Evaluating the Constructability of NUDECK Precast Concrete Deck Panels for Kearney Bypass Project

Nebraska Department of Roads (NDOR)

Project No. SPR-P1 (13) M336

February 2015

University of Nebraska Lincoln

NDOR Nebraska Department of Roads

Evaluating the Constructability of NUDECK Precast Concrete Deck Panels for Kearney Bypass Project

Nebraska Department of Roads (NDOR)

Project No. SPR-P1 (13) M336

FINAL REPORT

Investigator

George Morcous, Ph.D., P.E.
University of Nebraska – Lincoln

February 2015

Technical Report Documentation Page

Report No.	Government Accession No.	Recipient's Catalog No.
Title and Subtitle Evaluating the Constructability of NUDECK Precast Concrete Deck Panels for Kearney Bypass Project		1. Report Date February 2015
		2. Performing Organization Code
3. Author(s) George Morcous, Afshin Hatami, and Micheal Assad		4. Performing Organization Report No.
5. Performing Organization Name and Address University of Nebraska-Lincoln 1110 South 67th St. Omaha, Nebraska 68182-0178		6. Work Unit No.
		7. Contract or Grant No.
8. Sponsoring Agency Name and Address Nebraska Department of Roads Bridge Division P. O. Box 94759 Lincoln, NE 68509-4759		9. Type of Report and Period Covered Final Report
		10. Sponsoring Agency Code
11. Supplementary Notes		

12. Abstract

The first generation of precast concrete deck system, NUDECK, was implemented on the Skyline Bridge, Omaha, NE in 2004. The second generation of NUDECK system was developed to further simplify the system and improve its constructability and durability. The new generation consists of full-width full-depth precast concrete deck panels that are 12 ft long to minimize the number of deck panels and transverse joints, and consequently accelerate bridge construction. It also uses covered individual pockets and bundled shear connectors at 4 ft spacing to simplify panel and girder production and eliminate the need for deck overlay. Precast deck panels are pre-tensioned in transverse direction and post-tensioned in the longitudinal direction to enhance deck durability. Post-tensioning strands are placed underneath the deck panels (at the haunch area) to eliminate threading strands through ducts and grouting operations.

The objective of this project is to investigate the constructability of the 2nd generation NUDECK system through full-scale testing of two key features: 1) using self-consolidating concrete (SCC) to fill the gap between precast concrete deck panels and bridge girders as well as covered deck pockets; and 2) using the proposed deviators and anchorage block in the end deck panels for post-tensioning the bridge deck. These investigations includes evaluating the pumpability/pouring of the developed SCC mixture in mockup and full-scale specimens. Sequence of pumping/pouring of SCC as well as its quality control and quality assurance procedures are also demonstrated. The investigations also include pullout testing of the deviators and pushoff testing of the anchorage block used for post-tensioning to evaluate their performance and refine their reinforcement details.

1. Keywords: Precast Concrete, Deck Panels, Accelerated Bridge Construction, Prestressed Bridge Decks, Shear Connectors			
2. Security Classification (of this report) Unclassified	3. Security Classification (of this page) Unclassified	4. No. of Pages: 92	22. Price

Form DOT F1700.7 (8-72)

DISCLAIMER

ACKNOWLEDGEMENTS

This project was sponsored by Nebraska Department of Roads (NDOR), University of Nebraska-Lincoln (UNL), and Center for Transportation Infrastructure and Safety (CTIS) of the Missouri University of Science and Technology (Missouri S&T). The donation of test specimens by Precast Concrete Association of Nebraska (PCAN) is highly appreciated. Also, the efforts of faculty/staff and graduate/undergraduate students at UNL and Missouri S&T are acknowledged. Special thanks to Prof. Maker K. Tadros and Kamal H. Khayat for their support and encouragement.

TABLE OF CONTENTS

ABSTRACT

The first generation of precast concrete deck system, NUDECK, was implemented on the Skyline Bridge, Omaha, NE in 2004. The second generation of NUDECK system was developed to further simplify the system and improve its constructability and durability. The new generation consists of full-width full-depth precast concrete deck panels that are 12 ft long to minimize the number of deck panels and transverse joints, and consequently accelerate bridge construction. It also uses covered individual pockets and bundled shear connectors at 4 ft spacing to simplify panel and girder production and eliminate the need for deck overlay. Precast deck panels are pre-tensioned in transverse direction and post-tensioned in the longitudinal direction to enhance deck durability. Post-tensioning strands are placed underneath the deck panels (at the haunch area) to eliminate threading strands through ducts and grouting operations.

The objective of this project is to investigate the constructability of the 2nd generation NUDECK system through full-scale testing of two key features: 1) using self-consolidating concrete (SCC) to fill the gap between precast concrete deck panels and bridge girders as well as covered deck pockets; and 2) using the proposed deviators and anchorage block in the end deck panels for post-tensioning the bridge deck. These investigations includes evaluating the pumpability/pouring of the developed SCC mixture in mockup and full-scale specimens. Sequence of pumping/pouring of SCC as well as its quality control and quality assurance procedures are also demonstrated. The investigations also include pullout testing of the deviators and pushoff testing of the anchorage block used for post-tensioning to evaluate their performance and refine their reinforcement details.

1 INTRODUCTION

1.1 Background

The Kearney East Bypass is the first bridge project in Nebraska that uses the latest developments of NUDECK precast deck system (i.e. 2nd generation) and the only bridge with precast deck panels on prestressed concrete girders. These developments include widening the precast panel from 8 ft to 12 ft, using covered individual pockets at 4 ft spacing instead of continuous open channel, eliminating deck overlay, and placing post-tensioning strands underneath the deck panels. The former research project titled "Implementation of Precast Deck Panel NUDECK" focuses on the design and detailing of the new deck system to be implemented in the Kearney East Bypass (Morcous, et al., 2013). In order to evaluate the constructability of the developed precast deck system, simplify its construction, and improve its competitiveness against cast-in-place concrete deck, two key features need to be experimentally evaluated:

- Pumping self-consolidating concrete instead of expensive grouting materials to fill the girder haunch and deck pockets after panel erection and post-tensioning.
- Using post-tensioning strand deviators in the girder and custom made anchorage blocks in the end precast panel to raise the post-tensioning strands to mid-height of the deck.

1.2 Objective

The objective of this project is to experimentally investigate and demonstrate in a full-scale test setting the proposed pumping and post-tensioning procedures of the new precast concrete deck system. This demonstration will be presented to precast producers, bridge contractors, and concrete pumping suppliers to determine the most cost-effective method that ensures the success of these operations and eliminate any problems that might occur during bridge construction due to the unfamiliarity of the involved parties with the new developments. This experimentation has also two direct benefits: 1) ensures that the flowable concrete (self-consolidating concrete) proposed to fill the haunch and deck pockets can be pumped in a satisfactory and efficient manner; and 2) ensures that all specified post-tensioning hardware (deviators and anchorage block) are easy to fabricate/install, and functioning as expected.

2 PUMPING SCC

The objective of this investigation is to experimentally evaluate the pumpability of self-consolidating concrete (SCC) to fill the gap between precast concrete deck panels and bridge girders (i.e. haunch) as well as covered deck pockets. This includes developing SCC mixture(s) with specific requirements in terms of flowability, passing ability, stability, workability retention, and pumpability. Materials used in this investigation include: Type I Portland cement with specific gravity of 3.15; either class F or class C fly ash at 20% substitution of total cementitious materials, by mass; either CTS Komponent or Conex supplied from Euclid chemicals to reduce and/or control of shrinkage of the concrete; natural sand with specific gravity of 2.53 and absorption of 0.62%; and pea gravel of MSA of 3/8 in., specific gravity and absorption values of the gravel were 2.54% and 2.7%, respectively. Table 2.1 lists the proportions of the SCC mixtures developed for this application. For more information on mixture development, refer to Morcous and Khayat (2014).

Table 2.1 – Mixture composition of mixtures made with Komponent and Conex admixtures

Materials	SCC with Komponent		SCC with Conex	
	(lb/yd^3)	(fl.oz/yd^3)	(lb/yd^3)	(fl.oz/yd^3)
Type I Portland cement	570		600	
Class F fly ash (20% of total binder)	170		179	
Expansive agent (Komponent)	125		-	
Expansive agent (Conex)	-		87	
Total binder materials	865		866	
Water	285		285	
w/b	0.33		0.33	
Sand	1615		1615	
3/8 in. Pea gravel (coarse agg.)	1077		1077	
Superplasticizer 1 (Plastol 6200 EXT)		103.0		75.0
Superplasticizer 2 (Plastol 5000)		23.0		69.0
Set-retarder (Retarder 100)		33.0		33.0
VEA (Visctrol)		0		33.9
Air-entraining agent (AEA92)		1.4		1.4

2.1 Pumping Mockup Field Tests

Three mockup pumping campaigns were carried out in this project and are described below.

a. Pumping Mockup Field Test No. 1

In this test, SCC was optimized with the Komponent expansive agent for the placement of an element measuring 2 in. thick, 48 in. wide, and 48 ft long. The mockup simulates the haunch area between precast concrete girders and precast concrete deck panels that needs to be filled with SCC. The concrete was pumped using 2 in. diameter hose from one end of the test setup, as shown in Figure 2.1. The dimensions of this specimen are shown in Figure 2.2. This field test was conducted on May 24th, 2013 at the HyPoint laboratory at Missouri S&T. The SCC mixture had high flowability and adequate stability which is necessary for the challenging casting condition. At the beginning of pumping, concrete flowed very smoothly into the formwork without any signs of blockage or segregation. Yield stress and plastic viscosity rheological parameters determined using the ICAR rheometer were 7 Pa and 14 Pa.s, respectively, which indicate excellent flowability. However, due to the high pressure exerted by pumping, the formwork started to open and leak during pumping, as shown in Figure 2.3. The pumping process was then stopped without completing the test.

Figure 2.1 – Photo of the formwork and pump connection at one end

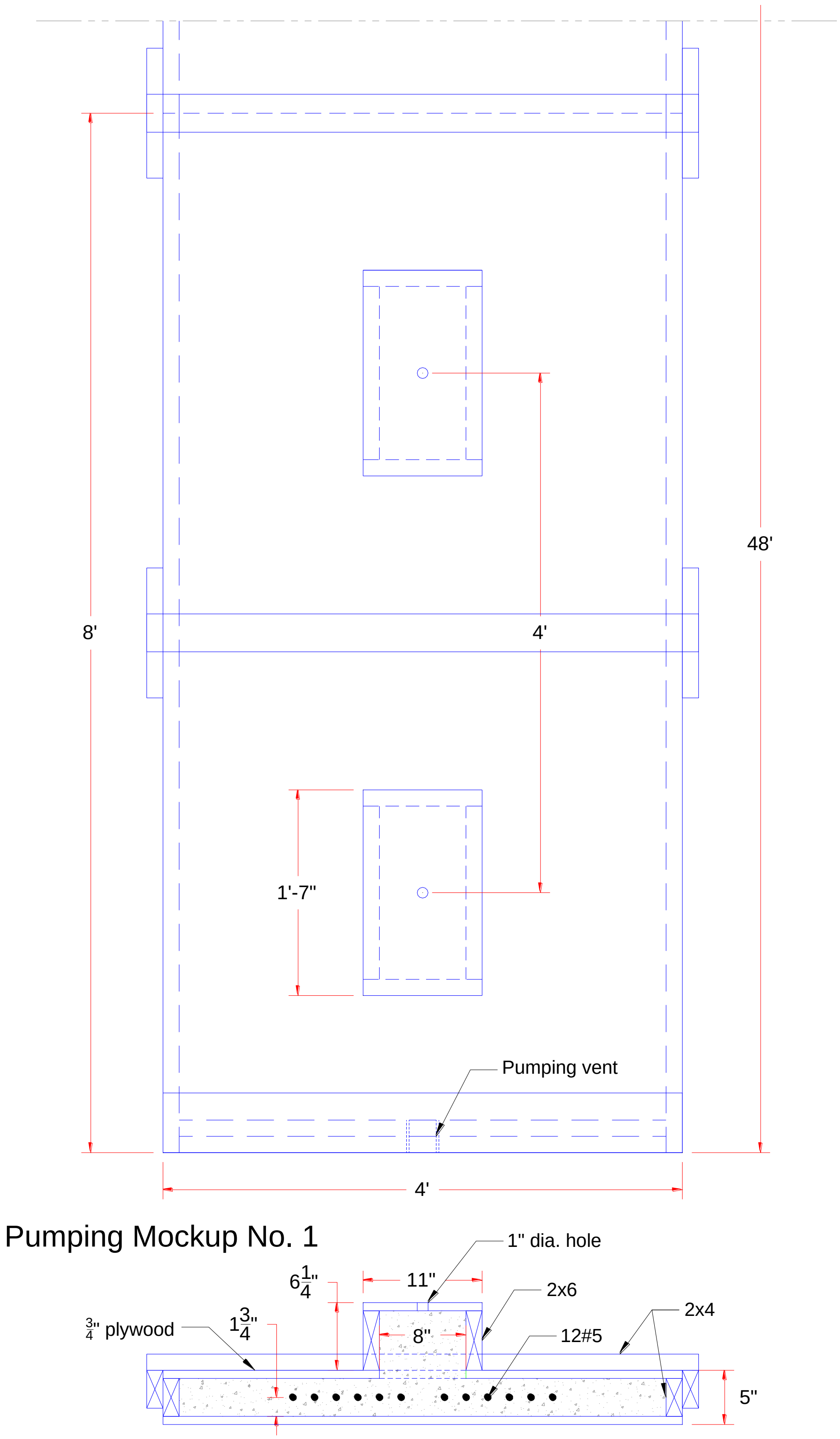


Figure 2.2 – Dimensions and reinforcement of the field test No. 1 specimen

Figure 2.3 – Leakage of concrete after formwork failure due to high concrete pressure at one end

The concrete was sampled to determine hardened properties in addition to workability and rheology. Fresh properties up to 90 minutes and compressive strength values at 1, 7, and 28 days of ages are summarized in Table 2.2. The optimized SCC mixture exhibited adequate slump flow and its retention up to 90 minutes and excellent resistance to bleeding which is required for static stability of the concrete. In addition, the optimized mixture had the spread difference between slump flow and J-ring flow of 0.4 to 0.5 in. and L-box ratio of 1 and 0.89 at 30 and 90 minutes, respectively. These values indicate excellent passing ability of the SCC mixture. The compressive strengths of the optimized mixture were greater than the targeted values, as presented in Table 2.2.

Table 2.2 – Fresh properties and compressive strength of SCC used for field test No. 1 (Komponent expansive agent)

Properties	Time after cement-water contact (min)				Target Value/Range
	30	90	120	180	
Slump flow (in.)	30.7	28.3	Due to the high pressure exerted by pumping, the test was stopped.		25.5 – 29.5
V-funnel (sec)	4.0	4.3			≤ 12
Air content (%)	3.6	2.1			-
L-box ratio (h_2/h_1)	1.0	0.89			0.8 to 1.0
J-ring (in.)	30.3	27.8			Diff. ≤ 2
Static bleeding	0	0			-
Yield stress (Pa)	7	-			-
Plastic viscosity (Pa.s)	14	-			-
Compressive strength at 1 day (psi)	3,340				-
Compressive strength at 7 days (psi)	5,960				3,500
Compressive strength at 28 days (psi)	8,060				6,000

b. Pump Mockup Field Test No. 2

From the lessons learned from the first mockup test, a more rigid formwork was designed to sustain the high pumping pressure for the second field testing. In this test, the wooden form was reinforced with many 2 x 4 in. lumbers, as shown in Figure 2.4. As in the case of the first field testing, the concrete was pumped from the one end point. A pressure indicator was installed to the form to evaluate the concrete rise in a chimney type of set-up, as shown in Figure 2.5. The mixture composition of the SCC used in the second field testing is summarized in Table 2.3.

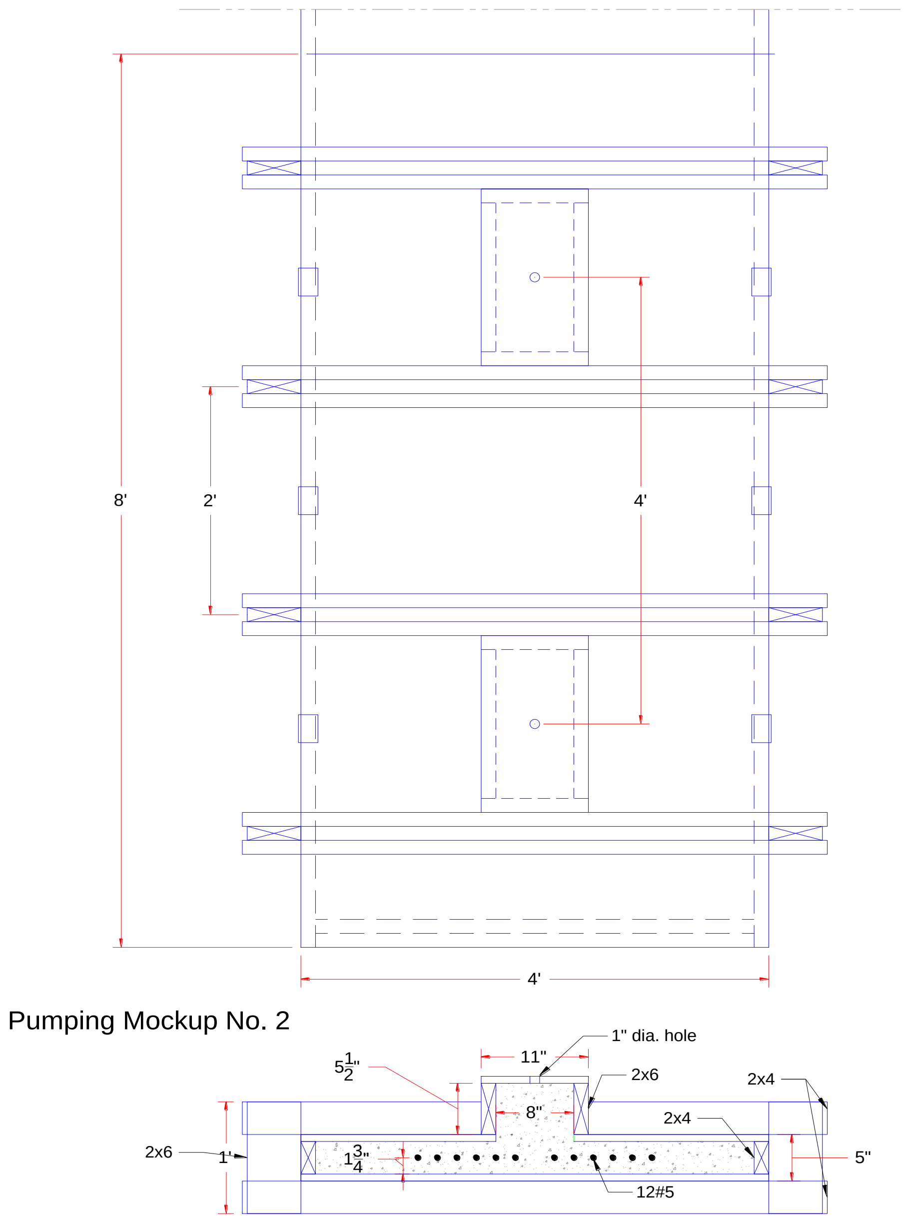


Figure 2.4 – Dimensions and reinforcement of the field test No. 2 specimen

Figure 2.5 – Photo of the formwork, pump connection, and pressure indicator at one end

Table 2.3 – Mixture composition of SCC used for the second field testing (Komponent)

Materials	US English unit		SI unit	
	(lb/yd^3)	(fl.oz/yd^3)	(kg/m^3)	(mL/m^3)
Type I Portland cement	570		338	
Class C fly ash	170		101	
Expansive agent (Komponent)	125		74	
Total binder materials	865		513	
Water	285 + 20		181	
Sand	1615		958	
3/8 in. Pea gravel (coarse agg.)	1077		640	
Superplasticizer 1 (Plastol 6200 EXT)		103.0		3,985
Superplasticizer 2 (Plastol 5000)		23.0		890
Set-retarder (Retarder 100)		33.0		1,277
VEA (Visctrol)		0		0
Air-entraining agent (AEA92)		1.4		54

Table 2.4 summarizes the fresh and hardened properties of the SCC mixtures. The slump flow of the concrete at the beginning of pumping was 26.5 in. (670) mm, which is within the acceptable range of the targeted values of 25.5 to 29.5 in. (650 to 750 mm). The flow value was lower than

that of the first field testing. However, a sudden set took place, and then the slump flow of the concrete at 60 minutes dropped to 19.3 in. (490 mm), which is not adequate to pump the concrete into the very restricted and long formwork. The concrete placement was stopped without the completion. The form was barely one third of the total length, as shown in Figure 2.6.

Table 2.4 – Fresh properties and compressive strength of SCC used for field test No. 2

Properties	**Time after pumping started (min)**				**Target Value/Range**
	0	**60**	**120**	**180**	
Slump flow (in.)	26.5	19.3	Placement was stopped due to sudden workability loss		25.5 – 29.5
V-funnel (sec)	4.2	6.3			≤ 12
Air content (%)	-	4.5			-
L-box ratio (h_2/h_1)	0.78	-			0.8 to 1.0
J-ring (in.)	27.5	17.7			Diff. ≤ 2
Static bleeding	0	0			-
Compressive strength at 1 day (psi)	3,840				-
Compressive strength at 7 days (psi)	6,410				3,500
Compressive strength at 28 days (psi)	7,895				6,000

Figure 2.6 – Photo of concrete flow in the field test No. 2 (sudden loss of workability)

Such sudden loss of workability is attributed to the fact that high temperature condition significantly accelerated the ettringite formation of Komponent which also consume considerable amount excess water that can contribute the flowability of the concrete. In addition, such high rate of ettringite formation may be speeded up in the presence of Class C fly ash at the high temperature. It is important to note that all the chemical admixtures were added to the mixture at the job site and the set-retarder may not be effective in controlling setting and workability retention, specifically at the high temperature. The concrete temperature of the second field test was higher than 95°F (35°C). More photos of the second field test are given in Appendix B. Deformations of concrete prisms subjected to different curing conditions (air-drying, sealed, and water-cured) were monitored and given in Figure 2.7. The SCC used for the second field test had about 150 μm/m of shrinkage at the age of 28 days under sealed conditions, which is similar to the exposure condition of actual concrete placed between the deck slab and bridge girder. In addition, the second field test revealed that control of concrete temperature is very critical especially for this type of SCC made with various chemical admixtures. Performance of such type of concrete is more sensitive to some variations in temperature, SP dosage, and water content. It is important to verify the robustness of this type of concrete.

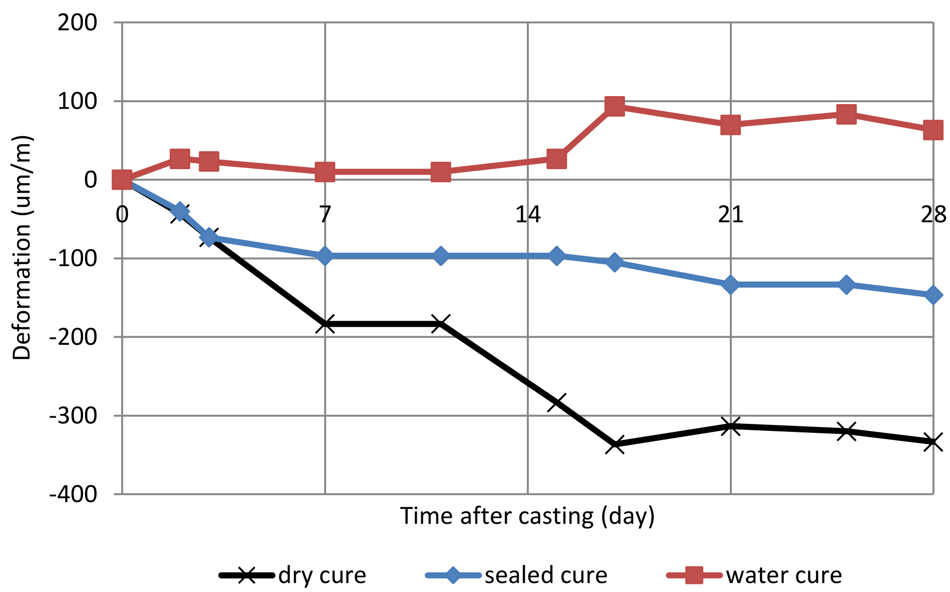


Figure 2.7 – Deformation of SCC used in field test No. 2 under different curing conditions

c. Pumping Mockup Field Test No. 3

For the field test No. 3 that was carried out on August 19th, 2013, the Conex expansive agent was used. Unlike the first and second field tests, all chemical admixtures were added at the batching plant, and additional adjustment of the SP1 was carried out at the job site to secure adequate slump flow. Concrete arrived at the job site approximately 30 minutes after the contact of the cement and water. After SP adjustment, the pumping of the concrete started from the one end point and was gradually continued to push the concrete to the other end of the form. Figures 2.8 and 2.9 show photo and schematic of the pumping line connection to the form as well as a detailed drawing of this connection.

Figure 2.8 – Pumping line connected to the one end of the formwork and concrete pressure indicator tower to monitor pressure build-up exerted on the formwork

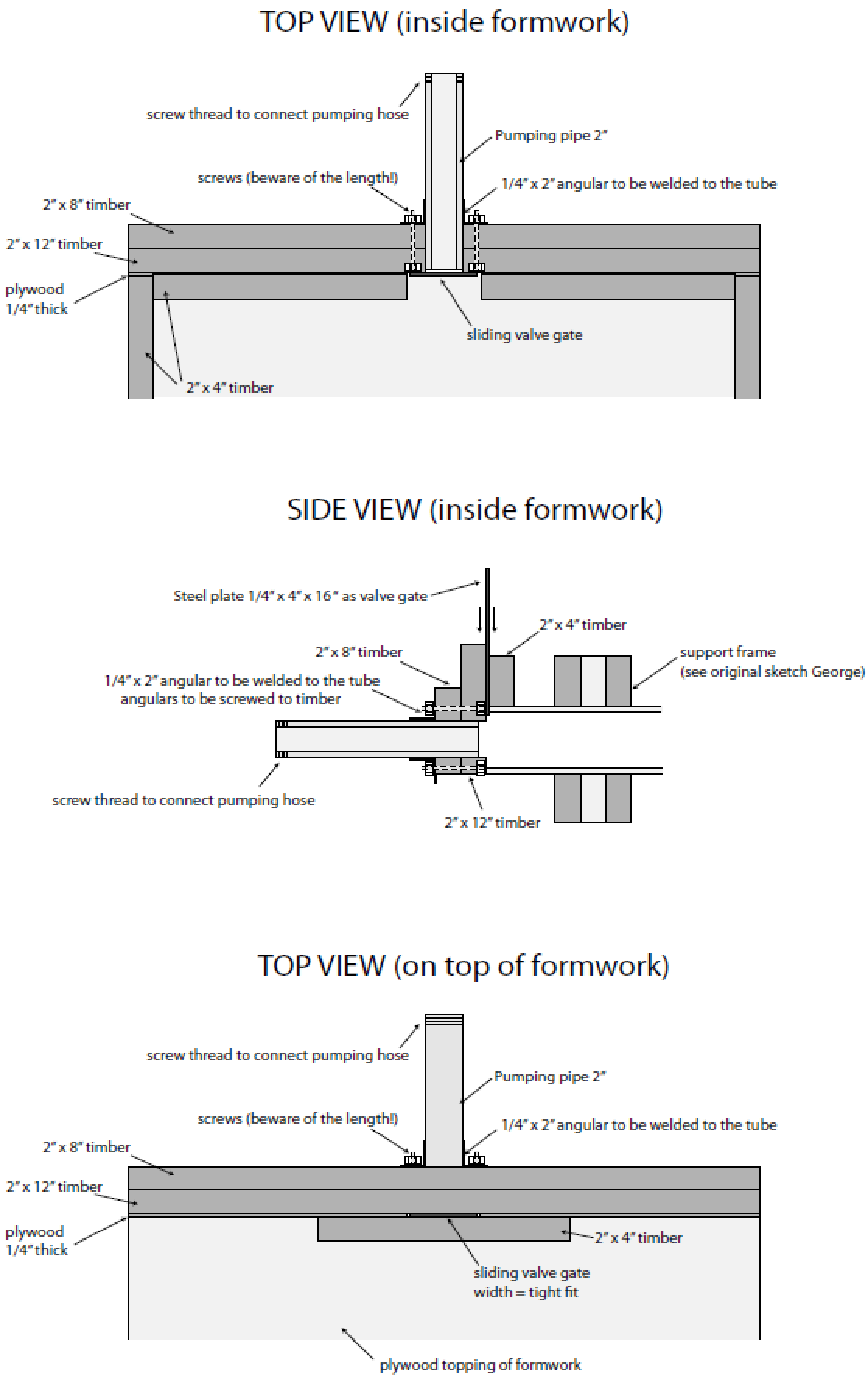


Figure 2.9 – Schematic of the pumping line connected to the one end of the formwork

The mixture compositions and test results of the SCC used in the third field test are summarized in Tables 2.5 and 2.6, respectively. A total of 4 yd^3 of concrete was arrived at the job site (HyPoint laboratory). After the first SP adjustment, slump flow and J-ring flow values of the concrete were 30.7 and 29.7 in. (780 and 755 mm), respectively. The pumping of the concrete started from the one end at the age of 40 minutes. At the initiation of the pumping, the concrete had excellent slump flow of 29.1 in. (740 mm), high passing ability of J-ring flow diameter of 29.5 in. (750 mm), and high resistance to segregation (sieve stability, SR index of 4.3% and static segregation of 3.5%). Concrete flowed smoothly into the form without any leakage or major problem.

Table 2.5 – Mixture composition of SCC used for field test No. 3 (Conex)

Materials	**Imperial unit**	
	(lb/yd^3)	**($fl.oz/yd^3$)**
Type I Portland cement	600	
Class F fly ash (20% of total binder)	179	
Expansive agent (Conex)	87	
Total binder materials	866	
Water, initial	292	
Sand	1615	
3/8 in. Pea gravel (coarse agg.)	1077	
Superplasticizer 1 (Plastol 6200 EXT)		75 (initial)
Superplasticizer 2 (Plastol 5000)		6.8
Set-retarder (Retarder 100)		33.0
VEA (Visctrol)		33.9
Air-entraining agent (AEA92)		1.4

Table 2.6 – Fresh properties and compressive strength values of SCC used for field test No. 3 (Conex expansive agent)

Time after cement-water contact (min)	**25**	**40**	**60**	**70**	**75**	**80**	**120**
Real time (hour:min)	9:25	9:40	10:00	10:10	10:15	10:20	11:00
Observation			Slump loss and high pumping pressure		Pumping stopped	Recovery of slump flow	
Action taken	Accepted	Start 1st pumping	Testing	Testing	½ gallon of water added to 2 yd^3 of concrete	½ gallon-water & 250 ml-SP1 added then start 2nd pumping	Testing
Concrete temperature (°F)	82.6	79.2	-	-	-	86.4	83.3
Slump flow (in.)	30.7	29.1	26	25.6	28	31.5	26.2
T-50 (sec)	0.59	1.09	1.10		0.84	0.4	1.47
V-funnel flow (sec)	-	3.94	-	-	-	3.0	3.46
Air content (%)	-	9	-	-	-	8	9
Unit weight (kg/m^3)	-	2180	-	-	-	2165	2175
L-box ratio (h_2/h_1)	-	0.86	-	-	-	0.86	0.88
J-ring (in.)	29.7	29.5	-	-	-	26.8	25.2
Column segregation, static segregation (%)*	-	3.5%					
Sieve stability, SR (%)	-	4.3%	-	-	-	-	-
VSI	1	0	0	0	0	1	0
Yield stress (Pa)	-	-	13	-	-	-	42
Plastic viscosity (Pa.s)	-	-	28	-	-	-	20
Compressive strength at 1 day (psi)	-						
Compressive strength at 7 days (psi)	3,760						
Compressive strength at 28 days (psi)	5,360						

* Static segregation was determined on the top and bottom sections in accordance with ASTM C 1610.

Slump flow determined at 60 and 70 minutes were 26 and 25.6 in. (660 and 650 mm), respectively. Due to the high loss of the slump flow, pumping pressure increased rapidly and reached to nearly the maximum allowable level before opening of the enclosed section, as presented in Figure 2.10. On the first pumping stage, the concrete flew up to about half of the total length of the element (24 ft out of the total length of 48 ft), as presented in Figure 2.11. The pumping process stopped, and ½ gallon of water was added to remaining concrete of approximately 2 yd^3. After the addition of another ½ gallon of water and 250 ml of SP1, slump flow of the SCC backed to 31.5 in. (800 mm) at 80 minutes, which was 40 minutes from the initiation of the first pumping. The pumping line was connected to the other end of the form. The second pumping process started at 80 minutes of age (Figure 2.12). The form was completely filled by the age of 100 minutes (Figure 2.13). It should be noted that there were few minor leaks on the form due to high pumping pressure.

Figure 2.10 – Photo of indication of nearly maximum pumping pressure on the formwork

Figure 2.11 – Photo of center of the form after the first pumping stage

Figure 2.12 – Photo of the second pumping stage

Figure 2.13 – Photo after the completion of casting from both ends

After 1 week from the casting, the forms were stripped, and visual inspection was carried out. As presented in Figure 2.14, the concrete slab did not have any visible voids or any major issue with the surface finish. In addition, all the chimneys that represent the pockets in the pre-fabricated bridge panels were completely filled, which indicates that the SCC can indeed fill all the shear keys between bridge decks and girder connections. There were some air pockets at the top surface of the concrete slab. The cast slab was cut into six separate sections in order to verify the aggregate distribution of each section. All cut sections exhibited very homogenous distribution of coarse aggregate without any segregation or defects. Detail photos of the visual inspections, including top, side, and section views are presented in Appendix A.

Figure 2.14 – Photo of overall appearance of the cast element

The developed SCC mixture has high flowability, adequate stability, and no shrinkage. The thin and long element was successfully filled using the developed SCC mixture made with w/cm of 0.34 and Conex expansive agent (10% by total mass of binder). However, the three field tests revealed that sharp reduction in flowability or slump flow with respect to time should be prevented in order to reduce high pumping pressure. In addition, viscosity of the concrete should be reduced to increase the flow rate of the concrete on the form and to prevent rapid structural build-up (thixotropy) which may cause sharp increase in pumping pressure with rest time. Therefore, the following recommendations were made to improve the flow properties of the developed SCC:

- Use of Class F fly ash instead of Class C fly ash
- Reduce/eliminate of VMA dosage
- Use of self-consolidating mortar (absence of coarse aggregate reduces viscosity)
- Reduce/eliminate expansive agent.

2.2 Full-Scale Pumping Test

In order to evaluate the constructability of pumping the developed SCC mixture for connecting precast concrete deck panels and I-girder, a full-scale pumping testing was conducted. The full-scale specimen consisted of 58 ft 10 in. long NU900 (3 ft deep) precast/prestressed concrete I-girder and five precast concrete deck panels (three typical panels + two end panels). The specimen was designed and detailed using the same procedures and details proposed for the construction of Kearney East bypass bridge project presented earlier. Figure 2.15 shows the concrete dimensions and reinforcing details of the NU900 girder specimen fabricated by Concrete Industries Inc., Lincoln, NE on June, 20, 2013. Figure 2.16 shows photos of girder fabrication presenting the shear connectors, post-tensioning deviators, and metal tabs similar to those designed for the Kearney East Bypass bridge project.

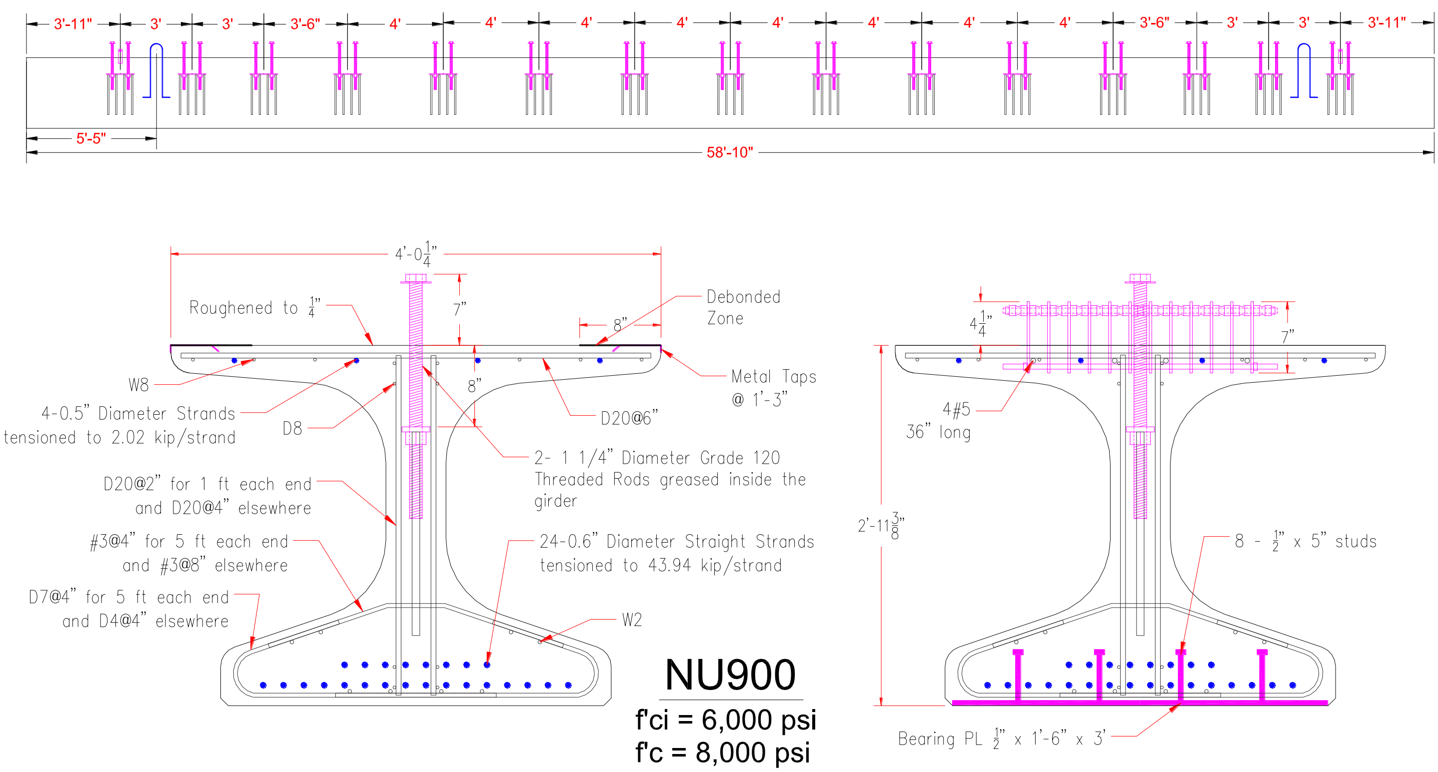


Figure 2.15 – Elevation view, middle cross section (left), and end cross section (right) of NU900 girder specimen

Figure 2.16 – Photos of girder fabrication

A total of five precast concrete deck panels were also fabricated to erect the full-scale specimen: three typical panels and two end panels to simulate actual bridge construction. The three typical panels were obtained by saw cutting a full size demonstration panel fabricated by Concrete Industries Inc., Lincoln, NE on April 25, 2013 as shown in Figure 2.17. The cutting layout resulted in three skewed panels that are 8 in. think, 12 ft long and 7 ft 8.25 in. wide as shown in Figure 2.18. Each panel has three pockets at 4 ft spacing: two pockets with lifting inserts (type A), and one pocket without lifting inserts (type B). The two end panels were also fabricated by Concrete Industries, Inc. in Lincoln, NE in a later date. Each end panel is 8 in. thick, 11 ft 4.75 in. long, and 7 ft 8.25 in. wide with 14° skew. End panels contain embedded anchor blocks for deck post-tensioning. Figure 2.19 shows the concrete strength of the specimen girder and deck components. Curing compounds were sprayed to the pre-fabricated girder and decks for curing.

Figure 2.17 – Photo of full size demonstration deck panel showing panel soffit and shear pockets

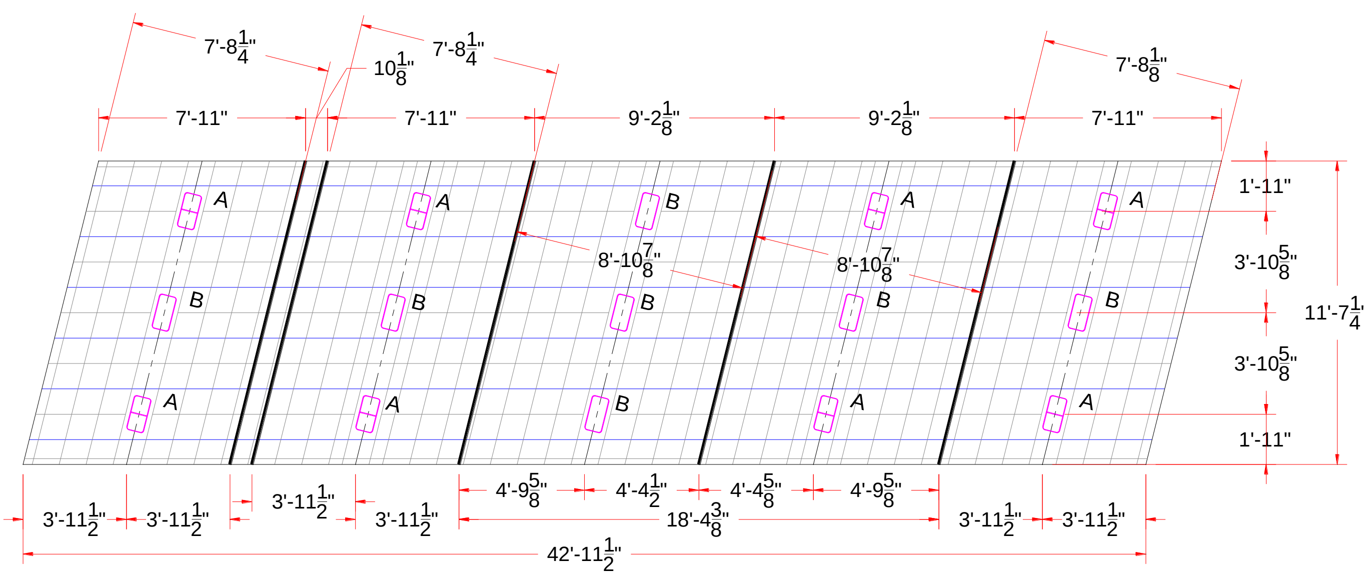


Figure 2.18 – Layout of panel saw cutting

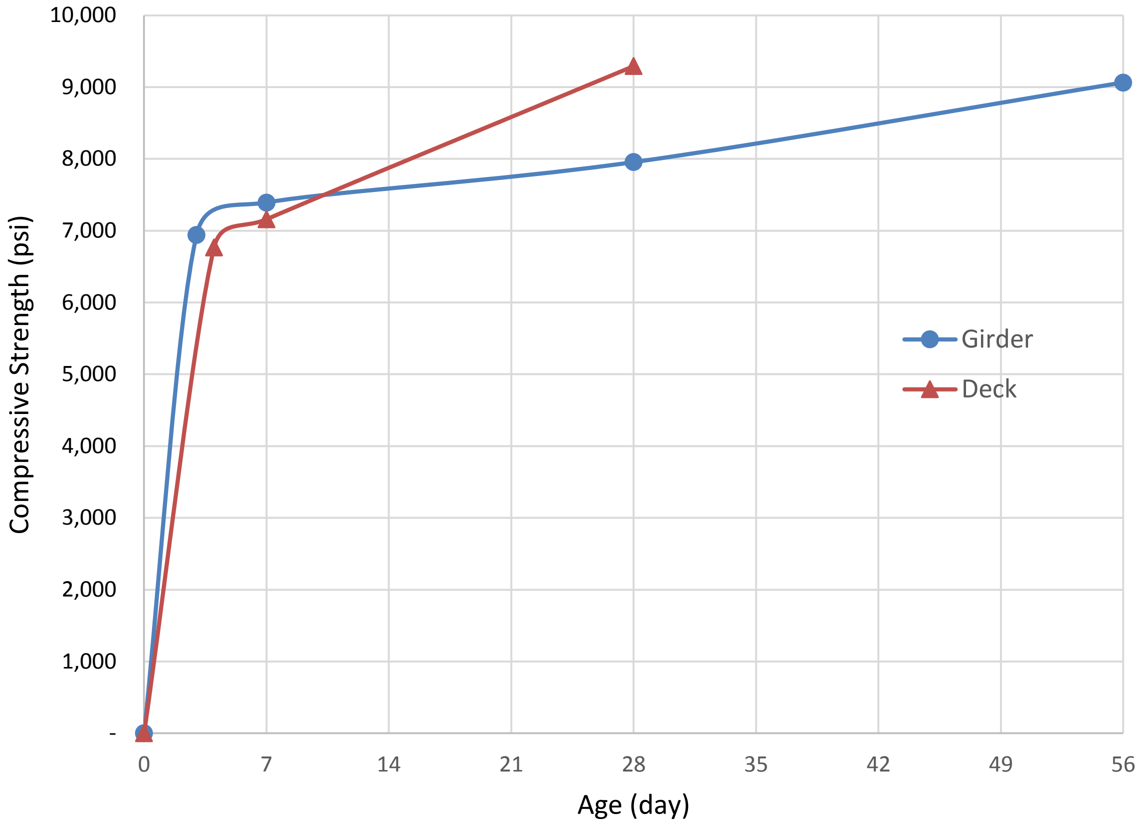


Figure 2.19 – Concrete strength of precast girder and deck panels

The NU900 girder specimen and the five deck panel specimens were shipped to UNL Structural Laboratory in Omaha for erection and testing. The steps followed in the specimen erection are shown below. Photos of these steps are shown in Appendix B, and a video of specimen erection can be seen at the following YouTube link: http://www.youtube.com/watch?v=Jky8gpaGhRc.

1. Place the girder on roller supports located at the girder ends to create a simple span of 57 ft 10 in.
2. Lay down 12-0.6 in. diameter post-tensioning strands on the top flange and thread the ends through the deviators at girder ends. Strands were 4 ft longer than the girder.
3. Install steel bent plates (or angles) used as deck support system by welding them to the metal tab inserts on the girder top flange. The height of the bent plates was adjusted to achieve at least 3 in. thick haunch and provide the required deck profile after considering deck deflection. These bent plates are also acting as side forms for haunch concrete.
4. Adjust the height of shear connectors to have an embedment in the deck of at least 5 in.
5. Attach compressive material (backer rod) to the top of the bent plates to prevent leakage.

6. Place precast concrete deck panels on the deck support system starting from the middle and moving outward.
7. Form the sides and bottom of transverse joints between adjacent deck panels using backer rod and wood forms.
8. Place the specified SCC mixture into transverse joints after cleaning and moistening them.
9. Place anchor plates, post-tensioning chucks, and bearing/bulkhead plates at the two end panels.
10. Post-tension the strands using mono-strand jack starting from the middle strands and moving outward in a symmetrical manner to minimize the eccentricity.
11. Pump the specified SCC from the pump sleeves welded to the bulkhead plate provided at girder ends until concrete overflows from the inspection vents.

The developed SCC mixture for connecting precast girder and deck panels was slightly revised to accommodate material availability in the state of Nebraska. Table 2.7 lists the composition and proportions of the revised SCC mixture that consists of 1PF cement (Type I cement pre-blended with 23% ± 2% Class F fly ash), 3/8" limestone aggregate, C33 natural sand (called 4110), and BASF admixtures. No expansive agents was used in the revised mixtures.

Table 2.7 – Composition and proportions of the revised SCC mixture

Component	Quantity	US Units
IPF Cement	866	lb/yd^3
Water	285	lb/yd^3
w/c	**0.33**	**N/A**
4110 Sand	1615	lb/yd^3
3/8 in. Limestone	1077	lb/yd^3
TOTAL AGG.	**2692**	**lb/yd^3**
HRWR (Glenium 3030)	4	oz/cwt
Retarder (Delvo)	4	oz/cwt
VMA (Rheomac 362)	4	oz/cwt
AEA (MB-AE 90)	0.2	oz/cwt
WRA (RheoTEC Z-60)	4	oz/cwt

A trial batch was conducted on September 27, 2013 to evaluate the performance of the revised SCC mixture and pour the transverse joints between adjacent deck panels shown in Figure 2.20. The mixture achieved an average slump flow of 29 in. (735 mm), as shown in Figure 2.21, J-ring slump difference less than 1 in., and VSI = 1.0. These values indicate that the revised SCC mixture has adequate flowability, passingability, and resistance to segregation. Several 4 x 8 in. cylinders were taken to evaluate the compressive strength and hardened visual stability index (HVSI) shown in Figure 2.22. The same mixture with no modifications will be used in the pumping test to fill the gap between the precast girder and deck panels of the same specimen.

Figure 2.20 – Transverse joint between adjacent deck panels.

Figure 2.21 – Slump flow of the SCC mixture used in filling transverse joints (VSI = 1.0)

Figure 2.22 – Coarse aggregate distribution in joint concrete (HVSI = 1)

Pumping test was conducted on October 18, 2013 using ready mixed concrete from Lyman Richey Co. and Hotz concrete pumping Co. Concrete was delivered with low flowability, therefore, several dosages of HRWRA were added to achieve an average slump flow of 27.8 in. as shown in Figure 2.23. Other workability properties are summarized in Table 2.8.

Figure 2.23 – Slump flow of the SCC mixture used in the pumping test (VSI = 0)

Table 2.8 – Workability properties of the pumped SCC

Test	Criteria	Time (min)		
		30	60	90*
Slump flow (in.)	26 - 30	27.75	25	30.5
Visual stability index (VSI)	0 - 1	0	0	1
J-ring slump spread difference (in.)	0 - 2	1		
Penetration (in.)	0 - 1	0.75		
Filling capacity (%)	80 - 100	96		
Column segregation (%)	0 - 10	5.2		
Long through segregation (%)	0 - 30	16.1		
Air content (%)	5 – 9			
Static yield stress (Pa)	N/A			49
Dynamic yield stress (Pa)	N/A			22
Plastic viscosity (Pa.s)	N/A			3.3

* Another dosage of HRWRA was added to an assumed quantity of concrete

Pumping started by using a ½ cubic yard of slurry to lubricate the hose and haunch area, then SCC was pumped from one end, and the flow of concrete from the 1 in. diameter holes at 4 ft spacing was monitored to ensure the filling of shear pockets. Pumping continued until the accumulated pressure caused uplifting of the specimen panels. Pumping stopped and proceeded from the other end until the haunch and pockets were completely filled, and vents were plugged. Below is a detailed sequence of events recorded in this investigation:

- 1:42 PM: 5 yd^3 of concrete arrived. Initial slump flow = 18 in.
- 1:52 PM: 1.5 fl.oz/cwt of HRWRA was added. Slump flow = 24 in.
- 1:56 PM: 1 fl.oz/cwt of HRWRA was added. Slump flow did not change significantly.
- 2:02 PM: 1 fl.oz/cwt of HRWRA was added. Slump flow = 27.75 in. Accepted.
- 2:05 PM: Pumping started at 35 to 50 bars in concrete pressure as shown in Figure 2.24. Concrete overflow from vents was stopped using plugs, as shown in Figure 2.25.
- 2:15 PM: Concrete leaked after reached 29.5 ft from the pumping point due to the high pumping pressure causing uplift of deck panels as shown in Figure 2.26. Pumping stopped.
- 2:37 PM: Pumping resumed from the other end as shown in Figure 2.27.
- 2:50 PM: 1.5 fl.oz/cwt of HRWRA was added to the remaining amount of concrete. Slump flow = 30.5 in.
- 3:04 PM: Pumping was completed.

A YouTube presentation of the pumping test can be seen at:
http://www.youtube.com/watch?v=8kLjKAfsylY

Figure 2.24 – Pumping SCC using 2 in. diameter hose

Figure 2.25 – Plugging 1 in. diameter vents

Figure 2.26 – Concrete leakage during pumping due to deck uplift

Figure 2.27 – Pumping concrete from the other end of the specimen

Figure 2.28 presents the compressive strength of the lab-mixed and ready-mixed SCC used in pouring the transverse joints and haunch respectively. The plot indicates the reproducibility of the proposed mixture. Figure 2.29 shows a cross section of the hardened haunch concrete after specimen testing, which presents the coarse aggregate distribution. This figure indicates the pumped SCC has adequate resistance to segregation.

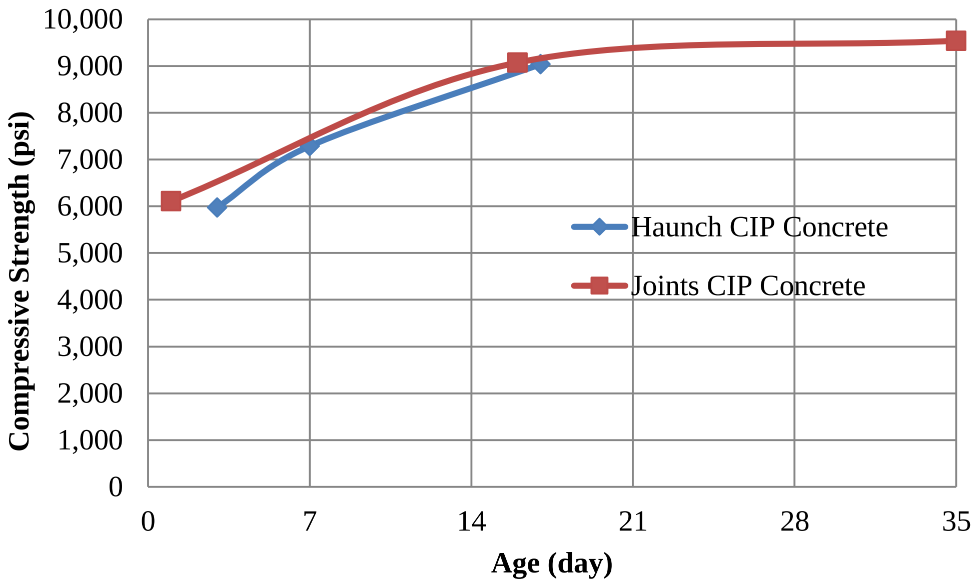
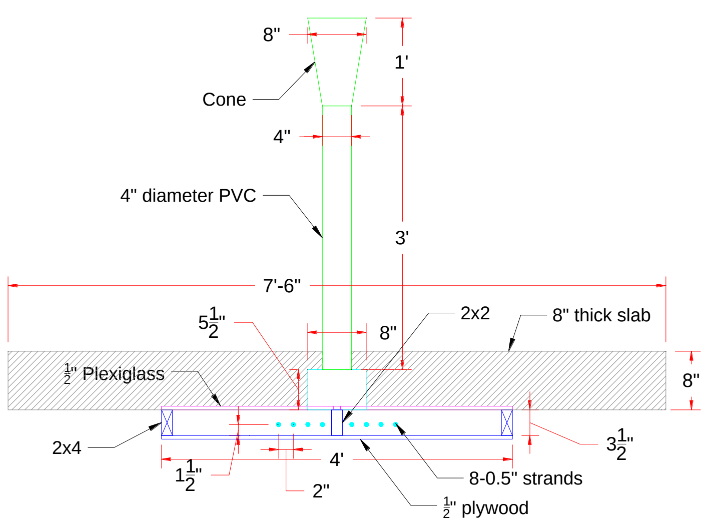


Figure 2.28 – Concrete compressive strength for CIP transverse joints and haunch

Figure 2.29 – Coarse aggregate distribution in haunch concrete (HVSI = 1.0)

3 MOCKUP POURING TEST

The challenge of pumping SCC into the haunch area from one end to the other one has indicated that it is not practical to pump SCC for a 334-ft long girder line. Despite the optimized rheological properties of the SCC, high pumping pressure would be required to ensure proper filling of the haunch and pocket areas, which can cause uplift of deck panels. This conclusion resulted in proposing a new approach, which is presented in this chapter. Testing was performed to evaluate the possibility of pouring SCC from a 4-in. diameter pouring port located in the middle of a 12 ft long deck panel. The goal is to completely fill the haunch area between deck panels and the supporting girder as well as the shear pockets within the deck panels without pumping SCC. The mockup pouring test aims to determine whether 12 ft (using only one pouring port per panel) is adequate to ensure a complete and efficient filling of the haunch and pockets. Figure 3.1 shows the cross section and plan views of the mockup specimen, which consists of: a) wood formed channel that is 16 ft long, 4 ft wide, and 3.5 in. thick; b) two deck panels with two shear pockets spaced at 12 ft; c) ½ in. plexiglass sheets covering the top of the channel between the panels to allow observing the concrete flow; d) 8-0.5 in. diameter strands lightly tensioned and located in the mid-height of the channel to simulate the post-tensioning strands used underneath the deck; e) several 2x2 lumber pieces to support the plexiglass and simulate the shear connectors located every 4 ft along the specimen; and f) 1 in. diameter vents to allow the air to escape while filling the haunch and pockets. Photos of the pouring mockup test specimen are presented in Figure 3.2.



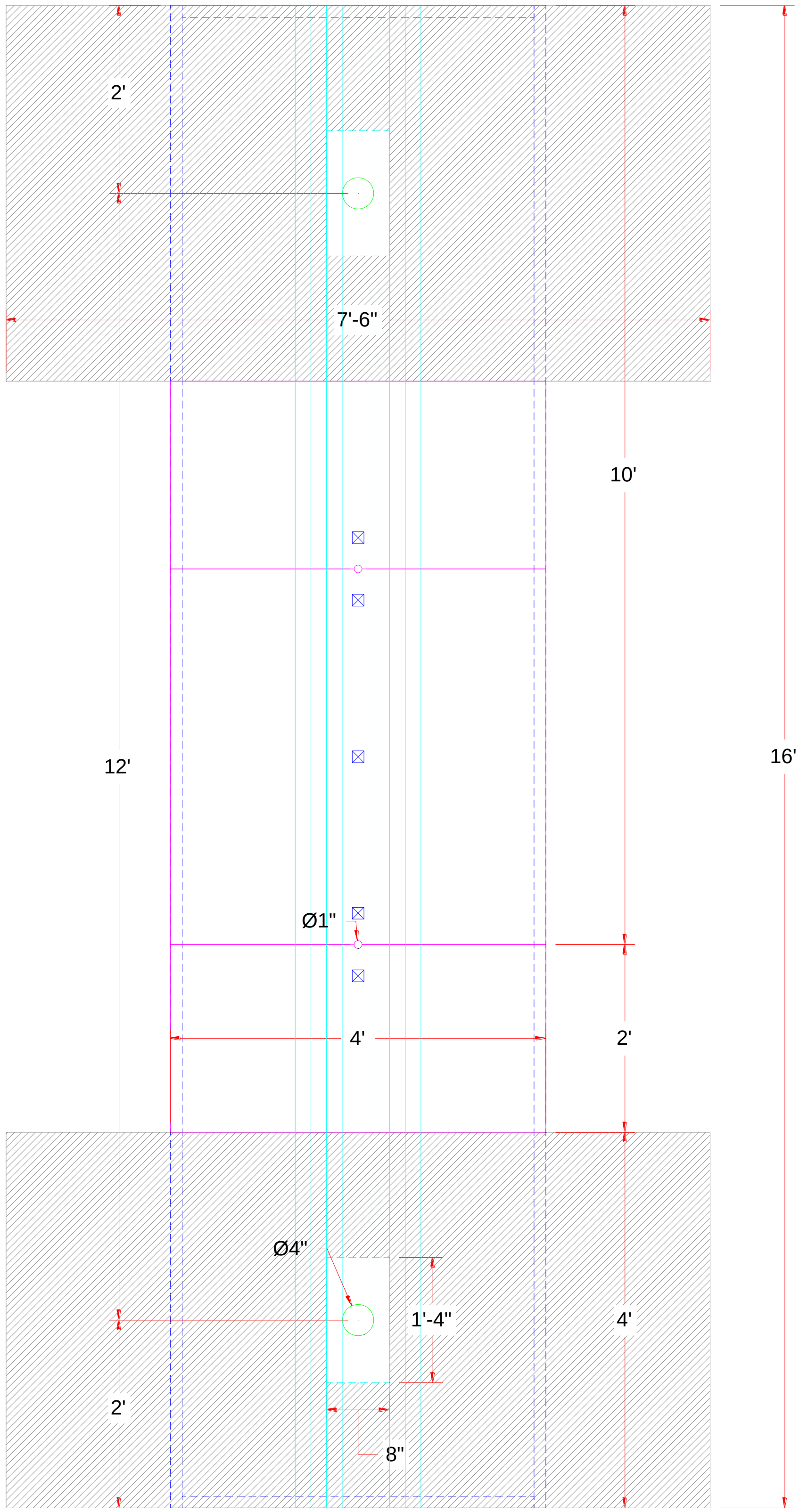


Figure 3.1 – Section and plan views of the mockup test specimen

Figure 3.2 – Photos of the pouring mockup test specimen

Table 3.1 lists the proportions of the SCC mixture delivered by ready mix on Nov. 18, 2013 at 11:30 am. The mixture initially had very low flowability, which required several additional dosages of HRWR and WRA as shown in Table 3.1 in order to achieve the required flowability. Table 3.2 lists the tests performed and the results of workability tests. Photos of slump flow and J-ring tests are shown in Figure 3.3 and photos of penetration resistance and air content tests are shown in Figure 3.4.

Table 3.1 – SCC mixture proportions

Component	Quantity per 1 cy	Units	Quantity per 3 cy
IPF Cement	866	lb/yd^3	2598
Water	285	lb/yd^3	855
w/c	**0.33**	**N/A**	**0.33**
Sand	1615	lb/yd^3	4845
3/8 in. Limestone	1077	lb/yd^3	3231
TOTAL AGG.	**2692**	**lb/yd^3**	**8076**
HRWR (Glenium 3030)	6	oz/cwt	156
AEA (MB-AE 90)	0.25	oz/cwt	6
WRA (RheoTEC Z-60)	0	oz/cwt	0
Added at the site			
HRWR (Glenium 3030)	5	oz/cwt	130
WRA (RheoTEC Z-60)	4	oz/cwt	104

Table 3.2 – Tests performed and their results

Conducted Test	ASTM/AASHTO Standard	Measured Parameter	Value	Criteria	Decision
Slump flow	C 1611 / TP 73	Average diameter (in.)	27.8	26 - 30	OK
Slump flow	C 1611 / TP 80	Visual stability index (VSI)	0	0 - 1	OK
J-ring	C 1621 / TP 74	Difference in slump flow and J-ring flow diameter (in.)	0.5	< 2 in.	OK
Penetration resistance	C 1712	Penetration (in.)	0.25	< 0.5	OK
Air content	C 231 / T 152	Percentage of air (%)	3.8%	5%-9%	Add more AEA
Static segregation	PP 58	Hardened visual stability index (HVSI)	0	0 - 1	OK
Compressive strength	C 39 / T22	Average 3-day strength (psi)	6,520	> 3,500	OK
	C 39 / T22	Average 28-day strength (psi)	11,860	> 6,000	OK

Figure 3.3 – Slump flow (top) and J-ring (bottom) tests (VSI = 0)

Figure 3.4 – Penetration resistance (top) and air content (bottom) tests

Concrete was poured using a large bucket and a custom-made 8 in. diameter chute, as shown in Figure 3.5, to easily pour the concrete into the cone/funnel used on top of the 4 in. diameter pipe. Figure 3.6 shows photos of the concrete flowing from one pouring port to the other, completely filling the channel and pockets, and encapsulating the strands in a very short time without trapping any air pockets. By the end of the test, concrete overflow at one of the transverse joints between Plexiglass sheets, as shown in Figure 3.7, because one of the screws holding the sheets was pulled out due to concrete pressure causing a gap between adjacent sheets. This was not a concern as it is not the case when adjacent deck panels are used. Table 3.3 shows the revised SCC mixture to account for the lack of entrained air and the need for a retarder. Figure 3.8 shows a photo of the hardened concrete after form stripping and cylinder testing. These photos show a uniform distribution of coarse aggregate across the section, which indicates a hardened visual stability index (HVSI) of 0. Figure 3.9 shows the compressive strength gain with time for SCC cylinders. A video of the mockup pouring test is posted in YouTube at: http://www.youtube.com/watch?v=85pAU3yFs9s

Figure 3.5 – The bucket and chute used in pouring SCC

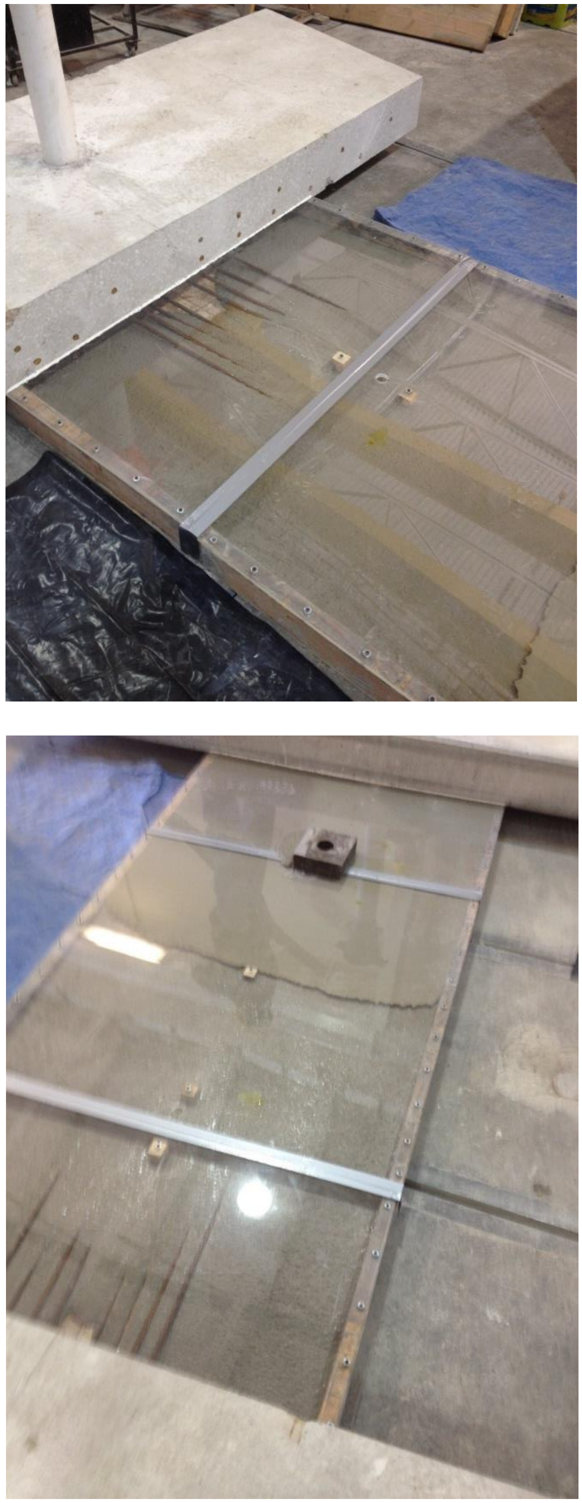


Figure 3.6 – SCC flowing from one port to the other and completely filling the channel

Figure 3.7 – Concrete overflowing at one of the joints between Plexiglass sheets

Table 3.3 – Revised SCC mixture proportions

Component	Quantity per 1 cy	US Units
IPF Cement	866	lb/yd^3
Water	285	lb/yd^3
w/c	**0.33**	**N/A**
Sand	1615	lb/yd^3
3/8 in. Limestone	1077	lb/yd^3
TOTAL AGG.	**2692**	**lb/yd^3**
HRWR (Glenium 3030)	6	oz/cwt
Retarder (Delvo)	4	oz/cwt
VMA (Rheomac 362)	0	oz/cwt
AEA (MB-AE 90)	0.4	oz/cwt
WRA (RheoTEC Z-60)	4	oz/cwt

Figure 3.8 – Coarse aggregate distribution in hardened concrete (HVSI = 0)

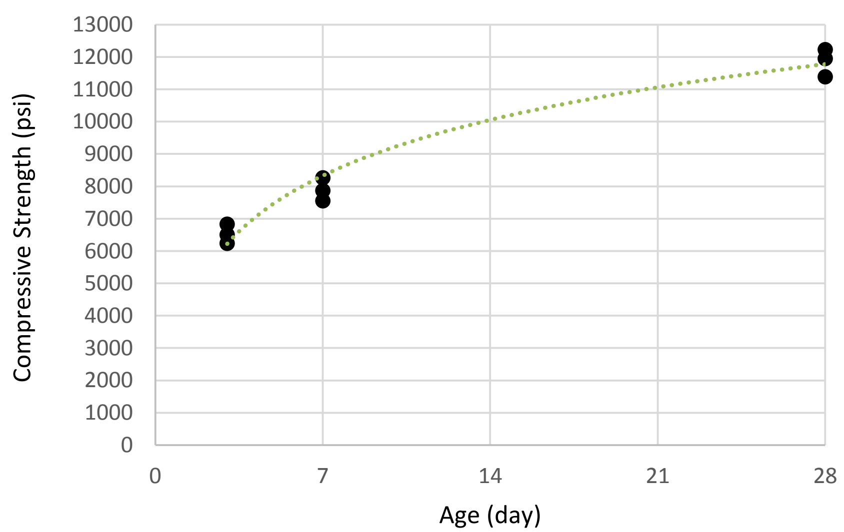


Figure 3.9 – Compressive strength test results of SCC

4 DEVIATOR PULLOUT TESTING

The longitudinal post-tensioning of the precast concrete deck panels is conducted using a new concept in the 2nd generation of NUDECK system. This concept eliminates the need for embedded post-tensioning ducts in the deck, couplers to connect these ducts, and threading post-tensioning strands through these ducts along the bridge length, which is a cumbersome and tedious task. Instead, post-tensioning stands are located in the haunch area between the girder top flange and the deck soffit. Deviators are located at the bridge ends to raise the post-tensioning stands to the level of the deck mid-thickness. These deviators are subjected to a pullout force that is equal to the vertical component of the post-tensioning force, which is equal to 12 (number of strands) x 43.9 (strand tension force) x 0.97 (instantaneous losses) x sin 10 (slope of the strands) = 88.7 kips.

The objective of this investigation is to determine the anchorage capacity of the post-tensioning deviators embedded in the girder top flange and the required detailing to ensure an anchorage capacity of at least 20% more than the required capacity (1.2 x 88.7 = 106.5 kips). It should be noted that the pullout force applied due to post-tensioning is a temporary force as the haunch will shortly be filled with concrete and the deviators will not be subjected to this force once the haunch concrete hardens.

Figure 4.1 shows the proposed design of the deviators, while Figure 4.2 shows views of deviator anchorage to the girder top flange. It was proposed to increase the top flange thickness by 1 in. at the end 8 ft of the girder to provide adequate embedment of the anchorage reinforcement used around the deviator, which are 10#5 - 4ft long bars, 2-0.6 in. strands, and D20@6”.

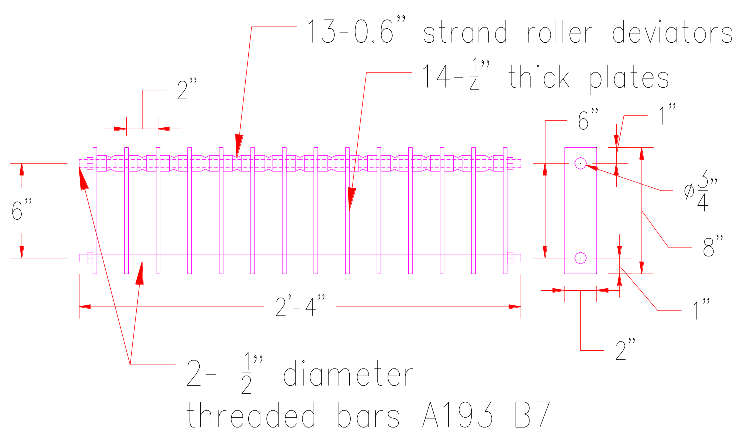


Figure 4.1: Deviators used for deck longitudinal post-tensioning

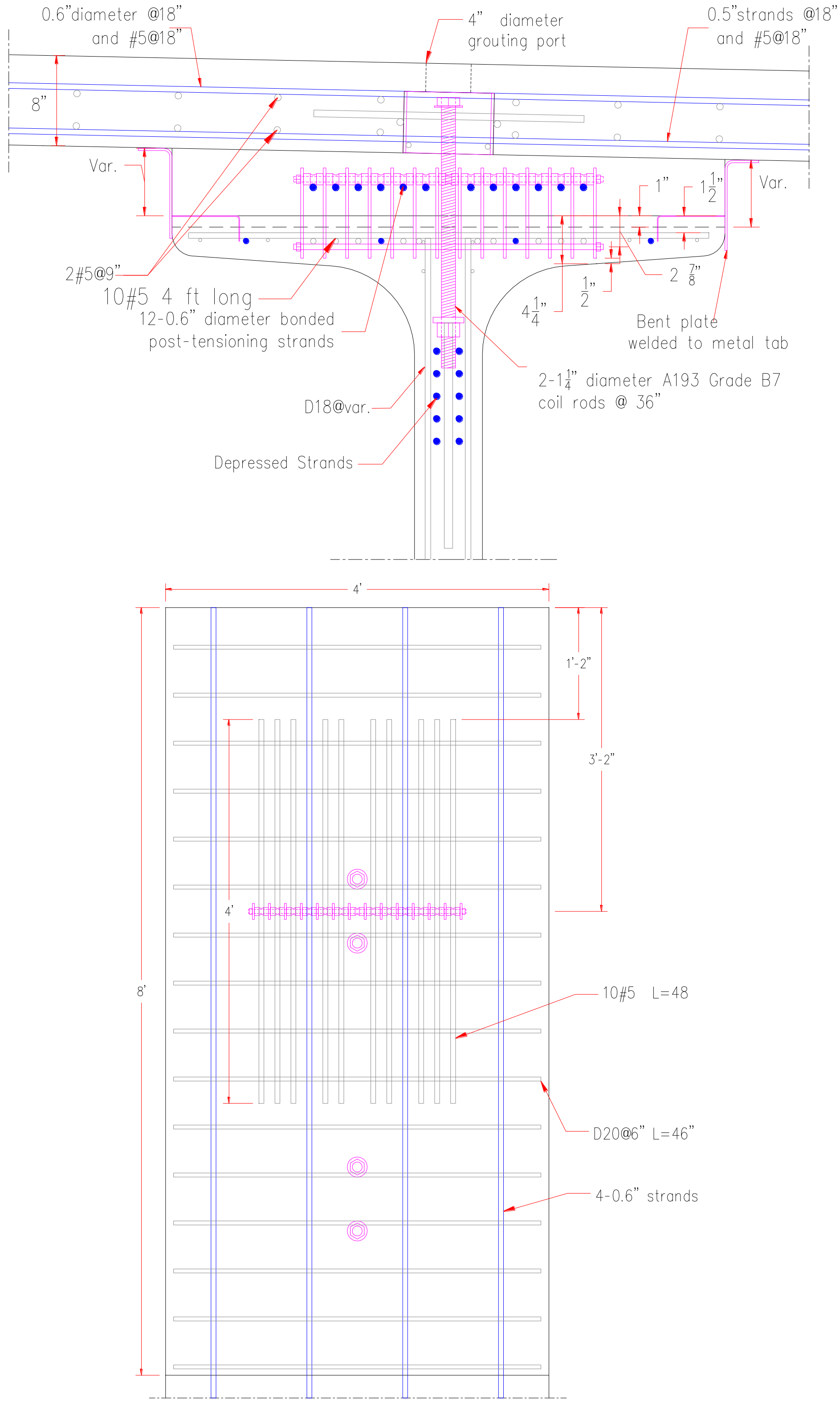


Figure 4.2: Cross section and plan views of the proposed deviator anchorage

To test evaluate the pullout capacity of the deviator, two specimens were designed as shown in Figure 4.3. Each specimen is 4 ft long, 2 ft wide, 7.5" thick, and contain deviators for only six strands (half of the girder deviators shown in Figure 4.2) due to the difficulty of pulling out 12 deviators simultaneously with the available testing equipment. These deviators will be pulled out using the custom made attachment shown in Figure 4.4. The target pull out force is 106.5/2 = 53.25 kips. The deviators were embedded 2 7/8" from the top surface of the specimen and anchored using 6#5 longitudinal bars and #4@6" transverse bars similar to the detail shown in Figure 4.2. Figure 4.5 shows the specimen before and after concrete placement.

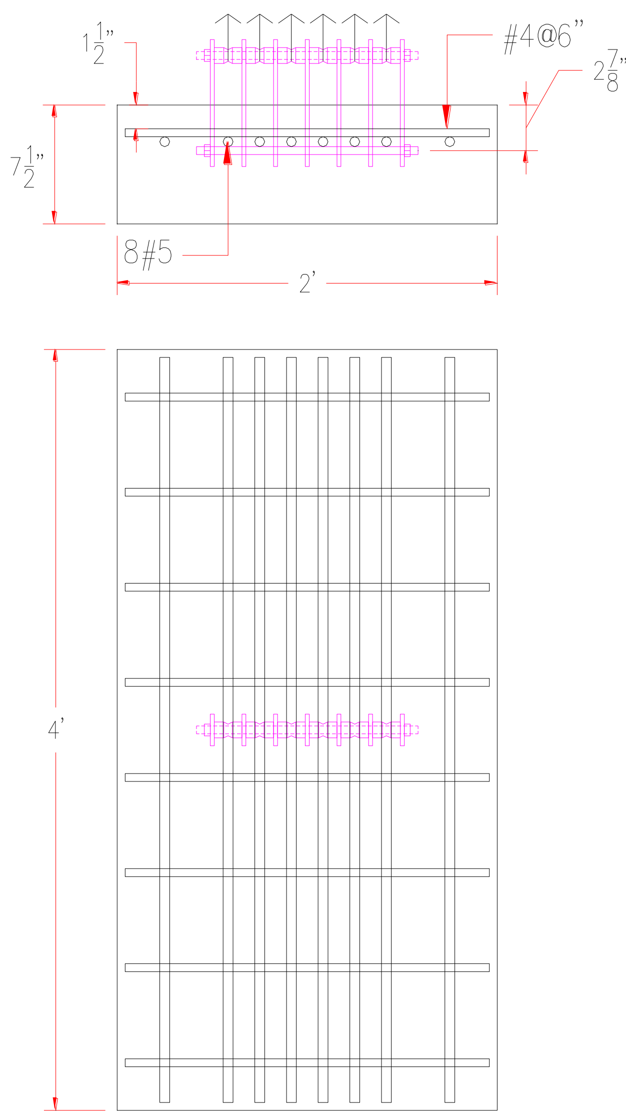


Figure 4.3: First specimen used for deviator pullout testing

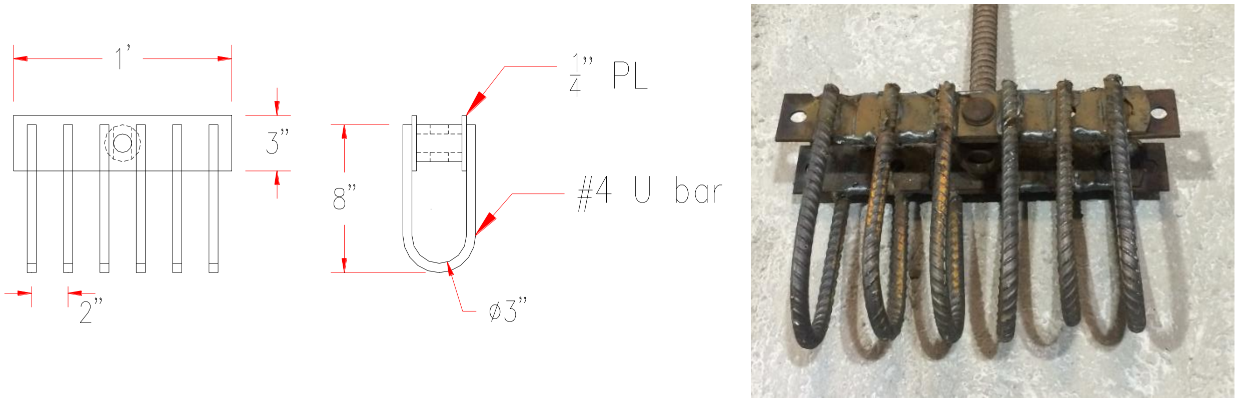


Figure 4.4: Attachment used for deviator pullout testing

Figure 4.5: Specimen photos before and after concrete placement

The first specimen was tested as shown in Figure 4.6 when the concrete strength was approximately 7 ksi. Figure 4.6 also shows the failure mode that took place at 25 kips, which is primarily splitting of the specimen due to lack of vertical reinforcement across the thickness. The second specimen was tested as shown in Figure 4.7 similar to the first specimen with the exception of the higher concrete strength (8 ksi) and the closer location of supporting beams (7” instead of 13.5”). Figure 4.7 indicates a similar failure mode, which took place at 32.5 kips. Figure 4.8 plots the loading history of the two specimens. The low pullout capacity of the two specimens indicated that the deviators anchorage and test specimens need to be re-designed.

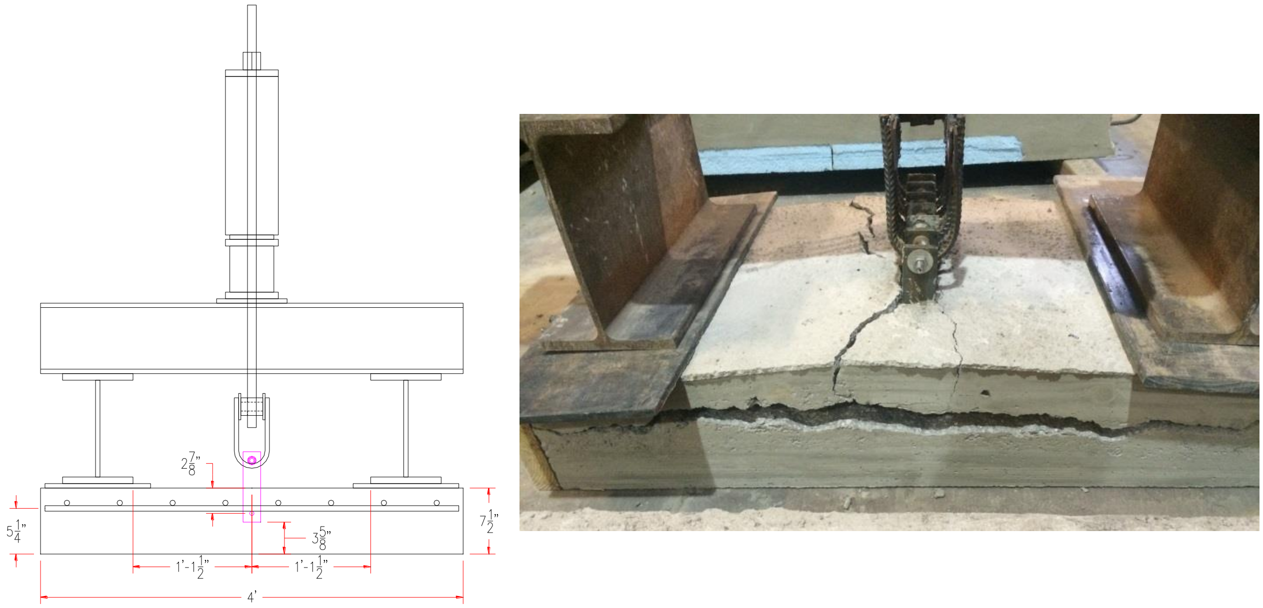


Figure 4.6: Test setup and failure mode of first specimen

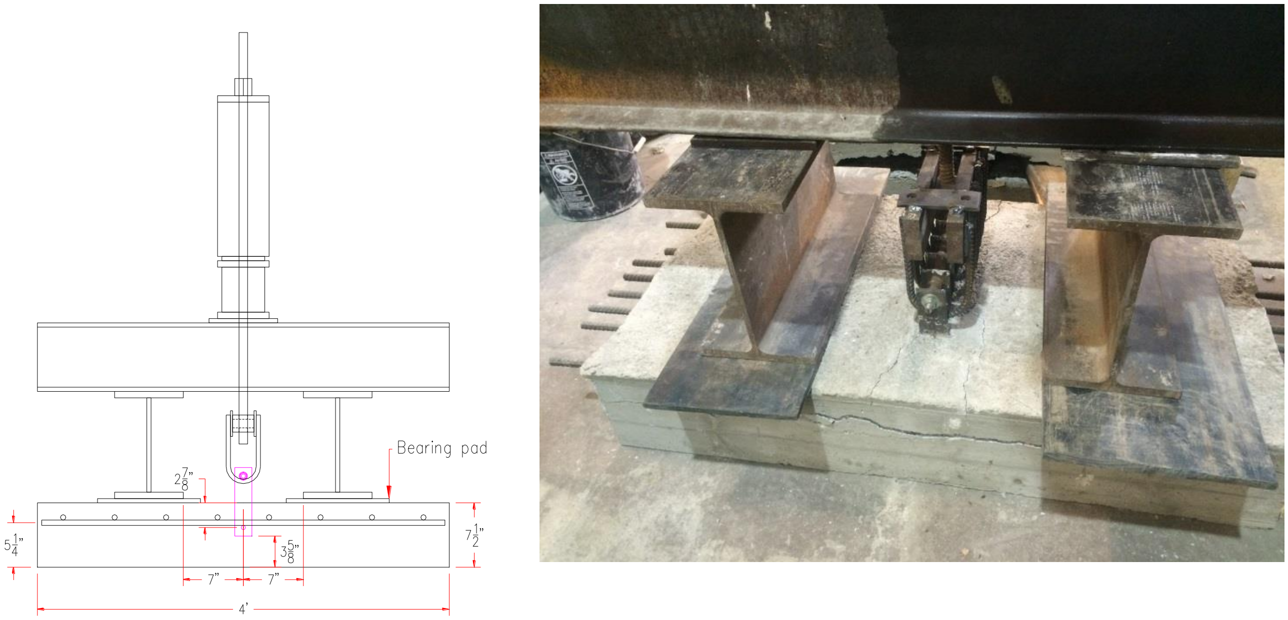


Figure 4.7: Test setup and failure mode of first specimen

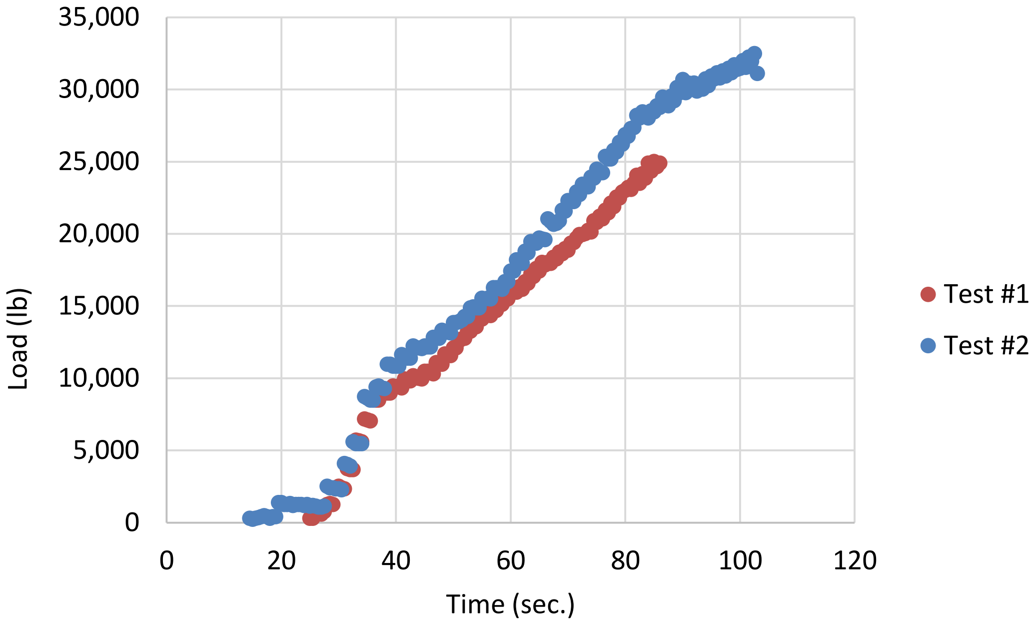
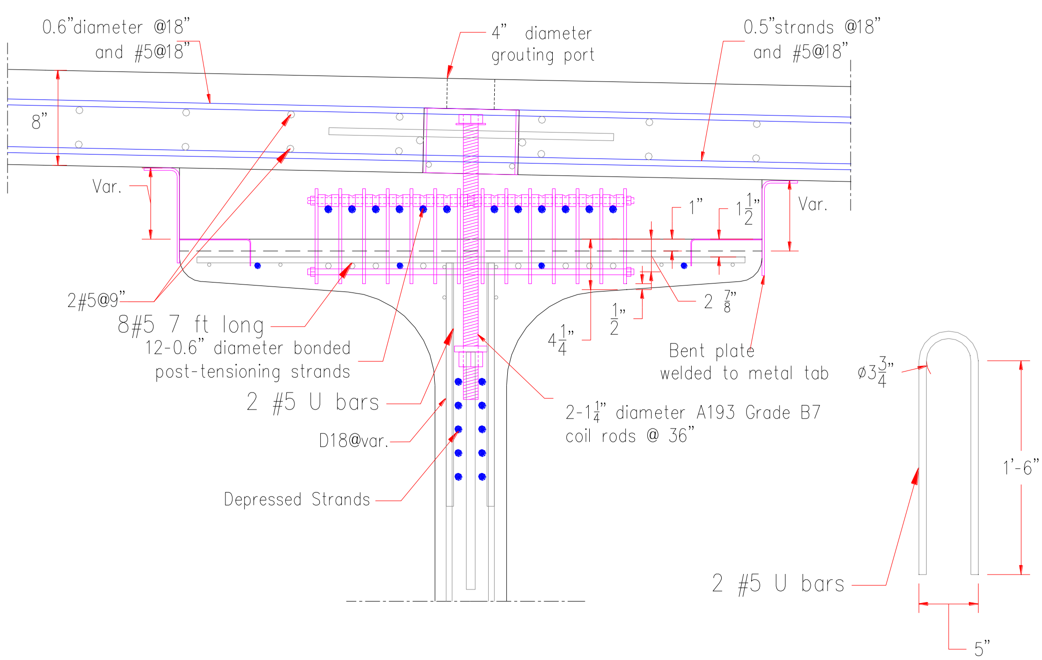


Figure 4.8: Pullout test results of the two specimens

Figure 4.9 shows views of the revised deviator anchorage to the girder top flange. This detail is similar to the one presented in Figure 4.2 with the exception of adding 2#5 U bars around the deviator and increasing the length of longitudinal #5 bars to 7 ft instead of 4 ft. These changes were proposed to enhance the deviators anchorage to the girder.



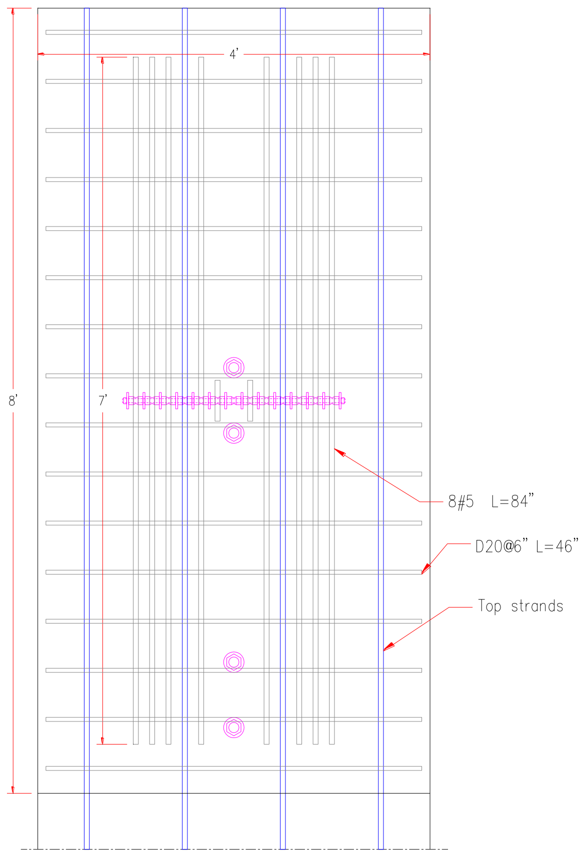


Figure 4.9: Cross section and plan views of the revised deviator anchorage

To test evaluate the pullout capacity of the revised deviator anchorage, one specimen with two deviators was designed as shown in Figure 4.10. The specimen is 8 ft long, 4 ft wide, 16” thick, and contains two six-strand deviators similar to those tested in the previous specimens. These deviators will be pulled out using the same custom made attachment shown in Figure 4.4. Each deviator was embedded 2 7/8” from the top surface of the specimen and anchored using 1#5 U bars, 5#5 longitudinal bars, and #4@6” transverse bars similar to the detail shown in Figure 4.9. Figure 4.11 shows the specimen before and after concrete placement.

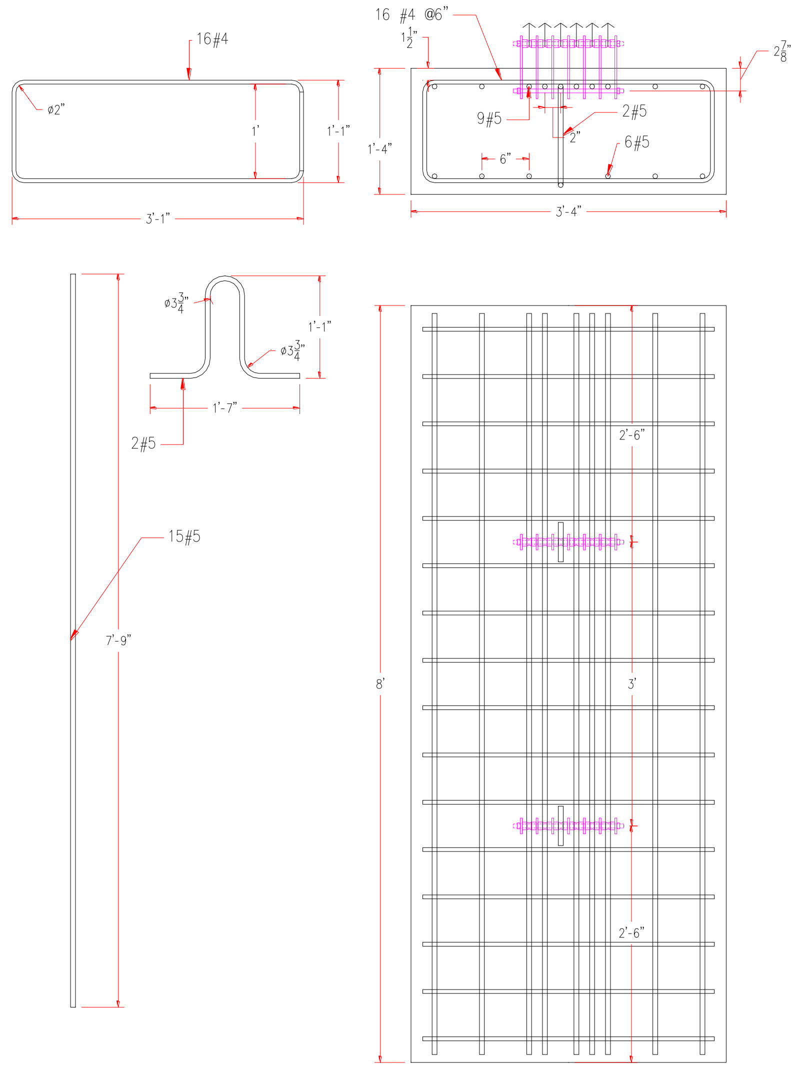


Figure 4.10: Dimensions and reinforcement details of the revised specimen

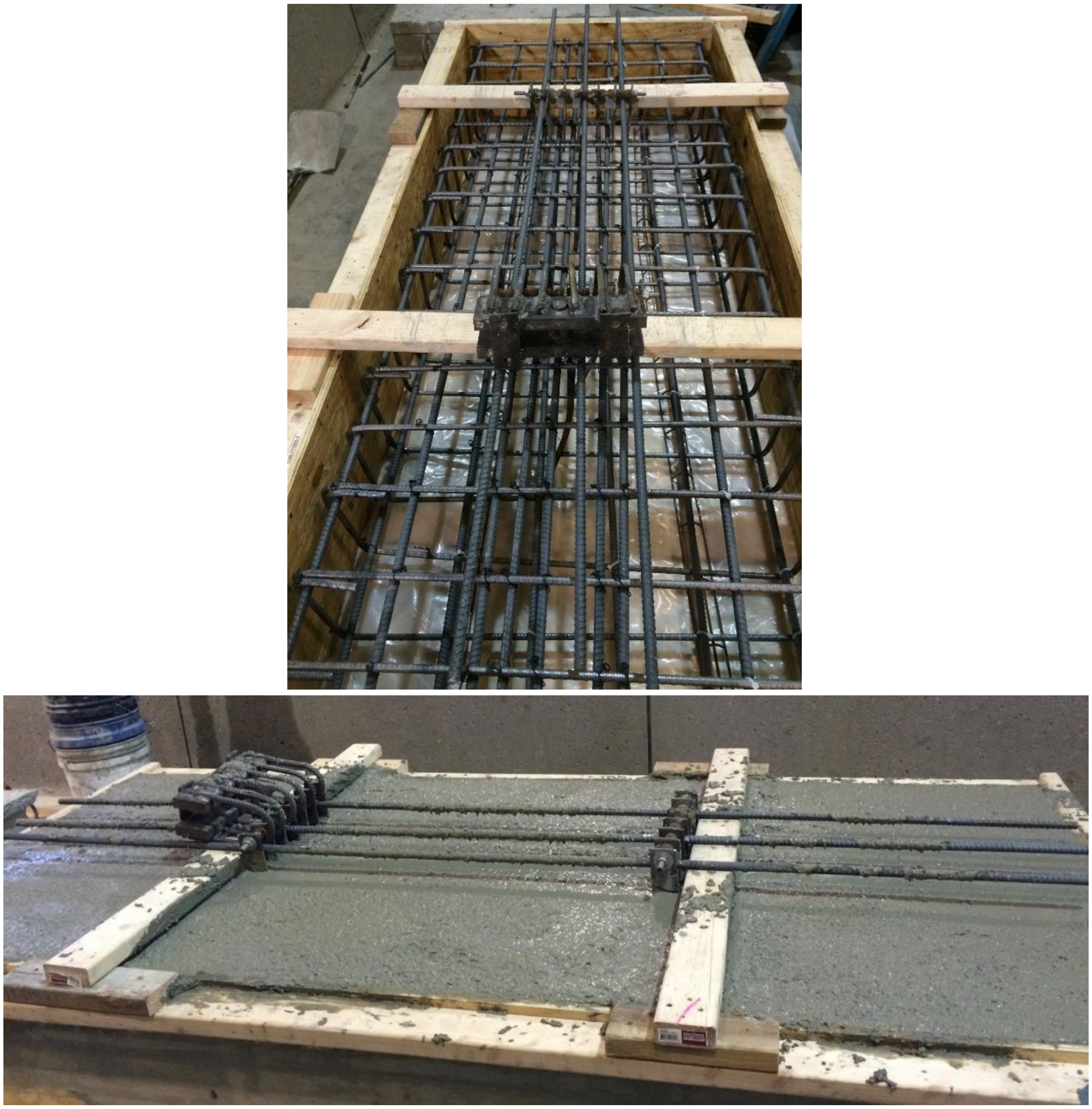


Figure 4.11: Specimen photos before and after concrete placement

The first test was conducted as shown in Figure 4.12 when the concrete strength was approximately 7.8 ksi. The pullout capacity reached was 55.4 kips and the threaded rod was sheared suddenly as shown in Figure 4.13. This was primarily due to the use of grade 36 ksi threaded rod rather than the specified one. The test was repeated after replacing the rod with high strength Grade 5 threaded rod, which resulted in the concrete breakout failure shown in Figure 4.14 at 52 kips. The second deviator was tested when the concrete strength was approximately 8.9 ksi. Figure 4.15 shows a similar failure mode, which took place at 60.4 kips. Figure 4.16 plots the loading history of the three tests specimens, while Table 4.1 summarizes the pullout test results for all specimens.

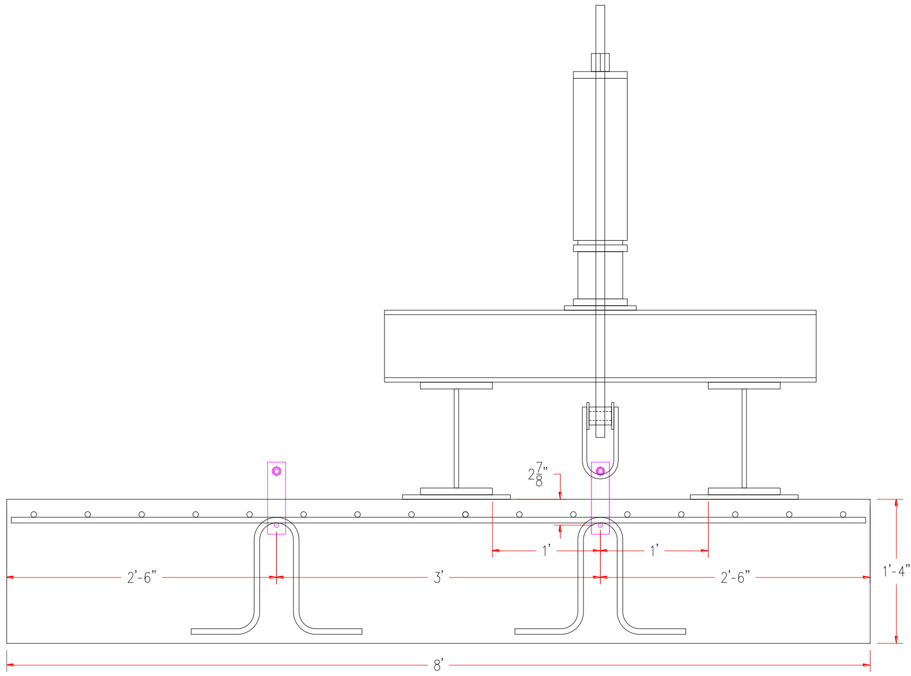


Figure 4.12: Pullout test setup

Figure 4.13: Rupture of the threaded rod

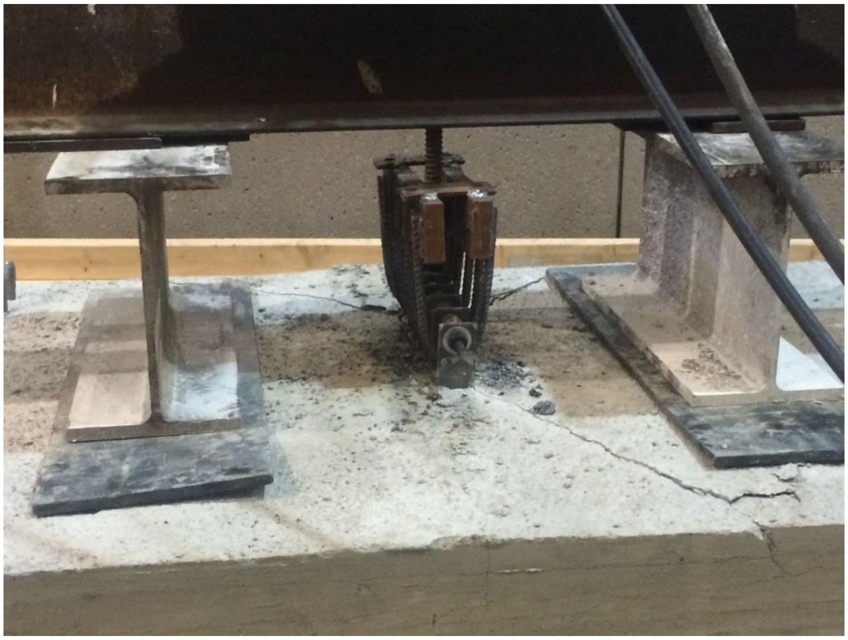


Figure 4.14: Failure mode of the first deviator

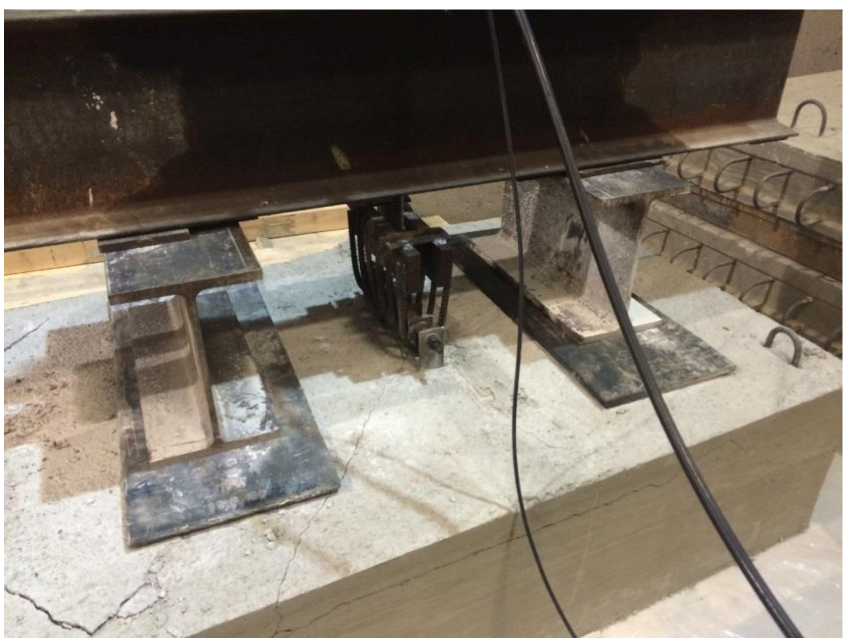


Figure 4.15: Failure mode of the second deviator

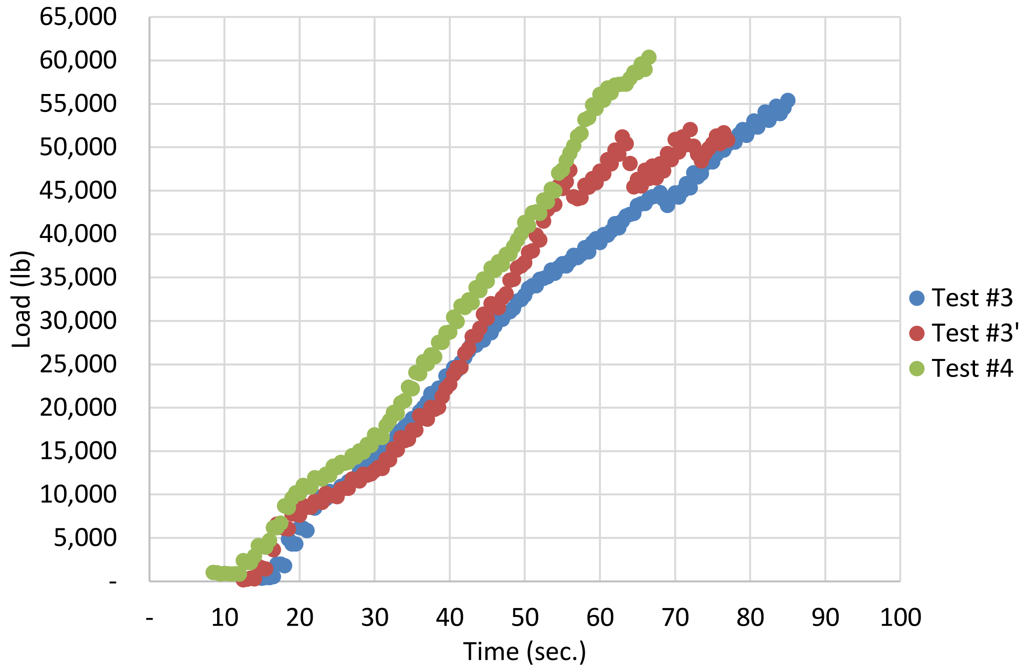


Figure 4.16: Pullout test results of the three tests

Table 4.1: Summary of all pullout test results

Detail	Test #	Test Date	Concrete Strength (ksi)	Ultimate Pullout (kip)	Average (kip)	Failure Mode
A	**1**	**11/10/2014**	7	25,015	**28,753**	Specimen splitting
	2	**11/11/2014**	8	32,490		Specimen splitting
B	**1**	**12/9/2014**	7.8	55,381	**55,929**	Shearing the rod
	2	**12/11/2014**	8	52,028		Concrete breakout
	3	**12/18/2014**	8.9	60,379		Concrete breakout

5 END PANEL TESTING

The end panels of the 2nd generation NUDECK system implemented in the Kearney East project are special panels designed and detailed to accommodate anchorage blocks for deck post-tensioning. Each end panel has five anchorage blocks (one at each girder line) as shown in Figure 5.1. Each anchorage block consists of welded steel plates and studs to transfer the post-tensioning force to the deck concrete. Figure 5.2 shows views and dimensions of the anchorage block and its plates, while Figure 5.3 shows a photo of the assembled anchorage block before installation.

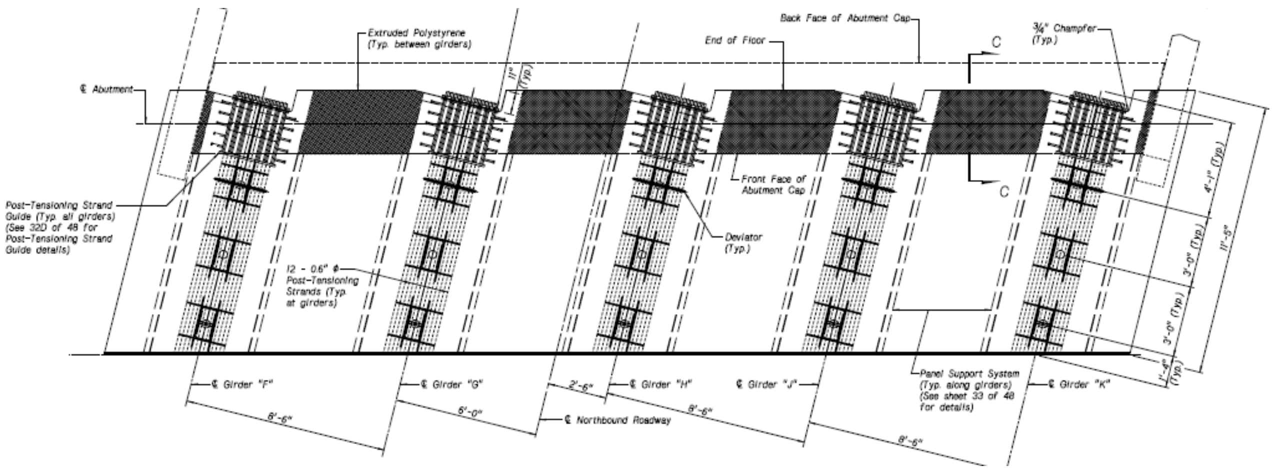


Figure 5.1: Plan view of the end panel and the five girder lines at the abutment location

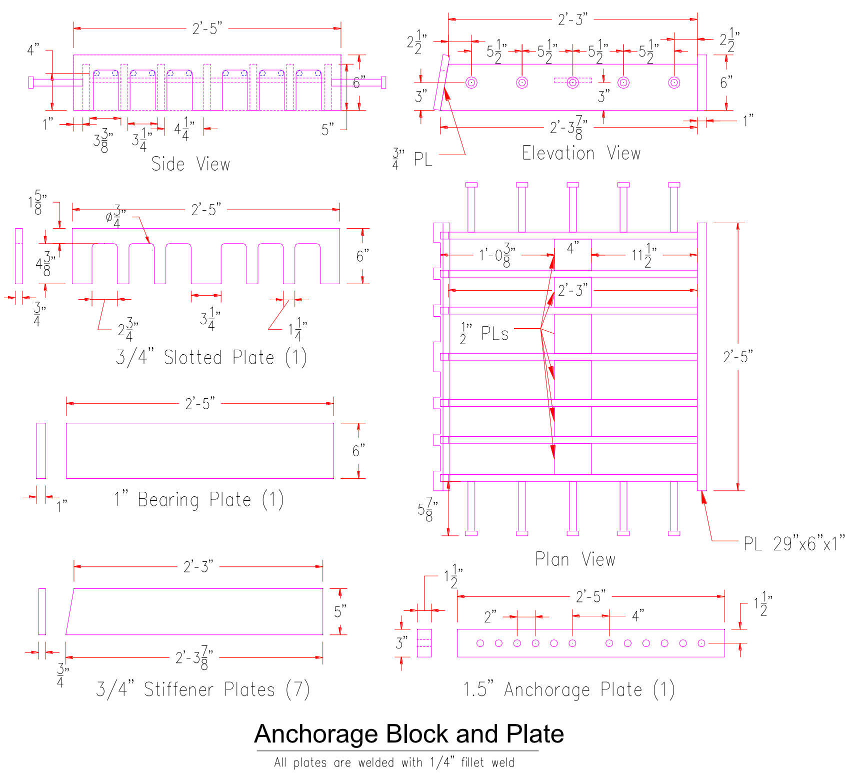


Figure 5.2: Dimensions and views of the proposed anchorage block

Figure 5.3: Photo of the anchorage block after assembly and before installation

The design of the anchorage zone of the end panel should meet the requirements of AASHTO LRFD provisions for local and general zone (AASHTO LRFD 5.10.9.2). This design was based on a refined elastic analysis using the finite elements (FE) program ANSYS R15.0. Figure 5.4 shows a 3D view of the modeled panel and the meshing scheme used in analyzing half of the panel due to the symmetry. Figure 5.5 shows the stress distribution in the anchorage zone of the concrete deck and steel anchorage block due to a horizontal force of 600 kips, which is 20% more than the horizontal component of the applied prestressing force after instantaneous losses. These stresses were considered in determining the reinforcement required to control deck cracking due to post-tensioning. Figure 5.6 show the proposed reinforcement for the anchorage zone.

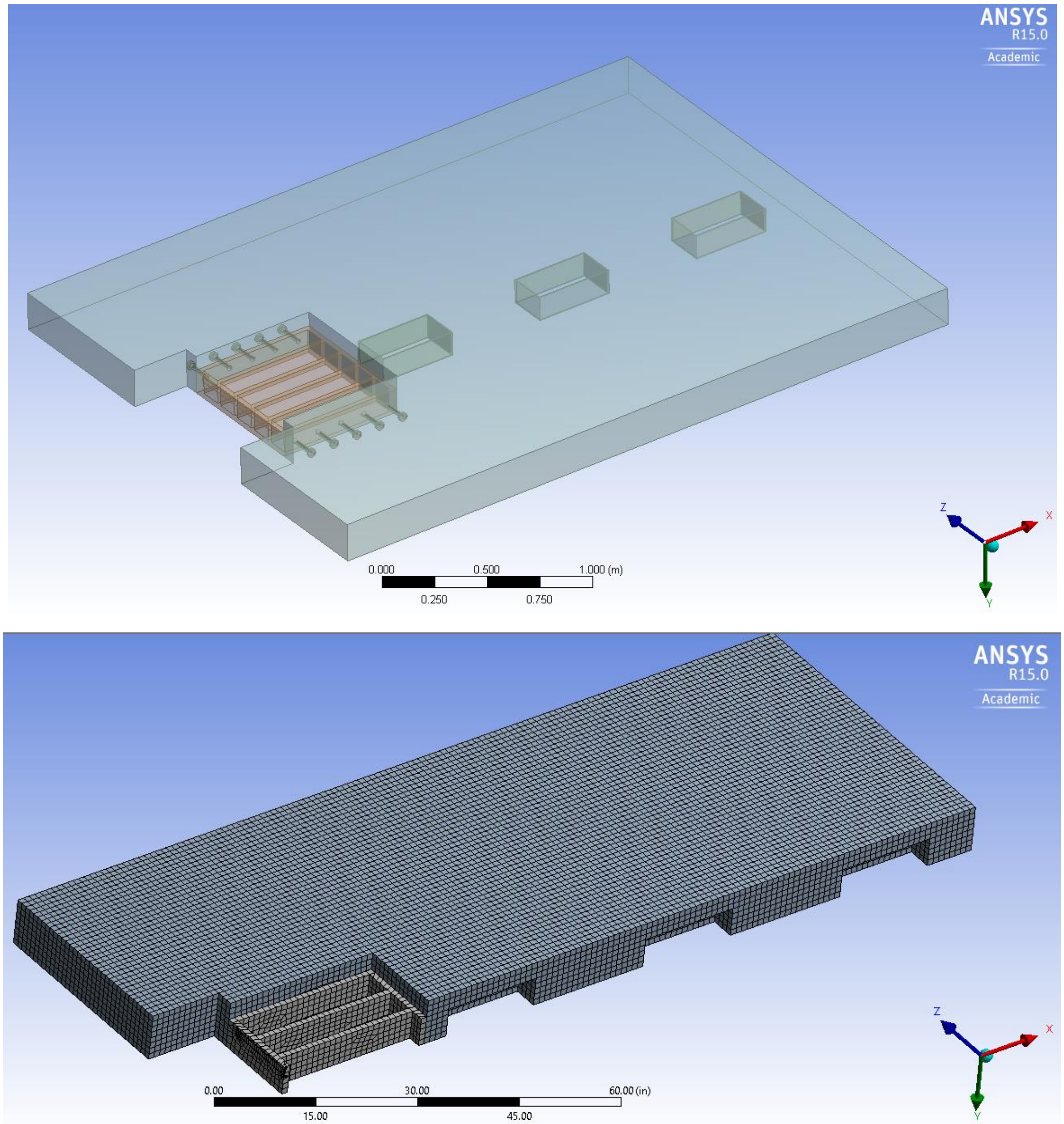


Figure 5.4: 3D views of the modeled panel and the meshing scheme

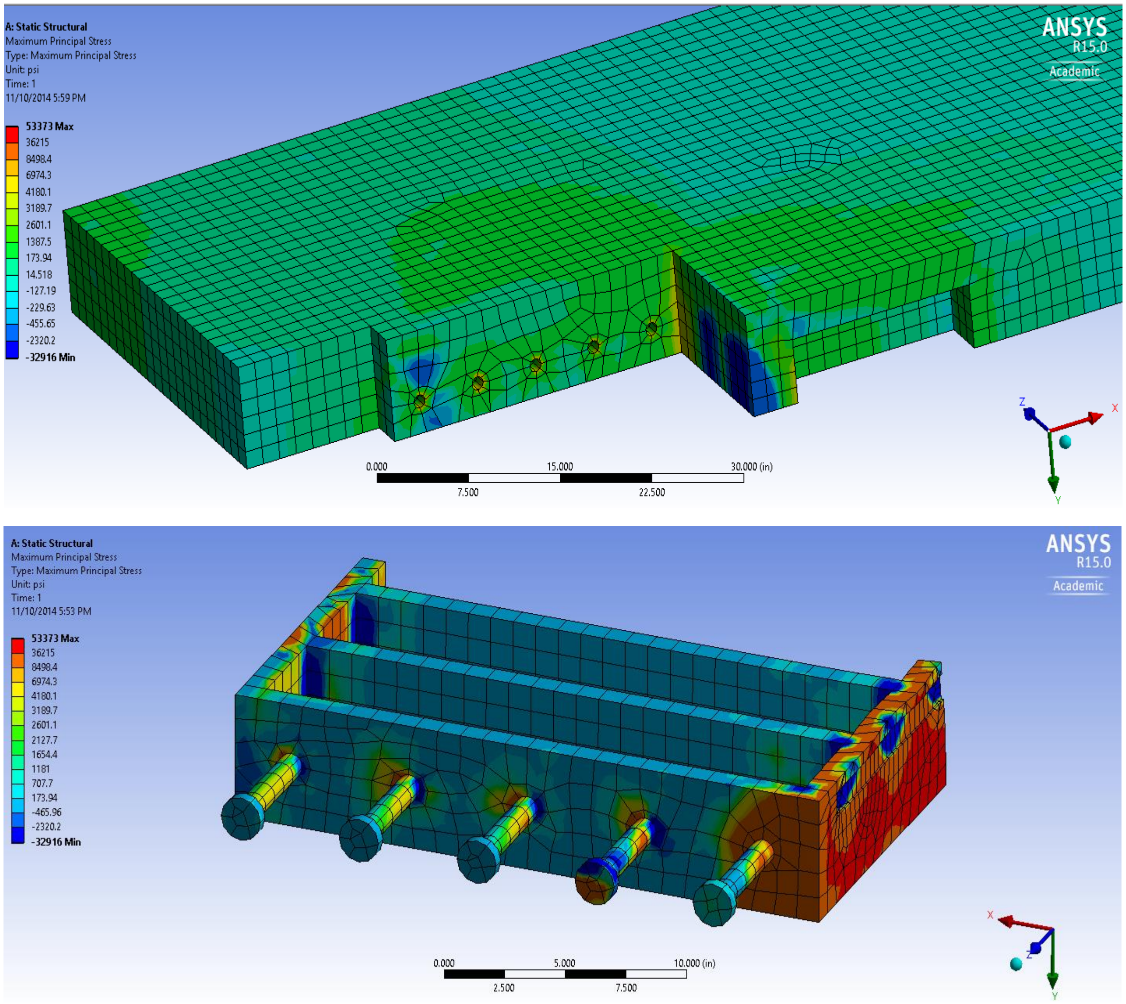


Figure 5.5: Stress distribution in the concrete deck and steel anchorage block

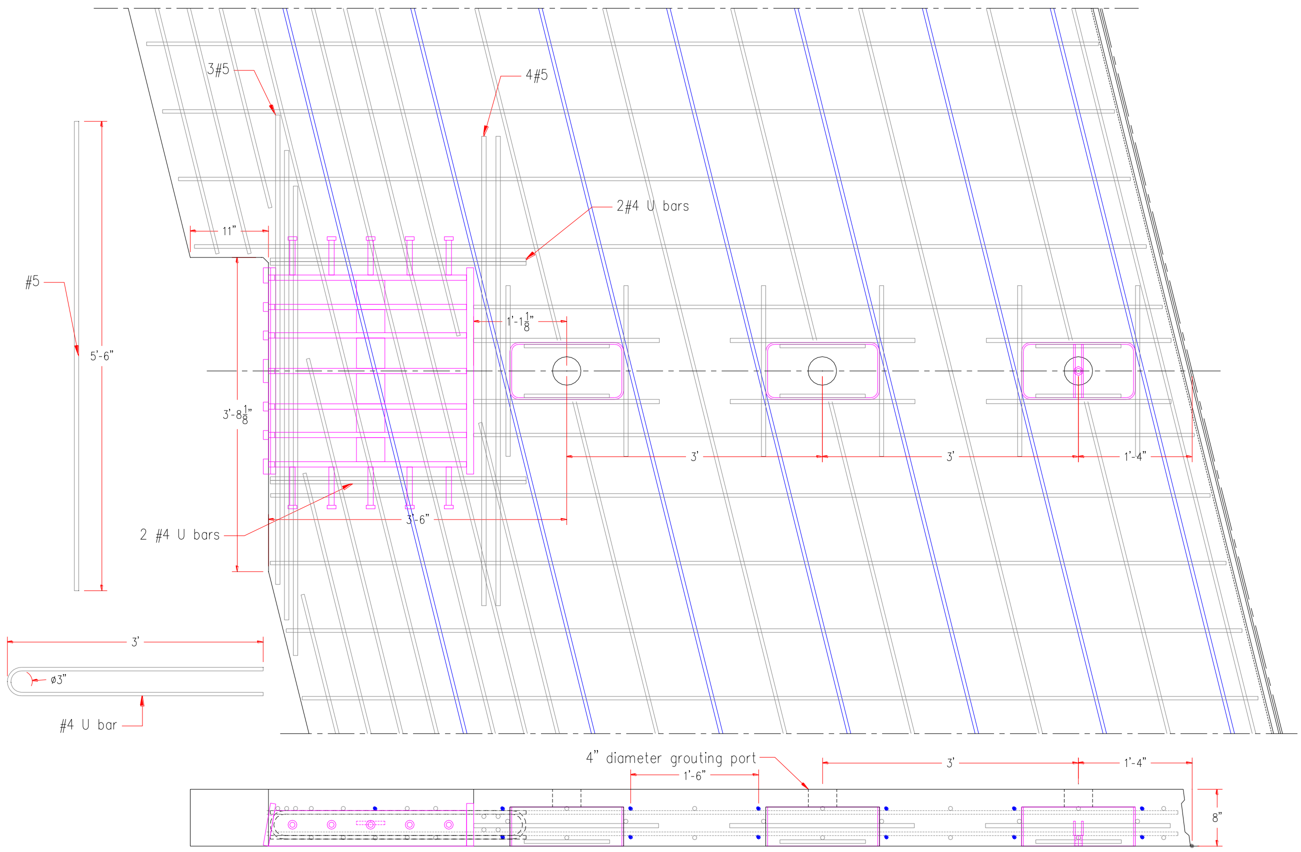


Figure 5.6: Reinforcement of the end panel and the anchorage zone

To evaluate the structural performance of the custom-made anchorage block and ensure that the provided reinforcement of the anchorage zone is adequate, a full scale specimen was produced by Coreslab Structures, Inc. in Bellevue, NE. and tested at UNL structural lab in Omaha, NE. The specimen is 8 in. thick and represents the tributary area of the deck around one anchorage block, which is 8.5' wide and 11' 6" long. The specimen was fabricated as a rectangular panel (no skew) and without transverse pretensioning to simplify fabrication and testing. Figure 5.7 shows the dimensions and reinforcement of the specimen, Figure 5.8 shows photos of specimen fabrication, and Figure 5.9 shows photo of the completed specimen during handling. To prepare the specimen for testing, several strain gauges were attached as shown in Figure 5.10 to measure the strain at the deck surface while loading. Three strain gauges were attached at the south end (transverse joint side) and three gauges were attached at the north end (anchorage block side).

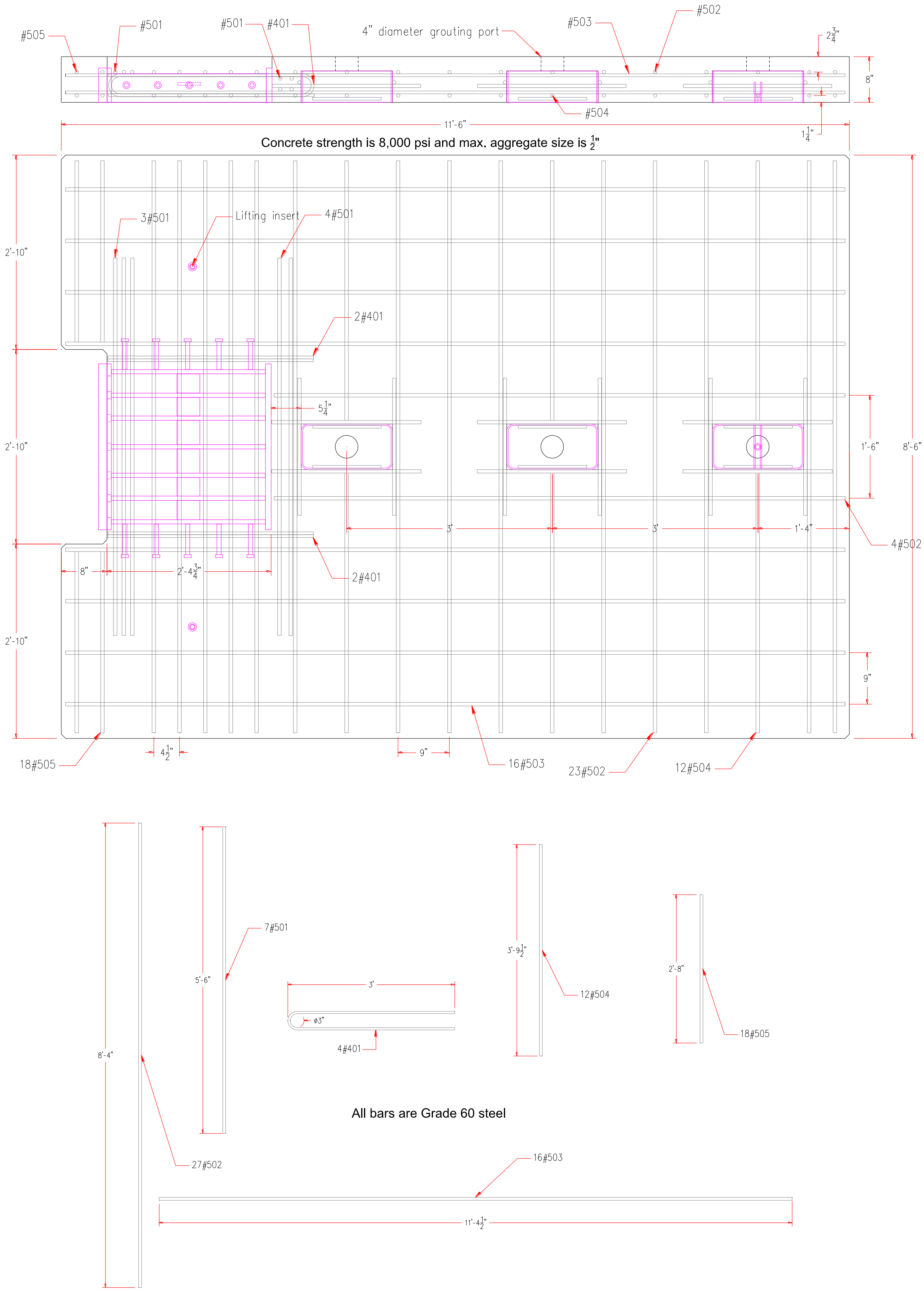


Figure 5.7: Dimensions and reinforcement of the end panel specimen

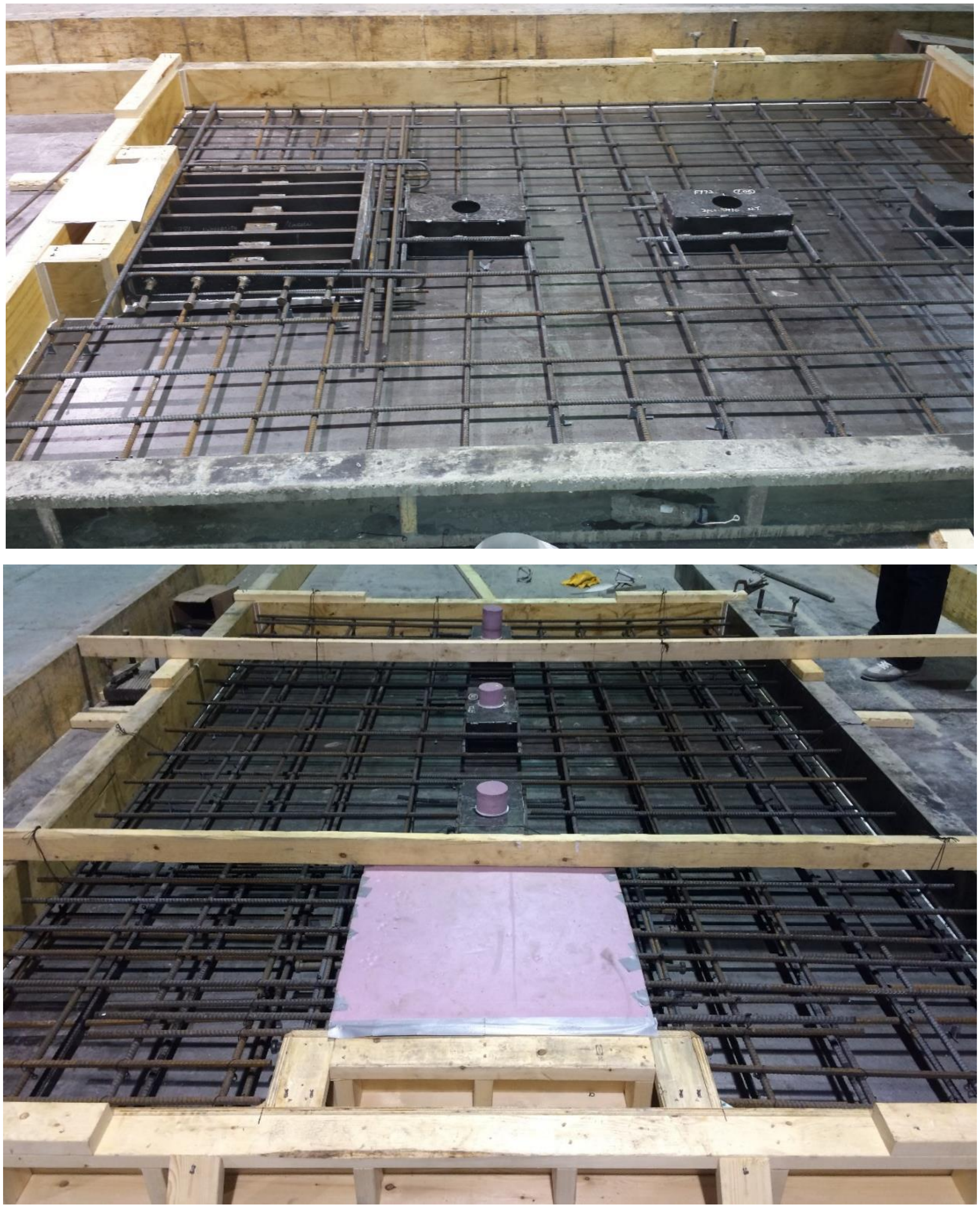


Figure 5.8: Photos of specimen fabrication

Figure 5.9: Photo of the completed specimen during handling

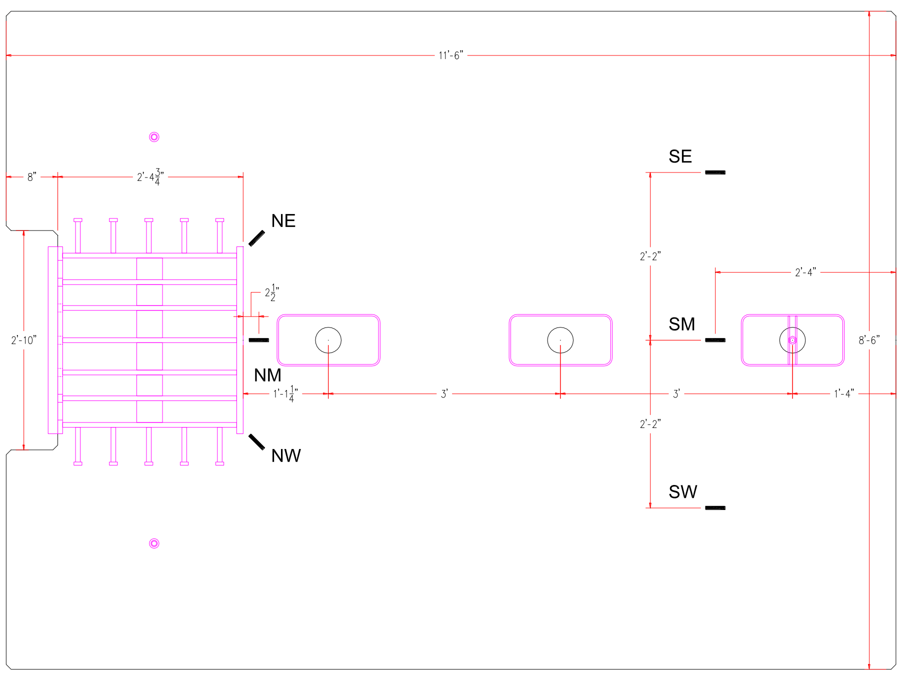


Figure 5.10: Plan view of the end panel specimen with strain gauges

Figure 5.11 shows the test setup that was initially used to load the end panel specimen up to 600 kips. In this setup, two hydraulic jacks were supported by a steel beam supported on a reaction wall, while the other end of the specimen was supported by two concrete blocks anchored to the floor. This setup did not work as the floor anchors were damaged at a load of 300 kips. Figure 5.12 shows the loading history of this incomplete test.

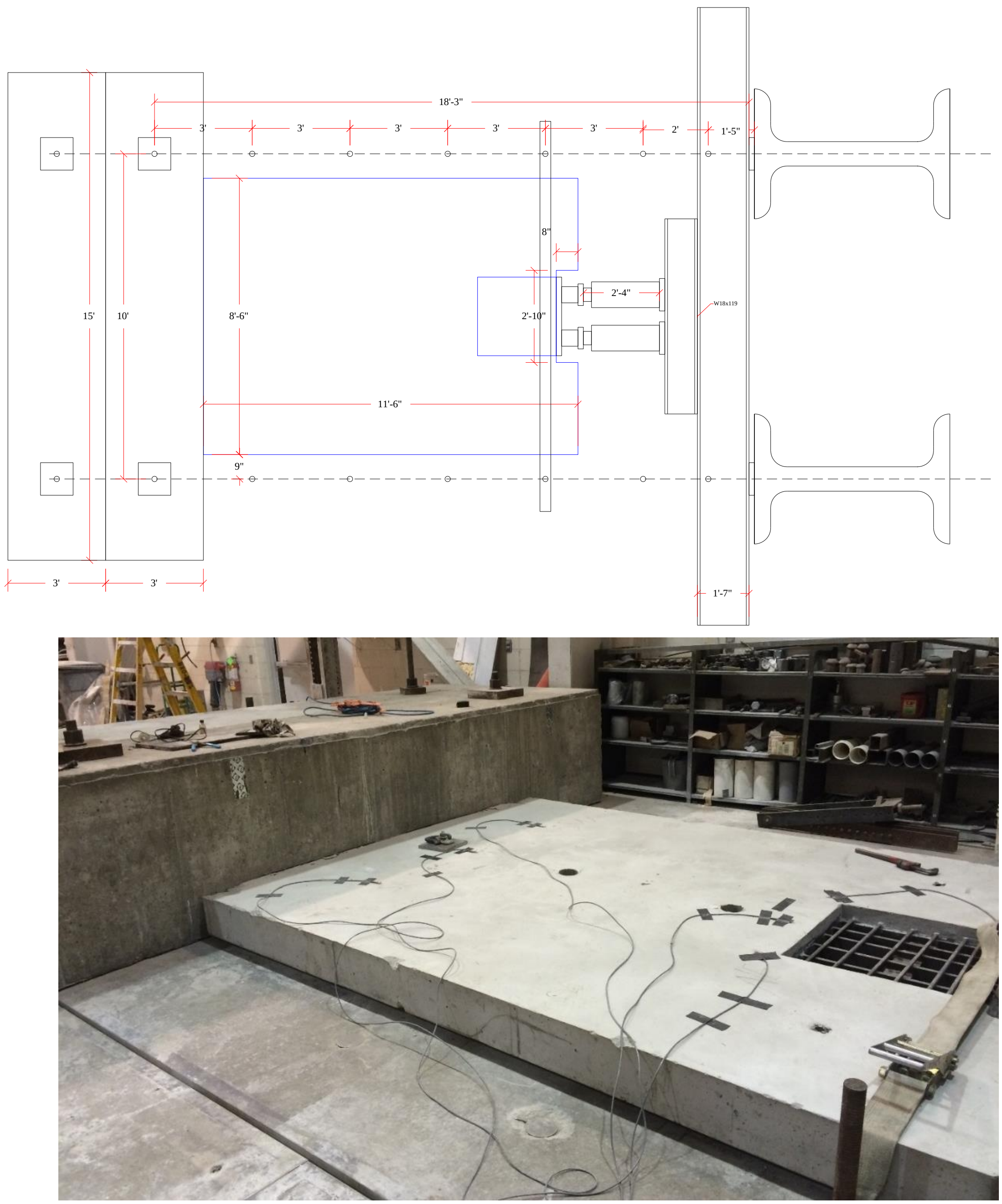


Figure 5.11: First test setup

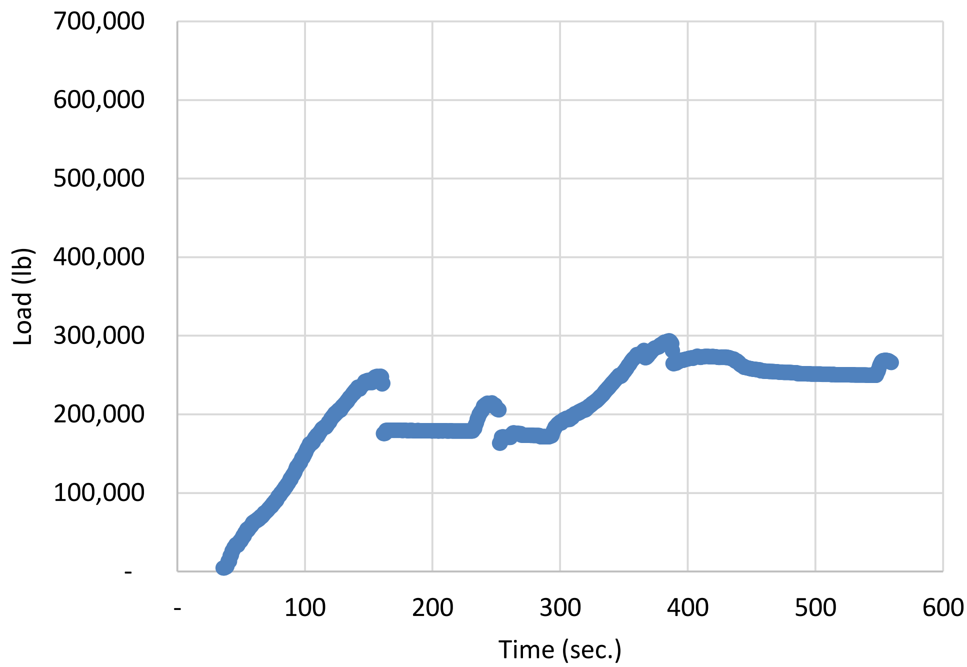


Figure 5.12: Loading history using the first test setup

Figure 5.13 shows the second test setup that was designed to avoid the problem encountered in the first test setup. In this setup, the same two hydraulic jacks were supported by one side of a horizontal steel frame, while the other end of the specimen was supported by the other side of the steel frame. Two threaded rods were used in each side of the specimen to strengthen the steel frame. This setup worked very well and the ultimate load of 600 kips was reached. Figure 5.14 shows the loading history of this completed test. Test was paused at 100 kip increments to visually inspect the specimen for cracking. No cracks were visually observed on the top surface of the anchorage zone even at 600 kip load as shown in Figure 5.15, which indicates the adequacy of the anchorage block design and proposed anchorage zone reinforcement. Figure 5.16 plots the average of three strain gauge measurements at both the south and north ends of the specimen during loading. These plots indicate the gradual increase of strain with loading and that the maximum strain is way below the ultimate compressive strain of the concrete. It should be noted that the specimen age at the time of testing was 42 days and the 28-day compressive strength of the specimen concrete was 5.45 ksi. Figure 5.17 shows views of the final end panel reinforcement and its girder connection.

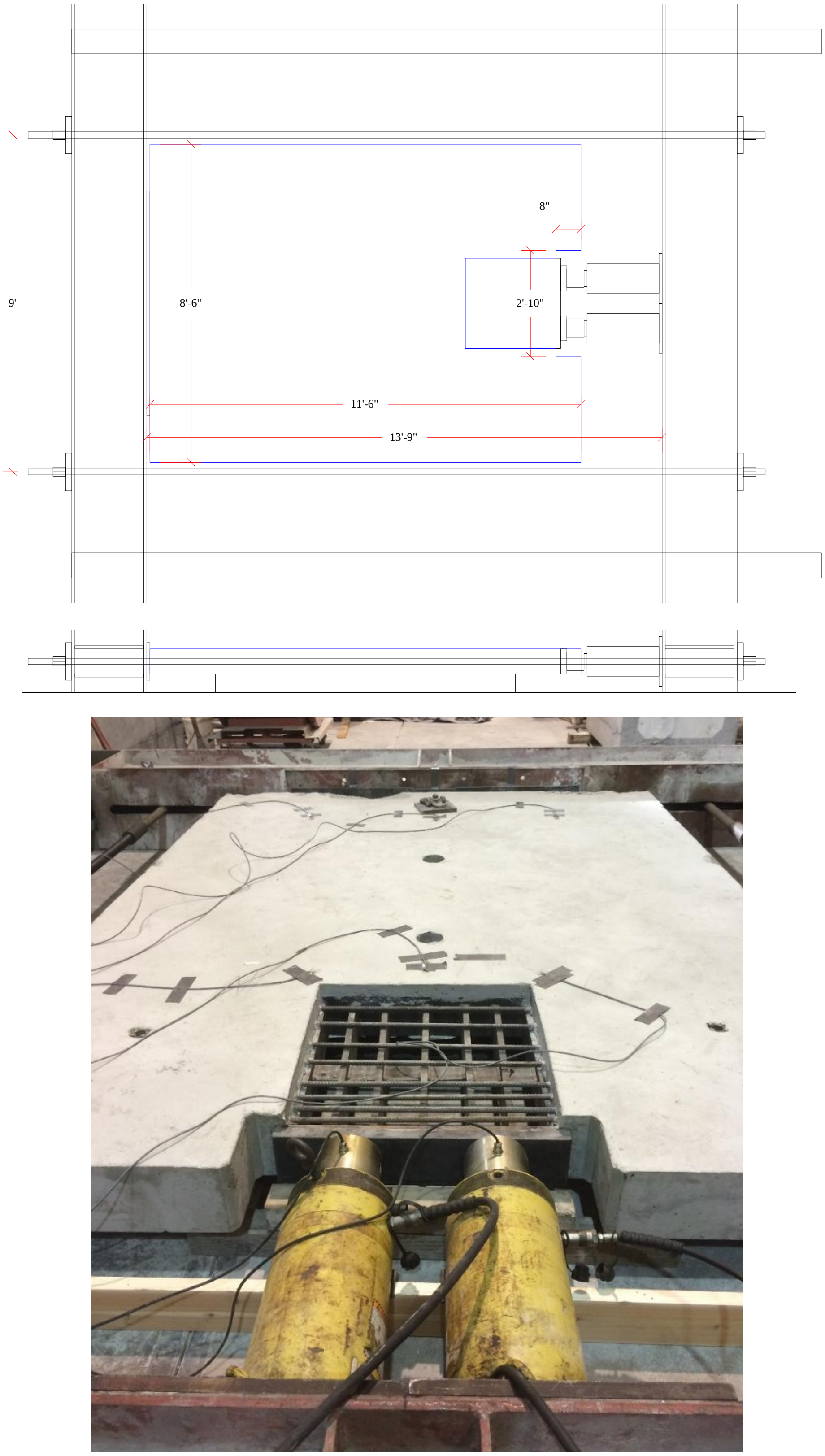


Figure 5.13: Second test setup

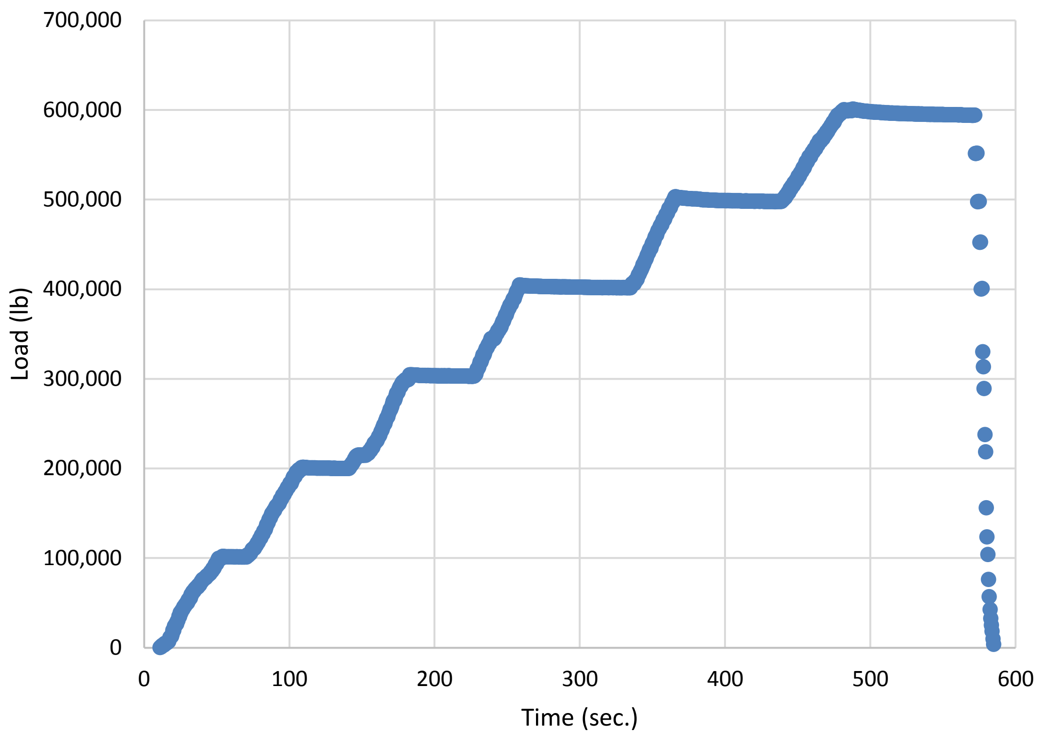


Figure 5.14: Loading history using the second test setup

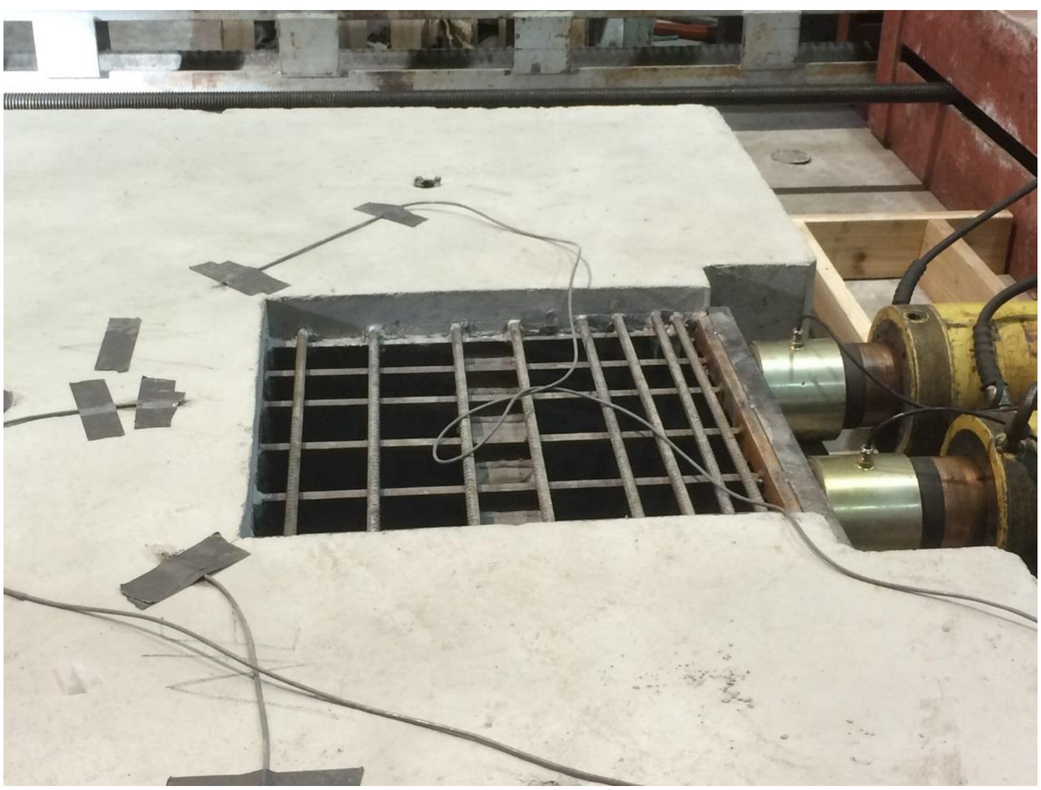


Figure 5.15: Photo of the anchorage zone when loaded at 600 kips

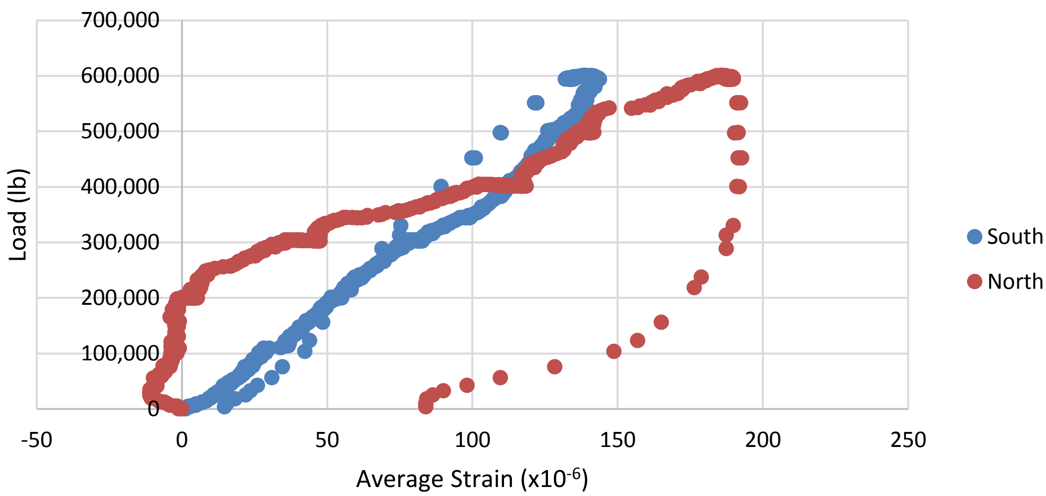


Figure 5.16: Average compressive strains at the north and south ends of the specimen

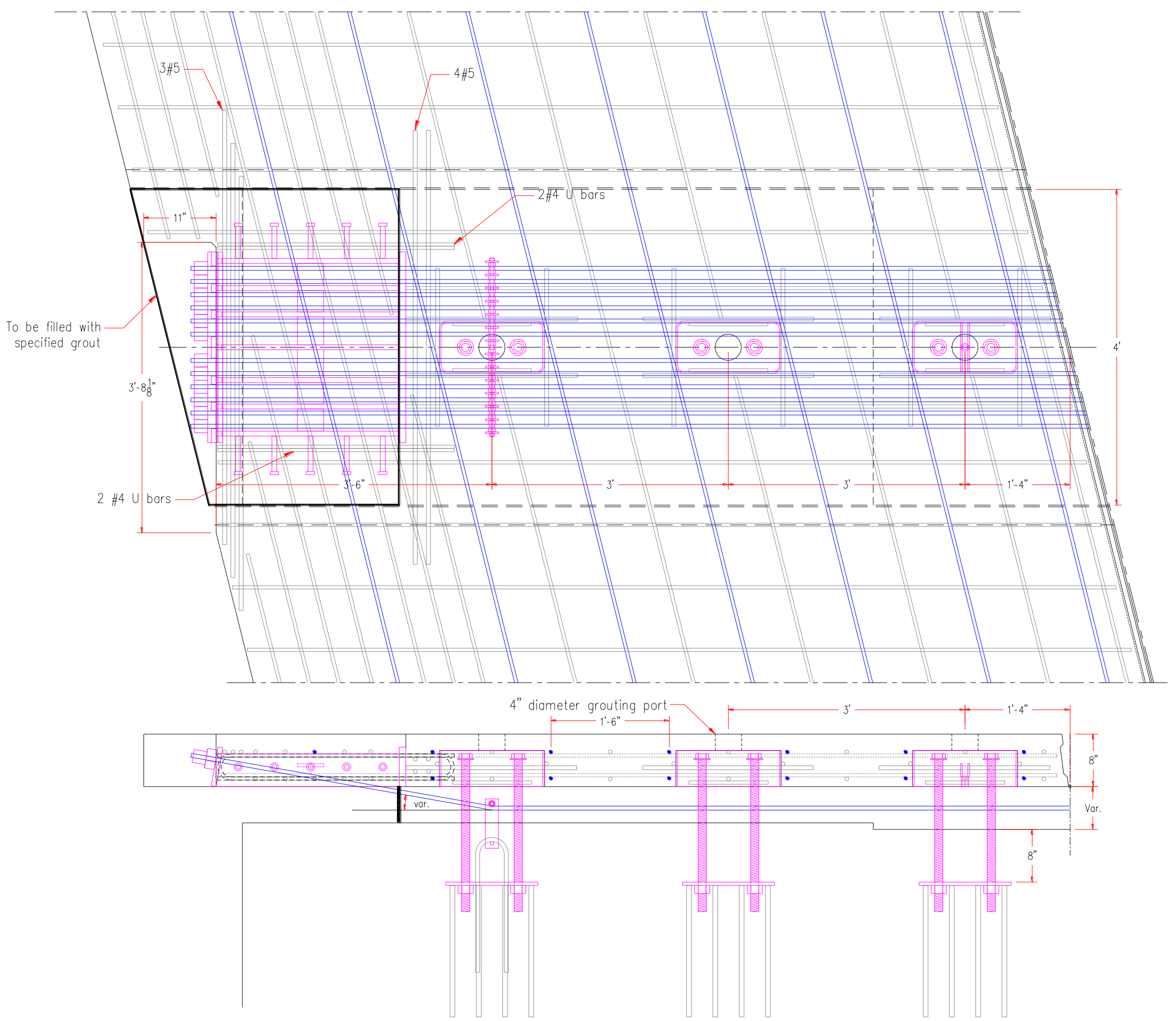


Figure 5.17: Reinforcement of end panel and its girder connection

6 CONCLUSIONS

This study is an integral part of another research project that aims at the design and detailing of a newly developed precast concrete bridge deck system (2^{nd} generation NUDECK system) for implementation in the Kearney East Bypass project. In this study, the pumpability of SCC for connecting deck panels and the supporting I-girders is investigated. Also, the special post-tensioning hardware (deviators and anchorage blocks) are produced and tested. Based on the outcomes of the several experimental investigations conducted in this study and the associated constructability experience, the following conclusions can be made:

1. A highly flowable and economical SCC mixture can be developed to fill the gap between precast concrete deck panels and bridge I-girders (haunch) as well as the shear pockets in deck panels while satisfying all workability, durability, and strength requirements.
2. Pumping SCC from one end of the bridge along the girder lines results in a significant increase in concrete pressure with distance due to the geometric complexity of volume that needs to be filled. The accumulated pressure could result in blow out of side forms or uplift of deck panels. Therefore, pumping should be restricted to short span bridges or when multiple pumping vents can be provided along the girder line.
3. Pouring SCC from 4 in. diameter holes spaced at 12 ft is a simple, efficient, and economical method for filling the haunch and shear pockets along each girder line without trapping air pockets around strands or at form corners.
4. The proposed design and detailing of the post-tensioning deviators has adequate anchorage to the girder top flange to resist a pullout force equivalent to the vertical component of the depressed post-tensioning strands with a load factor of 1.2 according to AASHTO LRFD.
5. The proposed design of the anchorage block and reinforcement of the anchorage zone are adequate to resist the bursting forces resulting from the horizontal component of the depressed post-tensioning strands with a load factor of 1.2 according to AASHTO LRFD.

7 REFERENCES

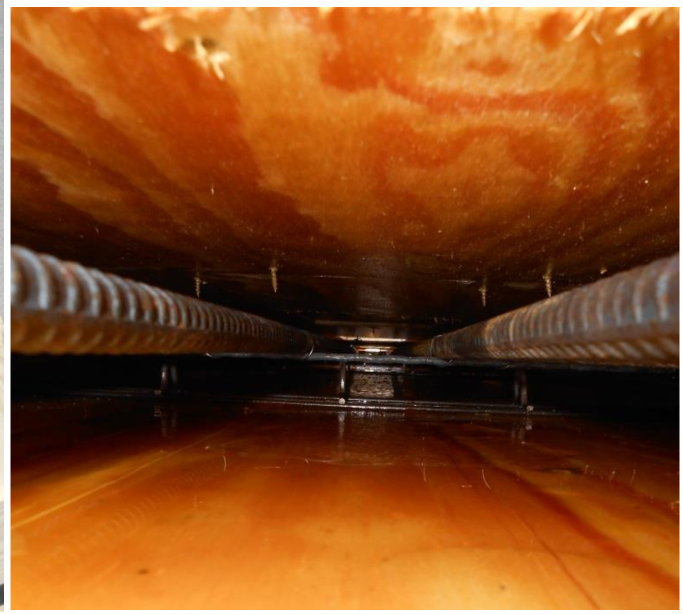
1. Morcous, G., Tadros, M. K., Hatami, A. (2013) “Implementation of Precast Concrete Deck System NUDECK (2nd Generation)” NDOR Technical Report M323, Dec.

2. Morcous, G., and Khayat, K. H. (2014) **“**Design and Performance of Self-Consolidating Concrete for Connecting Precast Concrete Deck Panels and Bridge I-Girders”,

8 APPENDICES

APPENDIX A: Pumping Mockup Tests

First Pumping Mockup Test



Second Pumping Mockup Test

Third Pumping Mockup Test

Section A)

Section B)

Section C)

Section D)

Section E)

Section F)

Section A)

Section B)

Section C)

Section D)


Section E)


Section F)


Section A)

Section B)

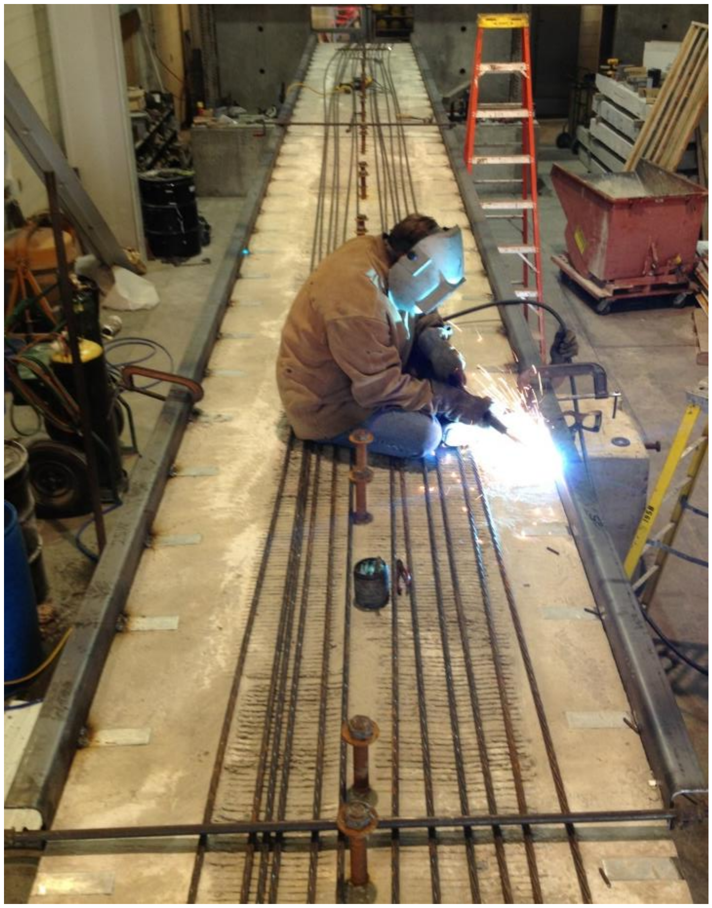
Section C)

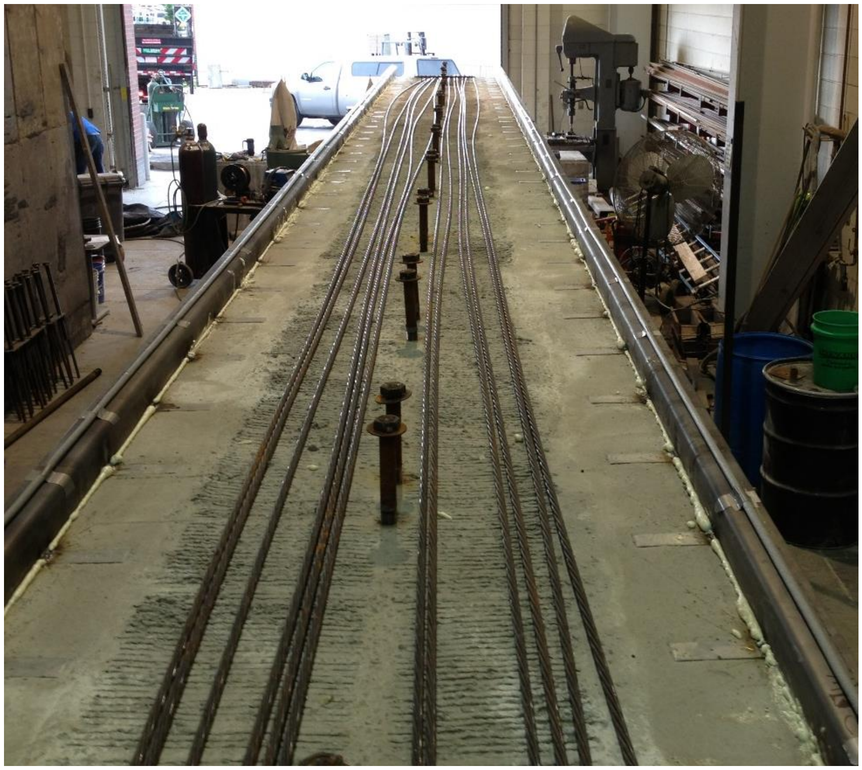
Section D)

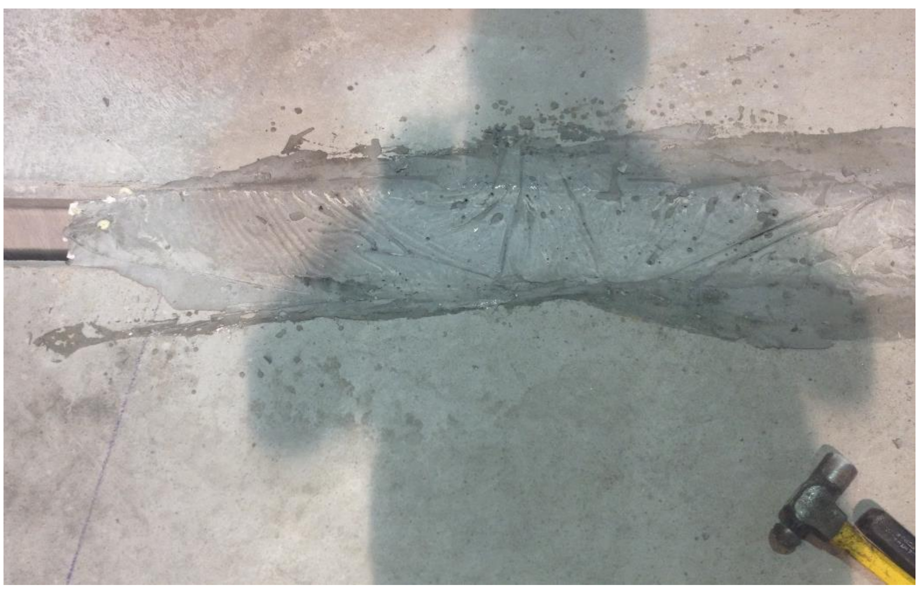
Section E)

Section F)

APPENDIX B: Fabrication of Full-Scale Specimen








F11243


#2