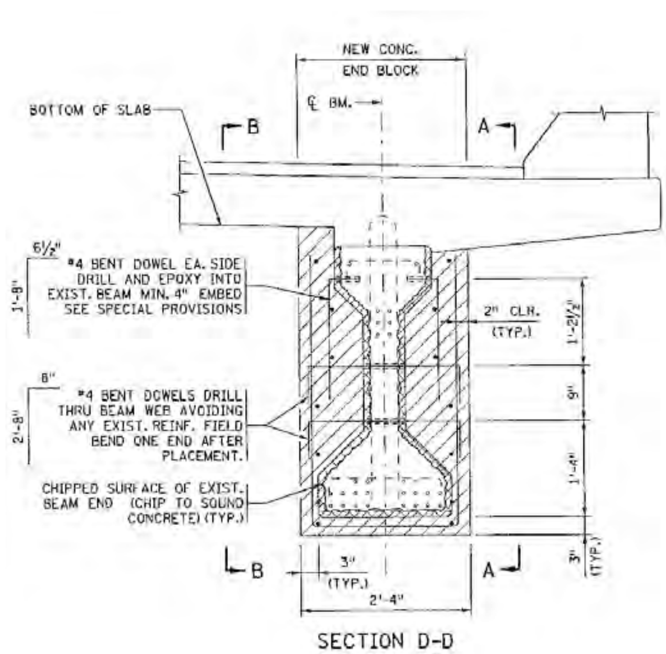


Figure 1.2 Repair – Section

Figure 1.3 Severely corroded girder at Pier 26

Figure 1.4 Damaged concrete and transverse reinforcement in girder at Pier 26

Figure 1.5 Exposed strand in girder at Pier 26

Figure 1.6 Supplemental reinforcement in place on external side of girder at Pier 26

Figure 1.7 Supplemental reinforcement in place on interior side of girder at Pier 26

Figure 1.8 Shotcreting of repair on girder at Pier 26

Figure 1.9 Shotcreting the bottom surface of the repair on girder at Pier 26

Figure 1.10 Finished repair on girder at Pier 26

CHAPTER 2: AS-RECEIVED CONDITION OF GIRDER ENDS

The four girder ends were delivered to the Galambos Structural Engineering Laboratory at the University of Minnesota in April 2017. Girders P2 and P4 (from the south end of the bridge) were received on April 11, 2017. Girders P24 and P26 (from the north end of the bridge) were received on April 13, 2017. In addition, four stacked neoprene bearings from Bridge 27568 were also delivered to the laboratory.

After the four girders were received, the stirrup locations were marked on the sides of the girders. The girder length, visible stirrup locations, stirrup condition, existing cracks and other damage were documented. Photographs of each girder were taken. Table 2.1 provides the cut lengths of each girder. Tables 2.2 through 2.5 document the location of the transverse reinforcement as measured from the end of interest, spacing of the transverse reinforcement, and visible condition of the transverse reinforcement. All girders had small amounts of residual deck concrete that required removal prior to deck casting and some minor concrete spalling in the top flange as shown in Figure 2.1. The thickness of the deck in the repaired region on Girders P4 and P26, which was left intact by the contractor was 12 in.

There was very little damage to the protruding loops of the transverse reinforcement in girders P4, P24, and P26. However, there were two transverse reinforcement loops missing and two broken in girder P2 as well as some bent loops, as shown in Figure 2.2.

On the cut-end of girders P2, P4, and P26 a horizontal crack was evident between the web and bottom flange that varied in length from 12 in. to 15 in. Figure 2.3 shows a typical representation of the crack location and extent.

Figure 2.4 illustrates the cracking at the cut-end of Girder P24. The horizontal crack located in the web about 6 in. above the bottom flange was about 18 in. in length. In addition, there were two vertical cracks that began in the web and extended through the top flange and one inclined crack near the cut-end that began about 8 in. above the bottom flange and extended through top flange. Girder P24 also has some minor concrete degradation and visible transverse reinforcement corrosion on the end of interest as shown in Figure 2.5.

Girder P4 was damaged on the repaired end (end of interest) on the diaphragm side (interior) which caused spalling of a portion of the repair and exposed horizontal reinforcement used in the repair as shown in Figure 2.6. The figure also shows a cold joint between the repair and the diaphragm. Figure 2.7 shows the cracking on the exterior side of the repair. Figure 2.8 shows an end view of the repaired end of Girder P4.

Girder P26 had a clean cut of the diaphragm on the repaired end and no visible cracks as shown in Figure 2.9. The figure also shows a cold joint between the repair and the diaphragm. Figure 2.10 shows the as-received cracking on the exterior side or the repair. Figure 2.11 is an end view of the repaired end of Girder P26, showing a small honeycomb void.

All four girders had a horizontal crack that varied in length from 12 in. to 24 in. just above the bottom flange at the diaphragm connection located 19 ft.-8 in. from the end, as shown in Figure 2.12. The crack ran through the two holes for the 3/4 in. diameter threaded rod diaphragm connection. Additional photographs documenting the entirety of one side of each girder can be found in Appendix B.

MnDOT engineers arranged for 1.65 in. diameter cores 3.2 in. long to be taken from the ends of the girders that were cut off by the contractor prior to disposal. Compression tests were performed on these cores by MnDOT staff. The measured compressive strengths for the for girders are shown in Table 2.6

Table 2.1 Cut Girder Lengths

Girder	Cut Length
P2	37’ 6”
P4	37’ 8”
P24	37’ 10.5”
P26	37’ 11”

Table 2.2 Location and Condition of Visible Stirrups in Girder P2

Location of Visible Stirrups as Measured from Girder End (in)	Stirrup Spacing (in)	Condition of Exposed Loop
6.25		ok
8.75	2.50	ok
10.50	1.75	ok
16.75	6.25	ok
26.00	9.25	ok
34.25	8.25	broken
52.50	18.25	ok
70.50	18.00	ok
89.00	18.50	ok
107.50	18.50	ok
126.50	19.00	ok
143.25	16.75	broken
161.00	17.75	ok
179.00	18.00	bent
197.00	18.00	ok
214.50	17.50	ok
232.75	18.25	ok
251.00	18.25	ok
269.50	18.50	bent
286.75	17.25	hoop missing
306.00	19.25	hoop missing
323.50	17.50	bent
342.50	19.00	ok
360.00	17.50	ok
378.50	18.50	bent
399.00	20.50	ok
415.75	16.75	ok
434.75	19.00	ok

Table 2.3 Location and Condition of Visible Stirrups in Girder P4

Location of Visible Stirrups as Measured from Girder End (in)	Stirrup Spacing (in)	Condition of Exposed Loop
51.50		ok
70.00	18.50	ok
87.75	17.75	bent
105.50	17.75	bent
123.50	18.00	ok
141.25	17.75	ok
159.25	18.00	ok
177.50	18.25	ok
195.75	18.25	ok
213.50	17.75	ok
231.25	17.75	ok
249.00	17.75	bent
267.00	18.00	ok
285.75	18.75	ok
303.50	17.75	ok
321.50	18.00	ok
339.00	17.50	ok
357.50	18.50	ok
376.50	19.00	ok
394.00	17.50	bent
412.75	18.75	ok
431.00	18.25	ok
449.25	18.25	ok

Table 2.4 Location and Condition of Visible Stirrups in Girder P24

Location of Visible Stirrups as Measured from Girder End (in)	Stirrup Spacing (in)	Condition of Exposed Loop
5.25		bent
7.00	1.75	ok
13.00	6.00	ok
16.50	3.50	ok
27.25	10.75	ok
34.50	7.25	ok
52.50	18.00	ok
71.00	18.50	ok
88.75	17.75	ok
106.00	17.25	ok
125.50	19.50	ok
142.00	16.50	ok
159.75	17.75	ok
177.50	17.75	ok
196.25	18.75	ok
213.50	17.25	ok
231.75	18.25	ok
250.50	18.75	ok
267.75	17.25	ok
286.00	18.25	ok
304.00	18.00	ok
322.00	18.00	ok
339.00	17.00	ok but with loop
356.00	17.00	ok but with loop
368.50	12.50	ok but with loop
381.50	13.00	ok but with loop
395.25	13.75	ok
414.50	19.25	ok
432.50	18.00	ok
450.25	17.75	ok

Table 2.5 Location and Condition of Visible Stirrups in Girder P26

Location of Visible Stirrups as Measured from Girder End (in)	Stirrup Spacing (in)	Condition of Exposed Loop
69.00		ok
87.75	18.75	bent
105.50	17.75	ok
123.50	18.00	ok
141.00	17.50	ok
159.50	18.50	ok
177.50	18.00	ok
195.50	18.00	ok
213.00	17.50	bent
232.00	19.00	ok
250.00	18.00	ok
267.00	17.00	bent
285.50	18.50	bent
304.00	18.50	ok
322.50	18.50	bent
342.50	20.00	bent
362.25	19.75	ok with loop
378.00	15.75	ok
396.50	18.50	bent
413.00	16.50	ok
431.75	18.75	ok
449.75	18.00	ok

Table 2.6 Girder Core Compression Strengths

Specimen	Compressive Strength (psi)
Core 1 Girder P2	5172
Core 2 Girder P2	5610
Core 1 Girder P4	5400
Core 2 Girder P4	5419
Core 1 Girder P24	7043
Core 2 Girder P24	6351
Core 1 Girder P26	7408
Core 2 Girder P26	6215

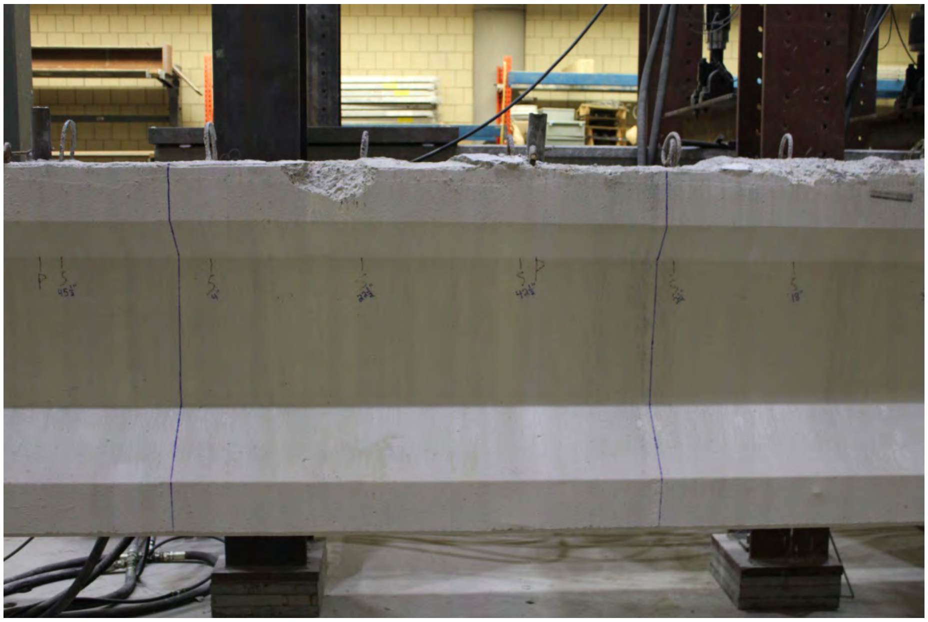


Figure 2.1 Typical top flange condition after slab removal

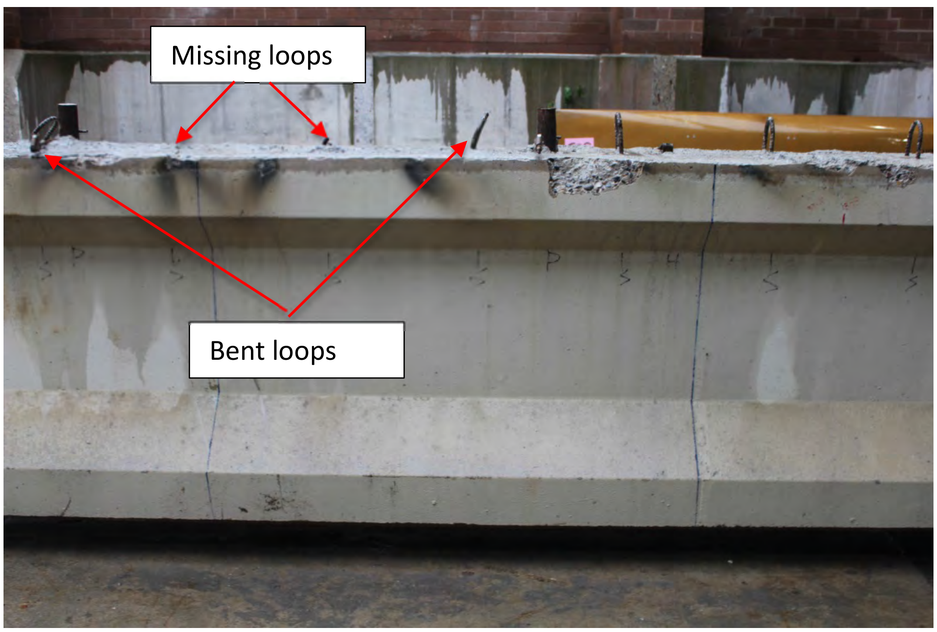


Figure 2.2 Damage to protruding part of transverse reinforcement Girder P2

Figure 2.3 Typical cut-end horizontal crack in Girders P2, P4, and P26 (marked in red)

Figure 2.4 Girder P24 cut-end cracking

Figure 2.5 Corrosion of transverse reinforcement at the end of interest on Girder P24

Figure 2.6 Damage to the repair on the diaphragm side of Girder P4

Figure 2.7 Girder P4 as-received cracking on the exterior side at the repaired end

Figure 2.8 End view of Girder P4 (repaired end)

Figure 2.9 Girder P26 on diaphragm side

Figure 2.10 As-received cracking on Girder P26 on exterior side

Figure 2.11 Girder P26 end condition

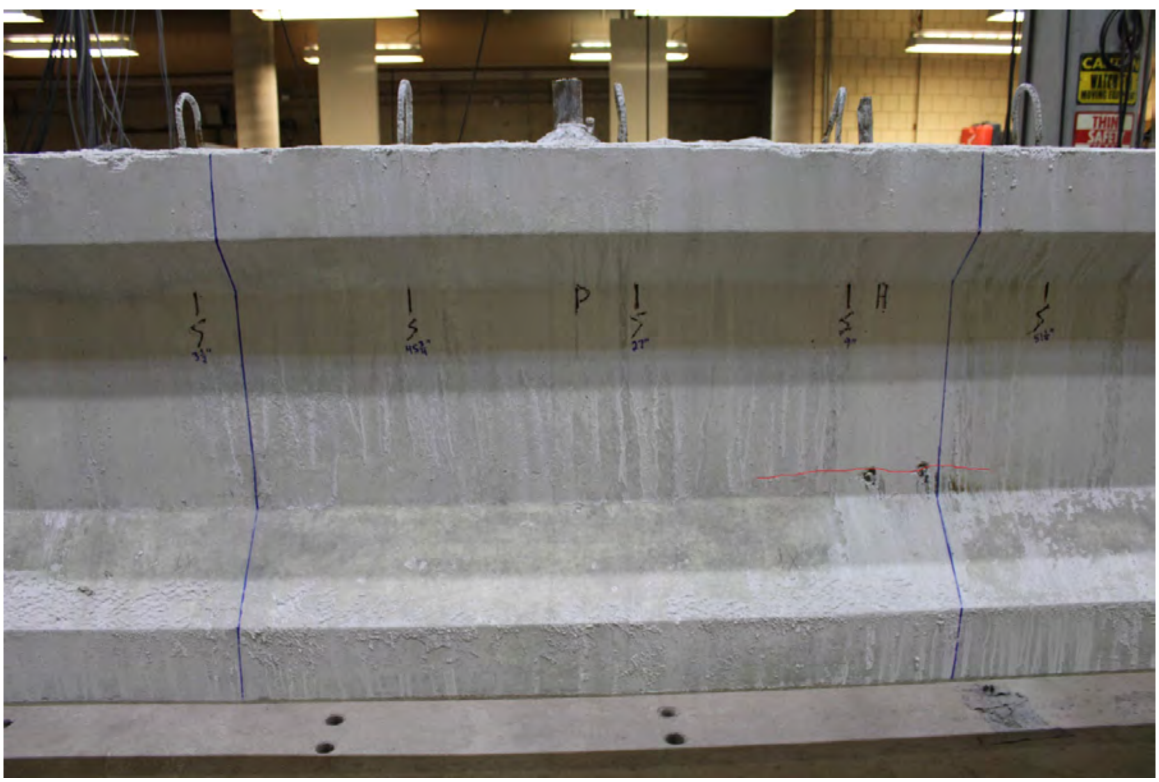


Figure 2.12 Typical horizontal crack near the diaphragm connection

CHAPTER 3: LABORATORY SETUP AND GIRDER TESTING

3.1 INTRODUCTION

The four girders were tested in three point bending to promote a shear failure using the 600 kip testing frame in the Galambos Structural Engineering Laboratory. This chapter describes the casting of the decks on the delivered girders, the testing setup, and the external shear reinforcement system used to ensure that the cut end of the girders did not fail in shear prior to the end of interest failing.

3.2 CONCRETE DECKS

The original decks on the girders were removed by the contractor in order to reduce the weight of the girders so that they could be lifted by the crane in the Galambos Structural Engineering Laboratory. However, analysis indicated that without the concrete deck, the girders would fail in flexure before they would fail in shear. The MnDOT Bridge office performed an analysis of the girders with a new cast-in-place deck section assuming a 1.3 overstrength factor for shear and a 10% reduction in the flexural strength. The 1.3 overstrength factor was based on previous MnDOT research on the shear strength of prestressed concrete girders (Dymond, 2016, Dereli, 2010). The 10% reduction in flexural capacity was applied to conservatively ensure a shear failure prior to a flexural failure. Based on an analysis by the MnDOT Bridge Office, a 12 in. thick by 14 in. wide concrete deck with a compressive strength of 10,000 psi was required to avoid flexure failure prior to the intended shear failure under three-point loading.

Because the girders with the repair were too heavy for the crane to lift after the new deck had been cast, the deck had to be cast on the repaired girders (P4 and P26) once they had been put into position under the 600 kip testing frame. In order to have two sets of good comparisons, a deck was cast on a repaired girder and the companion unrepaired girder using in a single batch of Readymix concrete. The first two girders that were decked were P2 and P4. Formwork for these two girders was installed on April 19, 2017 by volunteers from the Cement Masons, Plasterers, and Shophands Local No. 633 Union Apprentice Training Program. The formwork was just offset inward from the girder top flange edge and included 14 in. coil ties for upper support and ¼ in. rod and clamps to clamp the formwork at the bottom. The formwork was self-supporting due to the inward offset and required no external support. The deck was not reinforced or flared to match the width of the repair. Figure 3.1 shows the formwork setup.

The decks on girders P2 and P4 were cast on April 21, 2017 with help from the Cement Masons, Plasterers, and Shophands Local No. 633 Union Apprentice Training Program. Prior to casting, the top of the girder was wetted and kept moist until the deck was cast using Cemstone mix 10055 Readymix concrete. The mix design is shown in Figure 3.2. After casting, the top of the deck was sprayed with ConFilm, a temporary curing sealant, and then covered with a thin polymer sheet. The deck was kept continuously moist with water for 21 days after casting.

The decks on the other two girders, P24 and P26 were also cast together, after Girders P2 and P4 had been tested. Formwork for these girders was installed by University of Minnesota staff during the week of July 31, 2017. The deck for these two girders was cast by University of Minnesota staff on August 9, 2017. For these two girders, the curing sealant was not used, and the girders were simply covered with a thin polymer sheet and kept continuously moist for 21 days after casting.

For all four girders, the formwork was removed 14-days after casting, however wet-curing was continued for an additional 7-days. Curing was stopped at 21-days in order to install an external shear reinforcement system and to prepare the girders for testing. Figure 3.3 shows the cast deck from the cut end of the girder.

Cylinders constructed with deck concrete from the two pours were tested periodically. Results of these 4 in. by 8 in. cylinder tests are provided in Table 3.1

3.3 TESTING LAYOUT

The four girders were tested in the 600 Kip testing frame located in the Galambos Structural Engineering Laboratory. Figure 3.4 shows a schematic of the testing layout. Steel W14x145 Sections were used to support the girder and provide the reactions. The supports were set in a leveling grout and tied to the strong floor. The support-to-support span length for all four girders was 36 ft.-1.5 in., The sole-plate was located 7.5 in. from the end of interest. On the cut end, the center of the support reaction was placed a minimum distance of 9 in. from the cut end. The W14x145 support sections were placed at 12 ft.-7.5 in. and 23 ft.-6 in. from the center of the loading point to make up the span length. Bearing pads 12 in. x 24 in. x 3 in. thick, salvaged from Bridge 27568, were placed between the W14x145 sections and the girder bottom flange.

For documentation purposes, each girder was divided into eight 5 ft.-0 in. sections. Section 1 began at the repaired end, and Section 8 was located at the cut end. Vertical lines were drawn on the girder at the end of each section. The side of the girder was marked to show the location of transverse reinforcement (S), embedded pipes used for the original deck casting (P), hold down points (H), and lift hooks (LIFT). A dashed line was drawn on Girders P2 and P24 to indicate where the repair would have ended if P2 and P24 had been repaired. Figure 3.5 shows the section demarcation lines, the dashed line for where the end of the repair would have been located, the location of an embedded pipe, and the location of the transverse reinforcement on Girder P24 (unrepaired) prior to testing.

3.4 EXTERNAL SHEAR REINFORCEMENT

Because the girders were cut to length to bring into the laboratory, the cut end of the girder was not adequately reinforced for shear, as it had shear reinforcement appropriate for midspan and not an end span. To make up for the lack of internal shear reinforcement on the cut end of the girders, external shear reinforcement was added to preclude the cut end of the girder failing in shear prior to the end of interest.

The external shear reinforcement system used short steel beams above and below the girder connected together using four ¾ in. – UNC 10 ASTM A193 B7 threaded rod pretensioned to approximately 60 ksi. This system was installed at eight locations on the beam between the load point and the cut end of the girder, with an approximate spacing of 36 in., starting 24 in. in from the cut end. Figures 3.6 and 3.7 show the installed external shear reinforcement.

Conservatively estimating that each threaded rod in the external shear system provided 60 ksi of stress in resisting shear, the external shear reinforcement added an additional V_s of 91 kips (using d=0.8h, the composite h of 57 in., A_v or 4* 0.3 in^2, and s of 36 in.), or 114 kips (using the d_p after the harp point) in addition to the V_c and V_s from the existing internal stirrups. The predicted demand at the cut end was approximately 195 kips based on a conservatively estimated 500 kip load to cause shear failure in the repaired end. The existing V_c and V_s contribution was approximately 206 kips based on the MnDOT Bridge Office analysis.

3.5 TEST PROCEEDURE

The loading was applied using displacement control at a single point using the Galambos Structural Engineering Laboratory 600 kip MTS Model 311 Material Test Frame. A 9 in. x 22 in. x 1.5 in. neoprene pad and 11 in. x 20 in. x 1.5 in. steel plate were placed between the deck and the actuator. For all four girders, the initial loading rate was 0.06 in/minute (approximately 0.250 kips per second). For Girders P2 and P4, the load rate was increased to 0.09 in/minute after the 400 kip load pause, and increased again to 0.012 in/minute after the girder reached peak load. For girders P24 and P26 the load rate was increased to 0.09 in/min after the 400 kip load pause and maintained for the duration of the test. For all four girders, loading was paused every 25 kips, starting at 50 kips to monitor, mark, and document the cracking observed in the girder and take photos until it was deemed not safe to go near the girder. At this point loading was continuous until failure. Girder P4, the first girder, was tested on May 30, 2017. Girder P2, the second girder, was tested on June 15, 2017. Girder P26, the third girder, was tested on September 8, 2017. Girder P24, the last girder, was tested on October 6, 2017.

Table 3.1 Deck Cylinder Compressive Strengths

Specimen	Deck Concrete Age (days)	Capping	Number of Cylinders	Compressive Strength (psi)	COV (%)
P2/P4 Deck	21	Sulfur	3	10,500	9.7
P2/P4 Deck	28	Sulfur	6	11,100	3.9
P2/P4 Deck	39 (P4 test)	Neoprene	3	11,300	3.6
P2/P4 Deck	55 (P2 test)	Neoprene	3	12,800	2.8
P24/P26 Deck	7	Neoprene	3	8,800	1.9
P24/P26 Deck	14	Neoprene	3	10,100	4.0
P24/P26 Deck	21	Neoprene	3	10,800	2.7
P24/P26 Deck	29 (day before P26 test)	Neoprene	3	10,800	11.7
P24/P26Deck	58 (P24 test)	Neoprene	3	12,200	3.5

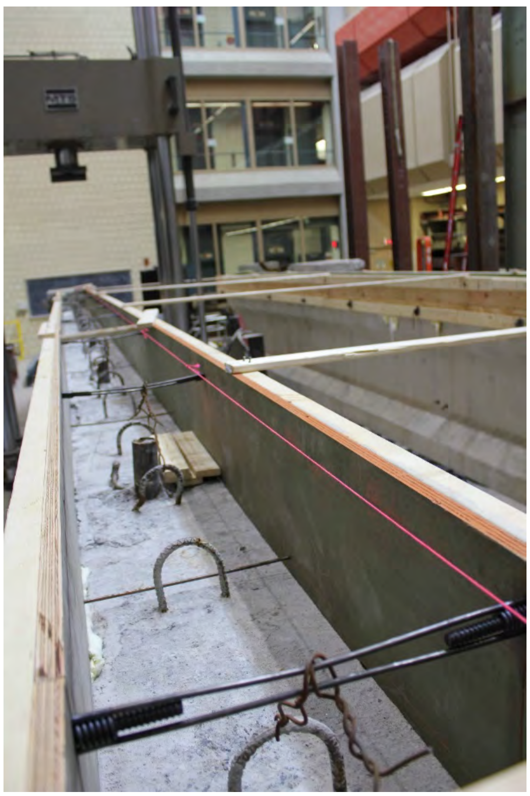


Figure 3.1 Installed formwork for the deck

CEMSTONE
ENGINEERING SERVICES

CONCRETE MIXTURE

DESIGNATION	VERSION
10055	Dec-9-15

COMPRESSIVE STRENGTH

10,000 psi at 28-Days

GENERAL INFORMATION

MIX TYPE:	Standard Mix
APPLICATION:	Interior
PUMPABILITY:	Pumpable Through a 4" Line

DESIGN

CEMENT,	(ASTM C 150)	698 lbs. (75%)	3.55 ft^3
FLY ASH,	(ASTM C 618)	232 lbs. (25%)	1.49 ft^3
SAND,	(ASTM C 33)	1,250 lbs. SSD	7.53 ft^3
3/4" DOLOMITE,	(ASTM C 33/#67)	1,650 lbs. SSD	9.76 ft^3
WATER,		270 lbs. = 32.4 gal.	4.33 ft^3
ENTRAPPED AIR CONTENT,		2.0 %	0.54 ft^3
			27.20 ft^3
MRWRA,	(ASTM C 494/TYPE A)	56 oz. (6.0 oz/cwt)	
HRWRA,	(ASTM C 494/TYPE F)	28 oz. (3.0 oz/cwt)	
STABILIZER,	(ASTM C 494/TYPE B)	28 oz. (3.0 oz/cwt)	
VMA,	(ASTM C 494/TYPE S)	56 oz. (6.0 oz/cwt)	

WATER-CEMENTITIOUS RATIO,	0.29
SLUMP, At Point of Truck Discharge	8.00 in.
CONCRETE UNIT WEIGHT,	150.8 pcf

TERMS, CONDITIONS AND NOTIFICATIONS

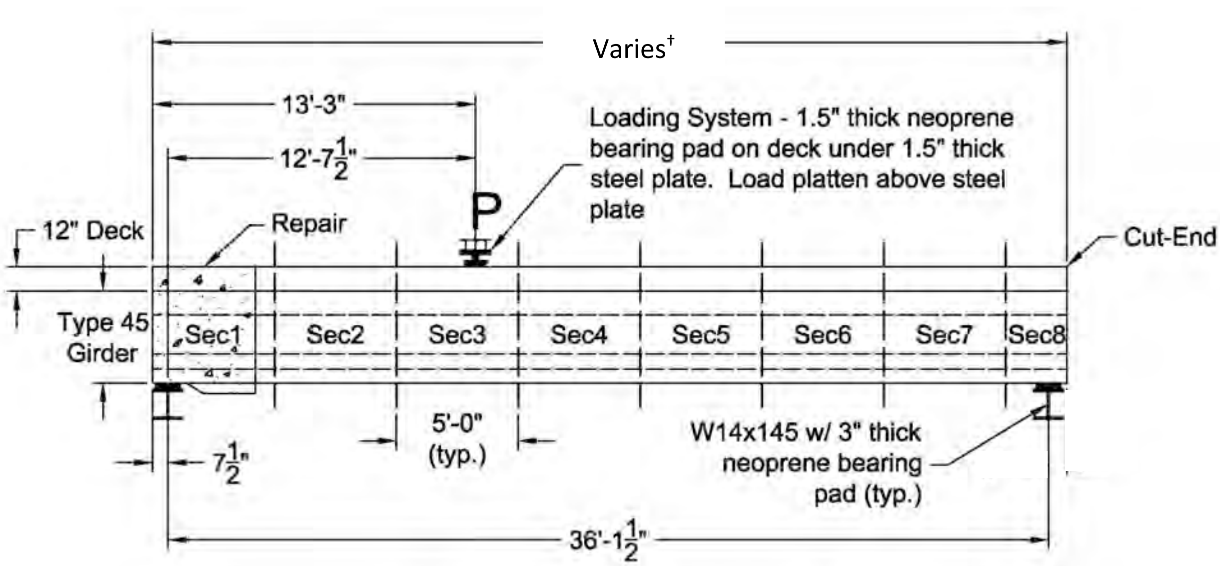
ADJUSTMENT: Material variation and job site conditions may require mixture adjustments to maintain water-cementitious ratio, slump, air content, and yield. The addition of other constituents and/or admixtures to this mix could cause the plastic and/or hardened properties to vary significantly from the above mixture design.
AIR CONTENT: If this mix contains no intentionally entrained air, the percentage of entrapped air listed above is an estimate based upon average conditions and can vary in either the plastic or hardened state.
DISCLAIMER: Cemstone disclaims and negates any warranty whatsoever of this concrete mix design if it is modified in any way or if it is provided too, or used by another concrete producer.
CONFIDENTIALITY: This concrete mix design is proprietary to, and confidential information owned by Cemstone. Its disclosure to a third party or entity, or permitting it to be used in competition with Cemstone, is strictly prohibited and will subject you to an injunction and damages.

PREPARED BY:

Kevin D. Heindel, P.E.

Figure 3.2 Cemstone Mix 10055

Figure 3.3 Cast deck – cut end



† Table 2.1 lists the overall lengths of each girder, which varied from 37’-6” to 37’-11”. The differences in length were made up by the overhang past the support on the cut end of the beam.

Figure 3.4 Layout of girder test setup

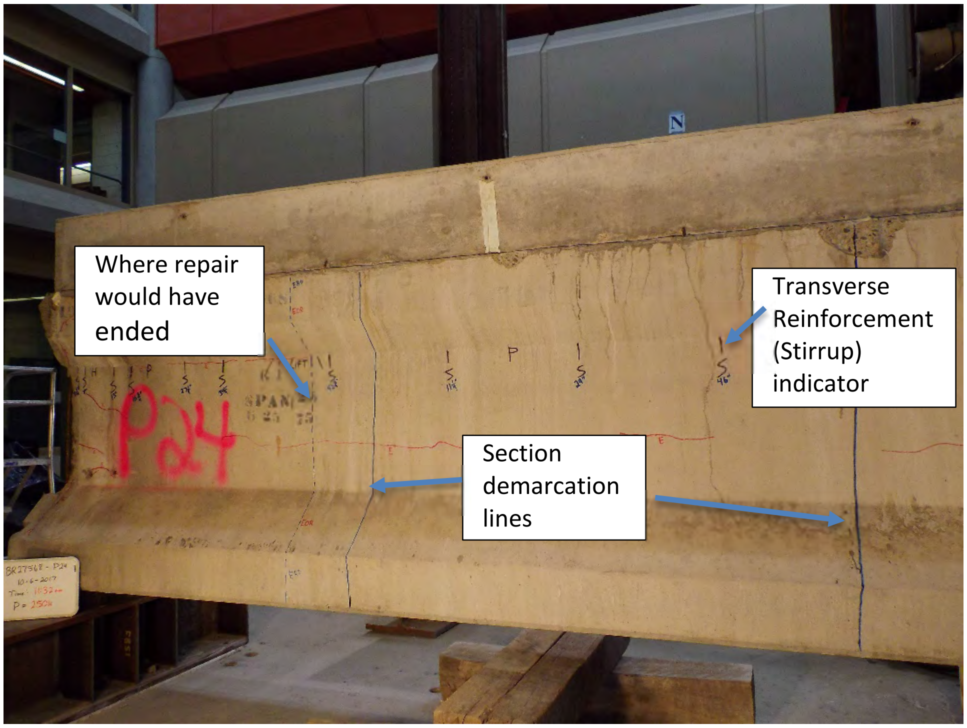


Figure 3.5 Girder P24 prior to testing showing solid blue lines dividing the girder into sections, a dashed blue line in the end section indicating where the repair ended on Girder P26, and the location of the transverse reinforcement

Figure 3.6 External shear reinforcement located between load point and cut end of girder

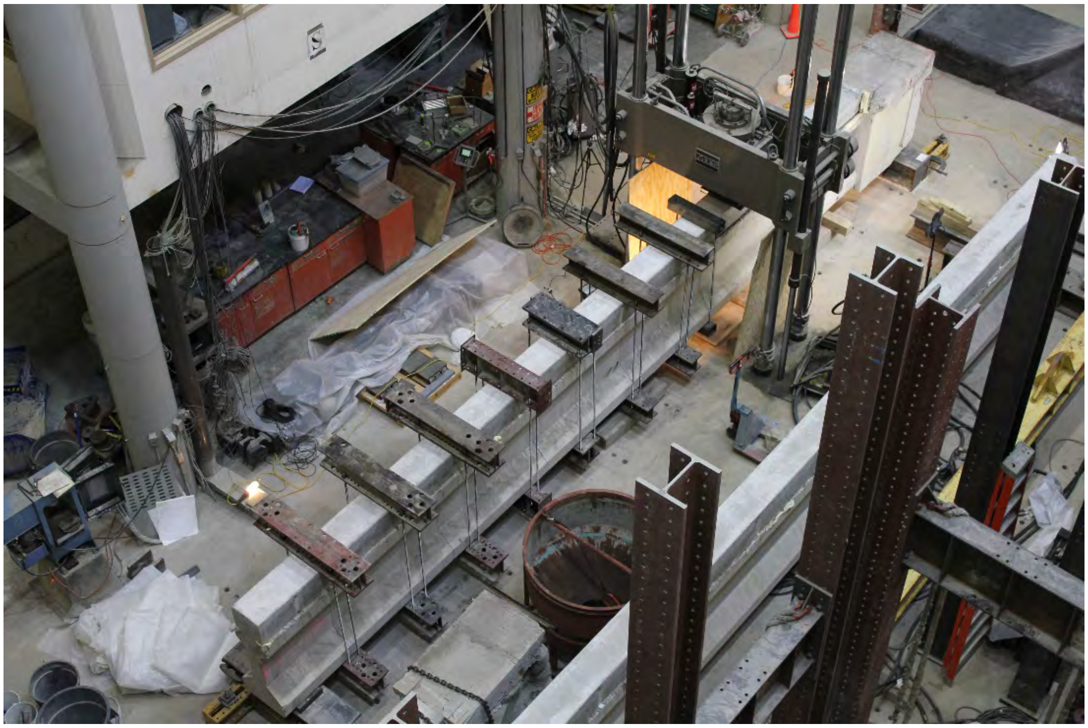


Figure 3.7 External shear reinforcement overview

CHAPTER 4: GIRDER TEST RESULTS AND OBSERVATIONS

The main goal of the testing was to determine if the shear capacity of the repaired girders was at least as large as the companion girders taken from the same bridge. In addition to determining the adequacy of the strength of the repaired girders, the behavior of the repair under loads that failed the girder was also of interest. To achieve these two goals, actuator load and displacement were measured during the test and at each load pause, the girders and the repairs were carefully inspected and photographed. A post failure analysis of this data was performed to achieve the goals of the project.

Table 4.1 summarizes the testing results for the four girders tested: first observed flexural crack, first observed web shear crack, peak applied load, and average measured girder concrete compressive strength from Table 2.6. Figure 4.1 shows the load versus displacement plots as recorded from the 600 kip testing machine load and displacement outputs for the four girders with Girder P2 and P4 in plot (a) and Girders P24 and P26 in plot (b). The girders are grouped by like girder concrete strength (Table 4.1). The recorded displacement also includes the settlement of the neoprene bearing and loading pads; hence the slope of these plots is not a true measure of the girder flexural stiffness. Girders P2 and P4 had average girder concrete strength less than Girders P24 and P26 (Table 4.1) which is likely what caused the increase shear strength in Girders P24 and P26 compared to Girders P2 and P4. There was good agreement between the failure loads of Girders P2 and P4, with the repaired girder having a slightly larger failure load. Likewise, there was good agreement between the failure loads of Girders P24 and P26, again with the repaired girder having the slightly higher failure load.

Table 4.2 summarizes the predicted and realized strengths for the four girders using both design (*fc’* = 5000 psi) and measured concrete strengths using unity load factors and resistance factors. The calculation is made at the critical section for the unrepaired girders (P2 and P24) and at the end of the repair for the repaired girders (P4 and P26). In all cases, the observed strength was larger than the predicted strength using either design or measured concrete strengths.

Figures 4.2-4.5 show the end of interest of Girders P2, P4, P24, and P26, respectively, at the fist load pause after observing web shear cracks. The higher initial observed web-shear cracking load for Girder P2 may have been due to the small cracks going unnoticed at the lower loads. The web-shear cracks in the unrepaired girders formed closer to the girder end than those in the repaired girders because of the additional local capacity added by the increased width of the shotcrete repair. Figures 4.6-4.9 show the test end of Girders P4, P24, and P26, respectively, at the last load prior to failure. In all four girders there was a well-developed system of web shear cracks prior to failure; however, the web shear cracks did not penetrate into the repair on Girders P4 and P26.

Figures 4.10-4.13 show the end of interest of Girders P2, P4, P24, and P26, respectively, after failure. In the repaired girders, P4 and P26, the web shear crack that ultimately opened and caused failure was located further into the girder than the end of the repair. However, on the unrepaired girders (P2 and P24), the web shear crack that grew into the failure was closer to the end of the girder than those cracks on the repaired girders, as can be seen by the relation of the bottom of the web shear crack with respect to the blue dashed line labeled EOR (for end of repair) in Figures 4.10 and 4.12. The blue dashed line was located 4’-1” from the end of the girder. The length between the end and the dashed line is indicative of where the repair was located on the companion girders and is provided as a way to compare location of cracks between the tested girder ends with and without repair. It appears that the repair increases the shear capacity in the area of the repair due to the increase in the web width. This forces the shear failure further into the beam, where the dead load shear is slightly smaller, increasing the net overall shear capacity of the girder. A full set of photographs taken during testing from two viewpoints for the unrepaired girders and three view points for the repaired girders at all load pauses can be found in Appendix A. No separation was noted between the shotcrete repair and the original girder in either Girder P4 or Girder P26.

Figures 4.14 through 4.48 show the crack drawings for each 5 ft. section along the south side of each girder documented at the end of the test. The drawings are annotated with the load level, in kips, when each crack or crack extension was observed. The drawings also show the location of fractured transverse reinforcement (stirrups), exposed prestressing strand, and crushed concrete. Three fractured stirrups were observed in Girder P2 (Figure 4.15). The web-shear crack that ultimately caused the failure was primarily located in Section 2 (between 5 and 10 ft. from the end of the girder), with some extension into Sections 1 and 3 (Figures 4.14-4.16). Three fractured stirrups were also observed in Girder P4 (Figure 4.25). The web-shear crack that caused failure in the girder was again primarily in Section 2 (Figure 4.25), with a small amount into Section 3 (Figure 4.26). The crack did not penetrate into the repaired portion of the girder (Figure 4.23). Four fractured stirrups were observed in Girder P24 (Figures 4.33 and 4.34). In this girder, the web shear crack that caused failure was located closer to the end of the beam than in the other three girders (Figure 4.32). The crack started less than halfway through Section 1 (Figure 4.32), and continued through the entirety of Section 2 (Figure 4.32). Three fractured stirrups were observed in Girder P26 (Figures 4.42 and 4.43). The web shear crack causing failure did not penetrate into the repair (Figure 4.40). During the load pause at 450k while testing Girder P4, the deck was observed to separate slightly from the girder top flange in Sections 2 and 3 as shown in Figure 4.49. This separation did not occur during any of the other tests.

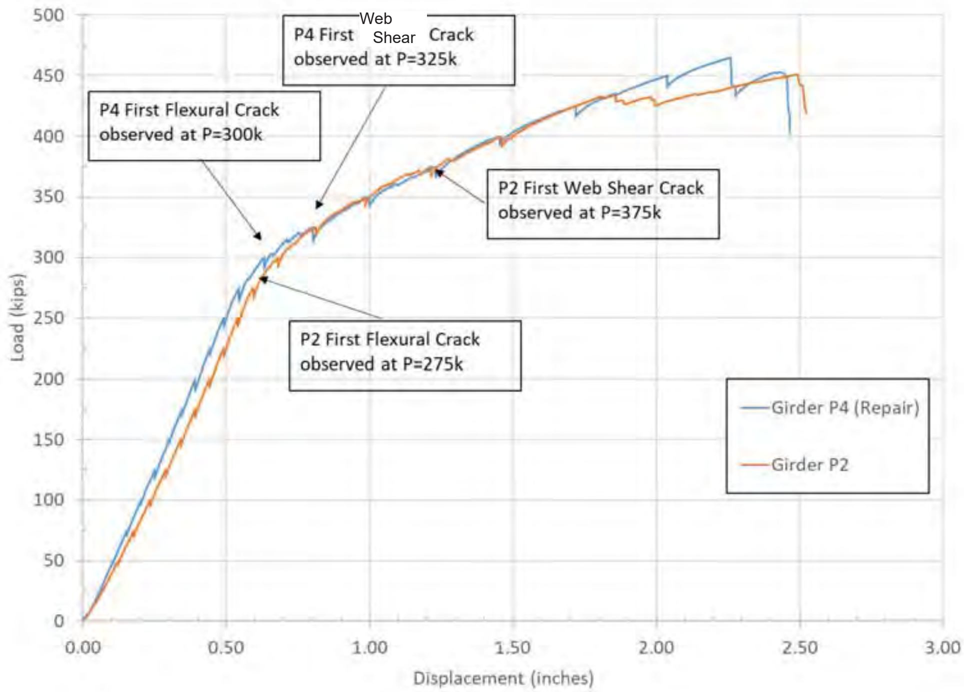
Table 4.1 Girder Test Results

Girder	Applied Load at First Observed Flexural Crack (Kips)	Applied Load at First Observed Web Shear Crack (Kips)	Peak Applied Load (Kips)	Average Measured Concrete Strength (psi)
P2	275†	375	451	5400
P4 (repaired)	300	325	465	5400
P24	275	325	492	6700
P26 (repaired)	275†	325	498	6800

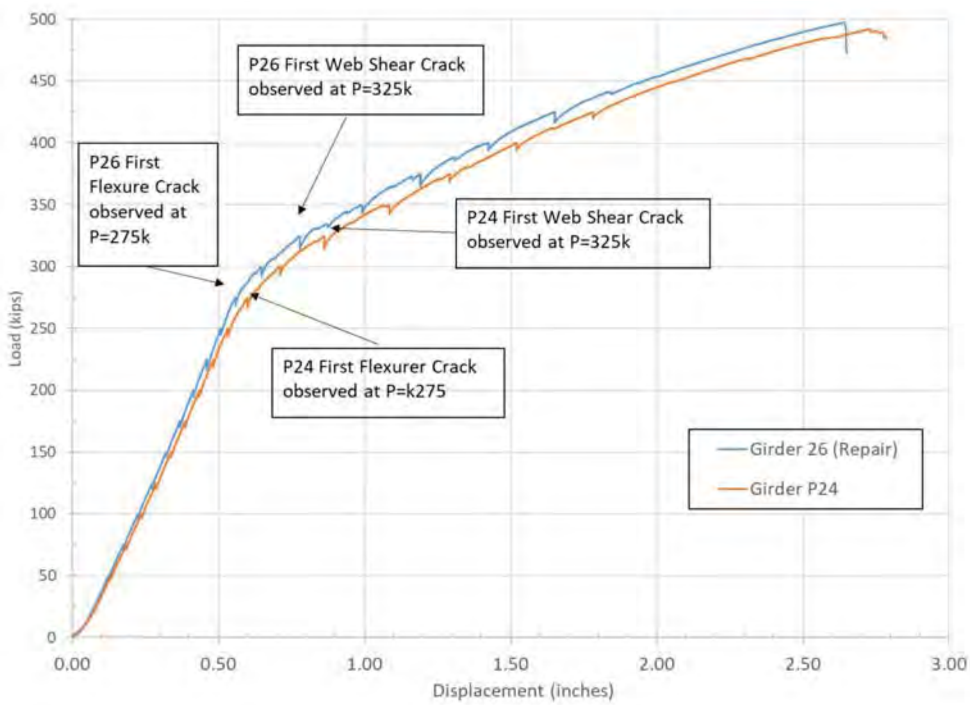
†Appeared on North Side of Beam only, first flexural crack on south side was at 300k

Table 4.2 Girder Test to Predicted Ratios

Girder	Vu/Vn based on design concrete strength	Vu/Vn based on measured concrete strength	Measured concrete strength used in calculation (psi)	Design Concrete Strength (psi)
P2	1.14	1.13	5172	5000
P4 (repaired)	1.17	1.15	5419	5000
P24	1.24	1.18	7043	5000
P26 (repaired)	1.25	1.18	7408	5000



a) Girders P2 and P4



b) Girders P24 ad P26

Figure 4.1 Applied load "P" versus displacement graphs

Figure 4.2 Girder P2 web shear cracking P= 375 k

Figure 4.3 Girder P4 web shear cracking P= 325k

Figure 4.4 Girder P24 web shear cracking P= 325k

Figure 4.5 Girder P26 web shear cracking P=325k

Figure 4.6 Girder P2 last load pause before failure, P=400k

Figure 4.7 Girder P4 last load pause before failure, P=450k

Figure 4.8 Girder P24 last load pause before failure, P=400k

Figure 4.9 Girder P26 last load pause before failure, P=425k

Figure 4.10 Girder P2 after failure

Figure 4.11 Girder P4 after failure

Figure 4.12 Girder P24 after failure

Figure 4.13 Girder P26 after failure

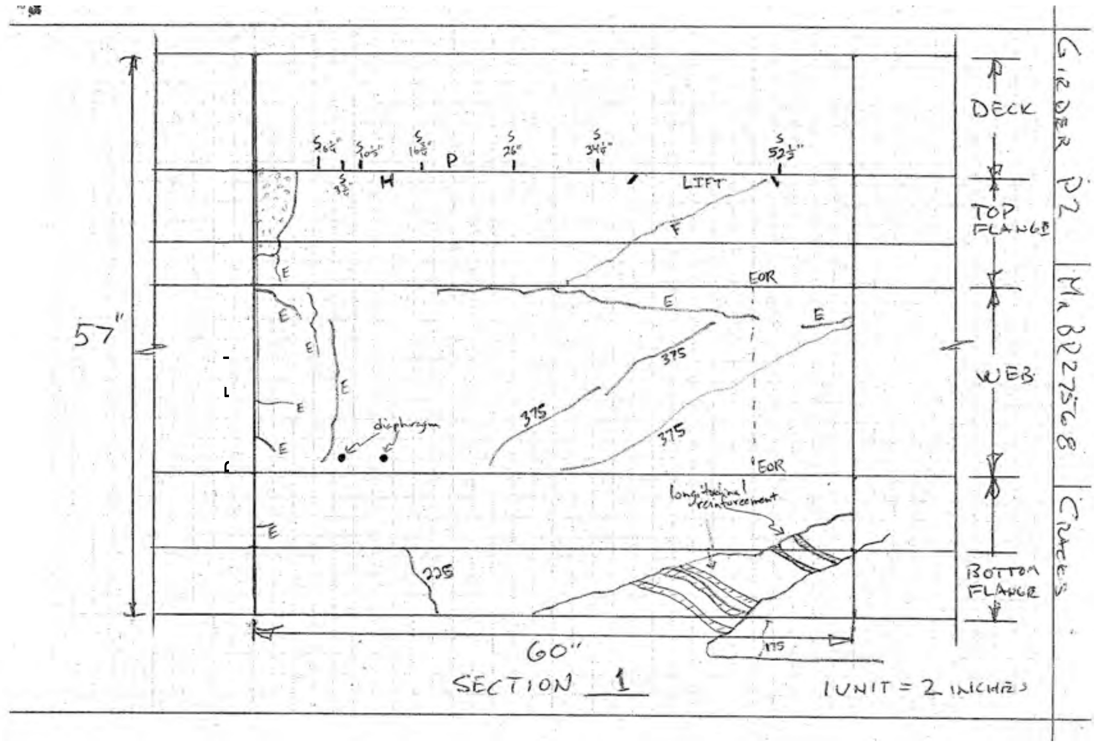


Figure 4.14 Girder P2 crack pattern – Section 1 (See Figure 3.4 for Section limits)

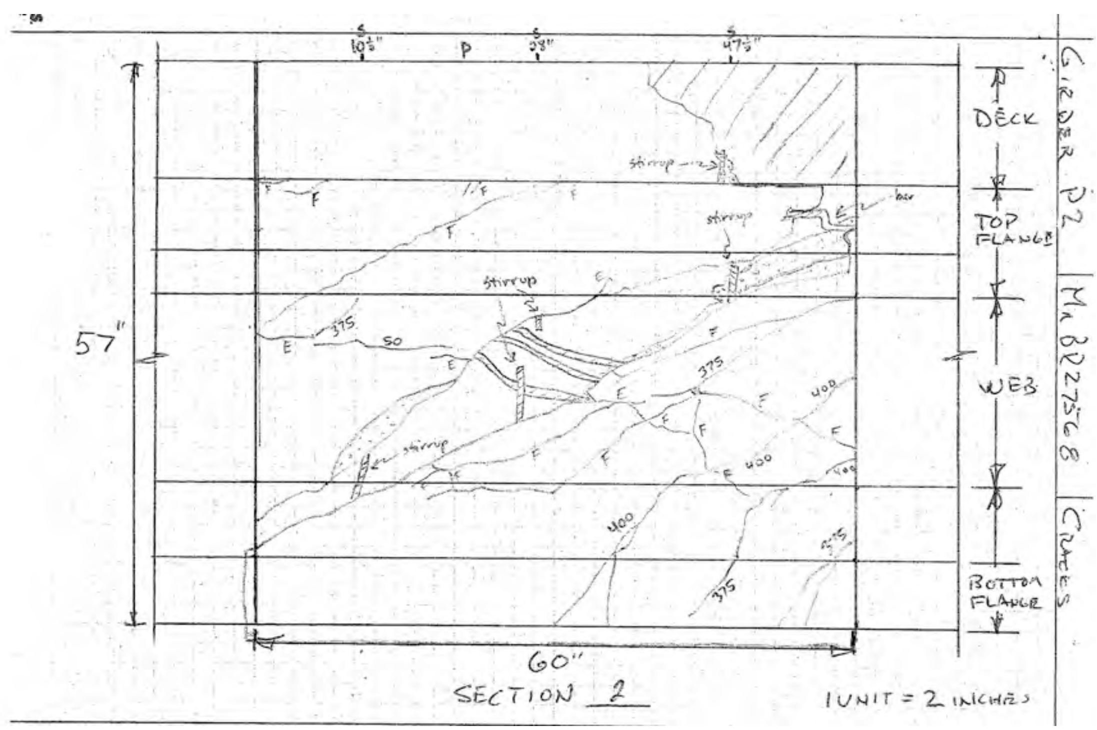


Figure 4.15 Girder P2 crack pattern – Section 2

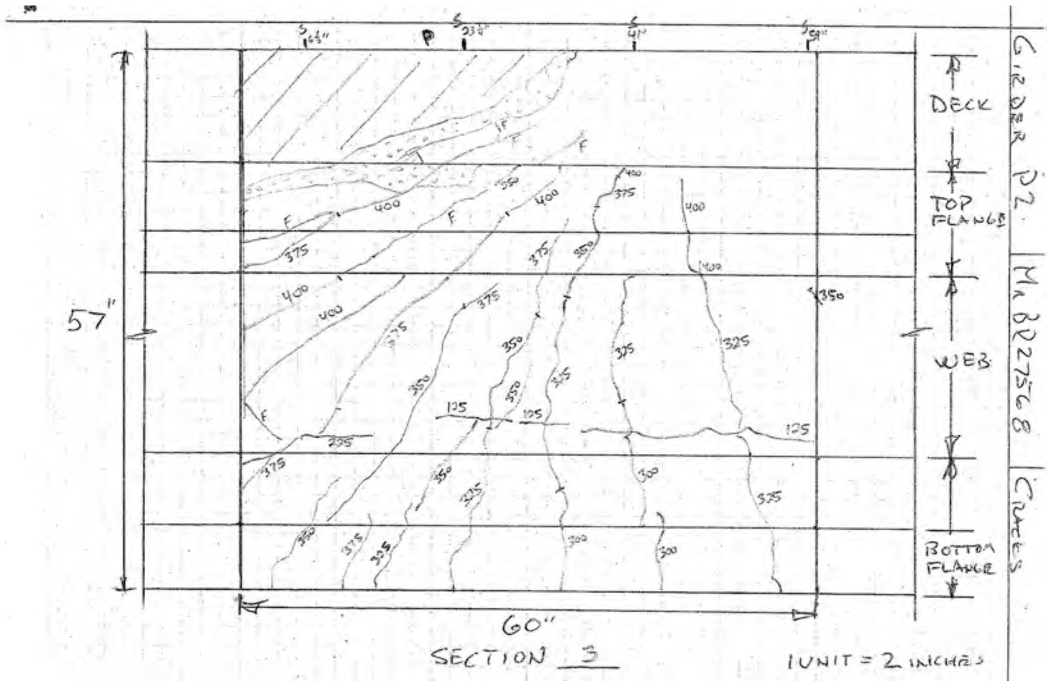


Figure 4.16 Girder P2 crack pattern – Section 3 (first flexural crack on South side at 300k)

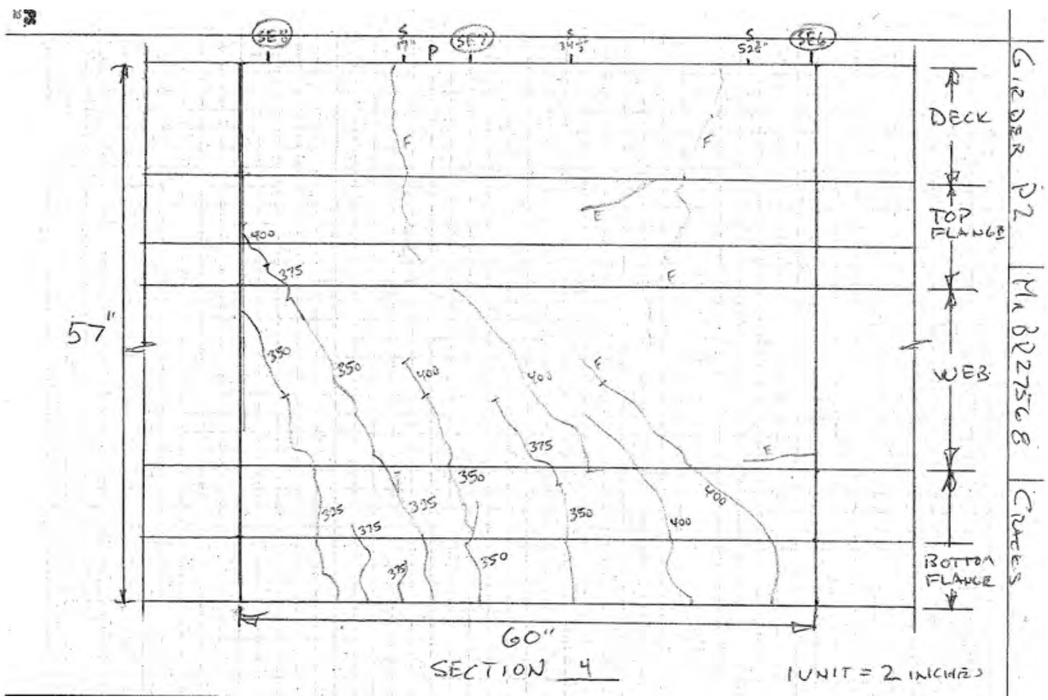


Figure 4.17 Girder P2 crack pattern – Section 4

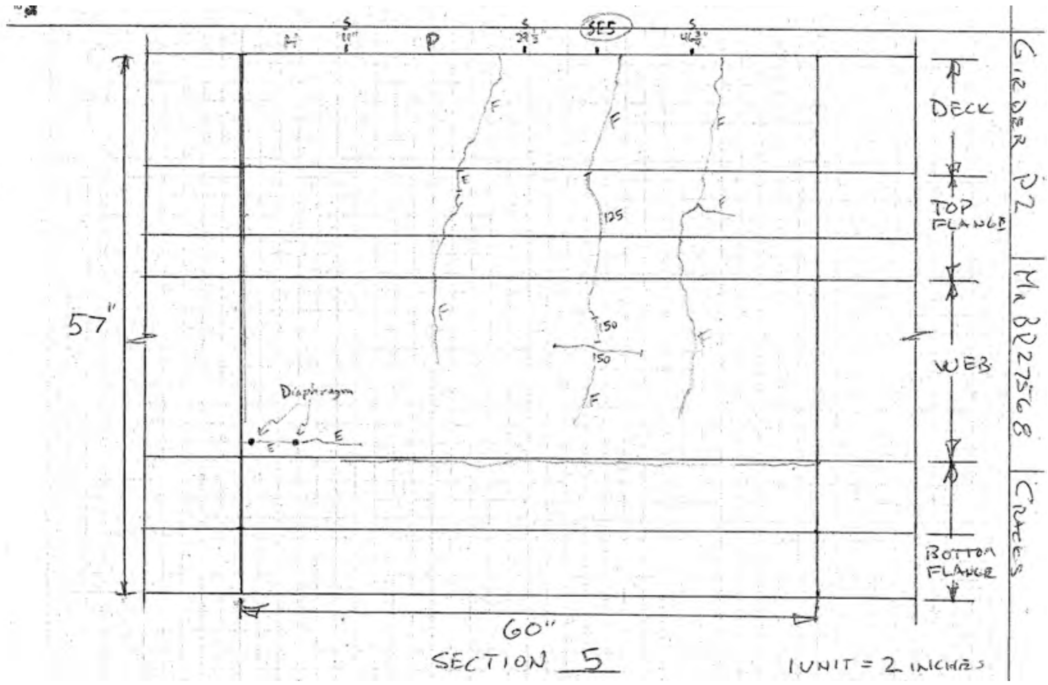


Figure 4.18 Girder P2 crack pattern – Section 5

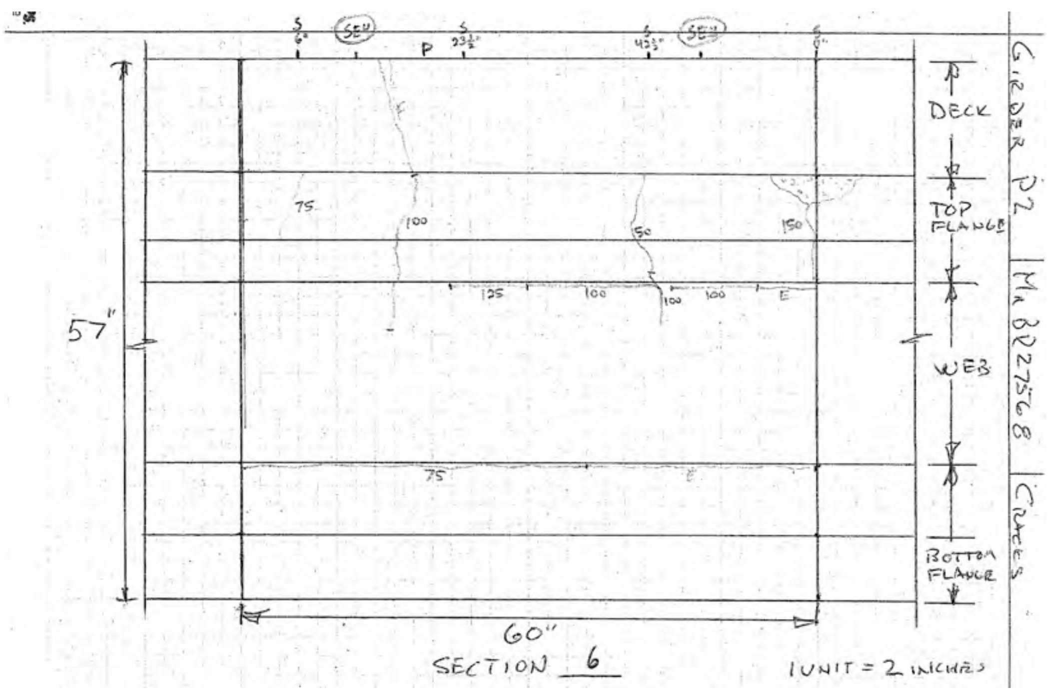


Figure 4.19 Girder P2 crack pattern – Section 6

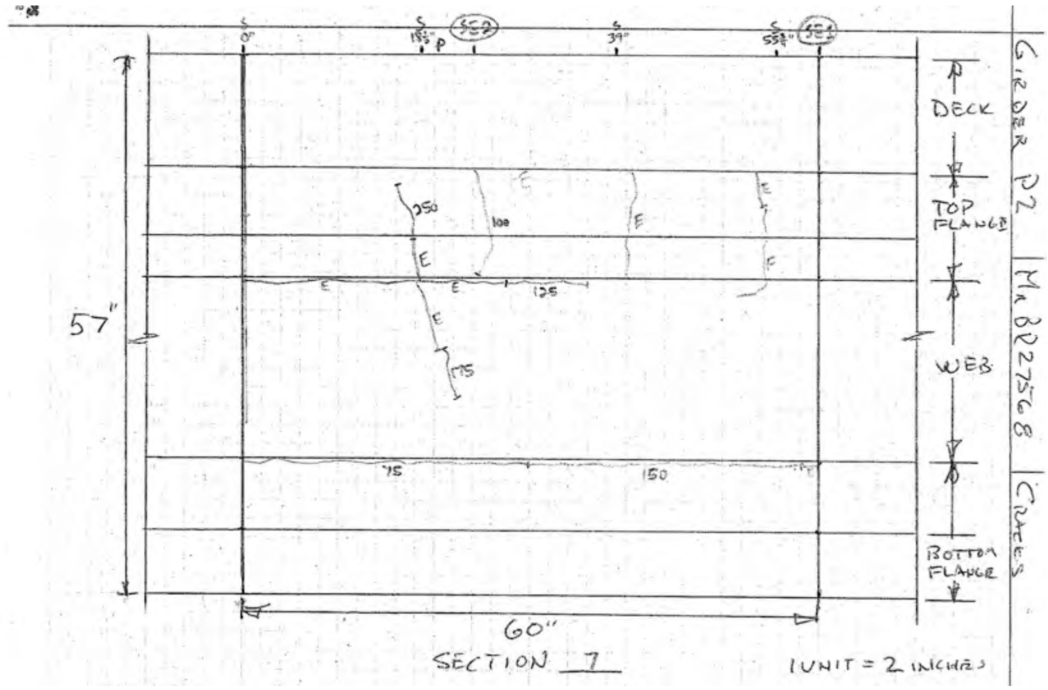


Figure 4.20 Girder P2 crack pattern – Section 7

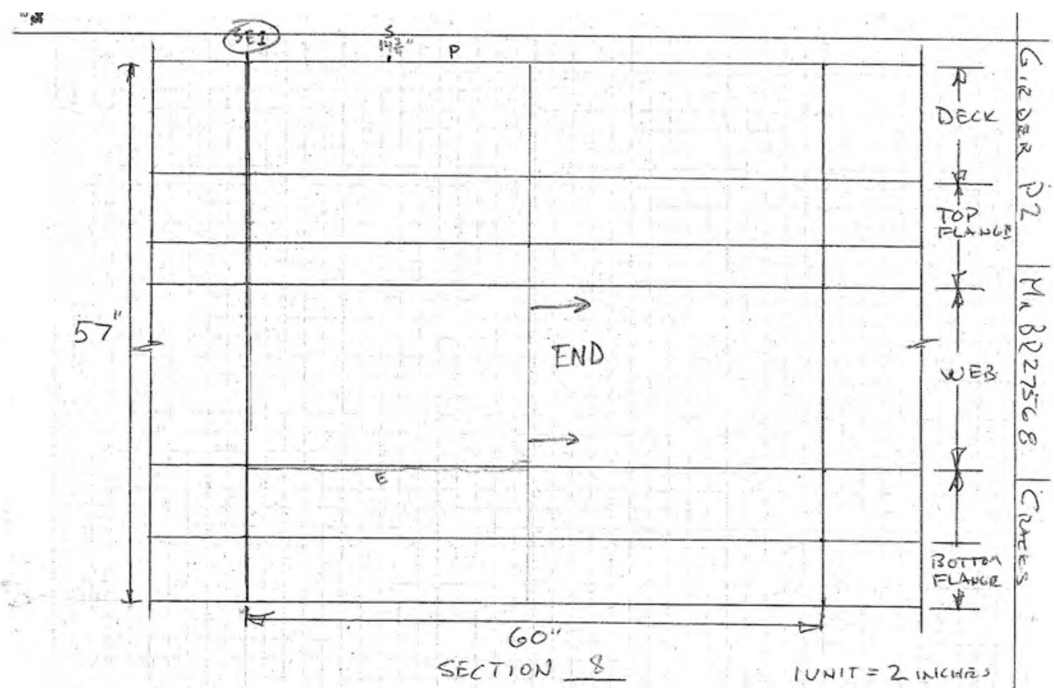


Figure 4.21 Girder P2 crack pattern – Section 8

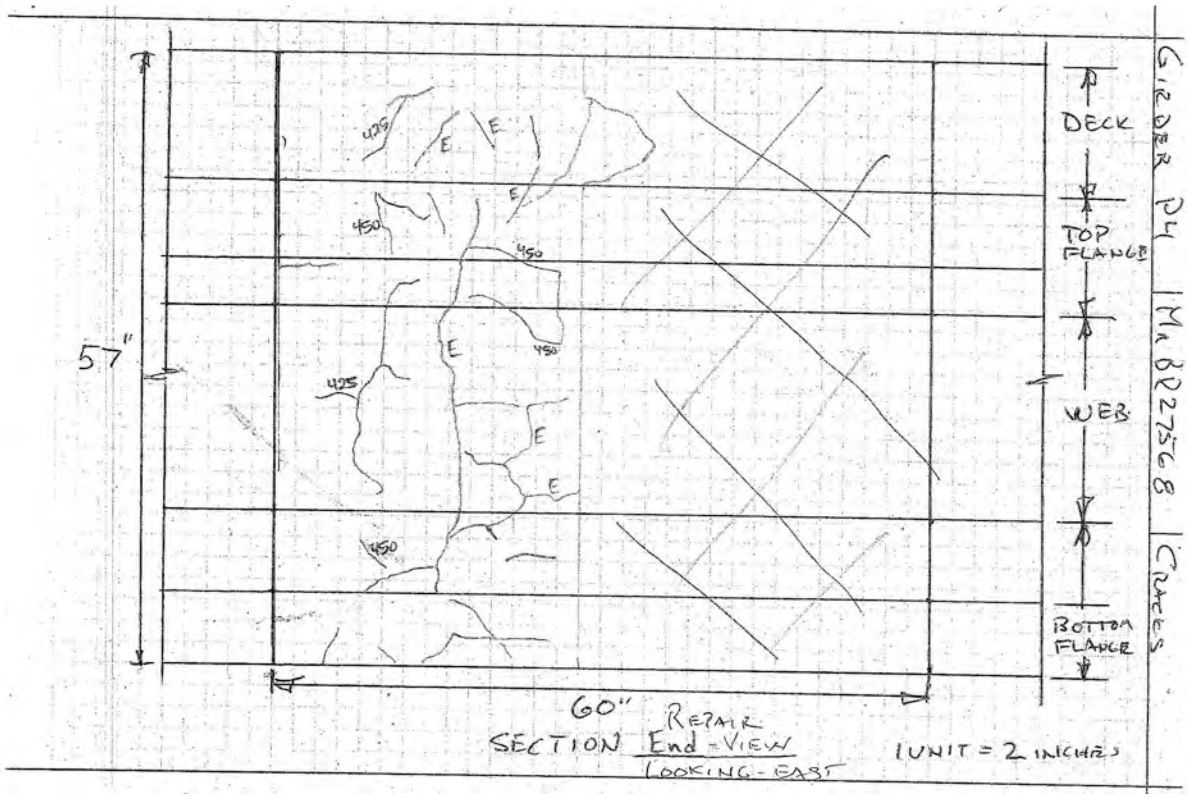


Figure 4.22 Girder P4 crack pattern – repair end view

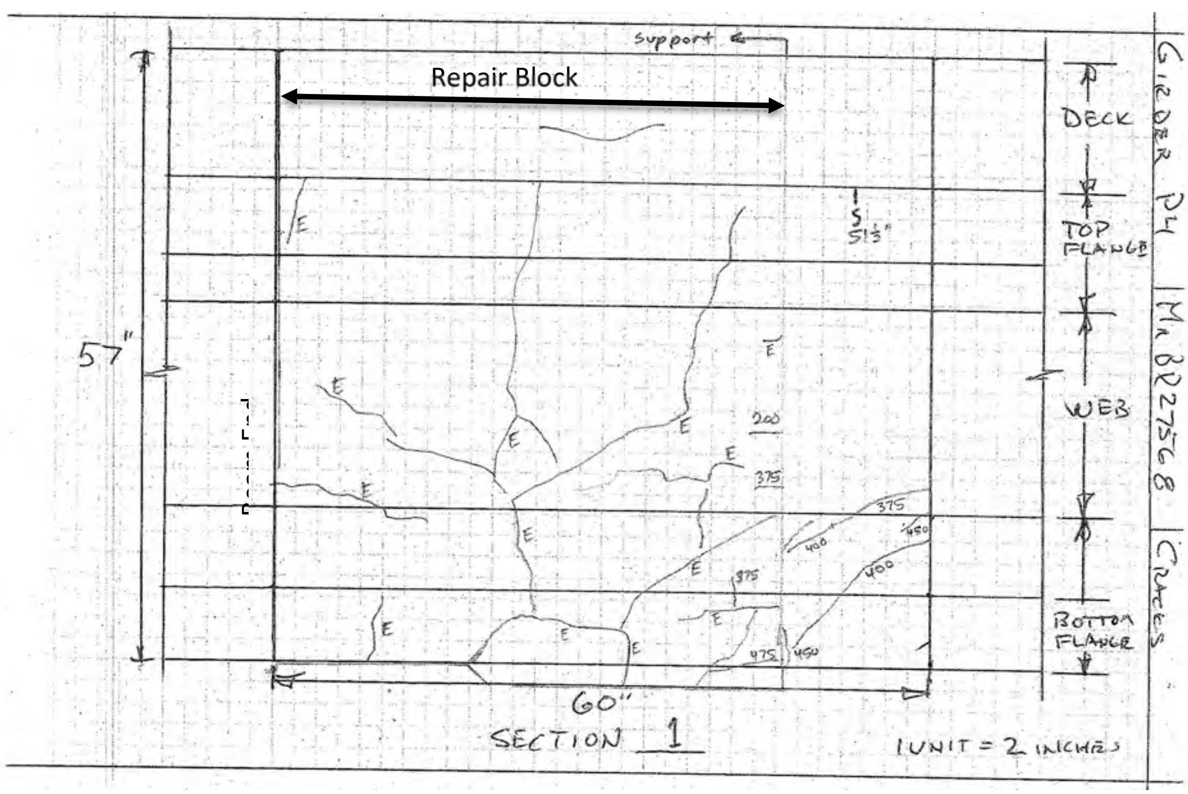


Figure 4.23 Girder P4 Crack Pattern – Section 1 (See Figure 3.4 for Section limits)

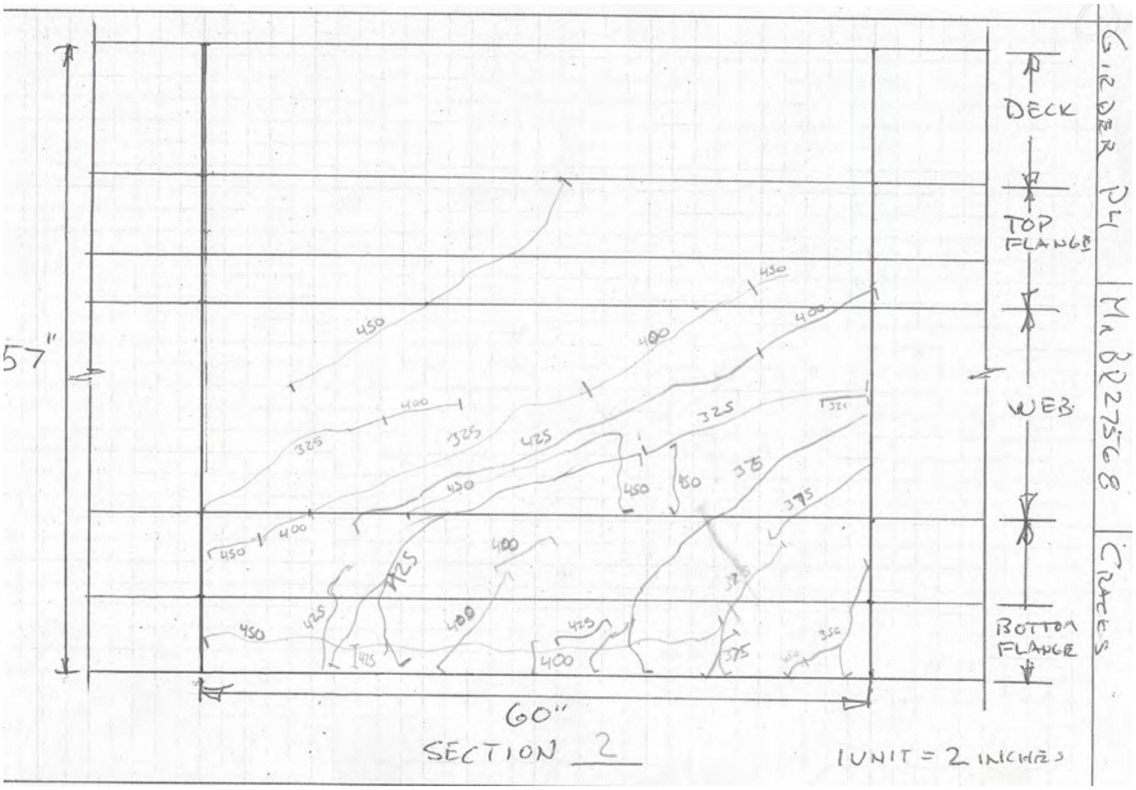


Figure 4.24 Girder P4 crack pattern – Section 2

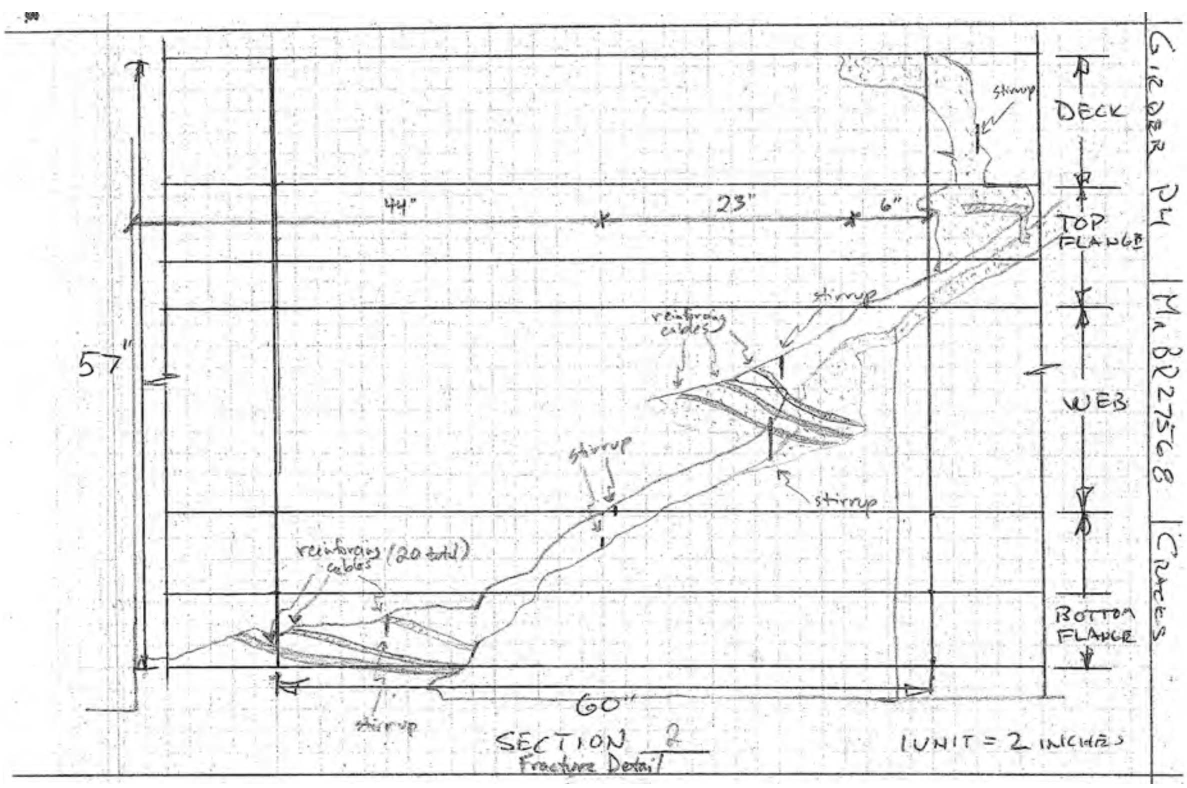


Figure 4.25 Girder P4 crack pattern – Section 2 (Bar Fracture Detail)

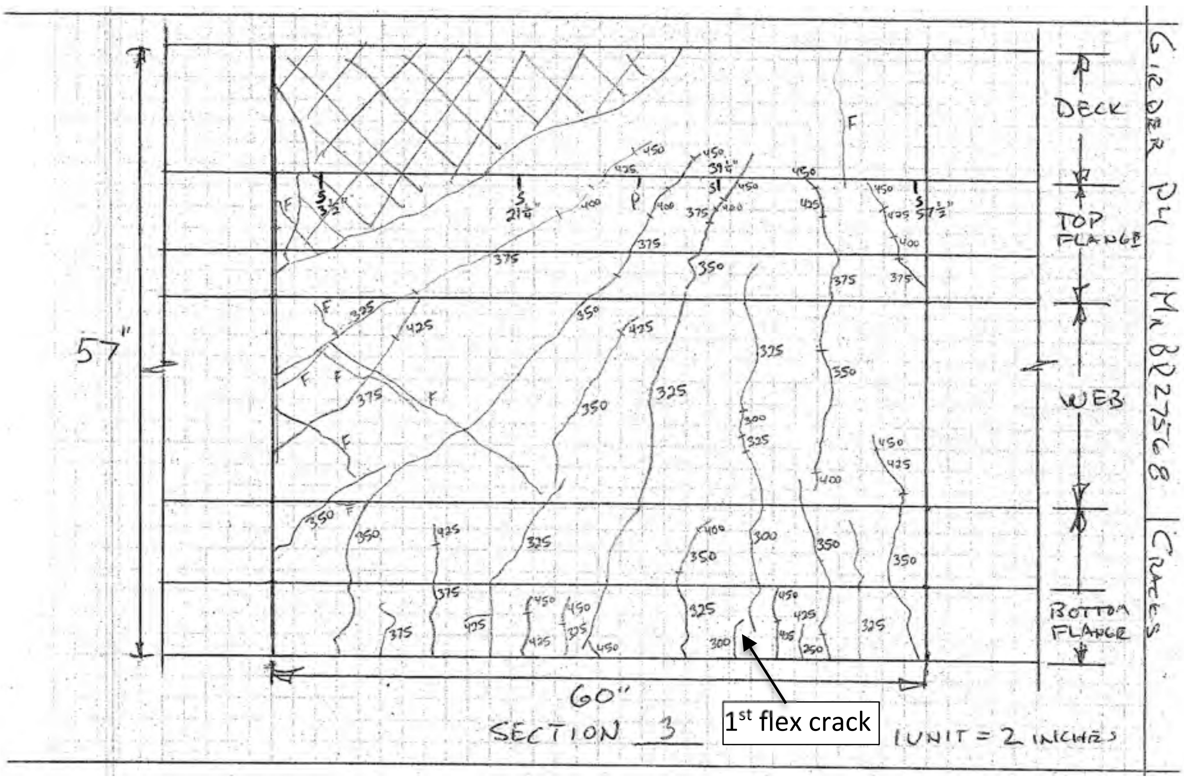


Figure 4.26 Girder P4 crack pattern – Section 3 (first flexural cracking at 300 k)

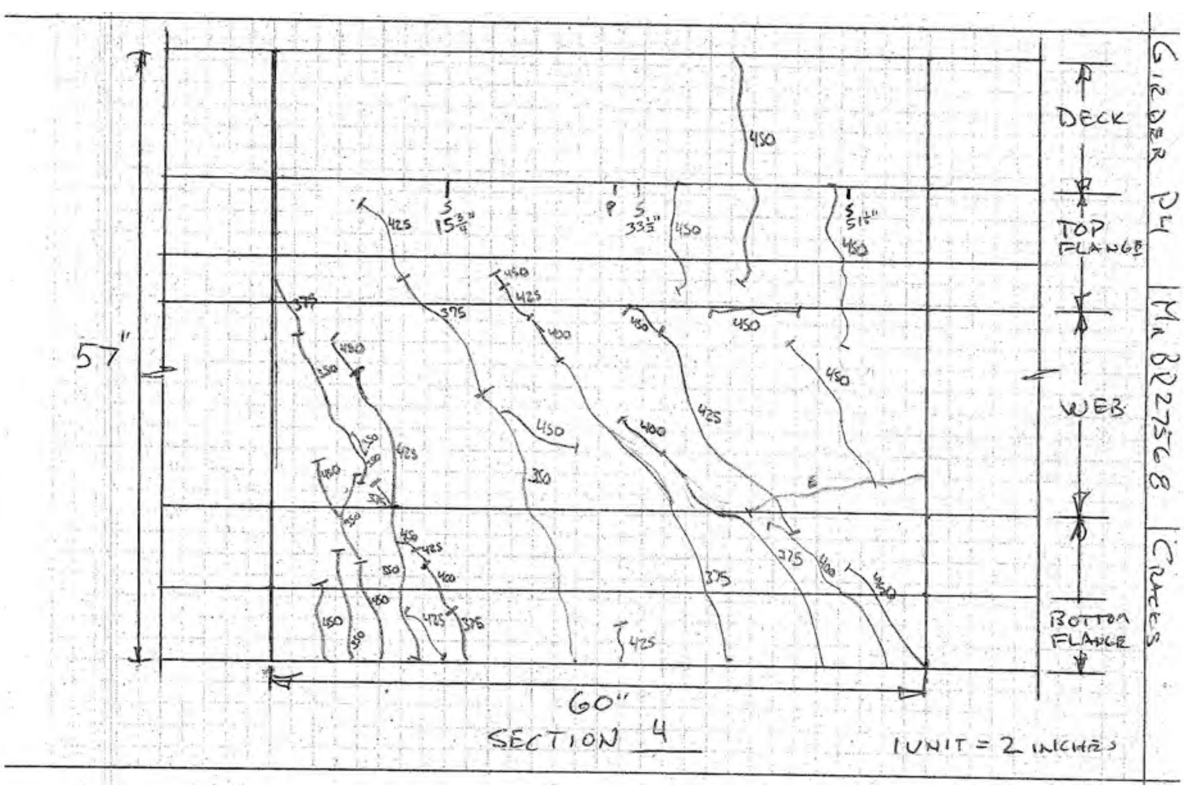


Figure 4.27 Girder P4 crack pattern – Section 4

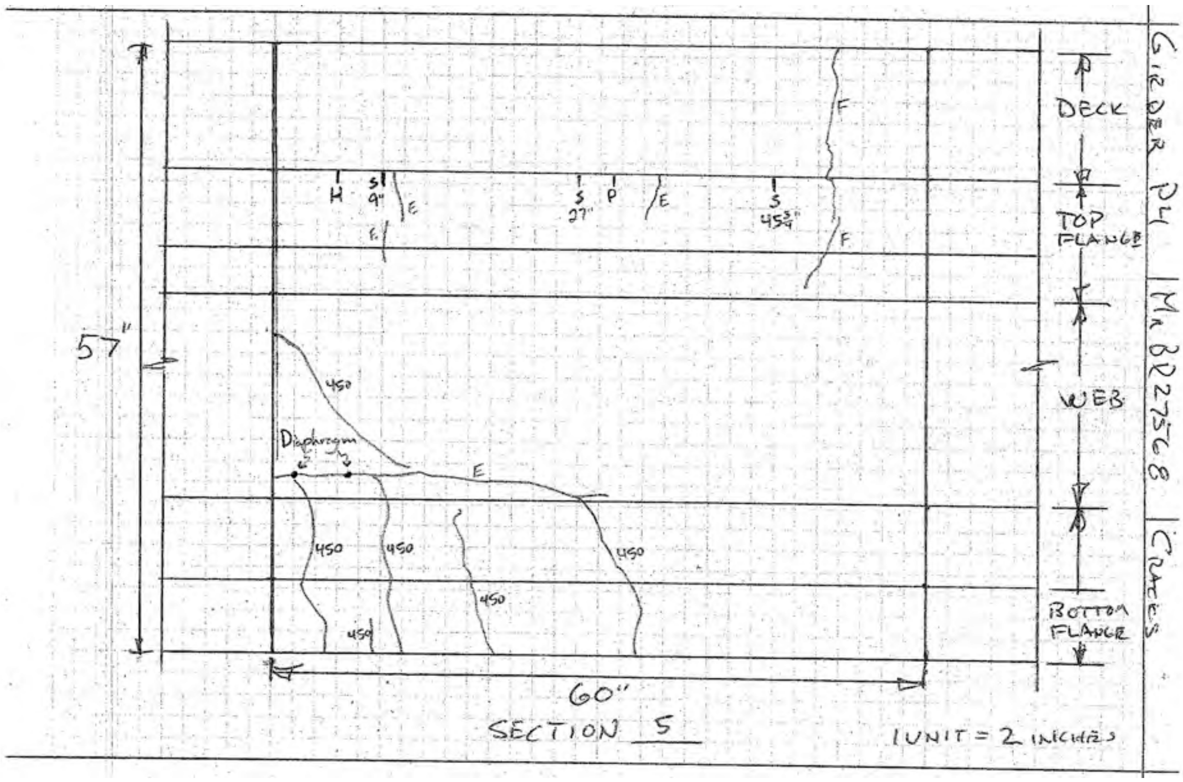


Figure 4.28 Girder P4 crack pattern – Section 5

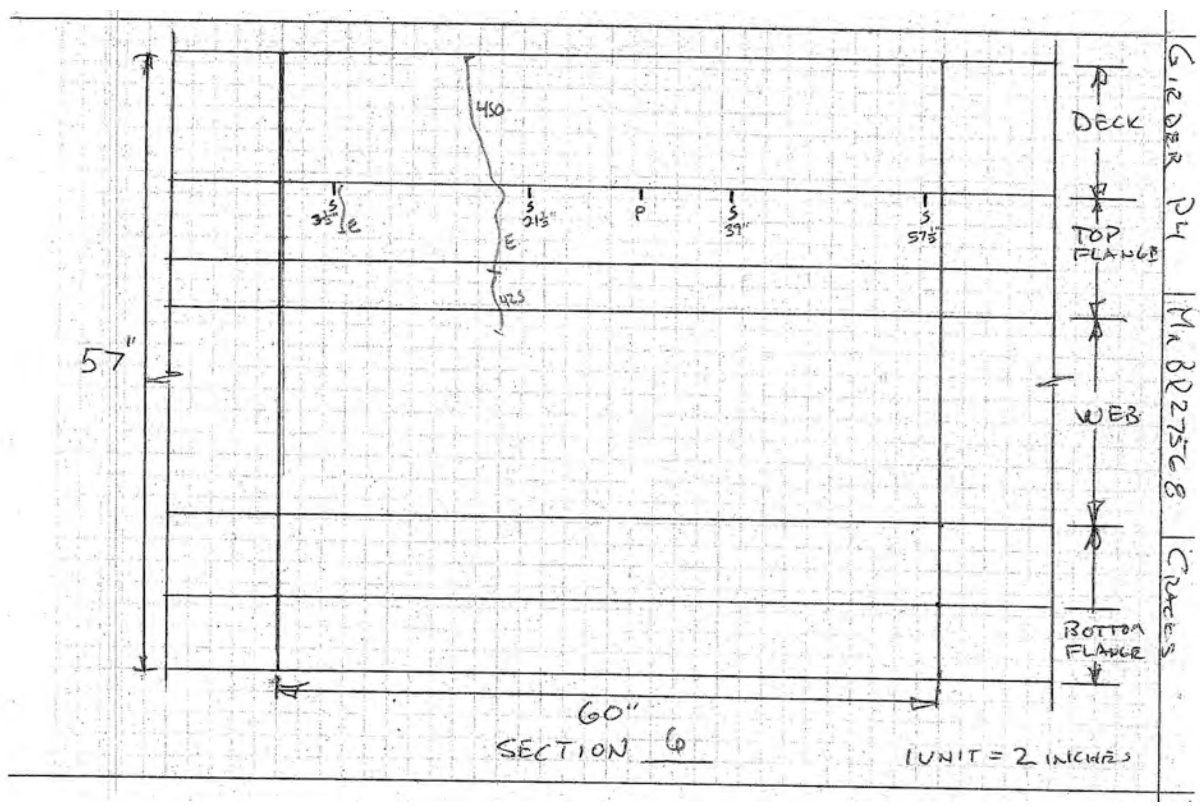


Figure 4.29 Girder P4 crack pattern – Section 6

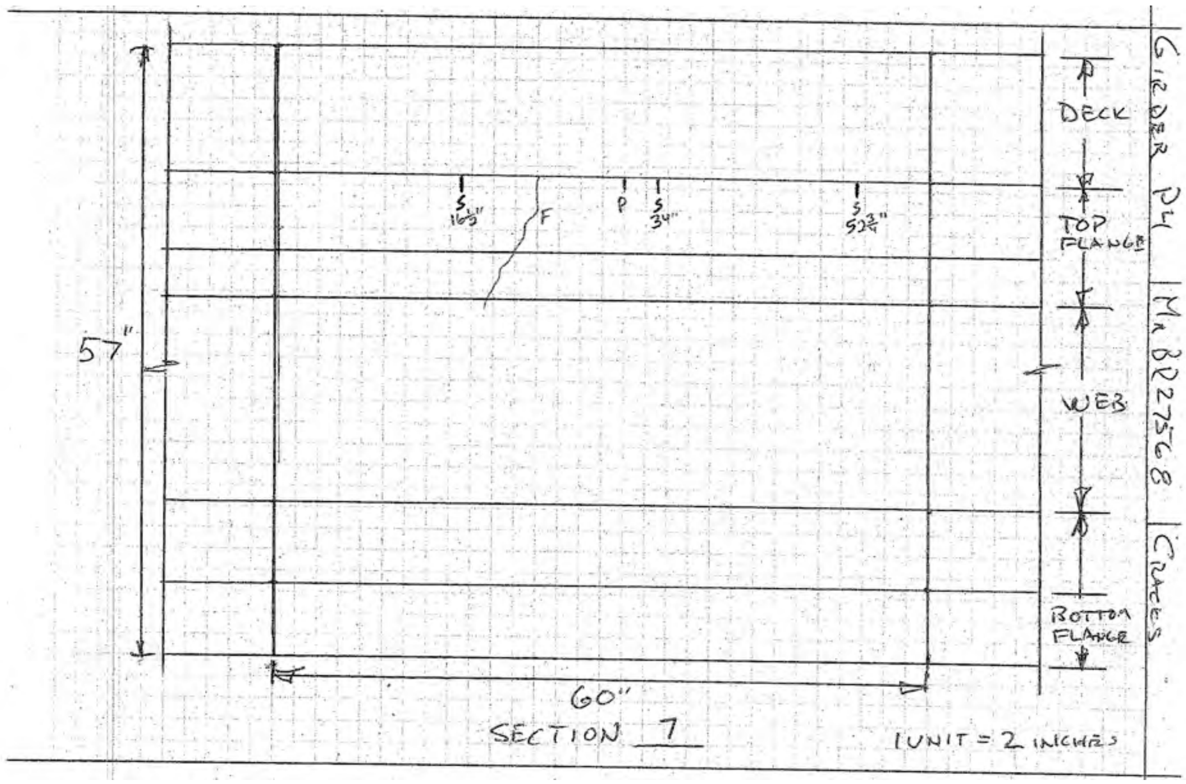


Figure 4.30 Girder P4 crack pattern – Section 7

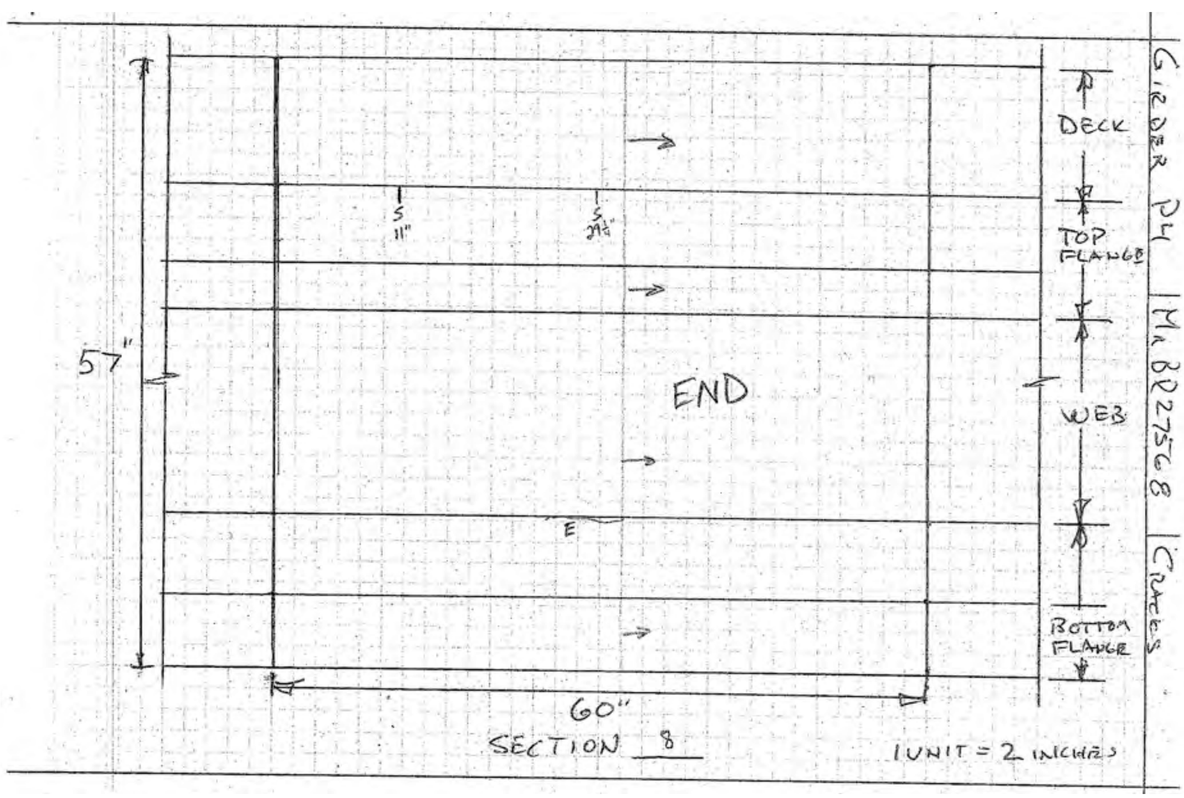


Figure 4.31 Girder P4 crack pattern – Section 8

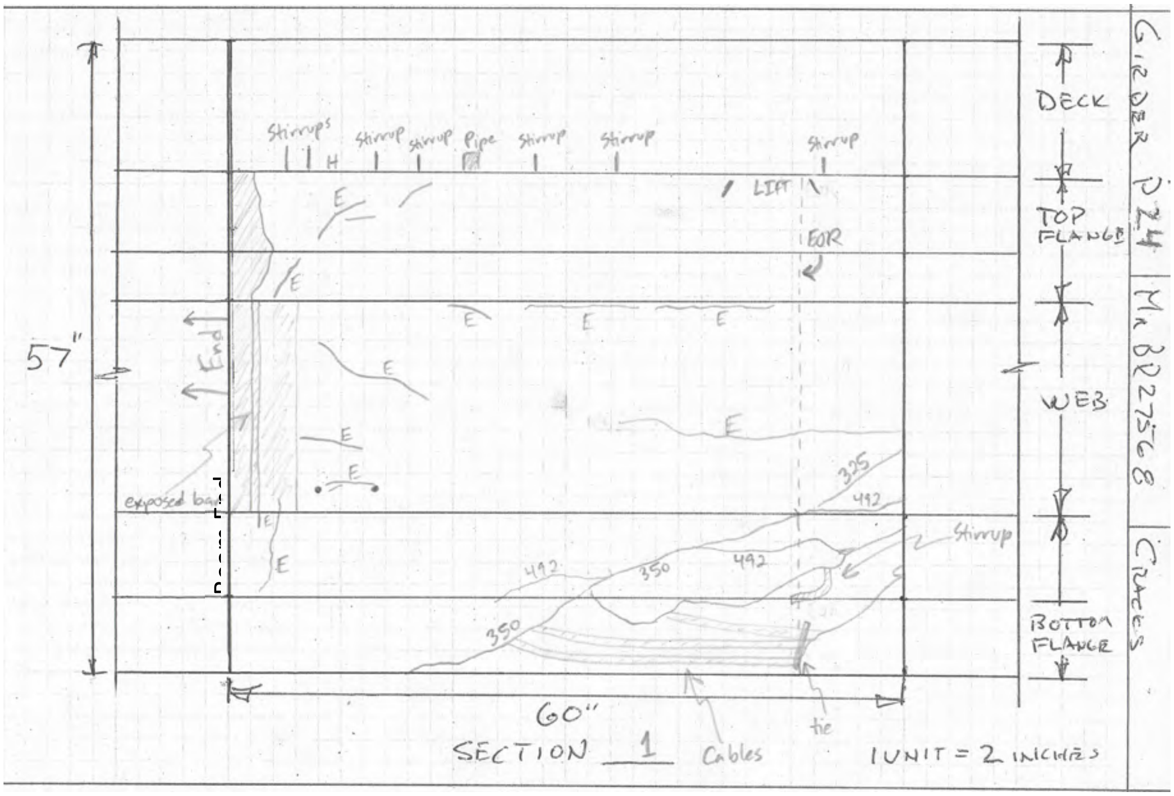


Figure 4.32 Girder P24 crack pattern – Section 1 (See Figure 3.4 for Section limits)

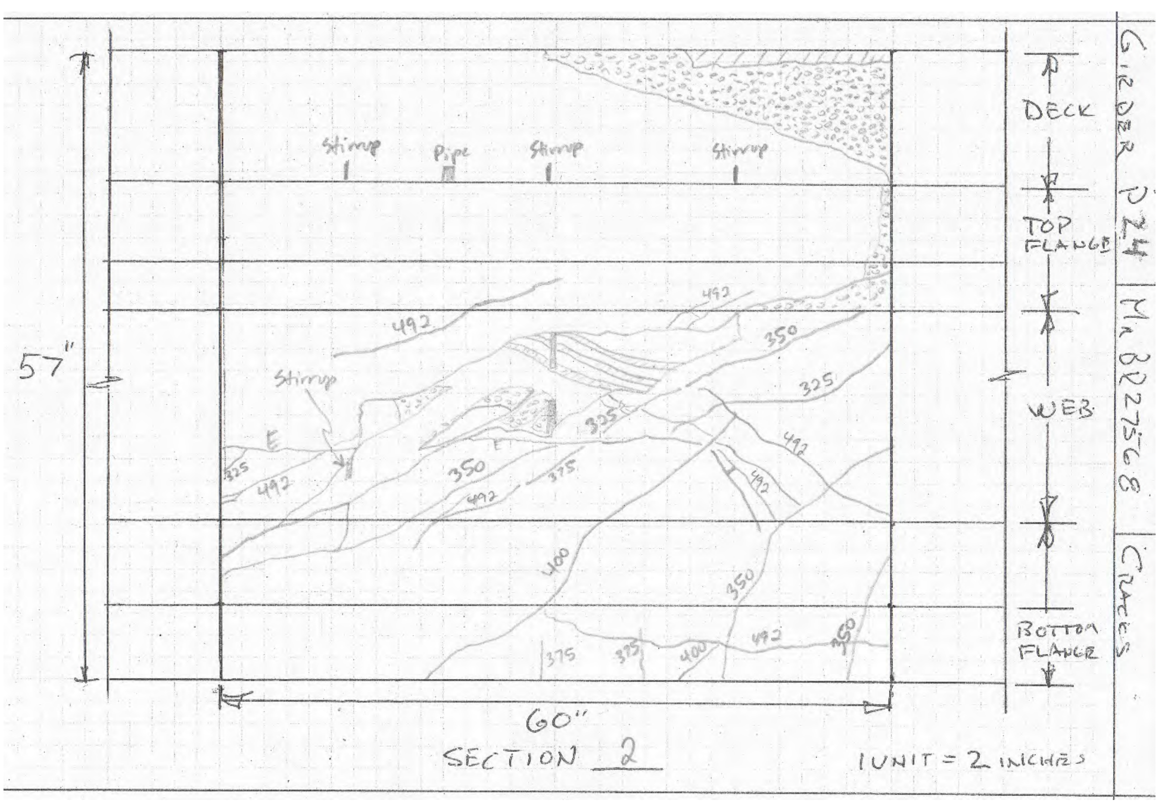


Figure 4.33 Girder P24 crack pattern – Section 2

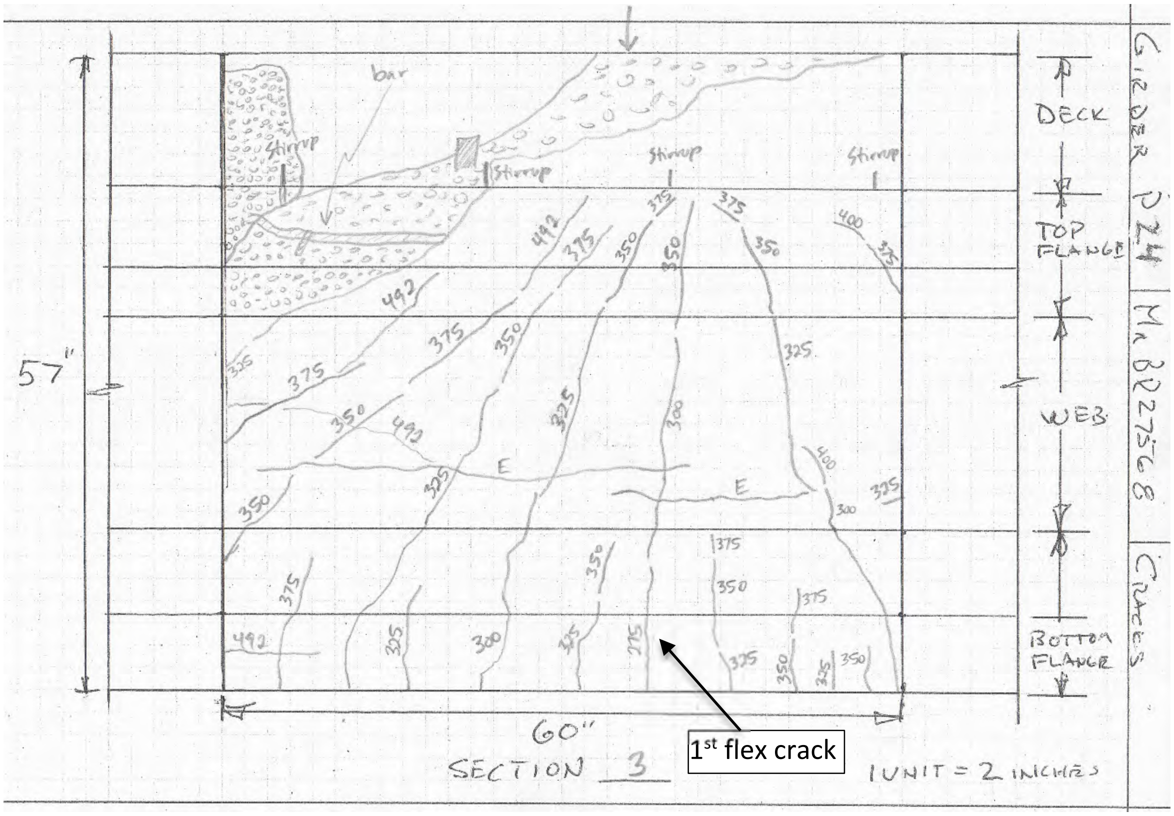


Figure 4.34 Girder P24 crack pattern – Section 3 (first flexural crack at 275k)

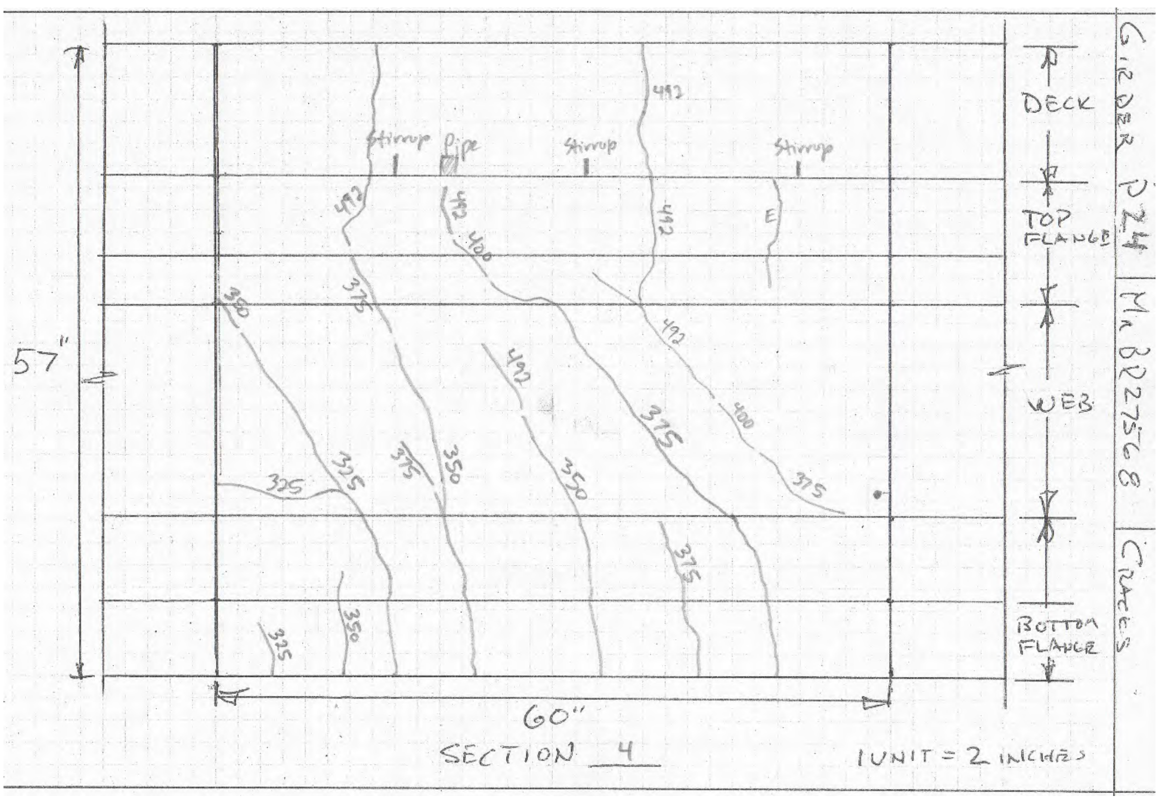


Figure 4.35 Girder P24 crack pattern – Section 4

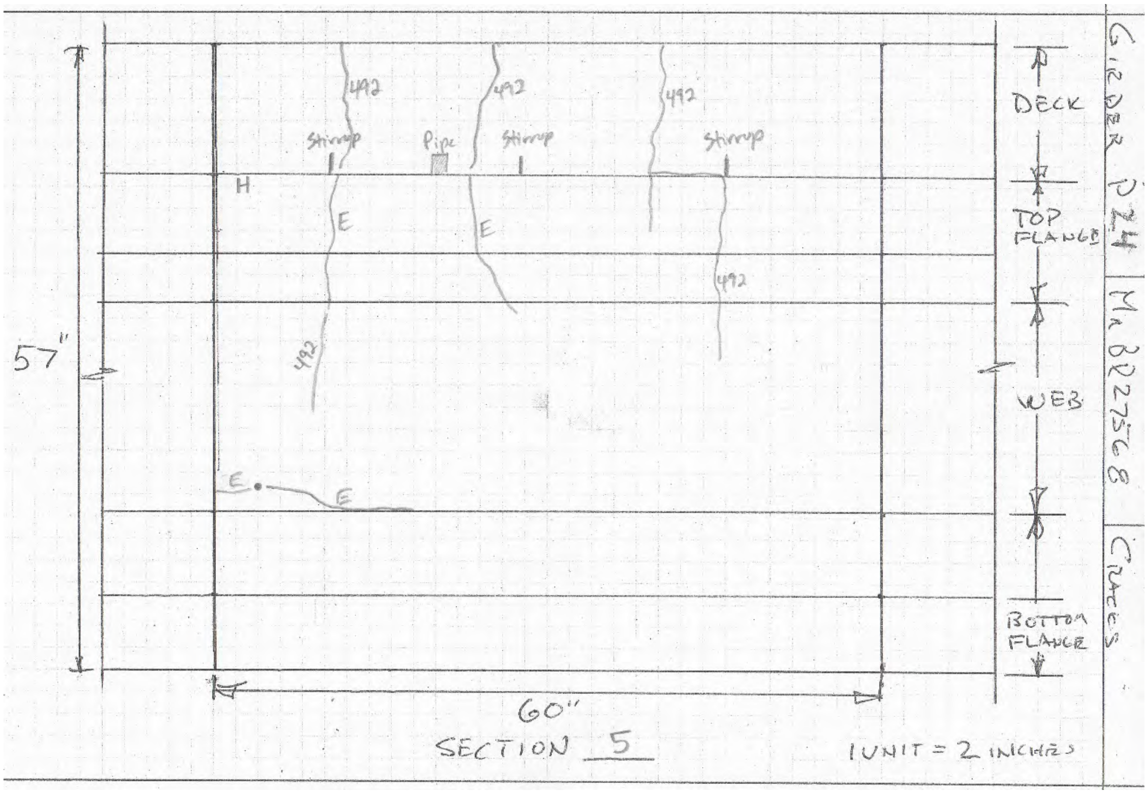


Figure 4.36 Girder P24 crack pattern – Section 5

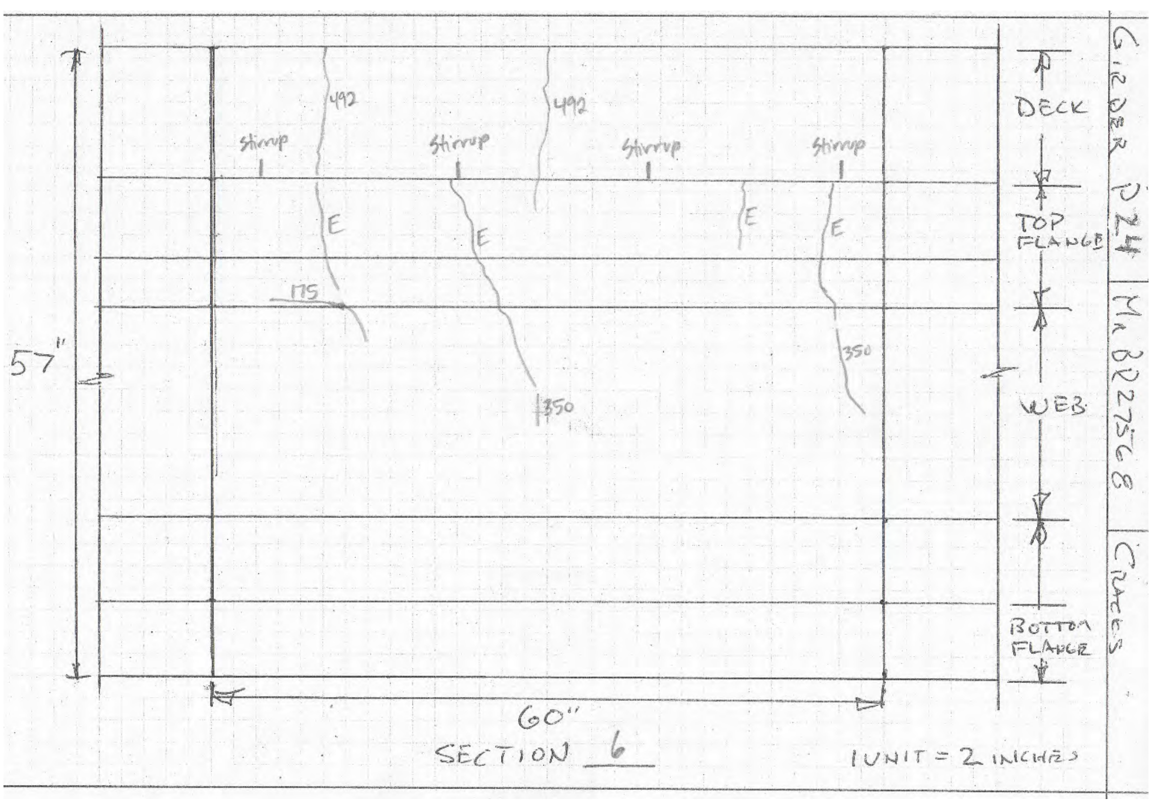


Figure 4.37 Girder P24 crack pattern – Section 6

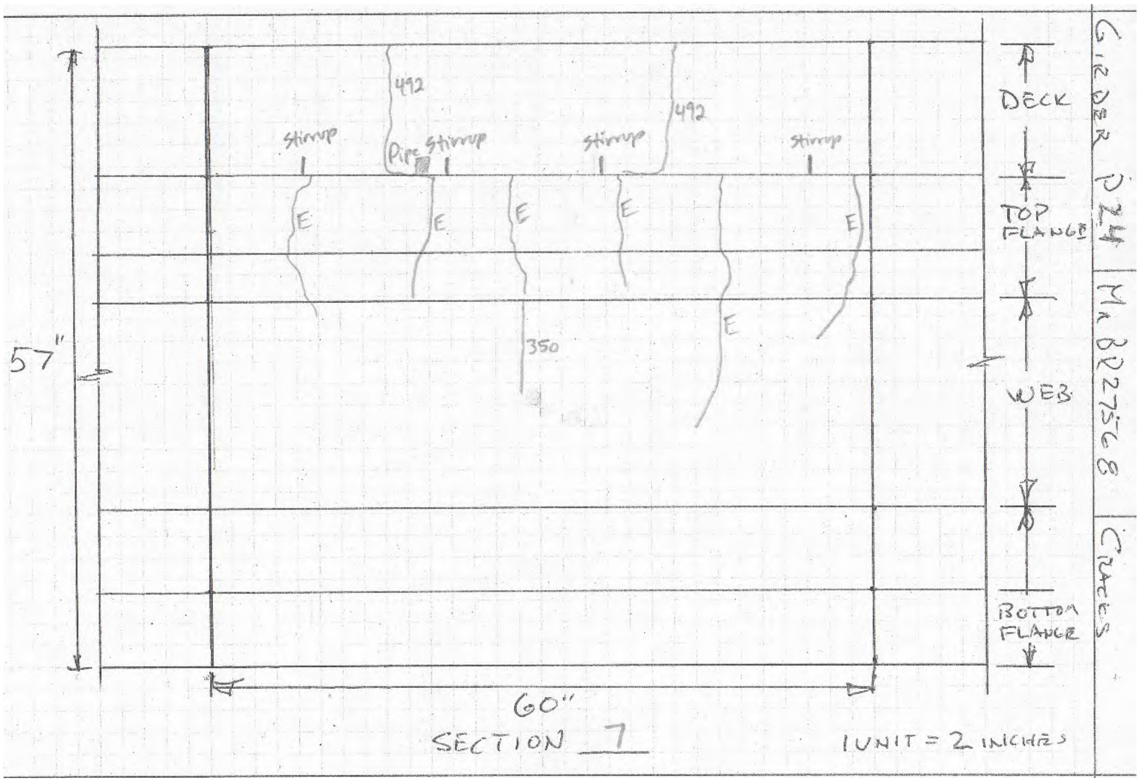


Figure 4.38 Girder P24 crack pattern – Section 7

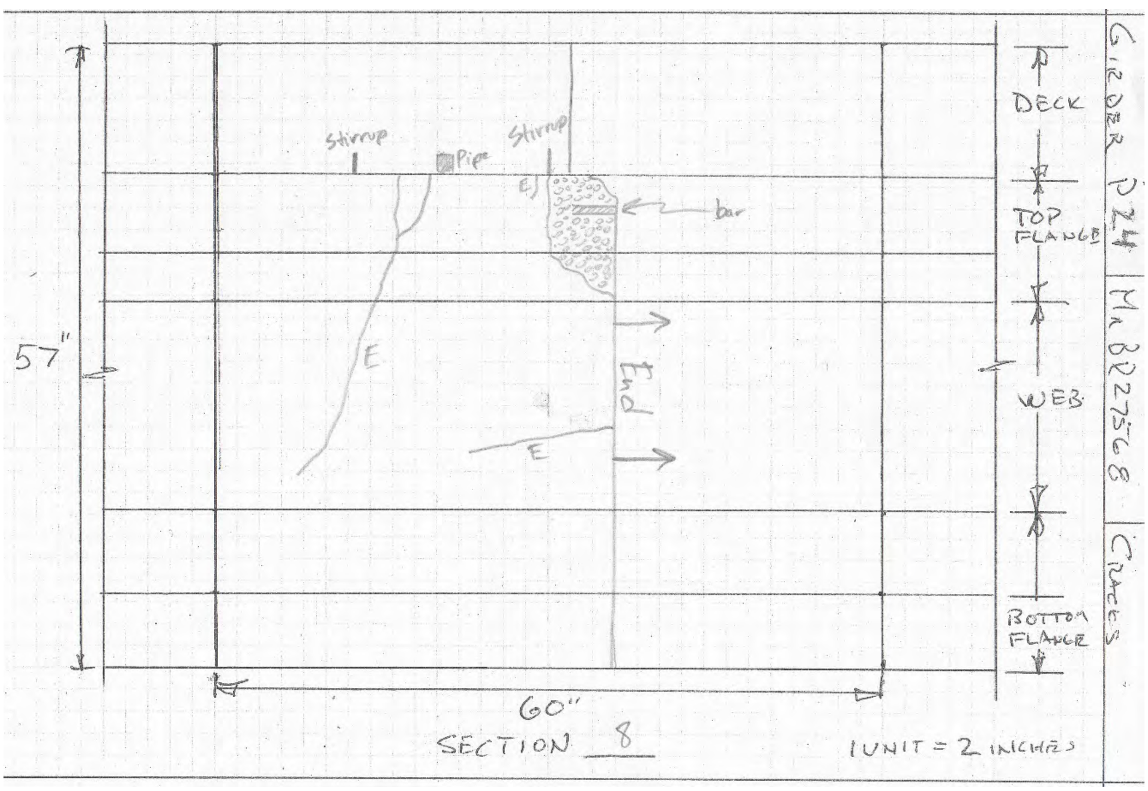


Figure 4.39 Girder P24 crack pattern – Section 8

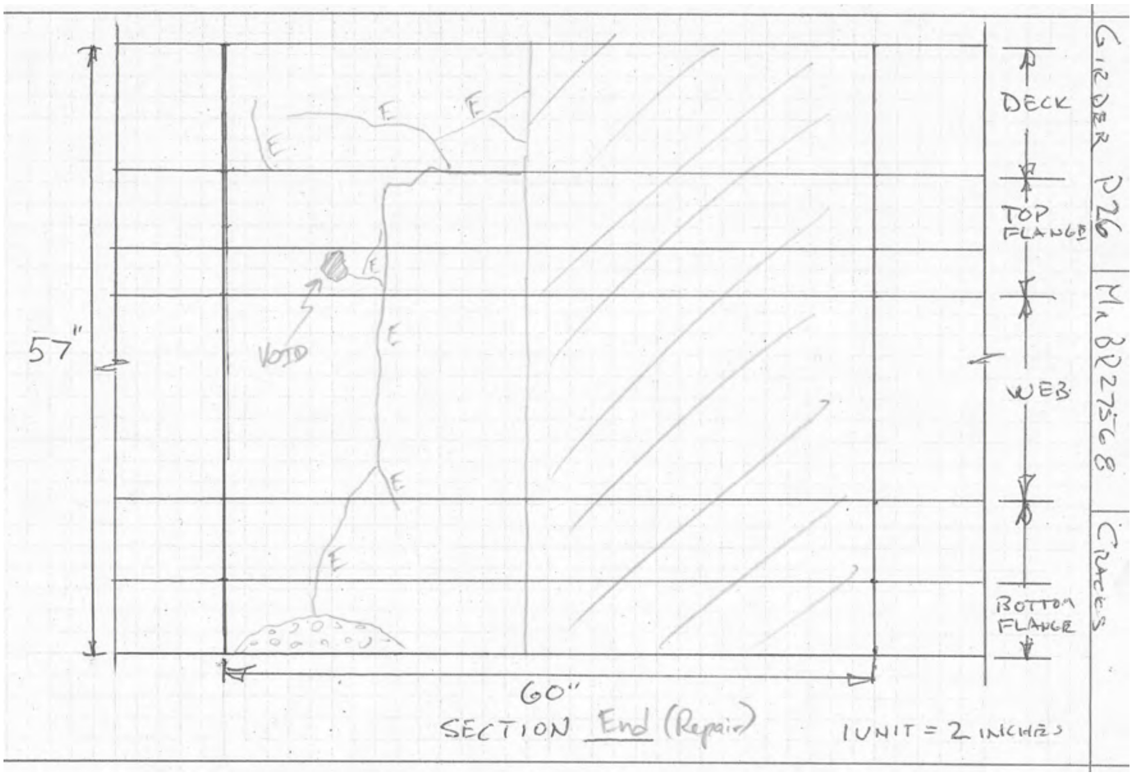


Figure 4.40 Girder P26 crack pattern – repair end view

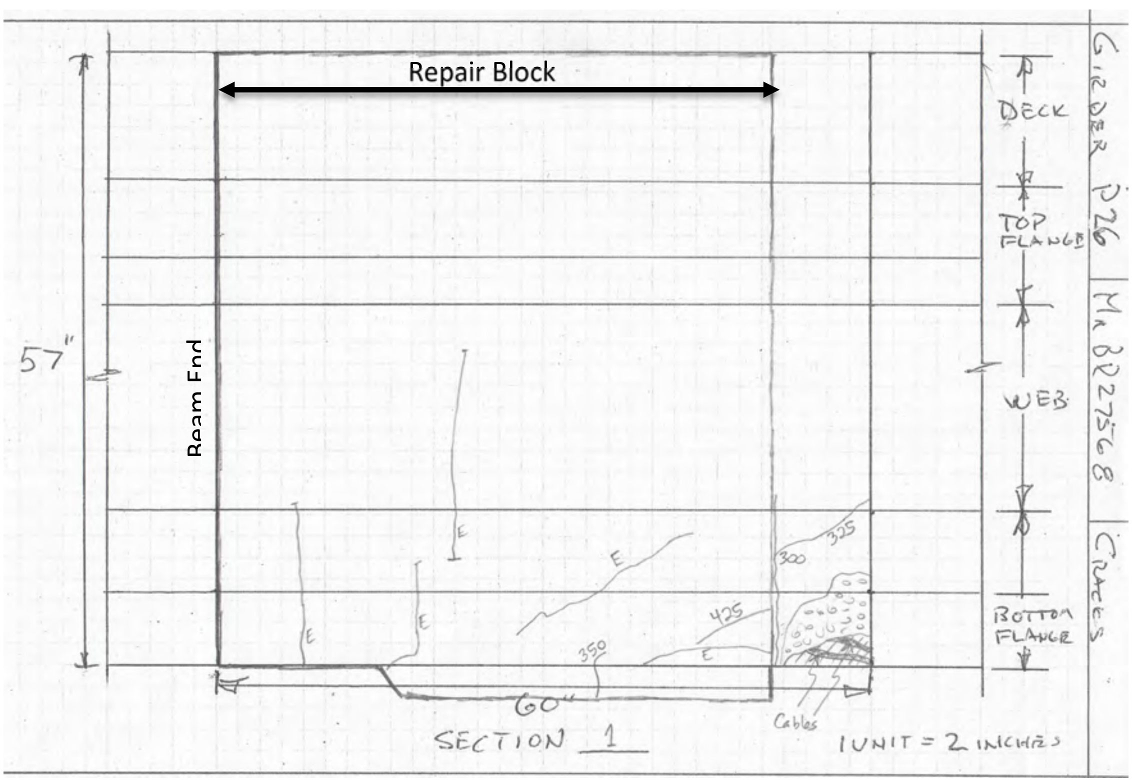


Figure 4.41 Girder P26 crack pattern – Section 1 (See Figure 3.4 for Section limits)

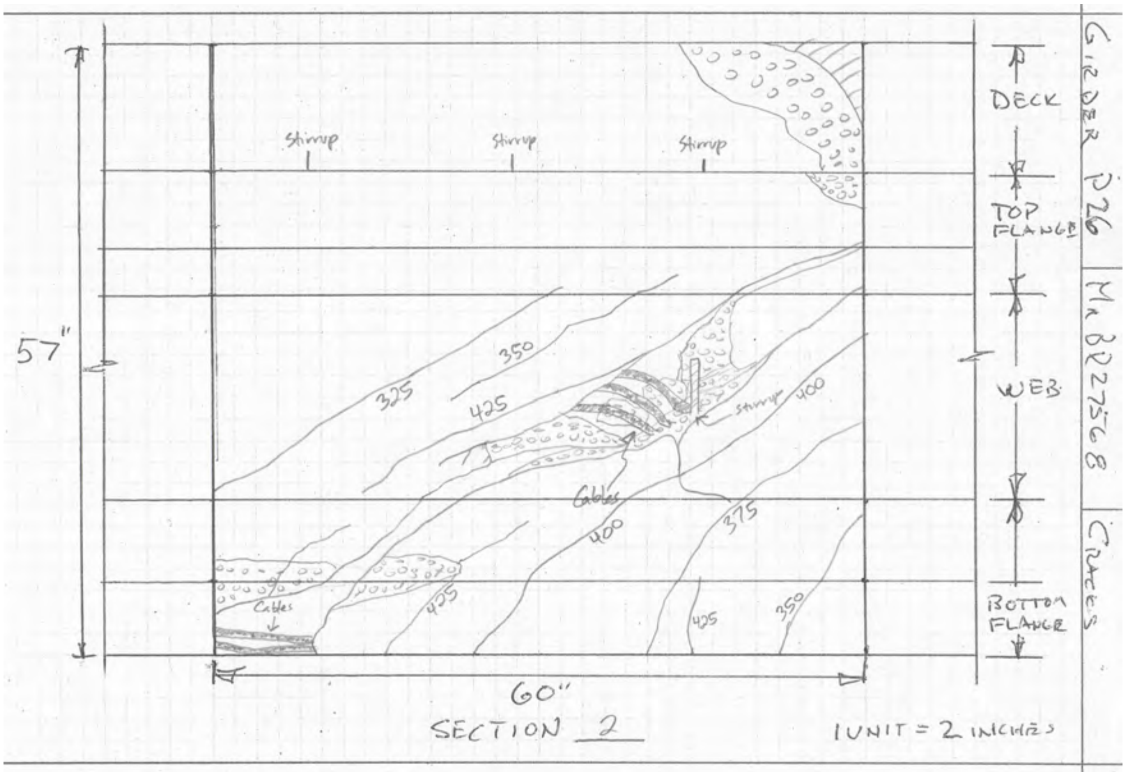


Figure 4.42 Girder P26 crack pattern – Section 2

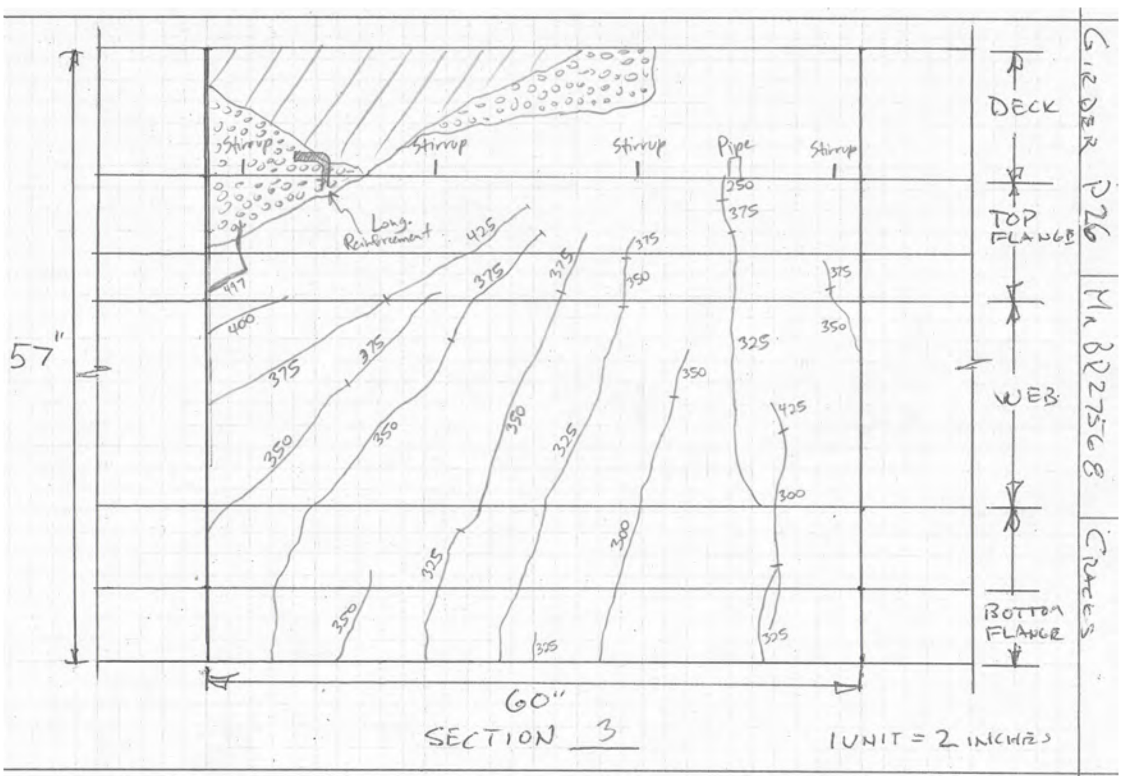


Figure 4.43 Girder P26 crack pattern – Section 3 (first flexural crack on South side at 300k)

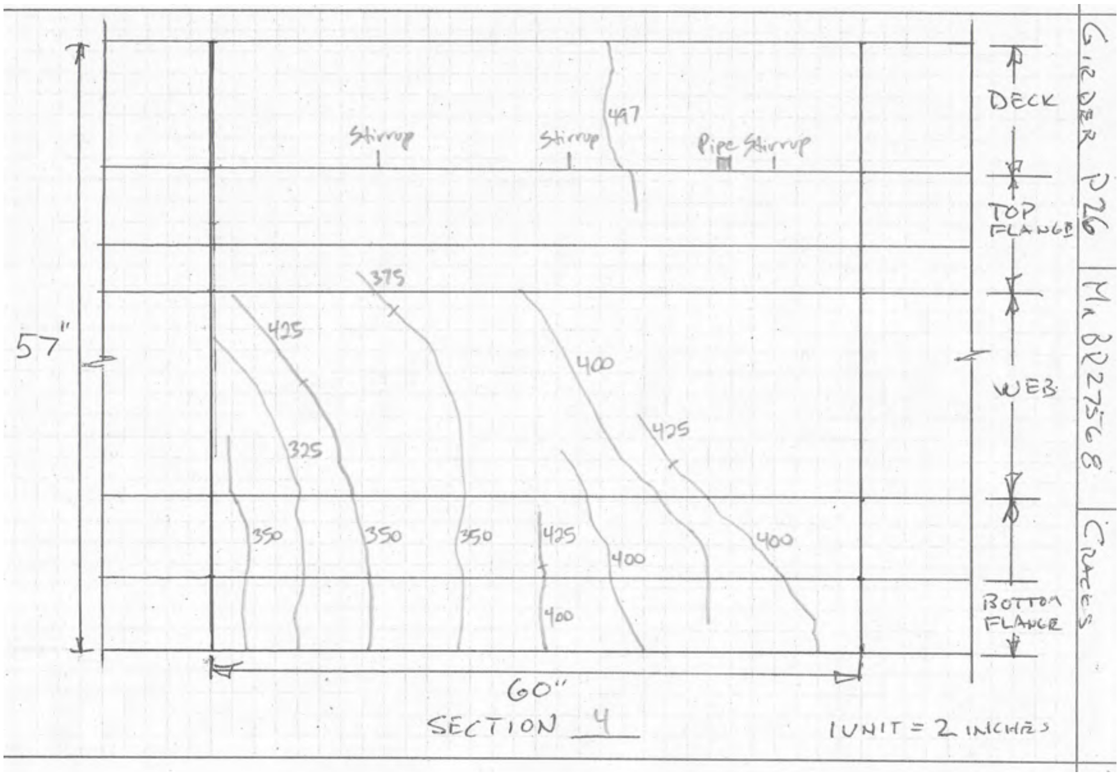


Figure 4.44 Girder P26 crack pattern – Section 4

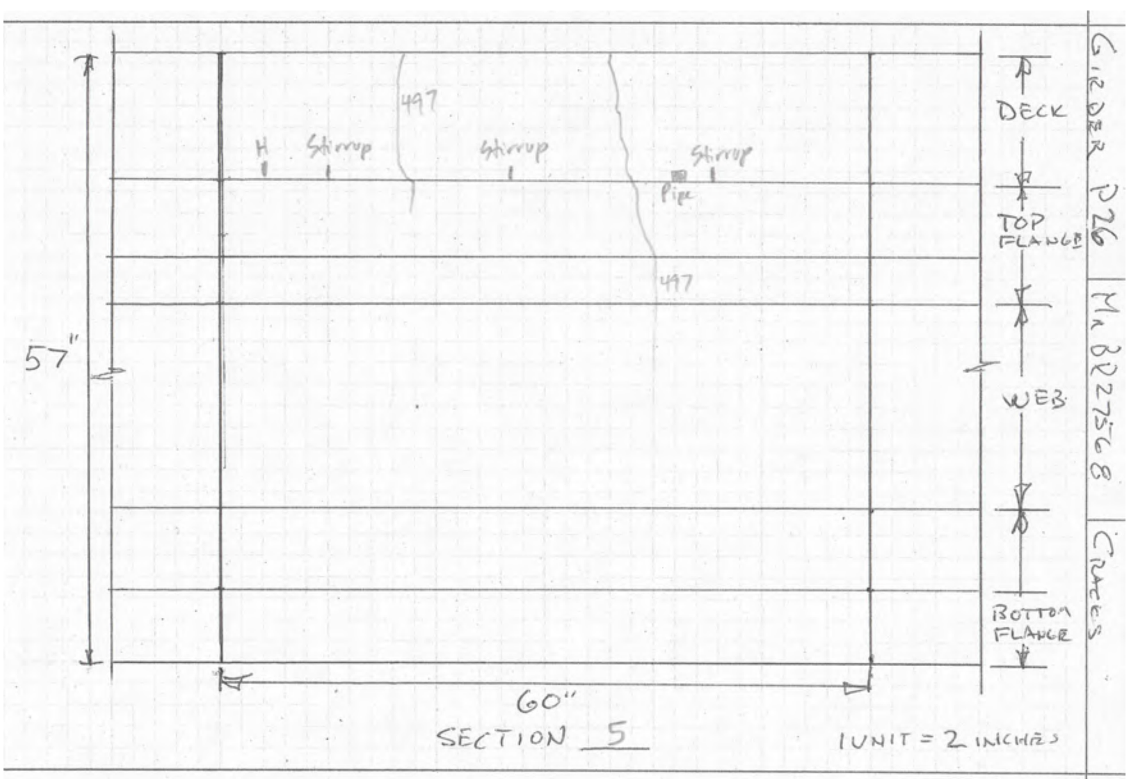


Figure 4.45 Girder P26 crack pattern – Section 5

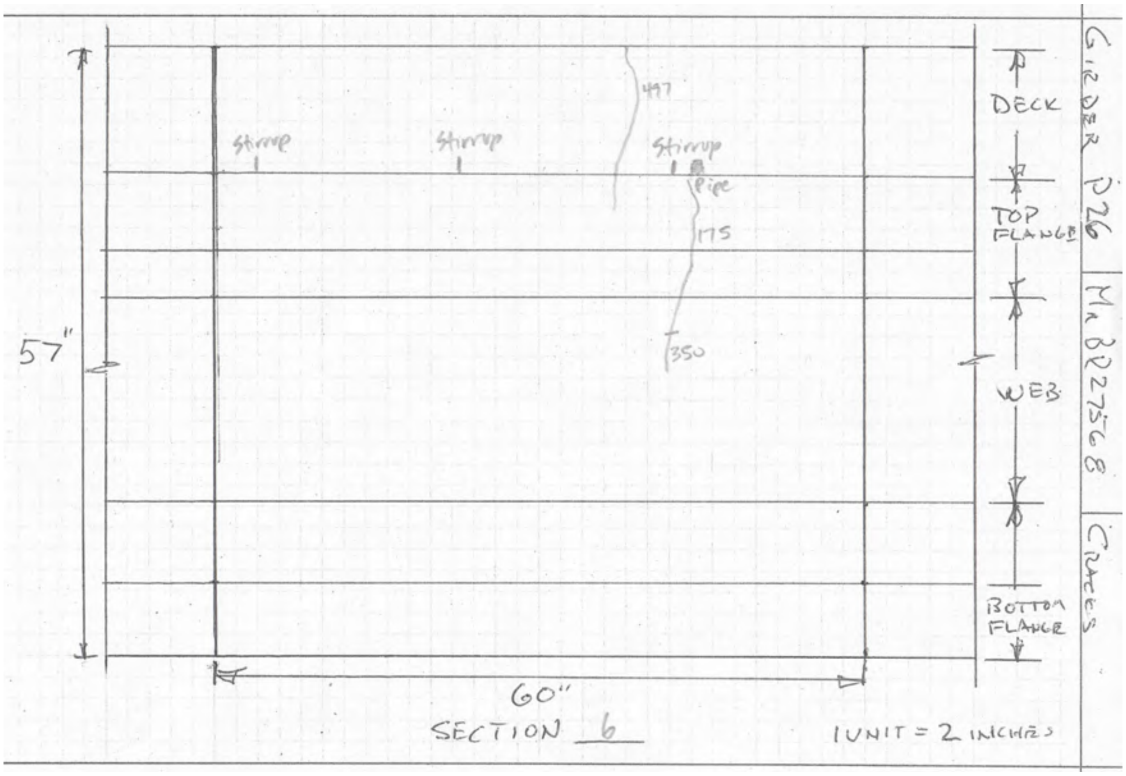


Figure 4.46 Girder P26 crack pattern – Section 6

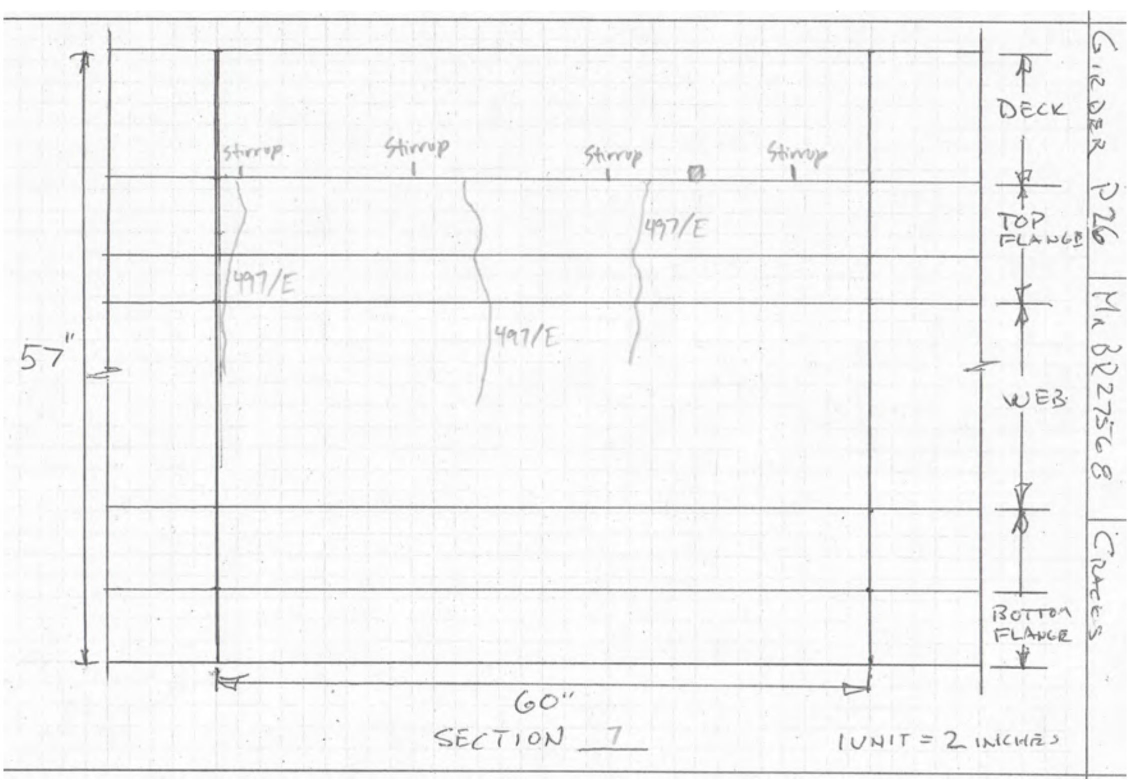


Figure 4.47 Girder P26 crack pattern – Section 7

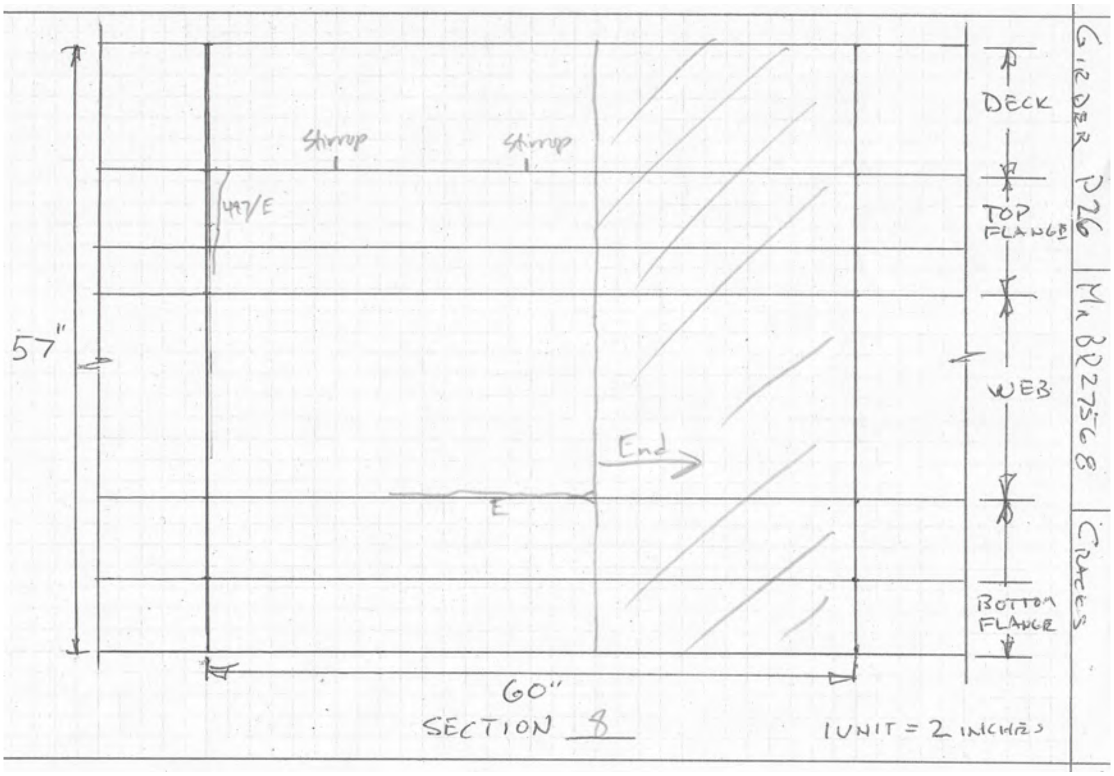


Figure 4.48 Girder P26 crack pattern – Section 8

Figure 4.49 Separation between the deck and top flange of Girder P26.

CHAPTER 5: SUMMARY AND CONCLUSION

The MnDOT Bridge Office was interested in determining if a prestressed girder end repair applied in 2013 on Bridge 27568 was effective at restoring the shear capacity of severely corroded prestressed bridge girders to its original state. Full-scale shear test of four girders taken from Bridge 27568 were completed at the University of Minnesota Galambos Structural Engineering Laboratory to investigate the efficacy of the repair.

Two sets of two girders that had similar girder concrete strength were salvaged from Bridge 27568 during demolition. One girder from each set had been repaired in the field by encasing supplemental reinforcement in shotcrete over the last approximately 4 ft. of the girder. The other girder from each set did not have this type of repair. Cores from Girders P2 (no repair) and P4 (repaired) had the same average concrete strength, 5400 psi. Cores from Girders P24 (no repair) and P26 (repaired) had average concrete strengths between 6700 and 6800 psi. The girders were cut to approximately 37.5 ft. by the demolition contractor and delivered to the University of Minnesota. Within the Galambos Structural Engineering Laboratory, 12 in. thick concrete decks were cast on the salvaged girders to ensure that the girders had sufficient flexural strength to withstand the shear test. An external shear strengthening system was used to ensure that the cut end of the girders would not fail in shear prior to the end of interest. The four girders were tested to failure in shear using the 600 kip testing machine in the Galambos Structural Engineering Laboratory. Girders P2(unrepaired) and P4 (repaired) failed at similar loads, with the unrepaired girder (P2) failing at a load 3% less than the failure load of Girder P4. Similarly, Girder P24 (unrepaired) and P26 (repaired) failed at even closer loads to each other, with Girder P24 failing a load 1.2% less than Girder P26. Upon completion of the tests, no new cracking was observed in the repair itself. The repaired section did not separate in any way from the girder during the testing.

From the close comparison in failure loads between the repaired and unrepaired girder in each set, it was concluded that the shotcrete repair investigated in this study returned the girder strength to that of the aged undamaged girders and is an effective rehabilitation strategy.

The ability to effectively repair corrosion damaged girder ends extends the useful life of prestressed concrete bridges. These repairs are significantly less expensive than beam replacement and represent conventional bridge repair construction. Bridge repairs that extend the useful life of existing structures almost always represent lower duration public impacts during construction than bridge replacements, and always at a lower cost. Experimentally demonstrating that the repair restores the girders up to the design strength enhances the safety of the bridge and provides MnDOT with a documented substantiated repair method that can be applied to other damaged prestressed concrete girder ends.

REFERENCES

Dereli, O., C. Shield, and C. French (2010). *Discrepancies in Shear Strength of Prestressed Beams with Different Specifications*. Minnesota Department of Transportation. Retrieved from http://www.cts.umn.edu/Research/ProjectDetail.html?id=2007032

Dymond, B., C. French, and C. Shield (2016). *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges*. Minnesota Department of Transportation. Retrieved from http://www.cts.umn.edu/Publications/ResearchReports/reportdetail.html?id=2550

APPENDIX A
MNDOT BEAM CONDITION BACKGROUND AND RESULTANT SHEAR COMPARISONS

by Paul Pilarski, P.E., MnDOT Metro North Region Bridge Construction Engineer

A.1 Bridge Background

Bridge 27568 was constructed by Hennepin County in 1975 while it was County Road 18. The structure was a 2,943 ft. long land bridge over swampy area where two previous roadbeds existed. The first bituminous paved roadway was about 4 ft. below the existing grade at the time of bridge repair. The upper paved roadbed was built onto approximately 4 ft. of fill atop the former bituminous roadbed. In 1975 the county elected to build a land bridge presumably to mitigate seasonal flooding of the roadway.

The structure was built with armored compression seals (Figure A.1) at even numbered piers over the 49-span structure. These compression seals do not adequately seal joints, especially where multiple joints exist within a bridge and there is potential for inconsistent movement within the structure. In addition, drainage scuppers were located at each expansion pier discharging chloride-laden runoff near piers onto the bituminous roadbed below. In 1987, the concrete beams were given an NBI rating of 6 with notes suggesting numerous small spalls on the fascia beams. The 1987 inspection report by the county also stated the joint material was pushed down under the barriers, the barrier cover/protection plates were rusted, and most joints were sand-filled. Details of the bridge layout, cross section and relevant fascia beam design are included in Figures A.2 through A.6.

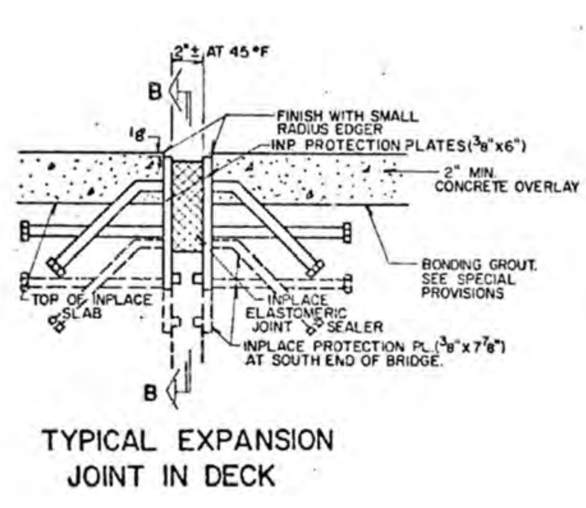


Figure A.1: Original expansion joint in service until 1995.

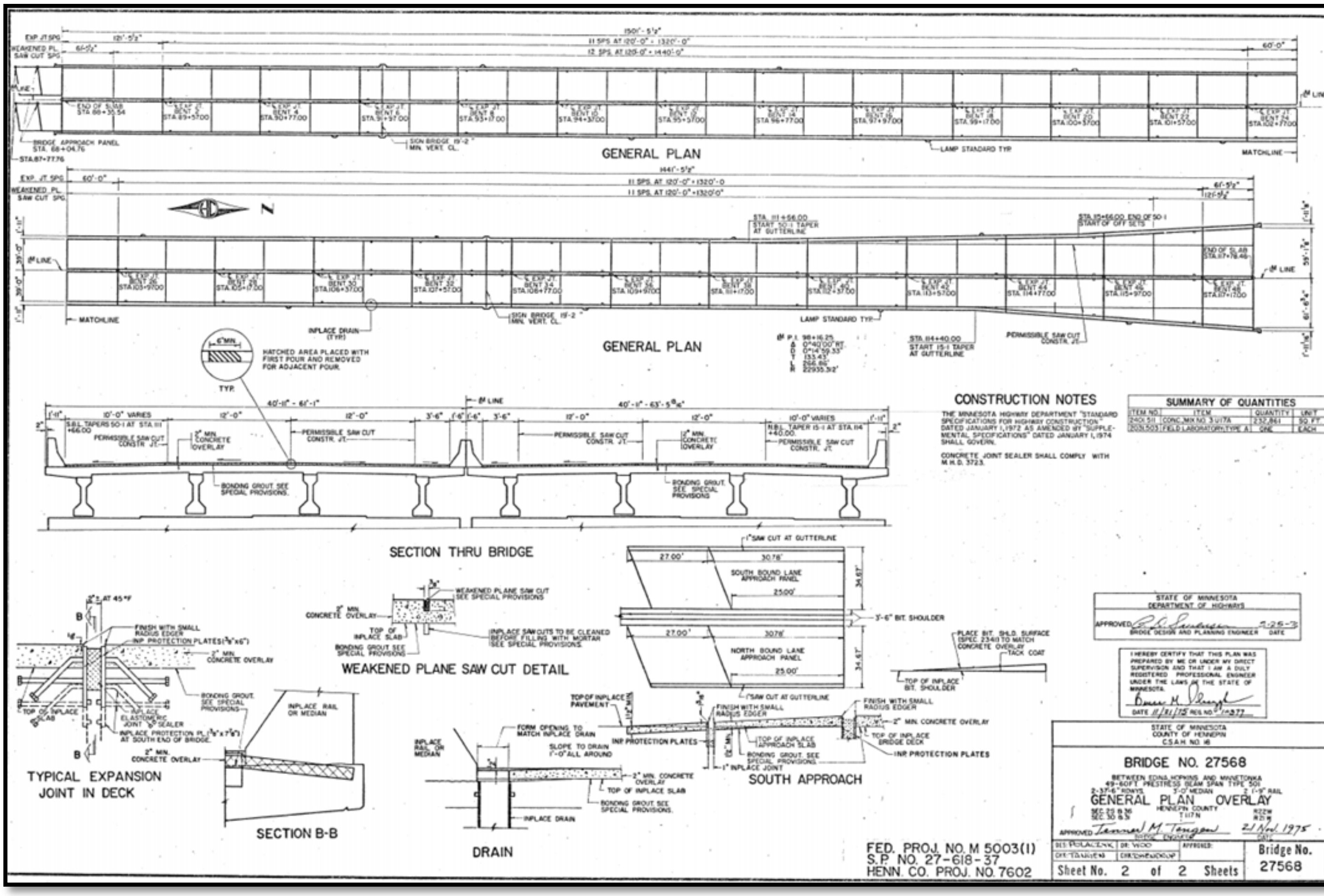


Figure A.2: Original Bridge plan and layout

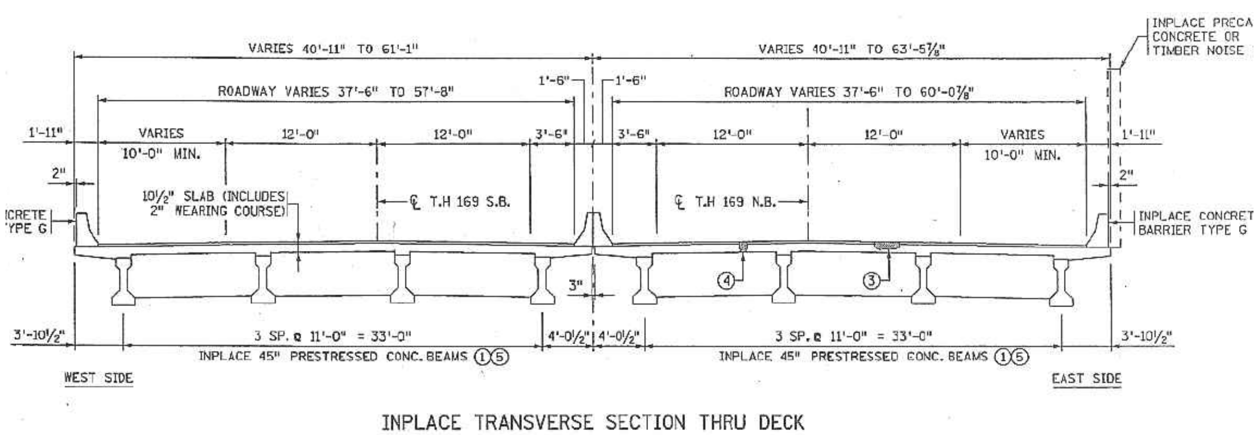


Figure A.3: Bridge cross section

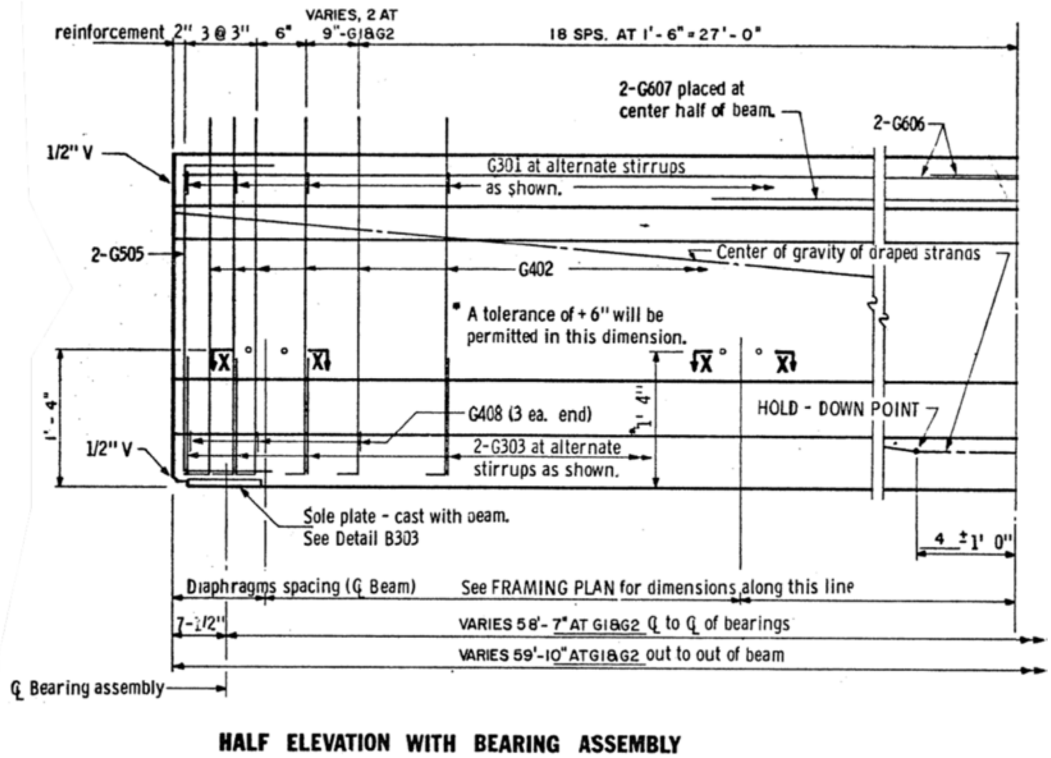


Figure A.4: Original plan fascia beam elevation view.

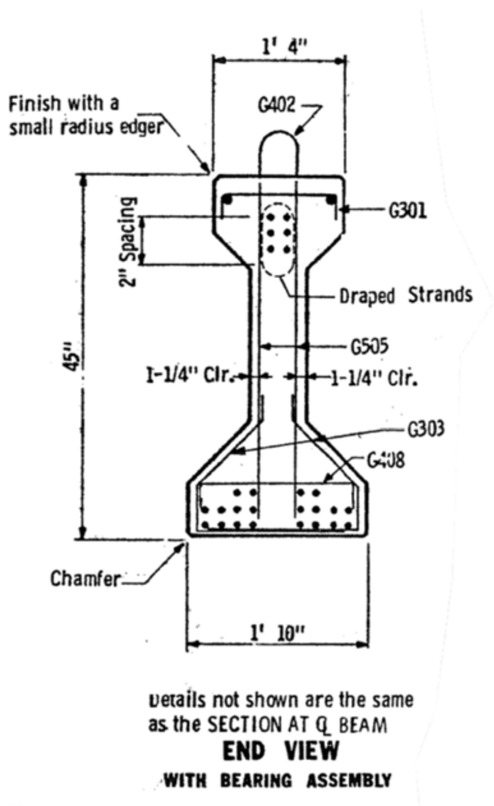


Figure A.5: Design beam end section at fascia beams.

MINIMUM CONCRETE STRENGTH - P.S.I.		
	① ③ f'ci	② ③ f'c
Computed Min. Concrete Strength	4310	5000
Required Min. Concrete Strength	4310	5000

① Minimum concrete strength at time of prestress transfer.

② Minimum concrete strength when curing can be discontinued and beam transported and installed.

③ Required minimum concrete strength shall be used. Computed minimum concrete strength is for information only.

Initial prestress
751842 lbs.
for each beam

Y DISTANCES (IN INCHES)			
	NO.	℄ SPAN	END
Straight strands	20	3.6	
Draped strands	6	5.0	38
Total strands	26	3.92	

Y = distance of Center of Gravity of strands from bottom of beam. All strands spaced 2" c-c, horizontally and vertically.

All strands 1/2" ϕ 270 kip, ultimate strength

Figure A.6: Original fascia beam design properties.

In 1988 the route became absorbed into MnDOT Trunk Highway 169. In 1995, MnDOT replaced bridge expansion joints with more robust strip seal style expansion joints. The damage had already been done, however, and the superstructure NBI dropped to 5 in 1995. By 2010 the superstructure was in need of repairs along with numerous piled bent caps which had absorbed the chlorides from the leaking joints over the 35 year history.

Scoping in 2010 suggested a combination of deck, superstructure and substructure repairs (See Figures A.7 and A.8). The repair contract was initiated in 2013 with two tiers of concrete repair quantities to manage potential overruns. Superstructure repairs in the contract were identified as Concrete Surface Repair, which most often takes the form of dry mix shotcrete with supplemental reinforcement where significant section loss is encountered. In addition, MnDOT included Thermal Sprayed Zinc, or metalizing, of the beam ends as a corrosion mitigation technique.

Figure A.7: Photo from October 2010 of Pier 4 beam end prior to final scoping

Figure A.8: Photo from October 2010 of Pier 26 beam end prior to final scoping

During the contract construction it was discovered that some beams had significant concrete damage, severe rebar and strand section loss. While the strand section loss posed low risk to the beam end shear capacity, stirrup losses and concrete cracking in the end region (See Figures A.9 through A.15 and Tables A.1 and A.2) demanded a more significant repair strategy than replacing lost concrete with shotcrete.

Figure A.9: Beam end at Pier 4 prior to preparation.

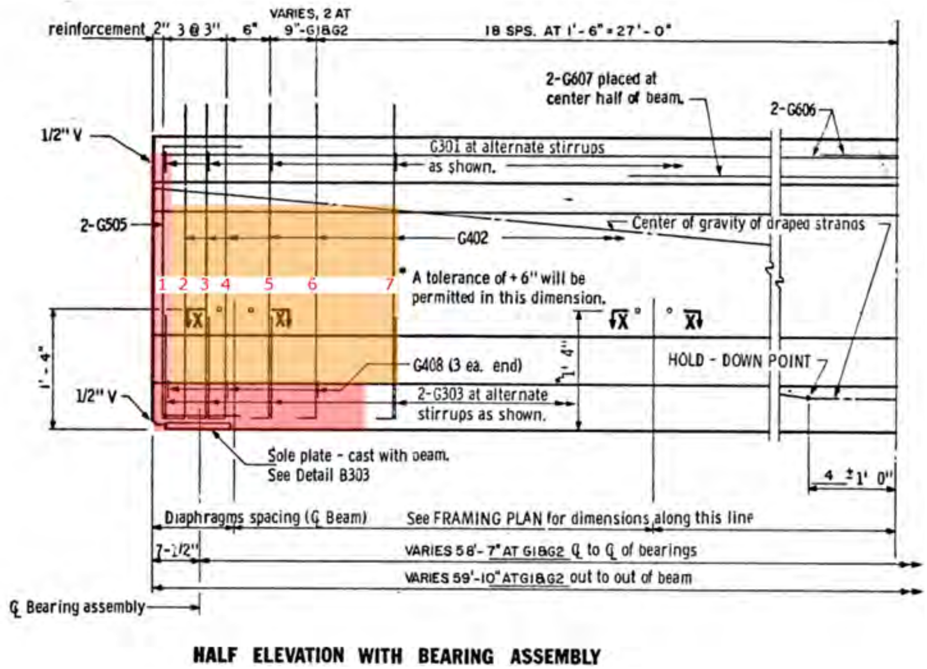


Figure A.10: Beam P4 (Fascia beam showing end at Pier 26)

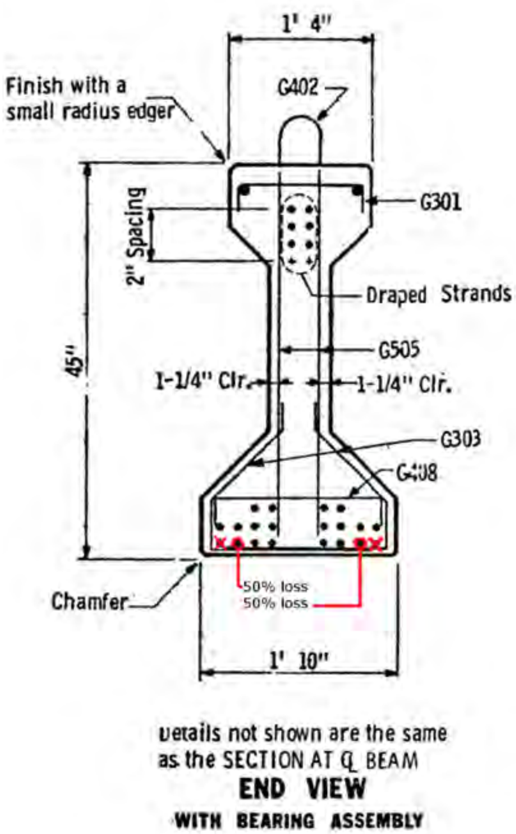


Table A.1: Stirrup losses estimated after cleaning steel. See elevation view for location.

Beam P4 stirrup losses (Areas include both faces of web in in^2)		
Stirrup location	Original Section	Est. Remaining Section
1	0.62	0
2	0.4	0
3	0.4	0
4	0.4	0
5	0.4	0.2
6	0.4	0.3
7	0.4	0.4

Figure A.11: Beam P4 estimated strand losses

Figure A.12: Sandblasted beam end at Pier 26 prior to issueing box-style repair design details.

Figure A.13: Underside of Pier 26 beam prior to preperation.

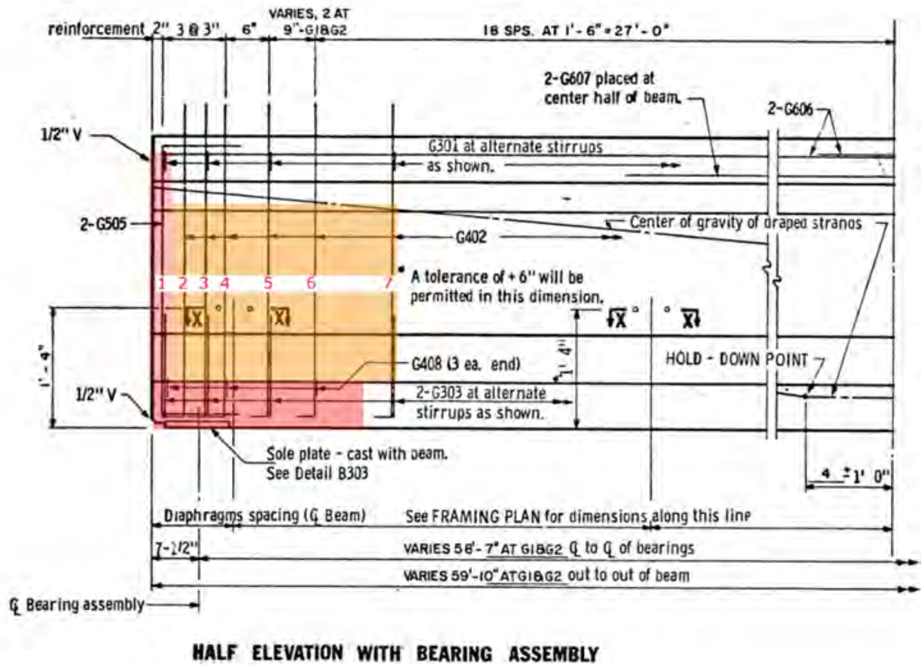


Figure A.14: Beam P26 (Fascia beam showing end at Pier 26)

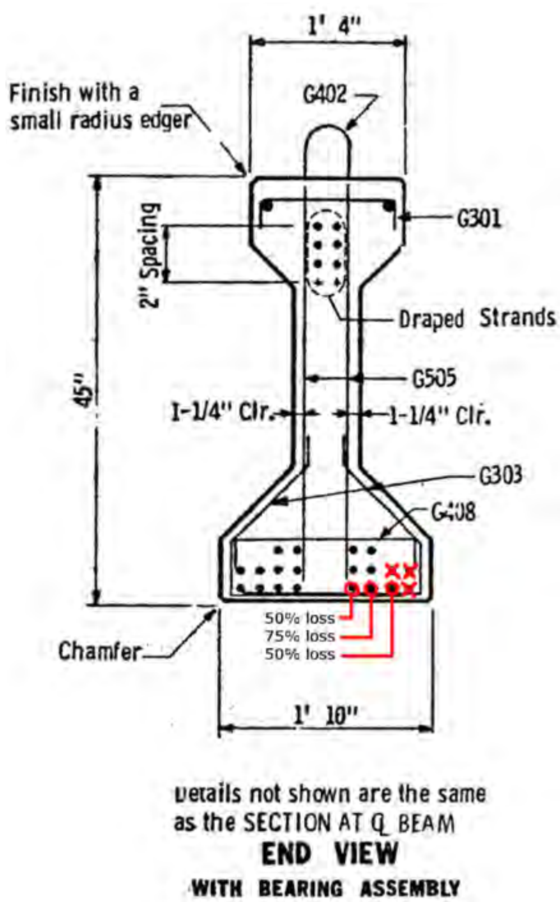


Figure A.15: Beam P26 estimated strand losses

Table A.2: Stirrup losses estimated after cleaning steel. See elevation view for location.

Beam P26 stirrup losses (Areas include both faces of web in in^2)		
Stirrup location	Original Section	Est. Remaining Section
1	0.62	0
2	0.4	0
3	0.4	0.1
4	0.4	0.1
5	0.4	0.2
6	0.4	0.2
7	0.4	0.2

Researching repairs implemented by other owners revealed that some encasement style repairs were developed by Michigan and FHWA in 1999. Details are found in Michigan research reports R-1373 and R-1380 entitled "Prestressed Concrete Beam End Repair," authored by Douglas Needham, PE with Michigan DOT (See Figure A.16). The Michigan repair technique utilized a conventionally reinforced CIP end block which was installed and tested in the laboratory. The repair material consisted of "Grade D latex-modified patch" mix. The conclusions of the report stated: "*From this experiment, we determined that exposing up to 305 mm of prestressing strands and 1 stirrup at the beam end does not result in significant prestress or shear strength loss.*"

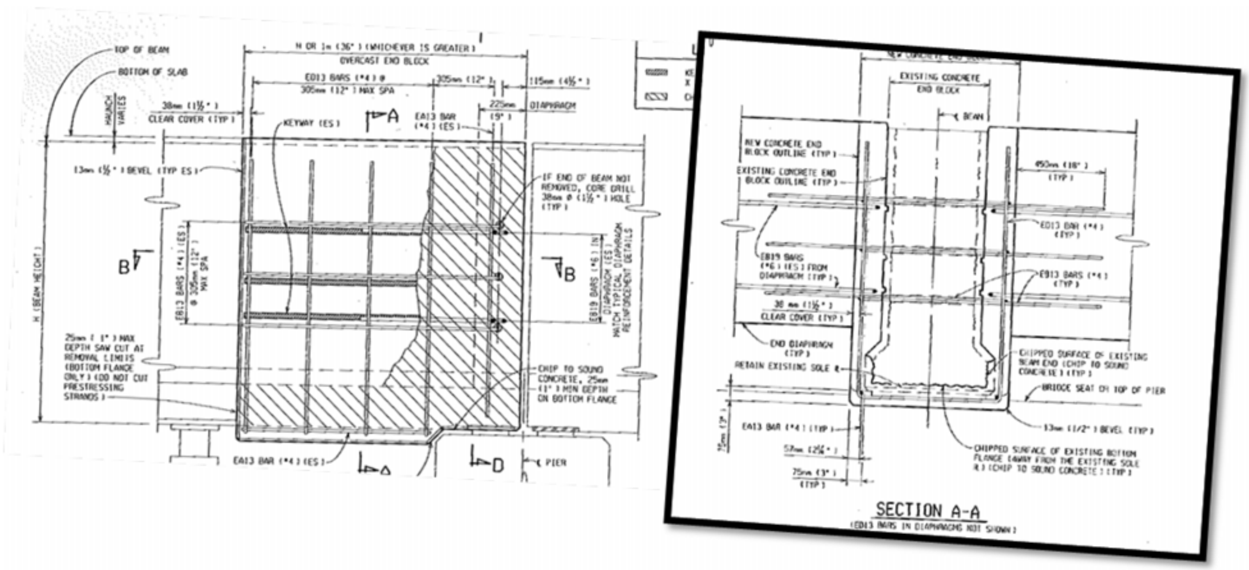


Figure A.16: Details from 1999 Michigan Report R-1373.

The Michigan research offered a possible strategy to the significant damage MnDOT was forced to repair within the construction project. There were large departures from the Michigan conditions and its conclusions, however. The MnDOT beams showed the following differences:

1. Stirrup losses exceeded the conclusions of the Michigan research.
2. Michigan research simulated a deteriorated beam end by chipping sound concrete and inflicting mechanical damage on intact stirrups. MnDOT beams suffered corrosion related damage with both degraded parent concrete and stirrup corrosion.
3. Michigan repairs were performed with full load relief in the laboratory. MnDOT would be performing the repairs in the field under traffic, albeit the locations being repaired were fascia beams.

Around the same time as this construction project, the MnDOT Bridge Office started questioning the long term behavior of shotcrete as a repair material, including needing an understanding of how to compute overall member resistance within reasonable accuracy. This is especially true when considering most members are not fully load relieved during the repair and curing. In MnDOT's perspective, there was inadequate research equating structural members repaired with differing repair materials to LRFD capacity equations. Some differences are that smaller aggregates are often used in repair materials and that the repaired member performance are very reliant on bond strength. The bond strength requirement is difficult to calculate and would vary based on the exact geometry repaired. In addition, there are several opinions within the concrete repair industry and within the MnDOT bridge office that shotcrete simply restores concrete cover and provided little structural benefit that should be relied upon.

While there were debates on these performance issues in the background, there were very few alternatives within the repair contract. Both form and pour style repair materials and high velocity dry-mix shotcrete were considered. It was decided that a high-velocity dry-mix air-entrained shotcrete including fibers for shrinkage reduction would provide the best chance for placement between strands and adequate bond strength. This material and method was already being used for pile bent repairs and lesser beam end repairs on the job. To mitigate concerns, the shotcrete-based block repair would be subjected to gradually increasing monitoring intervals for performance issues.

The repair was labeled as a “box end” repair, and execution required load relief of the beam end during the cure. The load relief requirement appeared sensible because there were no equations within the LRFD design specifications for shear capacity obtained through staged construction. It was thought that the beam end would perform most favorably in shear if the end was in a neutral stress state when repaired. In practice, however, the deteriorated beam end started cracking and splitting severely when subjected to load relief (See Figures A.17 and A.18). In response, the load relief was immediately switched to partial load relief only, where “partial” was considered to be 50% of the jacking pressure it took to see bearing lift-off. While the operation commenced there was concern that the continued presence of traffic would be detrimental to the repair performance and durability. There was a 10 ft. shoulder, and consequently traffic lanes near the fascia beam were not closed during the repairs.

The box end repair at Pier 4 was completed in Mid-October 2013 by Nick Senn, a certified ACI nozzleman employed by contractor PCI Roads, Inc. The shotcrete material was provided by King Concrete Industries, a bag mix designated MS-D1 SY. Pier 26 box end was completed on October 25th, 2013 (See Figures A.19 and A.20). Full dead load was restored after approximately 5 days from initial concrete placement, at which point the shotcrete gained the design strength of 5,000 psi. Due to the cooler fall overnight temperatures, heating and tarping was necessary but only a membrane curing agent was used (no water-based cure). Cracks within the repair appeared due to shrinkage and were marked and dated in mid-November. A follow-up mapping occurred at the end of December 2013, with some extensions of the original shrinkage cracks.

During spring 2014 MnDOT programming meetings there were many questions raised about the future of the bridge. Engineering judgement of past shotcrete performance placed the remaining life estimates at 7 -10 years for bridge replacement, with the caveat that bridge end of life performance was always difficult to estimate. Some concern was expressed about the timing of replacement due to not only the beam ends but other concerns surrounding the longevity of repairs made to the precast, prestressed piling. For several reasons, a bridge replacement was programmed through Design-Build procurement in the calendar year 2017.

A.2 MnDOT Research Initiative and Construction Experience

Due to the replacement scheduling, an opportunity to test the repaired girders alongside beams in good condition was pursued. To keep the costs of the research within constraints, no dedicated University research staff were assigned to perform calculations or detailed monitoring of the tested beams. The bridge repairs remained in service from October 2013 through bridge removal in March 2017 without serviceability issues. The replacement contractor, Ames Construction, salvaged the beams by change order, carefully removed the deck concrete outside the repair area, and sawcut the beam to length that the laboratory could handle.

This load testing research was the first attempt by MnDOT to answer questions surrounding the structural performance of field applications of shotcrete repair materials. The testing demonstrated that good adhesion exists with high-velocity dry mix shotcrete *even* when best practice might not have been followed with preparing the substrate. Such practice includes:

1. Removing any non-cementitious contaminants. In this case, the thermal sprayed zinc was inadvertently placed ahead on the repair substrate ahead of the shotcrete and not removed.
2. Prewetting the substrate and maintaining a saturated surface substrate for 12 hours minimum prior to shotcreting. No prewetting occurred outside the nozzle placement of shotcrete.
3. Curing with a 72-hour minimum wet cure. Only spray applied membrane cure was used in two coats.
4. Removal of all unsound concrete (Since the body of the web in the end region was so poor, there was no choice but to bury the unsound concrete within the box end repair)

While the above best practices were not followed, the load tests show good performance of the repair itself (See Figure A.21). Cracking was limited to shrinkage cracking. This shrinkage cracking largely arrested after the first few months with minor extensions within the first year in service. The shotcrete repair forced any failure to occur in the weaker reinforced zone outside the beam end, effectively behaving like partial end support due to the change in beam stiffness.

MnDOT has since installed four other installations of the box end repair at beam ends without the use of temporary support. Costs for the box end repair range between $5500 and $12,500 each location, depending on access and whether temporary shoring and jacking is included.

Figure A.17: Separation of concrete web concrete upon slow jacking from temporary support location. Separation was likely due to rebalancing of internal prestress forces. After the separation was reported the jacking pressure was immediately reduced to 50% dead load take-up from 100% dead load. The repair proceeded under 50% dead load take-up.

Figure A.18: Prepared beam end at Pie 26 prior to installation of mild rebar.

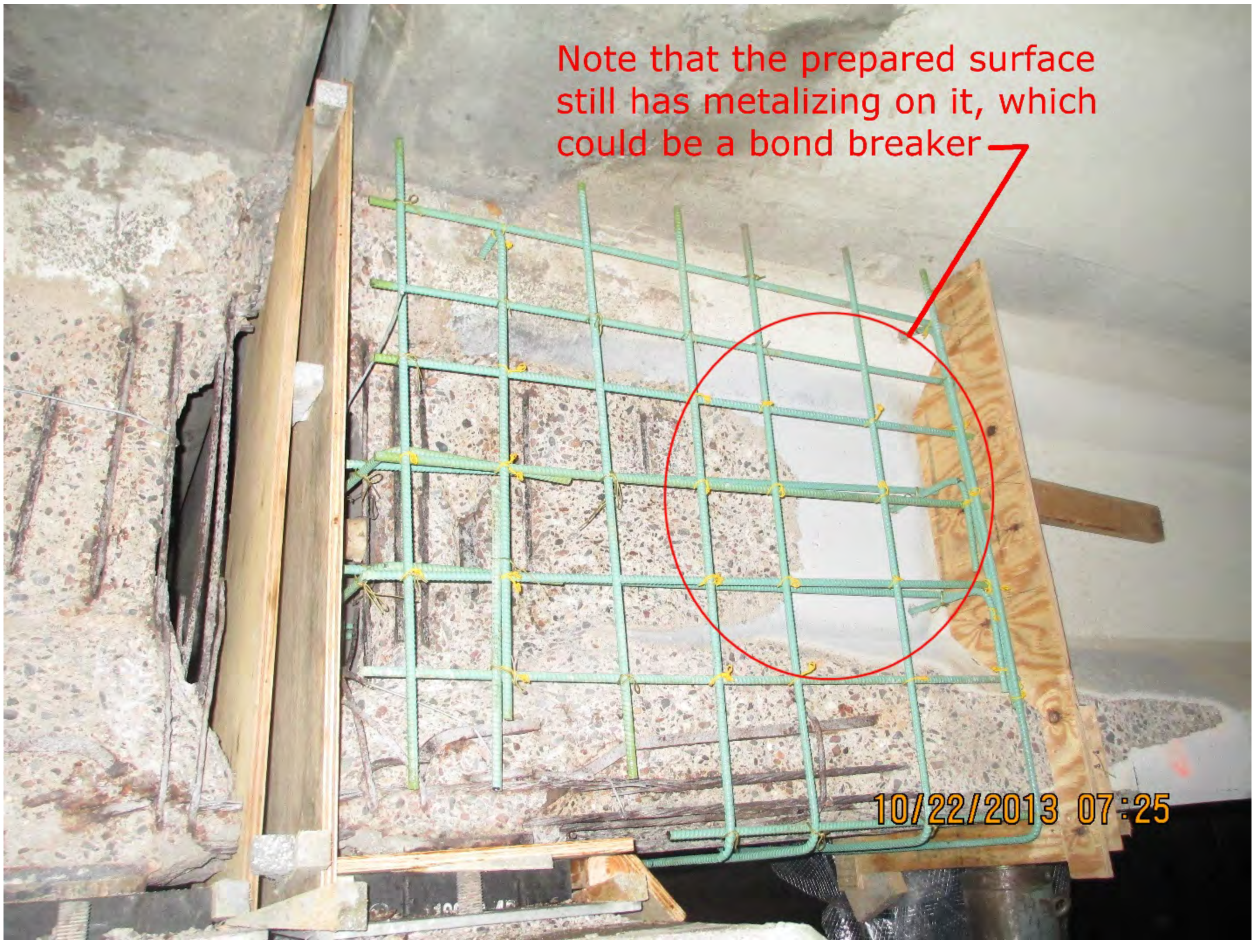


Figure A.19: Prepared Pier 26 beam end just prior to shotcreting.

Figure A.20: Inside face view of P26 beam end just prior to shotcreting.

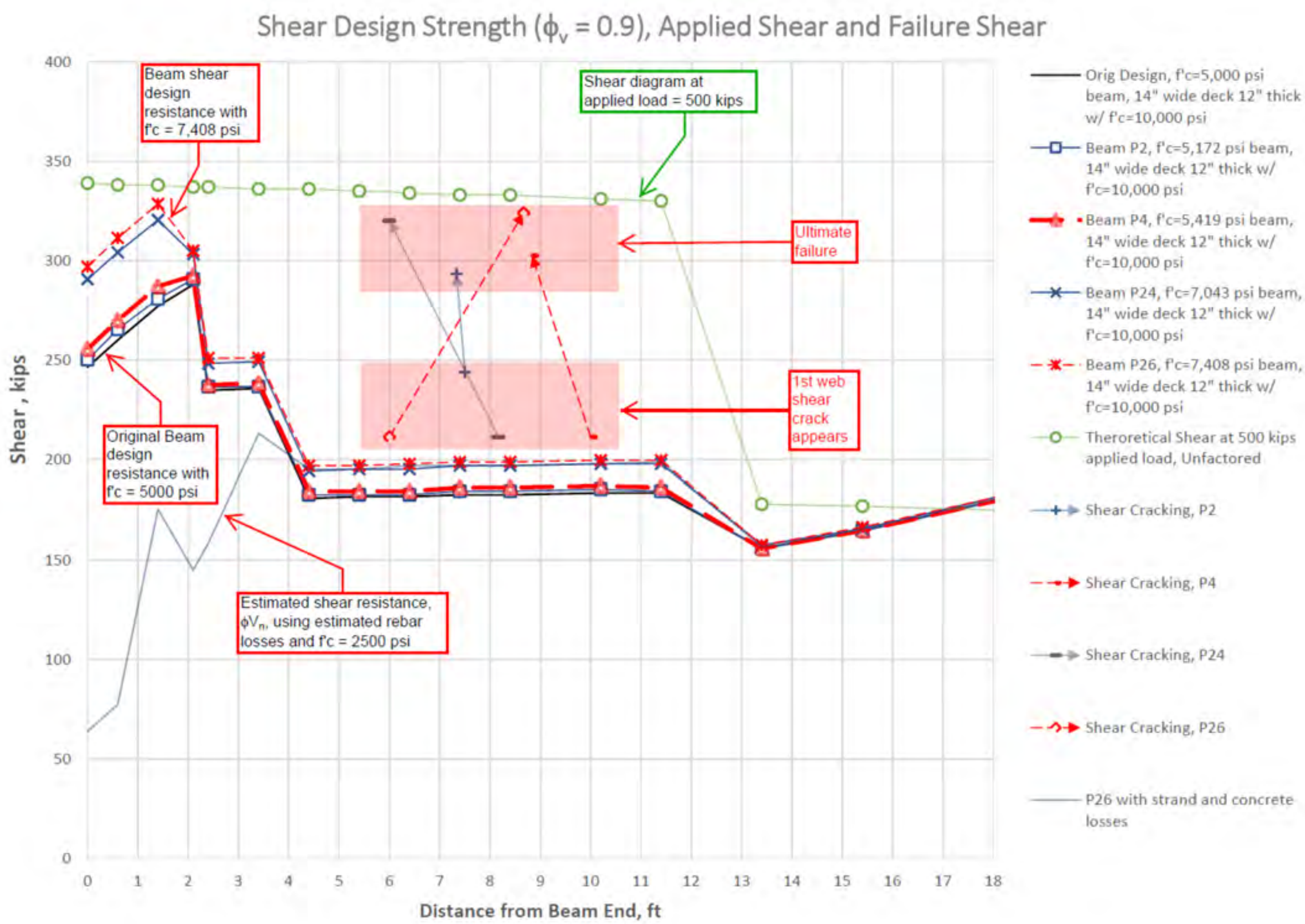


Figure A.21: Shear design resistance as compared to testing results.

APPENDIX B
AS-RECEIVED AND TESTING PHOTOGRAPHS

B.1 INTRODUCTION

This Appendix contains the series of photographs of Girders P2, P4, P24, and P26 taken when the girders were received to document the as received condition and during testing on both the north and south side of the girder, focusing on the tested end.

B.2 GIRDER P2

Figures B.1-B.7 show the condition of Girder P2 when it was received, prior to deck casting. Figures B.8-B.16 show pictures of Section 1 of the south side of Girder P2 during testing. Figures B.17-B.24 show pictures of Section 1 of the north side Girder P2 during testing.

Figure B.1 Girder P2 Prior to Casting – Section 1

Figure B.2 Girder P2 Prior to Casting – Section 2

Figure B.3 Girder P2 Prior to Casting – Section 3

Figure B.4 Girder P2 Prior to Casting – Section 4

Figure B.5 Girder P2 Prior to Casting – Section 5

Figure B.6 Girder P2 Prior to Casting – Section 6

Figure B.7 Girder P2 Prior to Casting – Section 7

Figure B.8 Girder P2 Prior to Casting – Section 8 (Cut-End)

Figure B.9 Girder P2 Southside Section 1-Section 2 *P*=250 kips

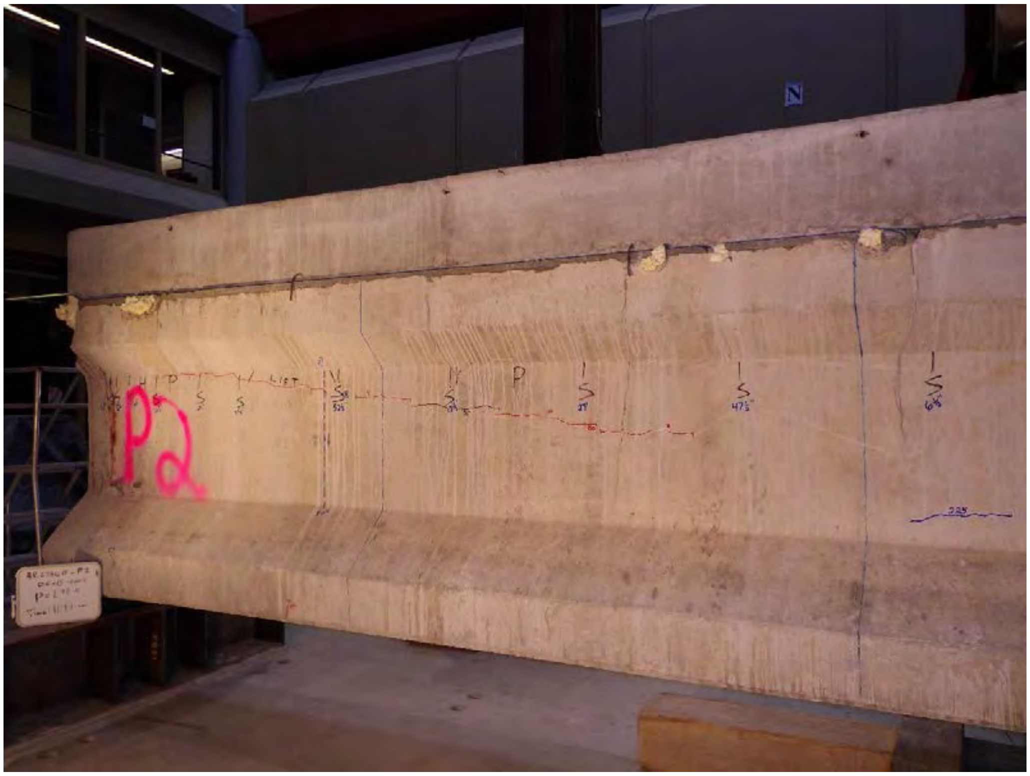


Figure B.10 Girder P2 Southside Section 1-Section 2 *P*=275 kips

Figure B.11 Girder P2 Southside Section 1-Section 2 *P*=300 kips

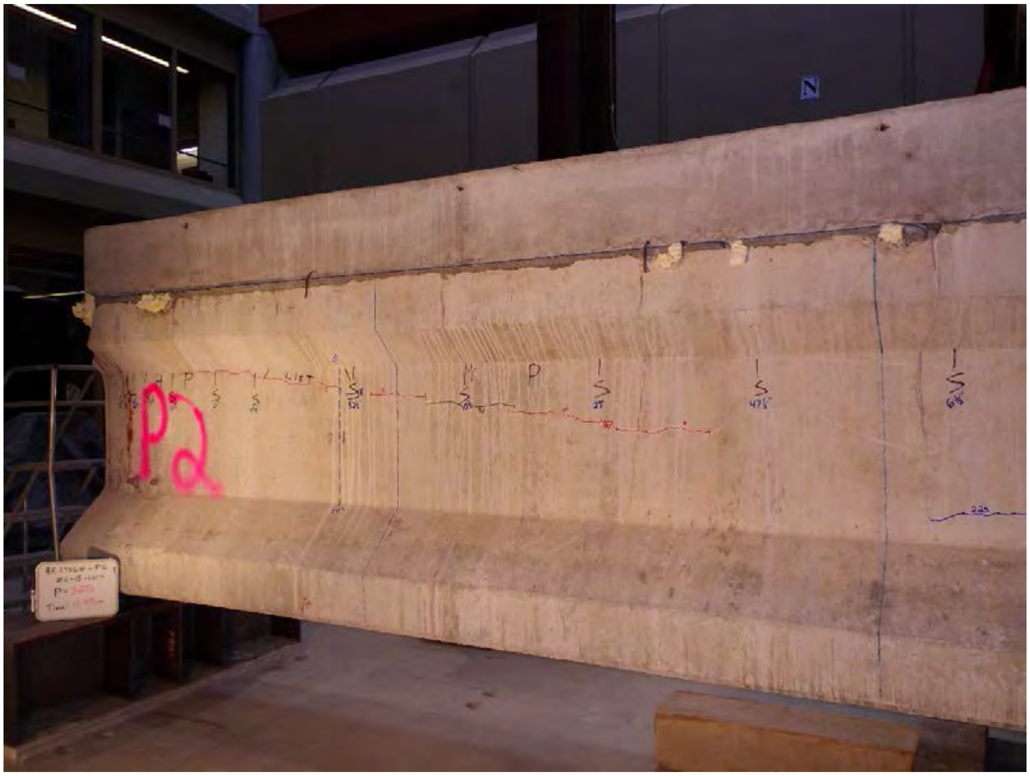


Figure B.12 Girder P2 Southside Section 1-Section 2 *P*=325 kips

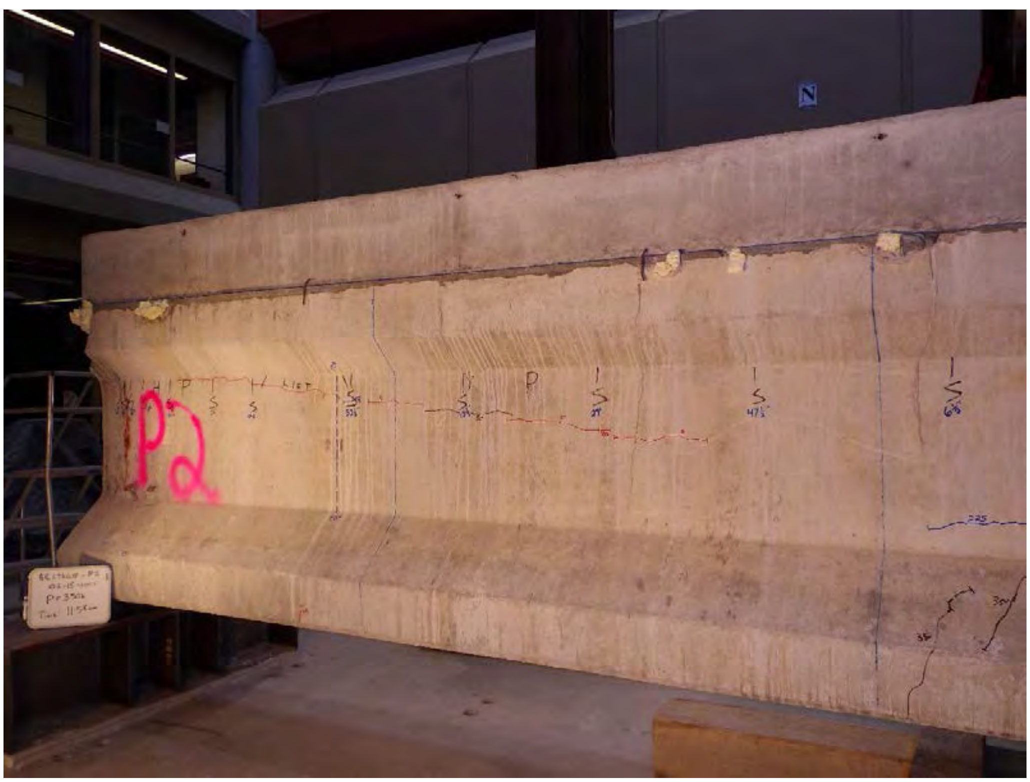


Figure B.13 Girder P2 Southside Section 1-Section 2 *P*=350 kips

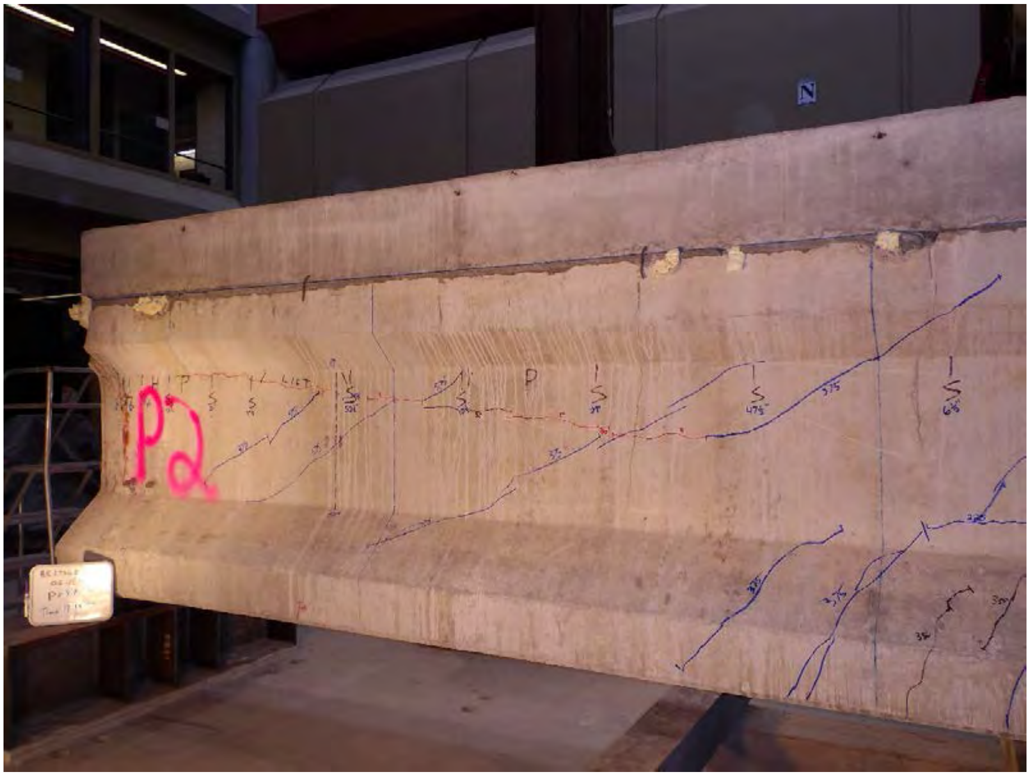


Figure B.14 Girder P2 Southside Section 1-Section 2 *P*=375 kips

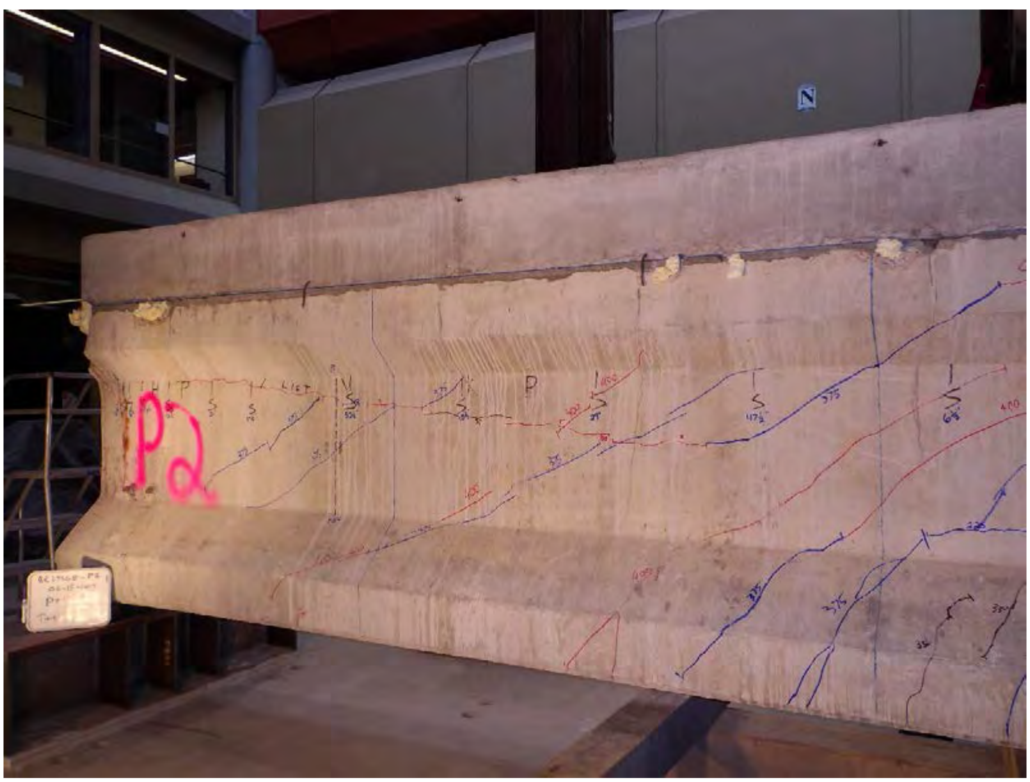


Figure B.15 Girder P2 Southside Section 1-Section 2 *P*=400 kips

Figure B.16 Girder P2 Southside Section 1-Section 2 Failure

Figure B.17 Girder P2 Northside Section 1-Section 2 *P*=250 kips

Figure B.18 Girder P2 Northside Section 1-Section 2 *P*=275 kips

Figure B.19 Girder P2 Northside Section 1-Section 2 *P*=300 kips

Figure B.20 Girder P2 Northside Section 1-Section 2 *P*=325 kips

Figure B.21 Girder P2 Northside Section 1-Section 2 *P*=350 kips

Figure B.22 Girder P2 Northside Section 1-Section 2 *P*=375 kips

Figure B.23 Girder P2 Northside Section 1-Section 2 *P*=400 kips

Figure B.24 Girder P2 Northside Section 1-Section 2 Failure

B.3 GIRDER P4

Figures B.25-B.33 show the condition of Girder P4 when it was received, prior to deck casting. Figures B.34-B.42 show pictures of Section 1 of the south side of Girder P4 during testing. Figures B.43-B.51 show pictures of Section 1 of the north side Girder P4 during testing. Figures B.52-B.56 show the Girder P4 repair during testing.

Figure B.25 Girder P4 Prior to Casting – Section 1 (Northside)

Figure B.26 Girder P4 Prior to Casting – Section 1

Figure B.27 Girder P4 Prior to Casting – Section 2

Figure B.28 Girder P4 Prior to Casting – Section 3

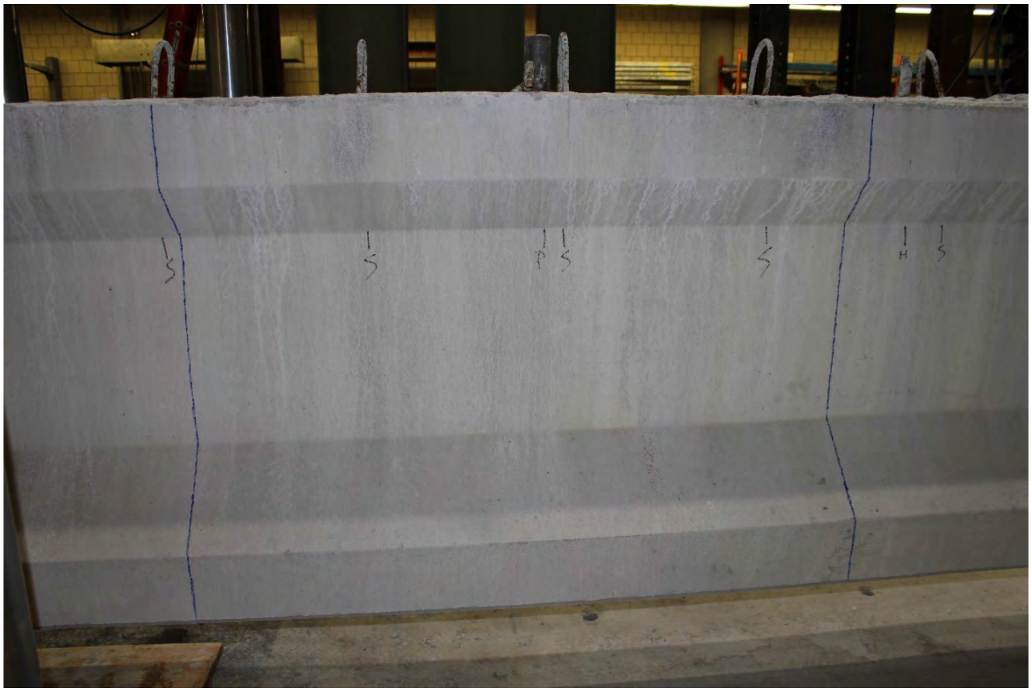


Figure B.29 Girder P2 Prior to Casting – Section 4

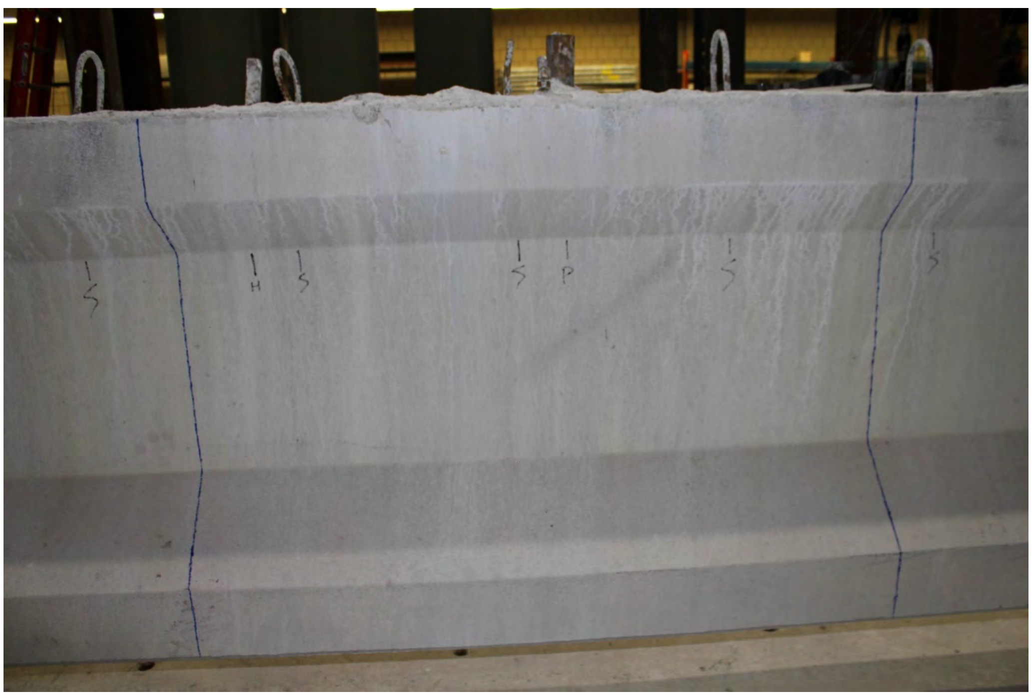


Figure B.30 Girder P4 Prior to Casting – Section 5

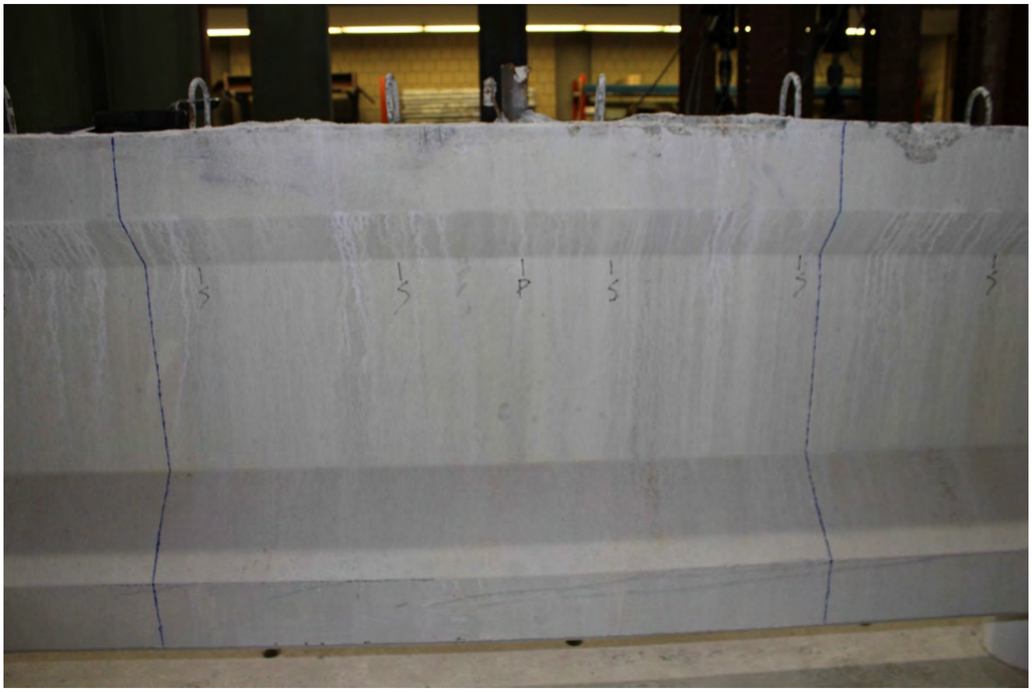


Figure B.31 Girder P4 Prior to Casting – Section 6

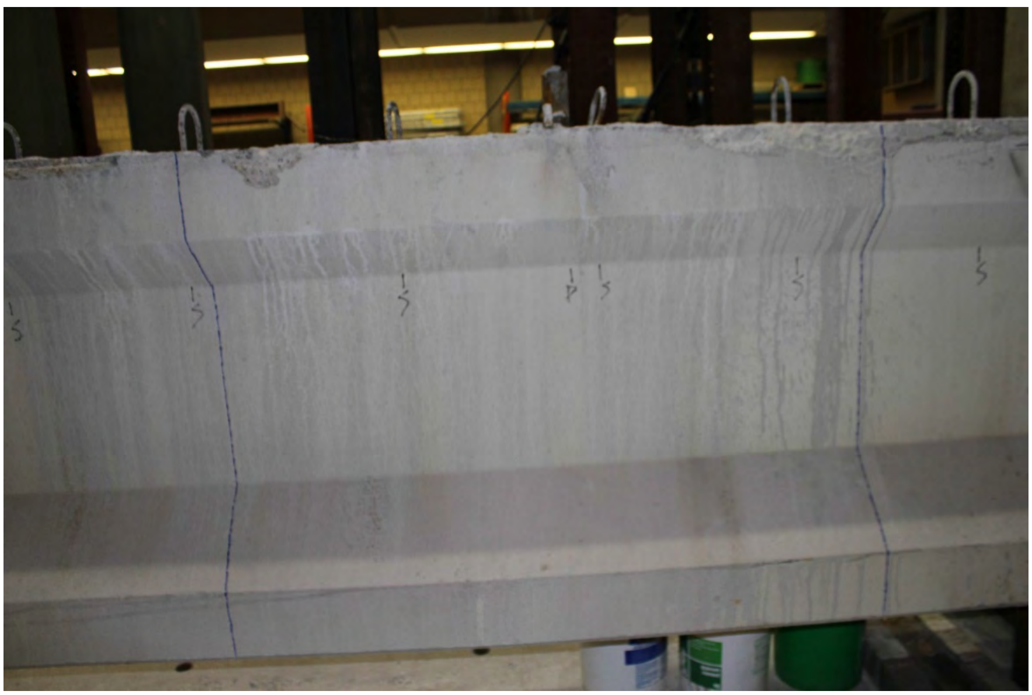


Figure B.32 Girder P4 Prior to Casting – Section 7

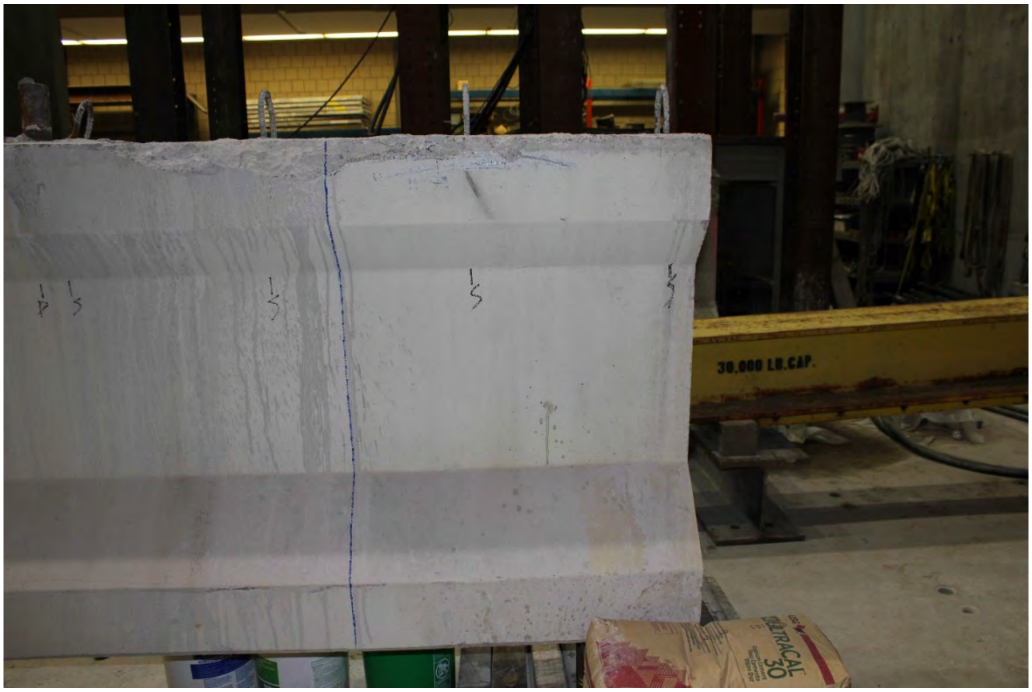


Figure B.33 Girder P4 Prior to Casting – Section 8 (Cut-End)

Figure B.34 Girder P4 Southside Section 1-Section 3 *P*=275 kips

Figure B.35 Girder P4 Southside Section 1-Section 3 *P*=300 kips

Figure B.36 Girder P4 Southside Section 1-Section 3 *P*=325 kips

Figure B.37 Girder P4 Southside Section 1-Section 3 *P*=350 kips

Figure B.38 Girder P4 Southside Section 1-Section 3 *P*=375 kips

Figure B.39 Girder P4 Southside Section 1-Section 3 *P*=400 kips

Figure B.40 Girder P4 Southside Section 1-Section 3 *P*=425 kips

Figure B.41 Girder P4 Southside Section 1-Section 3 *P*=450 kips

Figure B.42 Girder P4 Southside Section 1-Section 3 Failure

Figure B.43 Girder P4 Northside Section 1-Section 2 *P*=275 kips

Figure B.44 Girder P4 Northside Section 1-Section 2 *P*=300 kips

Figure B.45 Girder P4 Northside Section 1-Section 2 *P*=325 kips

Figure B.46 Girder P4 Northside Section 1-Section 2 *P*=350 kips

Figure B.47 Girder P4 Northside Section 1-Section 2 *P*=375 kips

Figure B.48 Girder P4 Northside Section 1-Section 2 *P*=400 kips

Figure B.49 Girder P4 Northside Section 1-Section 2 *P*=425 kips

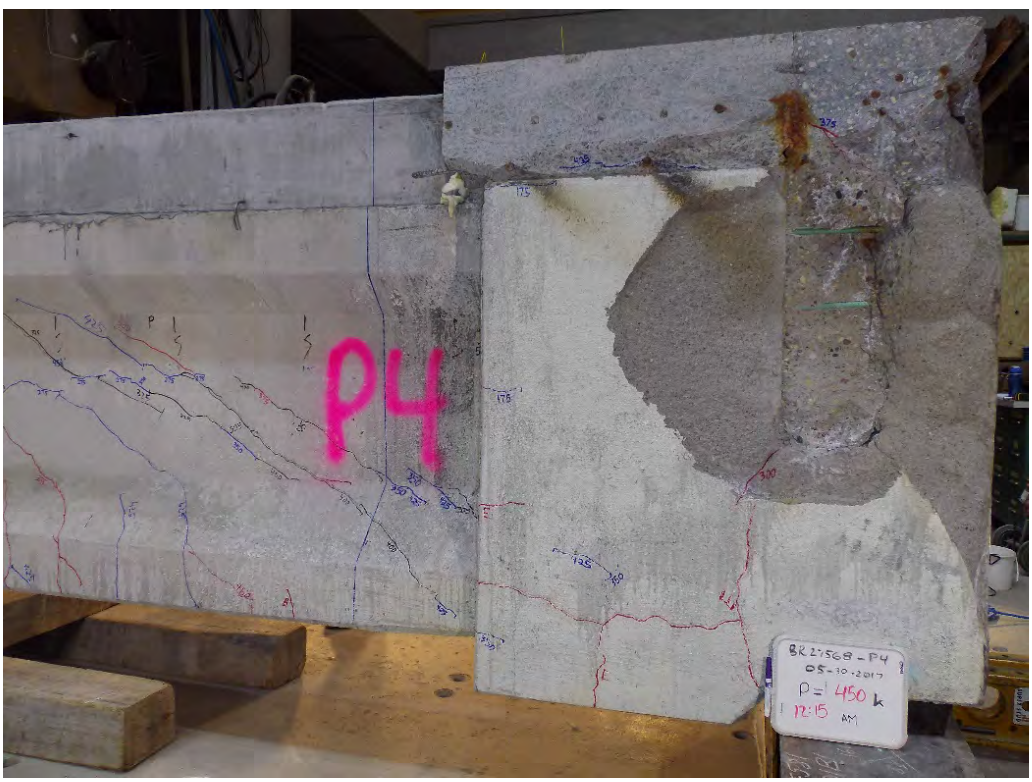


Figure B.50 Girder P4 Northside Section 1-Section 2 *P*=450 kips

Figure B.51 Girder P4 Northside Section 1-Section 2 After Peak = 465 kips (load incorrectly labeled on white board)

Figure B.52 Girder P4 Northside Section 1-Section 2 Failure

Figure B.53 Girder P4 Repair Section 1 *P*=300 kips

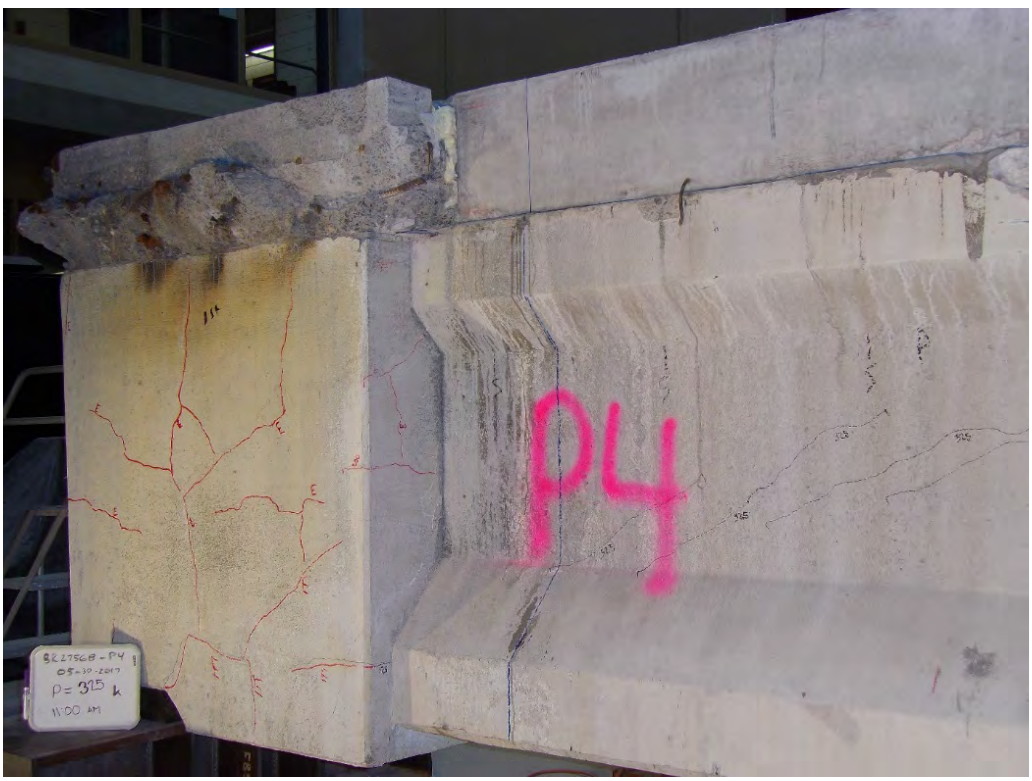
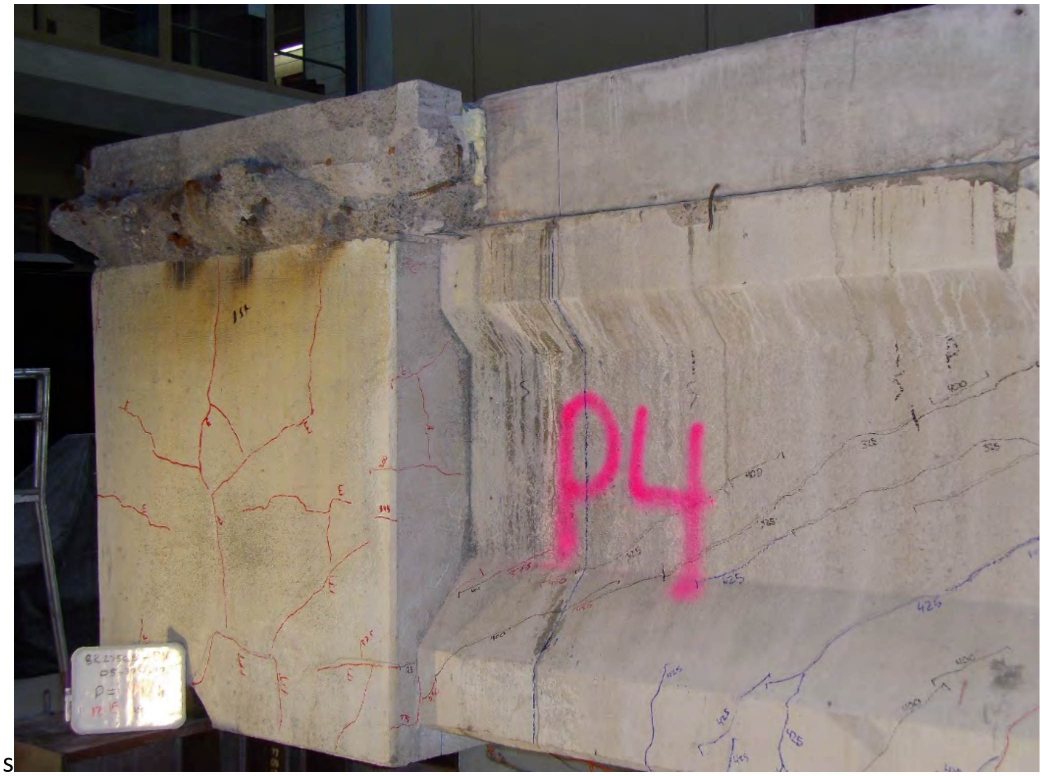
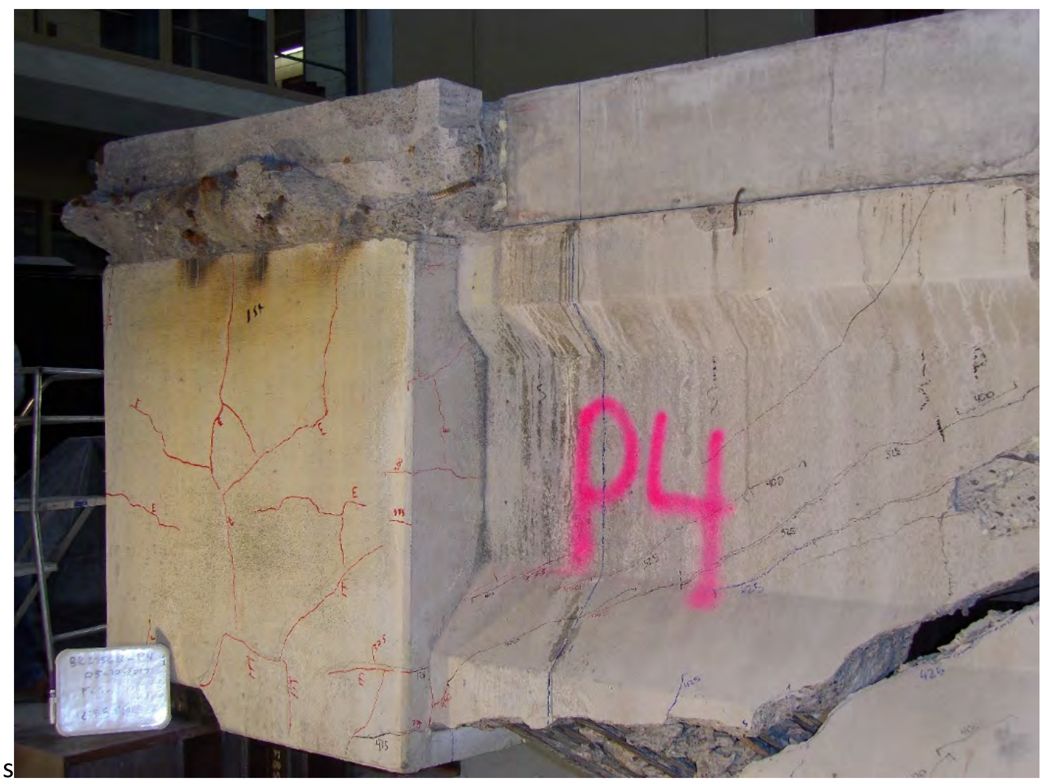


Figure B.54 Girder P4 Repair Section 1 *P*=325 kips



s

Figure B.55 Girder P4 Repair Section 1 *P*=450 kips



s

Figure B.56 Girder P4 Repair Section 1 Failure

B.4 GIRDER P24

Figures B.57-B.64 show the condition of Girder P24 when it was received, prior to deck casting. Figures B.65-B.73 show pictures of Section 1 of the south side of Girder P24 during testing. Figures B.74-B.82 show pictures of Section 1 of the north side Girder P2 during testing.

Figure B.57 Girder P24 Prior to Casting – Section 1

Figure B.58 Girder P24 Prior to Casting – Section 2

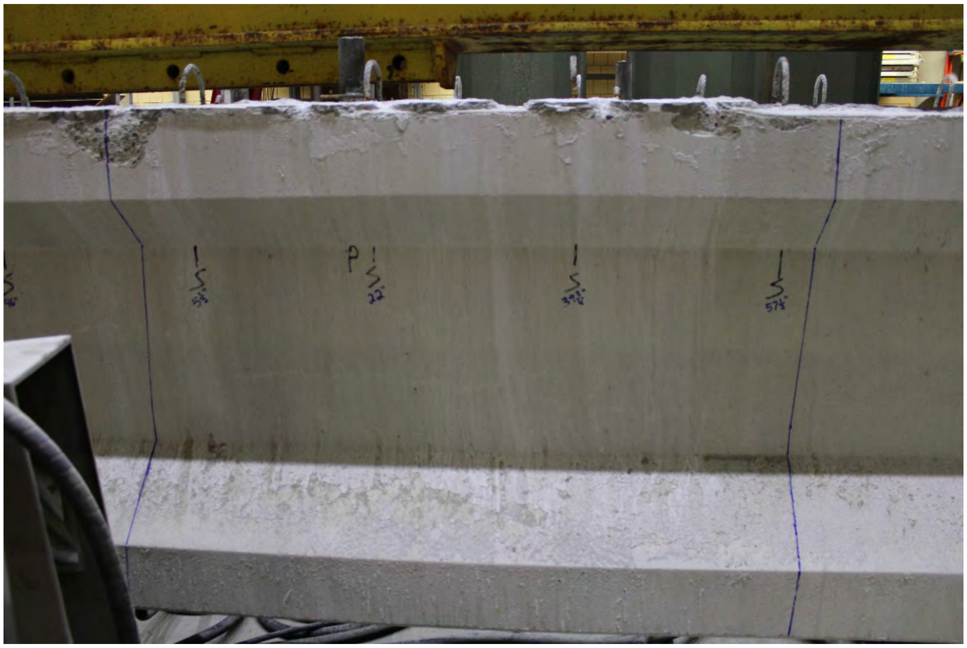
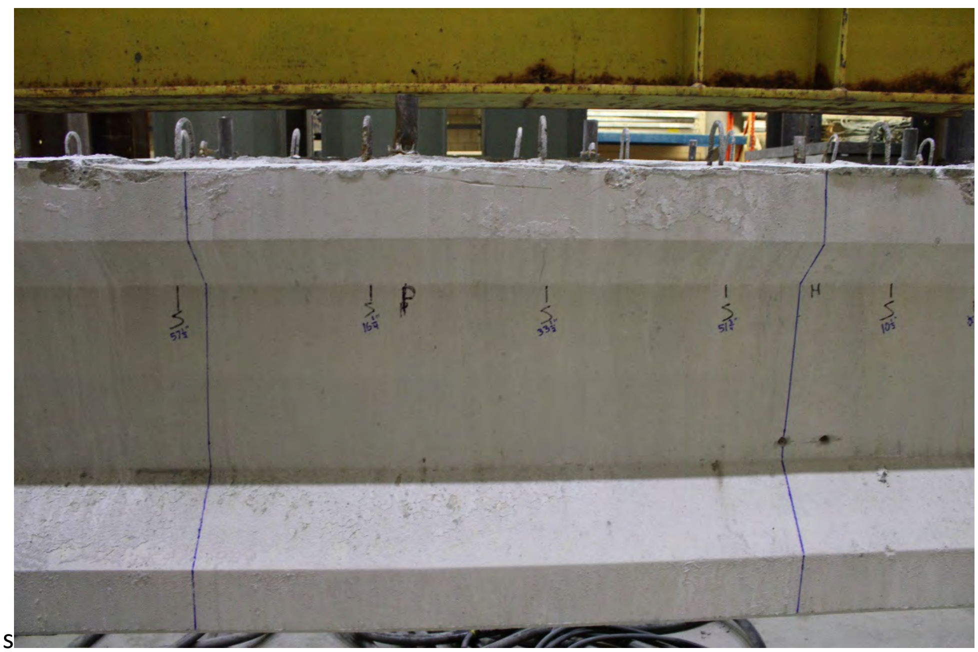


Figure B.59 Girder P24 Prior to Casting – Section 3



s

Figure B.60 Girder P24 Prior to Casting – Section 4

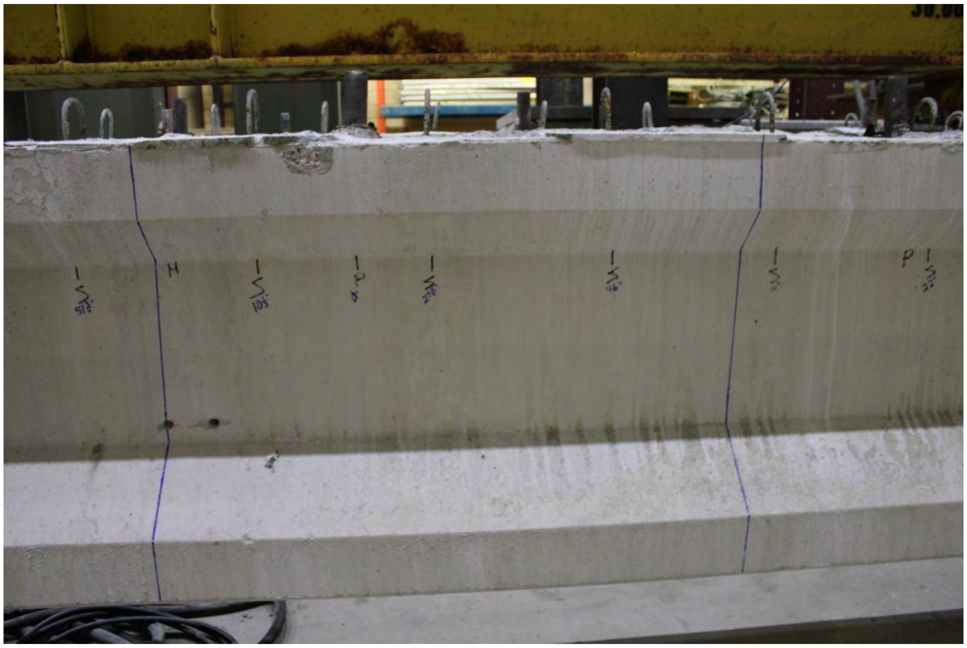


Figure B.61 Girder 24 Prior to Casting – Section 5

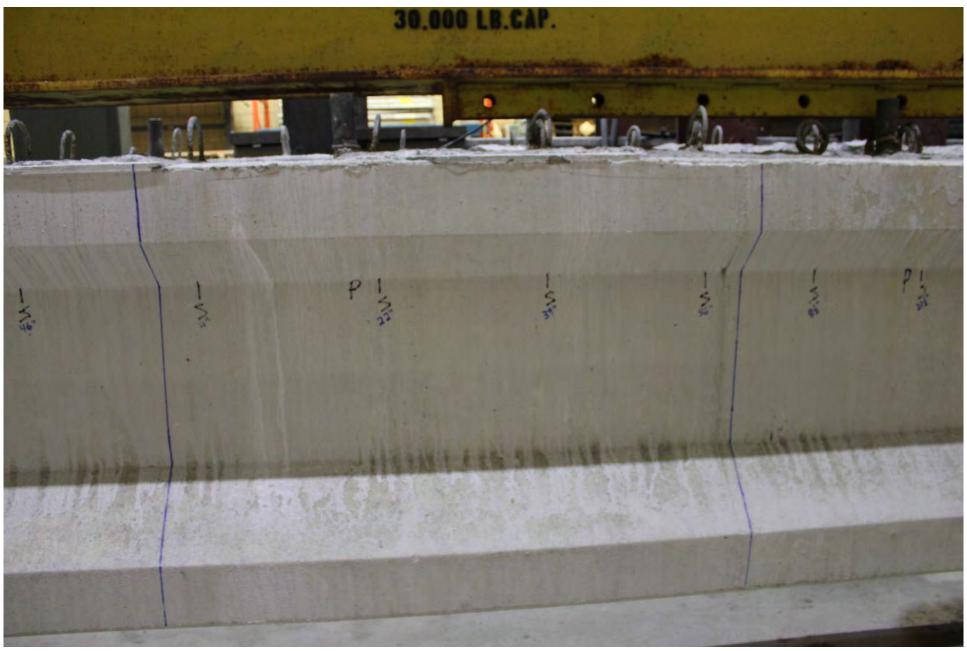


Figure B.62 Girder P24 Prior to Casting – Section 6

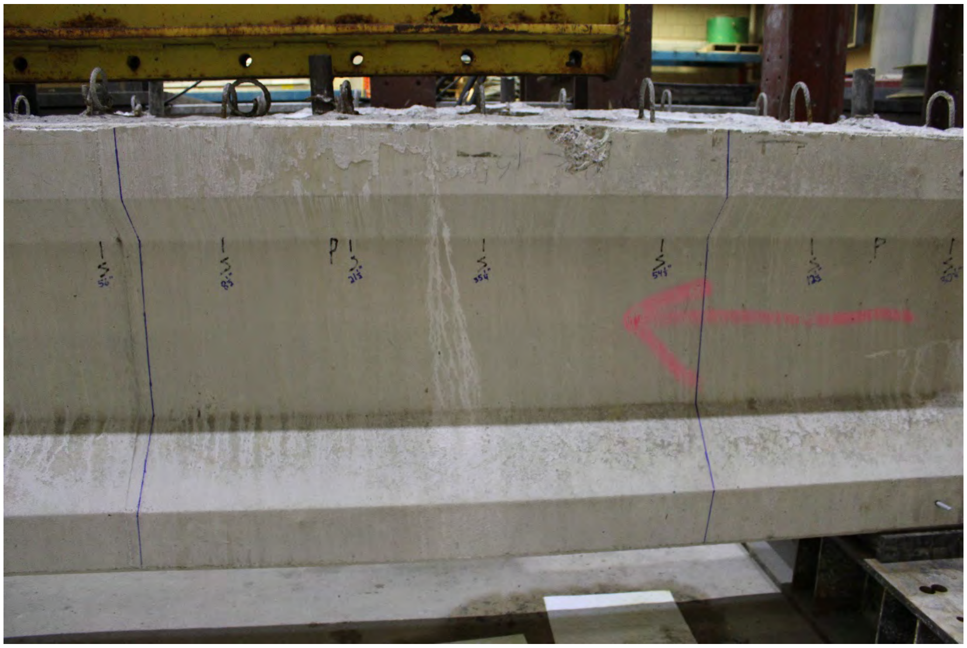


Figure B.63 Girder P24 Prior to Casting – Section 7

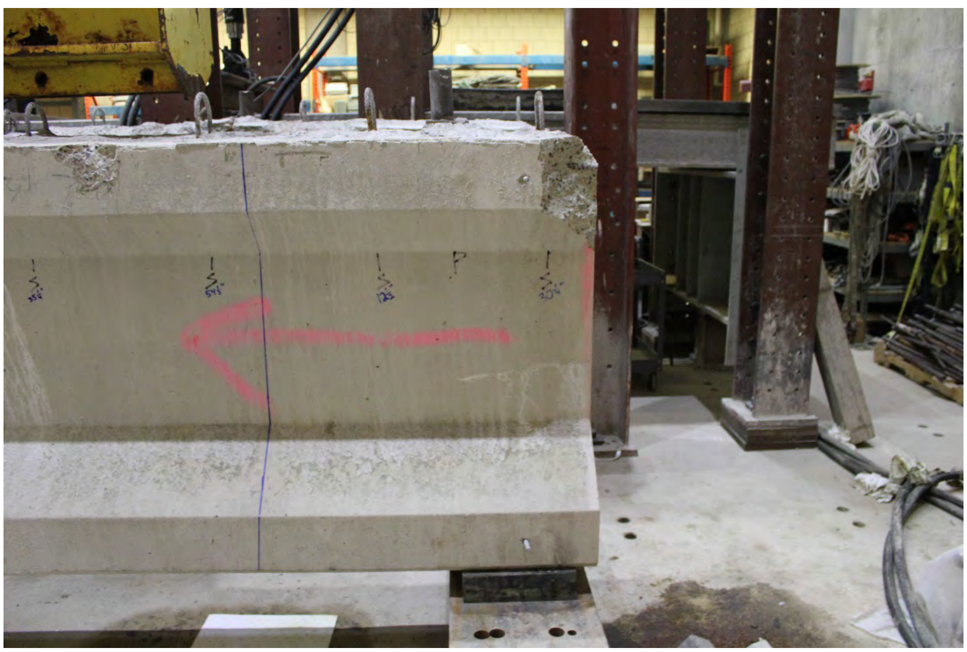


Figure B.64 Girder P24 Prior to Casting – Section 8 (Cut-End)

Figure B.65 Girder P24 Southside Section 1-Section 2 *P*=250 kips

Figure B.66 Girder P24 Southside Section 1-Section 2 *P*=275 kips

Figure B.67 Girder P24 Southside Section 1-Section 2 *P*=300 kips

Figure B.68 Girder P24 Southside Section 1-Section 2 *P*=325 kips

Figure B.69 Girder P24 Southside Section 1-Section 2 *P*=350 kips

Figure B.70 Girder P24 Southside Section 1-Section 2 *P*=375 kips

Figure B.71 Girder P24 Southside Section 1-Section 2 *P*=400 kips

Figure B.72 Girder P24 Southside Section 1-Section 2 P=425

s

Figure B.73 Girder P24 Southside Section 1-Section 2 Failure

Figure B.74 Girder P24 Northside Section 1-Section 2 *P*=250 kips

Figure B.75 Girder P24 Northside Section 1-Section 2 *P*=275 kips

Figure B.76 Girder P24 Northside Section 1-Section 2 *P*=300 kips

Figure B.77 Girder P24 Northside Section 1-Section 2 *P*=325 kips

s

Figure B.78 Girder P24 Northside Section 1-Section 2 *P*=350 kips

Figure B.79 Girder P24 Northside Section 1-Section 2 *P*=375 kips

Figure B.80 Girder P24 Northside Section 1-Section 2 *P*=400 kips

Figure B.81 Girder P24 Northside Section 1-Section 2 *P*=425 kips

s 

Figure B.82 Girder P24 Northside Section 1-Section 2 Failure

B.5 GIRDER P26

Figures B.83-B.91 show the condition of Girder P26 when it was received, prior to deck casting. Figures B.92-B.99 show pictures of Section 1 of the south side of Girder P26 during testing. Figures B.100-B.107 show pictures of Section 1 of the north side Girder P26 during testing. Figures B.108-B.115 show the Girder P26 repair during testing.

Figure B.83 Girder P26 Prior to Casting – Repair End View

Figure B.84 Girder P26 Prior to Casting – Section 1

Figure B.85 Girder P26 Prior to Casting – Section 2

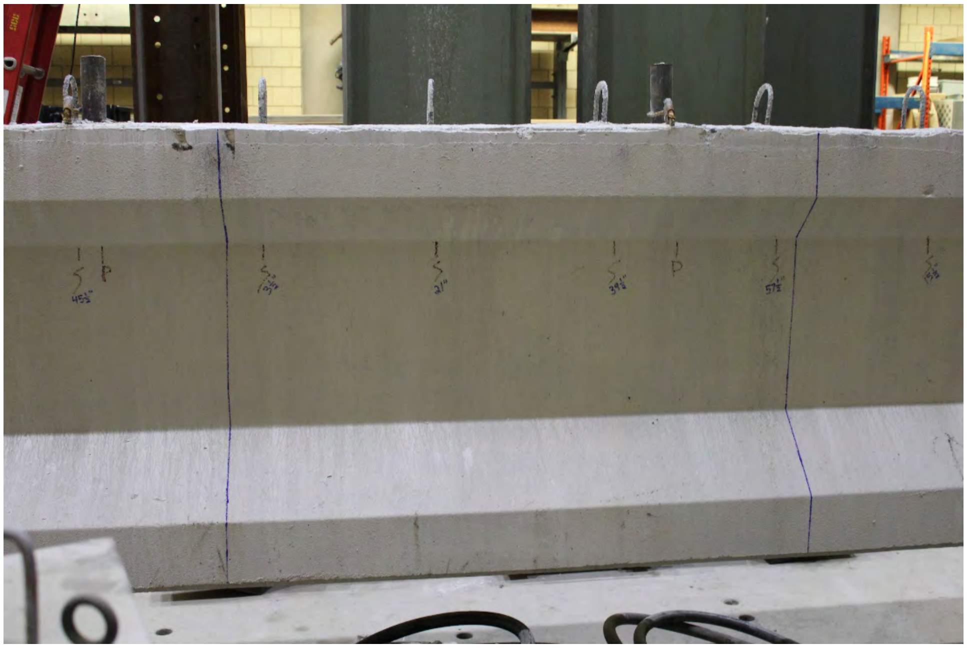


Figure B.86 Girder P26 Prior to Casting – Section 3

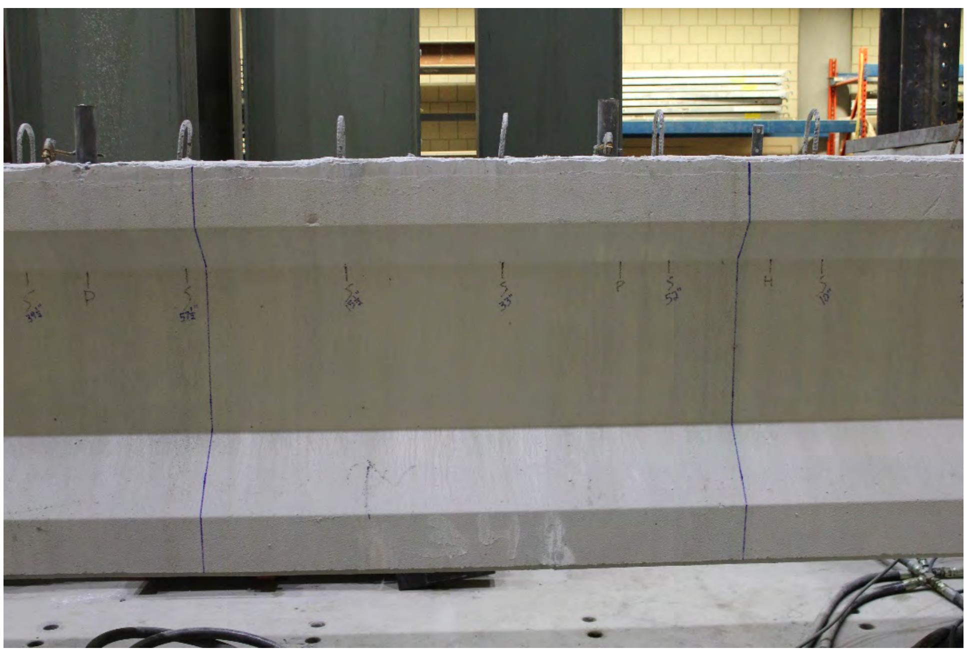


Figure B.87 Girder P26 Prior to Casting – Section 4

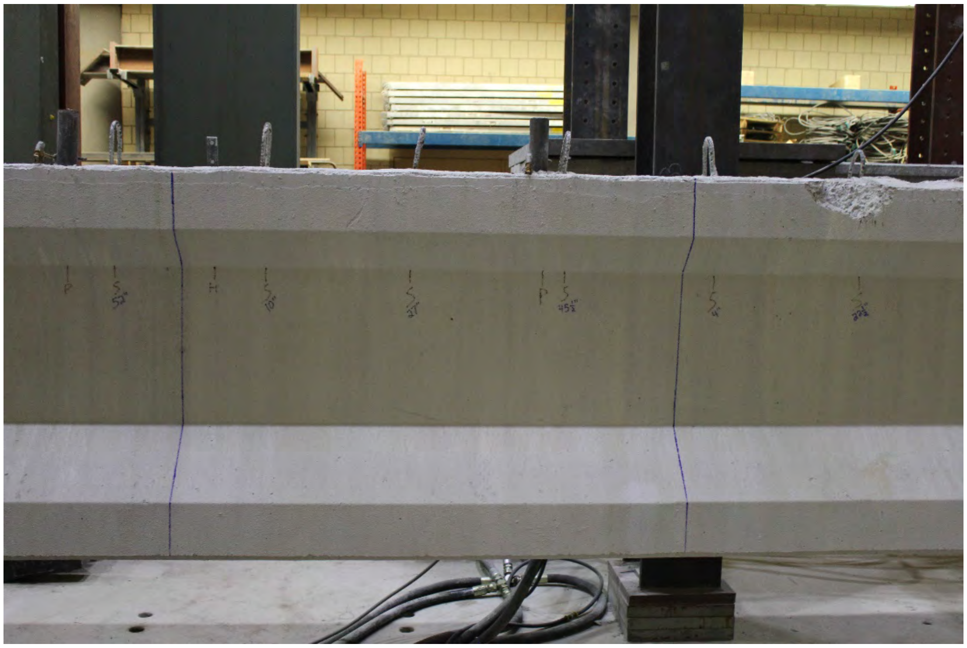


Figure B.88 Girder P26 Prior to Casting – Section 5

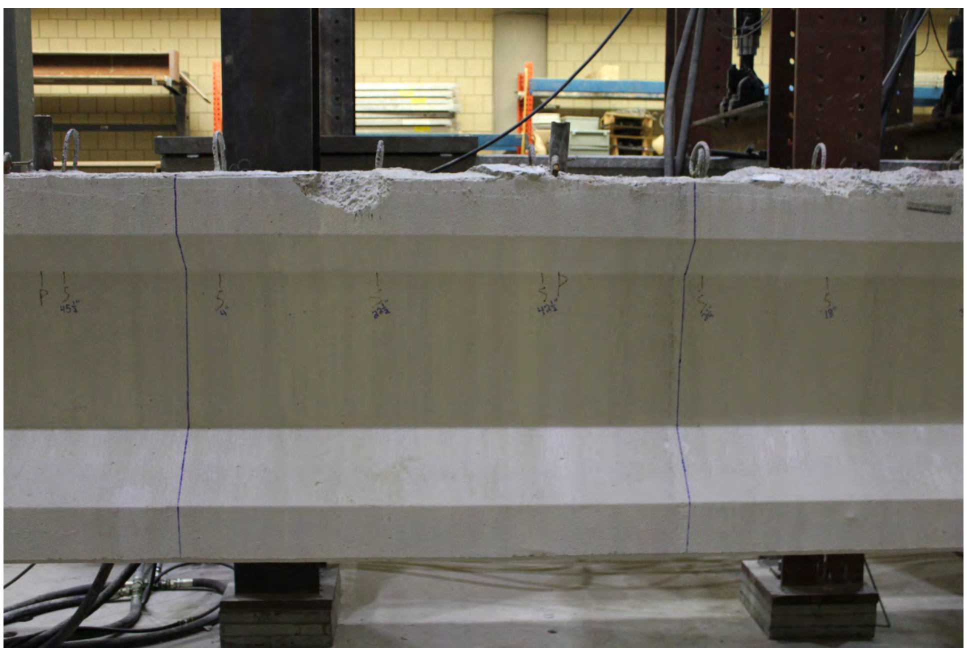


Figure B.89 Girder P26 Prior to Casting – Section 6

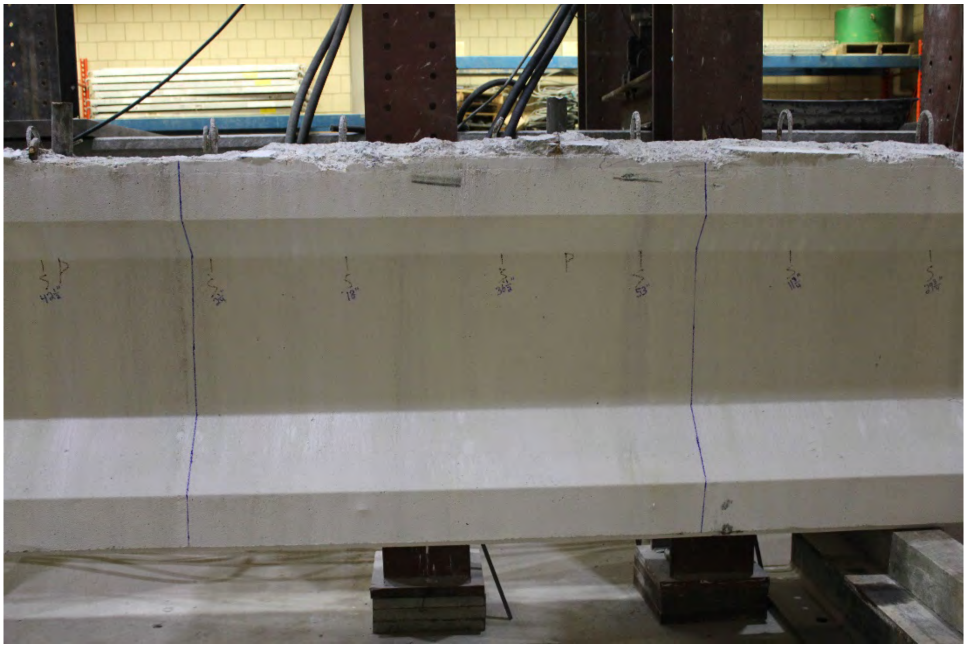


Figure B.90 Girder P26 Prior to Casting – Section 7

Figure B.91 Girder P26 Prior to Casting – Section 8 (Cut-End)

Figure B.92 Girder P26 Southside Section 1-Section 3 *P*=275 kips

Figure B.93 Girder P26 Southside Section 1-Section 3 *P*=300 kips

Figure B.94 Girder P26 Southside Section 1-Section 3 *P*=325 kips

Figure B.95 Girder P26 Southside Section 1-Section 3 *P*=350 kips

Figure B.96 Girder P26 Southside Section 1-Section 3 *P*=375 kips

Figure B.97 Girder P26 Southside Section 1-Section 3 *P*=400 kips

Figure B.98 Girder P26 Southside Section 1-Section 3 *P*=425 kips

Figure B.99 Girder P26 Southside Section 1-Section 3 Failure

Figure B.100 Girder P26 Northside Section 1-Section 2 *P*=275 kips

Figure B.101 Girder P26 Northside Section 1-Section 2 *P*=300 kips

Figure B.102 Girder P26 Northside Section 1-Section 2 *P*=325 kips

Figure B.103 Girder P26 Northside Section 1-Section 2 *P*=350 kips

Figure B.104 Girder P26 Northside Section 1-Section 2 *P*=375 kips

Figure B.105 Girder P26 Northside Section 1-Section 2 *P*=400 kips

Figure B.106 Girder P26 Northside Section 1-Section 2 *P*=425 kips

Figure B.107 Girder P26 Northside Section 1-Section 2 Failure

Figure B.108 Girder P26 Repair Section 1 – Section 2 *P*=275 kips

Figure B.109 Girder P26 Repair Section 1 – Section 2 *P*=300 kips

Figure B.110 Girder P26 Repair Section 1 – Section 2 *P*=325 kips

Figure B.111 Girder P26 Repair Section 1 – Section 2 *P*=350 kips

Figure B.112 Girder P26 Repair Section 1 – Section 2 *P*=375 kips

Figure B.113 Girder P26 Repair Section 1 – Section 2 *P*=400 kips

Figure B.114 Girder P26 Repair Section 1 – Section 2 *P*=425 kips

Figure B.115 Girder P26 Repair Section 1 – Section 2 Failure