Full-Depth Precast Deck Panel Testing

FDOT

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Overview

As accelerated bridge construction (ABC) becomes more prominent, the need for full depth precast deck panels has become evident. The Florida Department of Transportation (FDOT) recently designed a pilot project using precast bridge elements, including precast deck panels and precast bent caps. To gain knowledge and experience about the precast deck panels, the personnel at the FDOT M. H. Ansley Structures Research Center (SRC) formed, constructed and erected two full-sized precast deck panels to match the pilot project panel size. With this innovative idea, came a lot of questions, both on the installation procedure as well as strength and performance of the panels and their connection. In addition to building these two full sized panels, the SRC cast small specimens to test the panel to panel connection. The testing provided understanding of the behavior of precast panels, the process and procedure that will be used in the field during construction of the bridge, and introduced new challenges that were not as clear during the pilot project's design phase.

I. Full Size Mock-up of Precast Deck Panels

A. Introduction

The SRC cast two 43'-1" wide (transverse direction) by 8'-0" long (longitudinal direction) by 8.5" thick panels to match the pilot project panels. The two panels were supported on beams spaced at 9'-0" with 3'-6.5" overhangs at each end of the panels. Mild reinforcement was used on the east panel, while welded wire reinforcement (with a steel area equivalent to the mild reinforcement) was used on the west panel. Further details are presented below.

Part I had several objectives including identifying reinforcement bending and congestion challenges due to the shear pockets, lifting devices, leveling inserts, and the panel-to-panel joint. The second objective was to evaluate different lifting inserts and leveling bolts and their reinforcement requirements. Once bolts and inserts were selected, the stresses in the panel due to lifting and leveling procedures were evaluated. Once the panel was installed, dead load distribution was evaluated by reading torque measurements at each bolt as well as strain readings from strain gages placed throughout the panels. The final objective was to identify challenges when grouting the haunches and shear pockets. This included evaluating forming materials and the properties of the grout material.

B. Design and Design Assumptions

The precast deck panels were cast using FDOT Concrete Class II (Bridge Deck) concrete mix with a minimum compressive strength of 4500 psi. They were designed for a dead load due to self-weight and 15 pounds per square foot (psf) future wearing surface. They were also designed for a HL-93 Live Load, per AASHTO LRFD Bridge Design Specifications, 5th Edition. Three different cases were considered during design of the panels: lifting, leveling, and final condition. Each case had different loads, load factors, and dynamic load allowance (IM) factors. The design introduced some challenges with voids throughout the panel for the shear pockets, lifting inserts, and leveling devices. Stress concentrations in these areas were accounted for and appropriately reinforced.

There were ten shear pockets in one panel, two per beam line. The shear pockets, shown in Figure 2a, were approximately a 1'-2" by 1'-0" elliptical shaped void that tapered down to a 1'-1" by 11" ellipse at the bottom of the slab. This provided a void or pocket for the reinforcement projecting out of the beams. The shear pockets were reinforced with a No. 5 rebar on each side of the pocket, top and bottom, in the transverse direction. The leveling devices also created voids and reinforcement placement challenges. The leveling device allows elevation adjustments in the field when the panels are installed. There were also ten leveling devices, two per beam line. The part of the leveling device that was cast into the slab consisted of a standard one inch diameter steel pipe, a four inch square washer and a standard nut (Refer to Figure 2b). A cone shaped void was cast on top of this device to allow a bolt to be inserted and torqued during elevation adjustment or "leveling". The layout of the leveling devices and shear pockets are shown in Figure 1.

The panels were designed using a beam line model to account for final dead load condition. The live load moment values were taken from AASHTO LRFD Appendix 4A, Deck Slab Design Table. The panel was designed for Strength I and Service I per AASHTO LRFD 5th Edition. The overhangs of the panel were designed per the FDOT Structures Design Guidelines. For the leveling and lifting conditions, a finite element thin plate model was created in LUSAS of the 8'-0" by 43'-1" panel, shown in Figures 4a and 4b. Lifting and leveling cases were designed using an 8.5 inch panel, while final condition was designed for an 8 inch panel. The half inch may be lost in planing and grooving. As mentioned above, the final condition considered dead load (self-weight and future wearing) and live load (HL-93). The leveling condition considered panel self-weight and a 20 psf live load. The lifting condition only considered panel self-weight with an IM factor of 1.5.

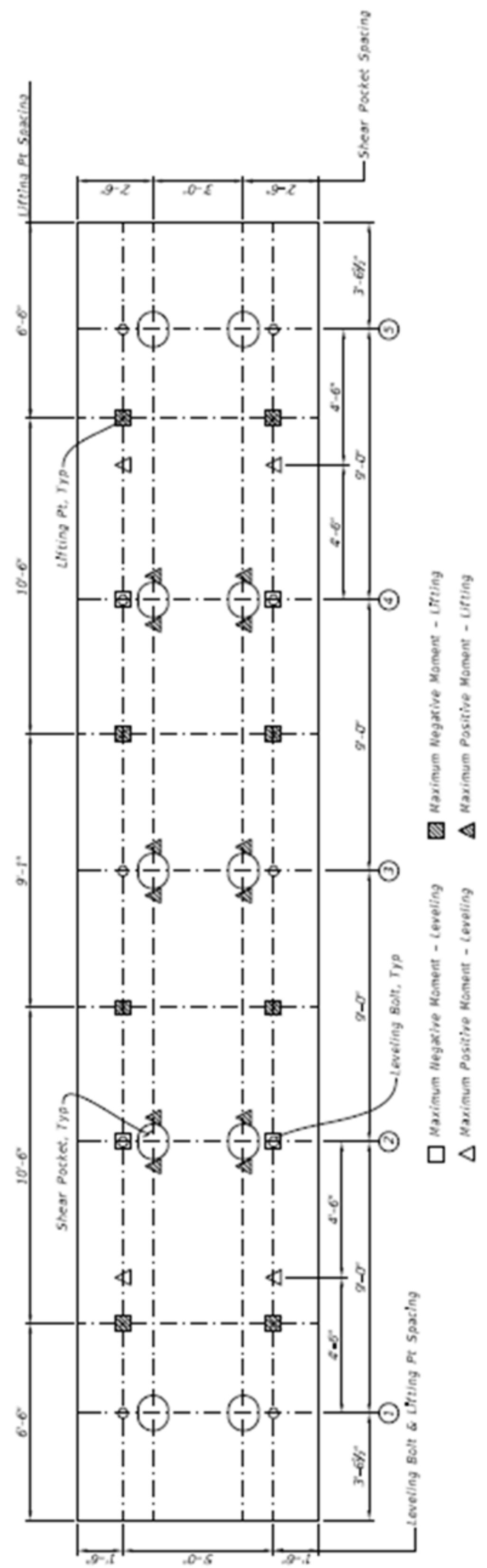


Figure 1 – Shear Pocket and Leveling Device Layout

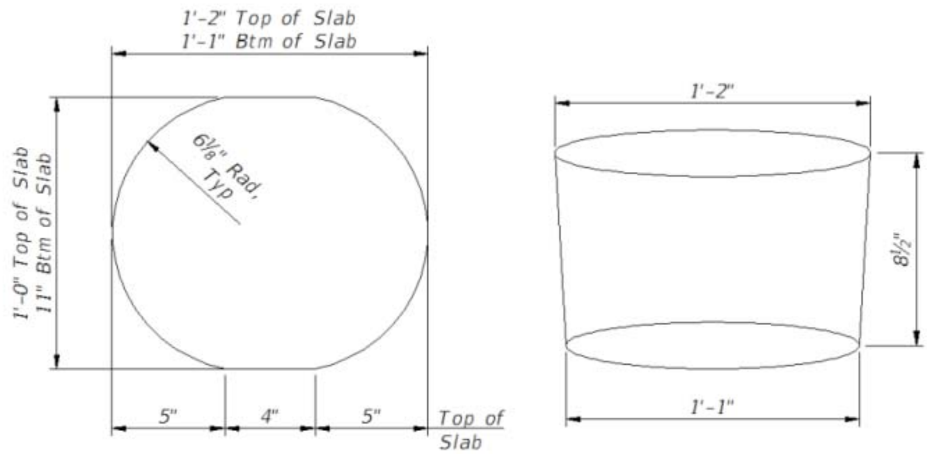


Figure 2a – Shear Pocket Details

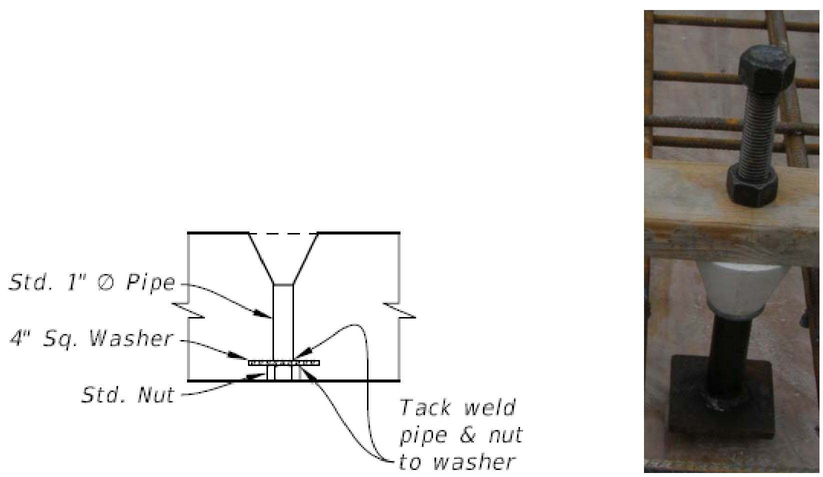


Figure 2b – Leveling Device Details

Once the analysis and design was complete, the final reinforcement required for the panel was No. 5 rebar at 6.5 inch spacing, top and bottom, in the transverse direction. The shrinkage and temperature reinforcing was No. 4 rebar at 12 inch spacing, top, in the longitudinal direction. The bottom longitudinal distribution steel was No. 4 rebar at 6 inch spacing (See Figure 3). Finally the overhang reinforcing was No. 5 rebar at 3.25 inch spacing. This reinforcement was for the final condition case. Additional steel was added in both the longitudinal and transverse direction for the lifting case. The reinforcing was placed to clear the shear pocket voids and leveling devices, while maintaining the same area of steel.

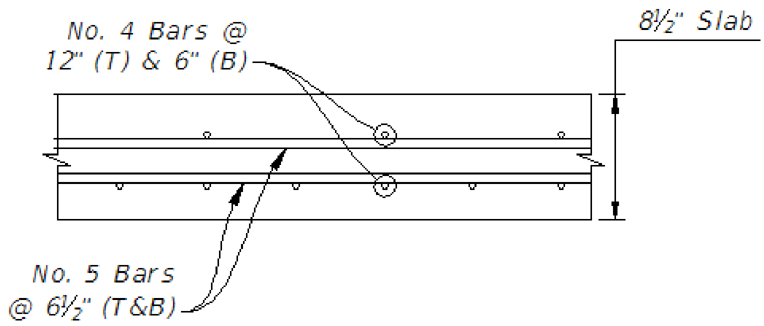


Figure 3 – Reinforcement Detail

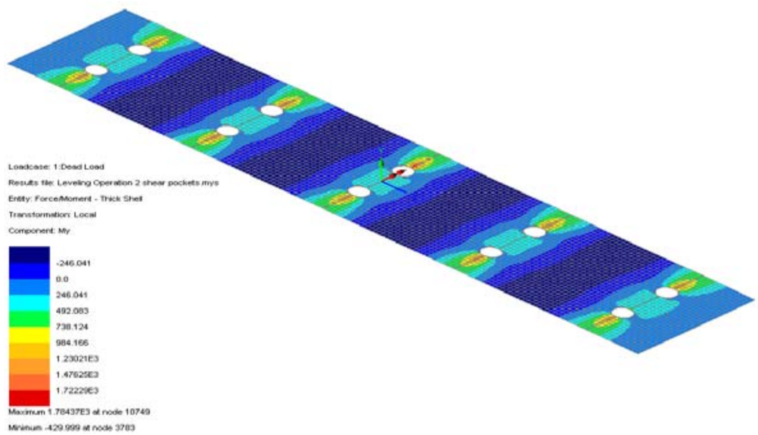


Figure 4a - FEM of Panel - Leveling Case with 10 Bolts in Contact Shown

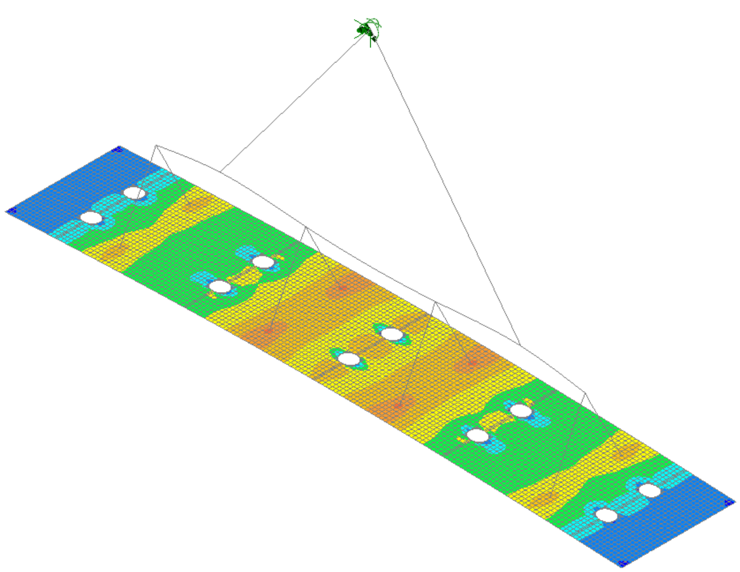


Figure 4b - FEM of Panel - Lifting Case with 8 Point Pick Support Shown

The leveling case was designed assuming only five bolts were in contact with the beams below. The lifting case with an eight point pick was designed for only six cables supporting the panel; one set of two interior cables were considered loose. This lifting condition required additional No. 4 bars to be added in the transverse direction at the top mat of the slab in the two interior spans (the negative moment locations for lifting). The steel stress for the bars in the longitudinal direction at the lifting device locations were also very high (48.6 ksi), resulting in one additional No. 4 bar at each side of each lifting device. See Appendix A, Precast Deck Reinforcement Layout for the final panel design and rebar layout.

The final component was the design of the panel-to-panel transverse joint connection. The design involved splicing two No. 4 hoop bars (Bar 4D) that projected out of each panel. The hoop bars were spaced at 6 inches, except at the shear pocket locations where they were 18 inches apart. To create the standard 180 degree hook, the longitudinal steel was placed at the top of the top mat and the bottom of the bottom mat of steel. The design of this connection was based on ensuring the same flexural capacity as provided by the distribution steel. The detail of the panel to panel transverse joint connection is shown in Figure 5. Small scale tests were performed to evaluate this connection. The results of the small scale testing as well as further discussion of this connection can be found in Part II of this report.

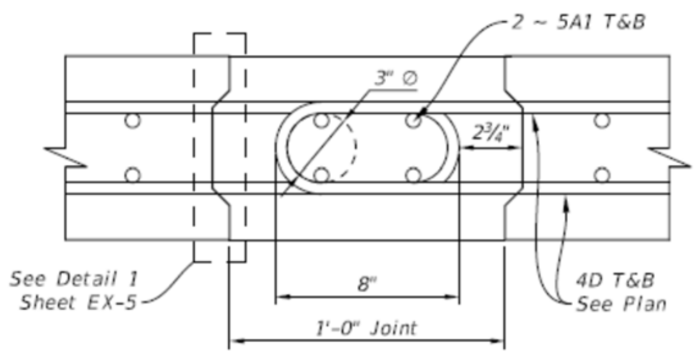


Figure 5 – Panel-to-Panel Joint Detail

C. Materials

The same concrete class used for a cast in place deck was specified for the precast deck panels, Class II (Bridge Deck) with a minimum compressive strength of 4500 psi. See Section G Testing for actual concrete strengths of each panel. The reinforcing for the panels with mild reinforcement was ASTM A615, Grade 60. The welded wire reinforcement was ASTM A496, Grade 70. The forming material, leveling and lifting devices, and grouting materials were evaluated based on strength (capacity), serviceability, availability and cost.

The haunch forming material was selected based on its compressibility and its ability to rebound, during the leveling operation. The two materials evaluated were polyethylene and polystyrene.

The polystyrene material did not rebound well, so only polyethylene was used. The polyethylene forming material is shown in Figure 6. Two products were investigated, Liquid Nails and Great Pro Wall, to determine how the polyethylene would be held down to prevent it from sliding off when the panels were being installed. Liquid Nails (or a generic equivalent) was used.

Figure 6 - Plan view of concrete beams with haunch forming material and shear reinforcement (Left); Side view of haunch forming material once panel is set (right).

The lifting device was selected based on capacity and minimum edge distance requirements. There were eight pick points, with a conservative assumption that only six cables would be in tension. The minimum compressive strength of the panels during lifting was specified as 3500 psi. The maximum calculated load one insert had to support was 11.54 kips. As part of the testing, two lifting cables were lengthened to simulate them becoming loose in the field. This will be discussed in further detail in the Testing section of this report. The lifting device used for the panels was the Dayton Superior P-94-S Fleet Lift S-Anchor, FL515 with a capacity of 12 kips and a minimum edge distance required of 30 inches (The panels only provided 18 inches from the lifting device to the edge). See Appendix B for Manufacturer's Specifications for the selected lifting device.

The leveling device, shown in Figure 2b, was made up of a steel pipe, a square washer, a nut and bolt. The bolts were designed assuming only five bolts were in contact with the beams below and carrying the entire load. The two bolt type's selected based on capacity and availability were an A193 B7 bolt or F1554 Grade 55 bolt. The bolts were one inch in diameter, to fit into the pipe, washer and nut. The bolts were 14 inches long to pass through the full thickness of the panel (8.5 inches) and haunch (4 inches maximum) and project out enough to still be able to turn. The bolts also needed machining (or a bolt cap) to create a rounded tip for bearing. Steel bearing plates, 6x6x1/4, were embedded into the top of the beams for the bolts to bear on.

D. Test Description & Setup

The SRC casted five concrete beams at a nine foot spacing. The three interior beams were 2.5 feet wide by 17 feet long (to accommodate two 8'-0" panels and a 12 inch closure joint). The outer two beams were 4 feet wide by 17 feet long, representative of the 4 foot top flange of the FIBs. All of the beams were sloped 2% in the transverse direction (43'-1" direction). The pilot project will have a 2% cross slope with the beam top flanges level. To recreate this slope, while the panels remain functional for the SRC, the beams were sloped and the panels remain level. The panels sloped down 1% longitudinally. It should be noted that the beams in SRC did not account for deflection as they were fully supported. See Figure 7 below.

The beams for the pilot project have two bars with a 180 degree hook spaced every three feet, which provide the shear interface connection between the panel and the beam. These hook bars fit into the shear pockets. The shear pockets are then filled with grout that will flow into the haunches, closing the gap between top of beam and bottom of panel. To emulate that condition, the SRC was able to drill and epoxy two bars with a 180 degree bend projecting out 6.75 inches (see Figure 6, Left). This would account for a maximum 4 inch haunch and a minimum of 2.75 inch embedment into the shear pocket. The concrete beams also differed in height, each creating a different haunch thickness. The haunch thicknesses ranged from a minimum of 0.5 inch to a maximum of 4 inches, similar to the pilot project plans. The objective was to determine if the grout would flow into these smaller haunch areas that are expected in the field due to tolerances and differential beam camber.

Figure 7 - Concrete Beams

The east panel was formed on an existing slab. Due to weather constraints, the SRC tied the steel for each half of the panel indoors and then brought out each half and laid it on the concrete and plywood forms. Splice bars were then added at the midspan of the panel as shown in Figure 9. The shear pocket voids were formed using an expanded polystyrene material. The goal was for the void forming material to maintain position during concrete placement without rigidly connecting it to the exterior formwork. The hoop bars were then tied to each longitudinal bar on the joint side of the panel. There were no real issues as far as bar congestion or bar placement, except for time. It took about five man days to tie all the steel and place it. The panel was cast with three edges flat and the west transverse edge with a 1.5 inch female keyway. The keyway detail is shown below in Figure 8.

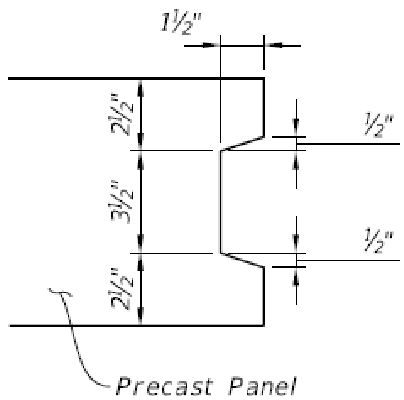


Figure 8 – Keyway Detail

The west panel was cast on top of the east panel. The SRC used welded wire reinforcement sheets comprised of 6.5x6 - D30.7/D19.7 with some additional mild reinforcement, to meet the design and spacing requirements described above. This was very similar to the east panel in that they placed each half and then added splice bars at the midspan of the panel. The hoop bars were tied to the longitudinal steel on the east end of the panel. Again congestion issues did not arise. This method was not as time consuming. Cutting, placing the sheets, and tying additional bars took about 1.5 man days. The shear pocket voids were formed with the same materials as described above.

Figure 9 - Splice at midspan of panels. East panel with mild reinforcement (left), West panel with WWR (right).

Figure 10 - The hoop bars (4D) in form.

The test setup included placing strain gages on the top of the panels at critical moment locations for two design cases: lifting and leveling. The final condition case was not within the scope of this testing. Gages were located six inches away from each leveling device and each lifting device, the negative moment (or tension) regions for each case. Additional gages were placed between each shear pocket and between each lifting device. The panel had two strain gages embedded in the panel on two transverse top bars beside the lifting devices. There were a total number of 32 strain gages including the two embedded gages. This setup was for the west panel, or Panel 2 (the second panel cast, but the first to be installed). The east panel, or Panel 1 (the first panel cast, the second to be installed), had a similar strain gage arrangement, however there were no embedded gages on the reinforcement. Panel 1 had a total of 30 gages, placed on the top of the panel in the same locations as Panel 2. See Appendix A for instrumentation plan.

The testing procedure involved casting two panels as discussed above and per the plans in Appendix A (Sheets EX-1 thru EX-4). Once the panels were cast and reached the appropriate strength, each panel was lifted at the pick points shown on the plans, and the strains were recorded. The panel was placed on the beams, with the leveling bolts preset to the anticipated heights. For this testing, the bolts were set 0.5 inch lower to prevent damaging the haunch material until final installation. Once the lifting cables lost tension (i.e. the leveling bolts were bearing on the beams), the leveling bolts were hand checked to determine how many bolts were in contact with the beam below. Initial torque readings were recorded. The leveling bolts were then adjusted to bring the panels to the proper predetermined elevations. The bolts were again torqued to bring the torque readings to approximately the same value within 20%. The panel was then lifted again (referred to in this report as the "return lift") and brought back to its original position. This same procedure was repeated five times. On the fifth cycle, the panels were set on the beams for their final position. The west panel was tested and installed on May 10, 2012 and the east panel on May 24, 2012.

E. Shear Pocket & Haunch Grout

The use of precast panels required the use of grout to fill the shear pockets, haunches, and to fill any voids created from the lifting devices and leveling bolts. For the testing in the SRC, the focus was solely on the shear pockets and haunches. This introduced the need for a precision grout, a type of non-shrink grout which provides extended working times and performs well for a fluid mix. These attributes were necessary to ensure proper grouting between the top of beam and bottom of precast deck panels and to ensure a proper connection at the shear pockets. The precision grout will also experience reduced shrinkage, reducing the likelihood of openings forming at the joints. The grout selected was Masterflow 928 by BASF, a high-precision mineral-aggregate grout. The minimum required compressive strength for the grout was 6750 psi.

The suggested grout sequence involved covering or restraining the shear pockets during grout placement to prevent upward expansion of the grout. For the two panels, restraining the shear pockets seemed achievable, however the pilot project had approximately 110 shear pockets per span. The grout sequence for the two panels was revised so that the shear pockets were not covered while grouting, but overflow was contained or controlled. The revised grouting sequence entailed placing vent tubes through the haunch forming material at low and high points of each beam haunch at the edge of the precast panel. Recall the panels were sloping 1% in the beam direction. Grout was poured into the lower shear pocket to begin filling the haunch. As grout began to fill the upper shear pocket, the pouring operation is stopped. The grout was then poured into the upper shear pocket to complete grouting of the haunch. The grouting operation continued until grout steadily exited the vent tubes at the high side, avoiding overflow of the lower and upper shear pockets. Details of grouting the shear pockets and haunches for the two panels in the SRC will be discussed further in the Testing section of this report.

To further understand the benefits of restraining the shear pockets during grouting, a small side test (known as Push-out Test for this Report) was performed. The objective of this Push-out Test was to determine the benefit of restraining the shear pockets while grout is curing versus no restraint. There were three unconfined specimens and three confined specimens.

The test involved pouring a similar volume of grout in a pipe having approximately the same area as the actual shear pocket and filled to the same depth. Then a load was applied concentrically to the specimen. The pipes used were round HSS16.000x0.500 and cut in approximately eight inch sections (range of 7.75" to 8.0625"). The pipe material was rigid and provided a consistent coefficient of friction on the contact surface. All specimens were filled completely with grout. The unconfined specimens were cured with a plastic wrap. Curing the unconfined specimens with plastic attempted to keep the moisture from escaping to keep shrinkage down, avoiding the grout pulling away from the inner edge of the pipe. The confined specimens were covered with a steel plate that was clamped down. Both specimen types are shown in Figure 11.

Figure 11 - Confined Specimens (Top) and Unconfined Specimens (Bottom)

Figure 12 - Push-out Test Setup

The grout was seven days old on the day of testing and had achieved a compressive strength above 8000 psi. The test setup can be seen in Figure 12. A 500 kip Universal Testing Machine (UTM) manufactured by MTS was used in the lab to apply the load. Because the eight inch tall pipe sections were completely filled, the reaction was transferred through steel pedestals underneath angle sections that had been welded to the outside wall of the HSS. The load application was transferred to the grout via a three inch thick steel, circular plate of diameter close to that of the inner diameter of the HSS pipe. Data was recorded at 10 Hz during testing at a target load rate of 100 lb/s. Four displacement transducers were on the upper load platen (one on each corner) to record downward displacement of the grout infill. Unconfined Test 1, 2, and 3 had a 3, 1.5, and 2.7 kip preload, respectively. Confined Test 1, 2, and 3 had a 2, 3.2, and 3.7 kip preload, respectively. The test set up allowed for 1.875 inches of vertical displacement of the grout plug before coming into contact with the lower platen.

The results of the test can be seen in the graph on Figure 13. There is no consistent evidence that restraining (Confined) the shear pocket prevents upward expansion. Nor is it clear that unconfined shear pockets will not expand upward.

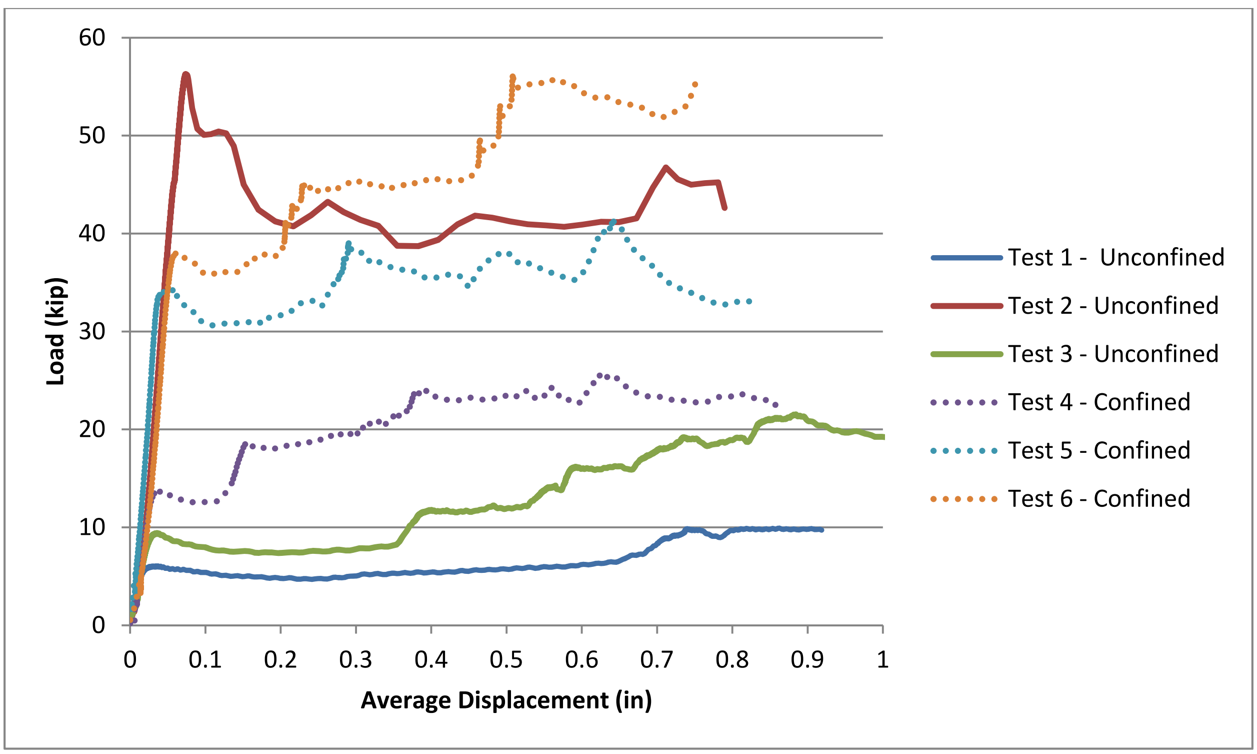


Figure 13 - Load-Displacement Graph for Push-Out Test

F. Construction Sequence

The following construction sequence was based on the Precast/Prestressed Concrete Institute (PCI) *Full Depth Deck Panels Guidelines for Accelerated Bridge Deck Replacement or Construction, Second Edition*, with some minor modifications to fit this specific panel design and testing needs. The sequence is as follows:

1. Cast the panels (addressed in detail in the sections above)
2. Clean surfaces of shear keys
3. Preset the leveling bolts to anticipated height
4. Form haunches between the top of the existing beams and the bottom of the deck panel
5. Erect panel
6. Adjust leveling devices on deck panels to bring panels to grade
7. Torque all leveling bolts to approximately the same value (within 20%)
8. Pre-wet specified shear pockets. Grout all haunches and shear pockets with non-shrink grout
9. Remove leveling bolts after haunches have been grouted and cured
10. Grout block outs after removal of bolts
11. Install reinforcement, apply epoxy, and pour transverse closure joints
12. Remove the entire haunch forming material to allow for inspection of grouted haunch

The construction sequence above provided a general guideline from which to start. Once the panels were installed and grouted in the SRC, certain aspects of the sequence were revised due to labor intensity and time.

G. Testing

Several tasks were accomplished before the panels were installed including ordering torque wrenches, renting two cranes and having dunnage available. Prior to the test date, the elevations of the embedded plates in the beams were recorded. The panel was pressure washed and all dust and water removed from all voids. The forming material was precut and installed on the beams (Figure 6). All of the preset leveling bolt elevations were determined. Two different sized shackles, used for the lifting straps to replicate a loose cable, had to be on hand. And finally all leveling bolts were installed in the leveling device assembly and had to be free to turn.

Day 1 - Installation of Precast Deck Panel 2

Prior to the crane arrival, the data acquisition (DAQ) box and cables were connected to the center of the panel using two Tapcon concrete screws for the box and tape for the cables. The cabling was laid out to not interfere with lifting. The 19 day concrete compressive strength of the panel was about 6200 psi. The testing was divided into three parts: Lifting, Leveling, and Return Lift. These three parts make up one sequence, there were four full sequences. The fifth sequence only included Lifting and Leveling, this was when the panel was installed in its final position (the fifth sequence is equivalent to the process for the pilot project). The initial top surface temperature of the panel was taken at 9:38 am, about 30 minutes prior to testing and it averaged 72.3 degrees F. The ambient temperature reached a high of 89.1 degrees F and there was some rain the night before or early that morning.

The test started at 10:04 am on May 10, 2012. The first lifting sequence followed the plans (see Appendix A) with all cables engaged and all ten leveling bolts preset to make contact. The lifting assembly included a spreader beam with two vertical cables above connected directly to the cranes, and four cables below, that looped around to create eight pick points. See Figure 14 below. The crane lowered the spreader beam to connect the shackles to the lifting straps. Once the shackles were connected, the DAQ was started for Lifting. The crane began lifting the spreader beam removing slack in the straps. The straps and shackles were inspected to verify proper installation. The panel was lifted a few inches and the leveling bolts were preset to the predetermined heights.

The cranes lowered the panel onto the beams. Once the panel was set, the tension in the cables was released without disconnecting the rigging. This completed the first part of sequence one. The DAQ was stopped and restarted to initiate Leveling. The bolts were inspected by hand turning each one to determine if any were loose. For this first sequence, three bolts were loose, the bolts on: Beam Line 1 West, Beam Line 3 West, and Beam Line 5 East. The bolts were hand turned until they made contact with the steel plate below. The panel elevation measurements were taken and recorded. The leveling bolts were adjusted to meet the required elevations. The torque was measured for all ten bolts. The average of the torque readings for the ten bolts was calculated. The bolts with the lower torque values were torqued to within 20% of the calculated average. Then the torque was measured again for all ten bolts, to see if the values redistributed. This was done six different times for this first sequence. The average torque measurements ranged from 30.4 foot pounds to 44 foot pounds.

Figures 14 - Starting on the upper left and going clockwise: (a) Lifting the panels from the form; (b) Lifting; (c) Leveling; (d) Return Lift.

The recorded torques for all ten bolts never came within 20%; the closest that was achieved for the first sequence was within 34%. Dunnage was placed prior to the Return Lift, to allow for the leveling bolts to be adjusted for the remaining sequences. Once Leveling was complete, the DAQ was reset for Return Lift. Because the rigging was never disconnected, the cranes lifted the panel and placed it back to its starting point, on the dunnage.

The second sequence introduced uneven interior lifting straps. To understand what would happen should one interior cable lose tension, or be slightly longer in length, an additional shackle was added at each end of the lifting cable. The two arrows in Figure 15 point to the two shackles. Beyond this variable change, the procedure was the same as Sequence 1. Once the panel was set and the bolts were hand inspected, there were three loose bolts that had to be hand turned to make contact. This sequence only went through three torque measurements in lieu of the six performed during Sequence 1. The average torque ranged from 33.3 foot pounds to 36.2 foot pounds. And similar to Sequence 1, having all bolts torqued to within 20% was not achieved (the best for Sequence 2 was within 110%).

Figure 15 - Sequence 2 Lifting

For the third sequence, the extra shackles were taken off and the original shackles were returned to the cable and lifting devices. For the third sequence two leveling bolts were preset to intentionally not make contact with the beams. This accounts for camber variability of the beam(s) or errors when presetting the bolts. The bolts left loose were the east bolt on Beam Line 2 and the west bolt on Beam Line 4. The same procedure was followed as the two sequences before. Once the panel was set on the beams and Leveling commenced, hand inspection determined that there were four loose bolts including the two that were left loose intentionally, and bolts on Beam Line 5 East and Beam Line 3 West. Again they were hand tightened and the torque measurements were recorded. Sequence 3 had three readings averaging between 34 to 36.5 foot pounds with the torque readings within 76% of each other.

Sequence 4's objective was similar to that of Sequence 3, except four bolts were preset to intentionally not make contact with the beam. The bolts that were left loose were the two bolts on Beam Line 2 and the two bolts on Beam Line 4. This created a clear span of 18 feet. Again the same procedure was followed as the previous sequences. The five loose bolts per the hand inspection included: Beam Line 2 East, Beam Line 2 West, Beam Line 4 East, Beam Line 4 West and Beam Line 5 East. The torque measurements were recorded twice for this sequence averaging 38.25 foot pounds with the torque readings within 100% of each other.

The final sequence was closer to what would be expected in the future pilot project: Lifting and Leveling. All cables were engaged and all ten leveling bolts were preset to make contact. In the prior sequences the leveling bolts were set lower from the top (longer in the bottom), to prevent damaging the haunch forming material. During Sequence 5, the bolts were set for the panel to make full contact with the form material and the bolts to make full contact with the steel plates embedded in the beams. The Lifting and Leveling procedures were the same as the four previous sequences. During hand inspection, the bolts that were loose were the east bolt on Beam Line 3, and both bolts on Beam Line 4. This final sequence went through several different torqueing procedures attempting to torque the bolts within 20% of the average, but only achieved 69%. The torque values and their relationship to the strain readings will be further discussed in the Results. The test ended at 4:07 pm. The last temperature reading of the top surface of the precast panel was taken at about 3 pm and averaged about 100 degrees F.

Day 2 - Installation of Precast Deck Panel 1

Day 2 of testing followed the same format as Day 1. The testing started at 9:52 am on May 24, 2012. The precast panel temperature was measured at about 70 degrees F. There was no rain but the ambient temperature reached a high of 95 degrees F. Due to the difficulty experienced on Day 1 trying to preset the leveling bolts while the panel was elevated by the crane, the panel was set on dunnage. Once this initial step was done, the leveling bolts were preset, and the Lifting process began. As seen on Day 1, each sequence with the exception of Sequence 1 and 5 had a variation to the typical condition: Sequence 2, an uneven interior lifting strap; Sequence 3, two leveling bolts intentionally set to not make contact; and Sequence 4, four leveling bolts intentionally set to not make contact. The testing ended at 3:07 pm. The panel's final top surface temperature reading was over 100 degrees F.

There was a new challenge that arose when installing this panel for each sequence. The hook bars from Panel 2 (the previously installed panel) and the hook bars from Panel 1 now had to be aligned. The panel had to be leveled both transversely and longitudinally, to maintain a 12 inch closure joint throughout. This was done, as seen in Figure 16, by placing a tape measure at each end of the joint as well as a level.

The Leveling procedure on Day 2 was consistent for each sequence. The initial torque readings were measured, recorded, and averaged. For the first sequence, the bolts were hand turned to determine any loose bolts. Three bolts were loose (Beam Line 3 West, Beam Line 4 East, and Beam Line 5 East). The bolts were hand turned until they made contact with the steel plate below. The torque was measured for all ten bolts. The average of the torque readings for the ten bolts was calculated, and the average torque was 40.6 foot pounds. All the bolts were torqued to 35 foot pounds, starting with the bolts with the lower torque values. The torque for all of the bolts were then measured again.

Figure 16 - Lining up the panels while maintaining a 12 inch closure pour.

The second sequence introduced the uneven interior lifting straps. Once the panel was set on the beams the bolts were hand inspected, there were no loose bolts that had to be hand turned to make contact. The average torque measured was 31.5 foot pounds. The bolts were all torqued to 30 foot pounds. The bolts would be torqued to this value for the rest of the sequences; only sequence one was torqued to 35 foot pounds. The torque was then re-measured.

For the third sequence, the extra shackles were taken off and the original shackles were returned to the cable and lifting devices. The variable changed in the third sequence was two leveling bolts preset to intentionally not make contact with the beams. The bolts left loose were the east bolt on Beam Line 2 and the west bolt on Beam Line 4. The same procedure was followed as the two sequences before. Once the panel was set on the beams and Leveling commenced, hand inspection determined that there were four loose bolts including the two that were left loose intentionally. Again they were hand tightened and the torque measurements were recorded. Sequence 3 had 2 readings averaging 39.1 (initial measurement) and 24 foot pounds (final measurement).

Sequence 4 was similar to Sequence 3, except four bolts were preset to intentionally not make contact with the beam. The bolts that were left loose were the two bolts on Beam Line 2 and the two bolts on Beam Line 4. Again the same procedure was followed as the previous sequences. There were five loose bolts per the hand inspection including bolts on: Beam Line 2 East, Beam Line 2 West, Beam Line 4 East, Beam Line 4 West and Beam Line 5 East. The torque measurements were recorded twice for this sequence averaging 35.2 (initial measurement) and 23.3 foot pounds (final measurement).

Figure 17 - Hand tightening the bolts during Leveling on Day 2.

During Sequence 5, the bolts were set for the panel to make full contact with the form material and the bolts to make full contact with the steel plates embedded in the beams. The Lifting and Leveling procedures were the same as the four previous sequences. Both bolts on Beam Line 3 and the bolt on Beam Line 2 East were loose during hand inspection. The torque measurements were recorded averaging 33 (initial measurement) and 22.2 foot pounds (final measurement). The torque values and their relationship to the strain readings will be discussed further in the Results.

Some differences from Day 1 and Day 2 include: The gages for the panel installed on Day 1 were placed three to four days prior to the testing. The panel on Day 2 was instrumented the day before it was installed. Another difference was the type of tape used to hold down the gages. On Day 1 the gages were taped down with a heavy black tape, whose temperature reading measured about 20 degrees F higher than the panel. Day 2 instrumentation was held down by a thinner blue tape, with temperature readings about 6 degrees F higher than the panel. Because the gages on the panel installed on Day 1 were exposed to temperature for several days, as well as additional heat due to the type of tape, the strain readings may reflect some of these differences, however it is not conclusive.

Horizontal (Longitudinal) Push Test

A small side test was performed on Panel 2, the west panel, to determine how much force it would take before the panel moved longitudinally. This is a concern during field placement, with both a vertical curve and possible camber effects. The Horizontal Push Test consisted of placing an actuator at the mid-span of the panel, pushing off of the existing slab, and pushing onto the flat 8.5 inch side of the panel. There were two string potentiometers (i.e. displacement gages) at each end of the panel (0’-0” and 43'-1" from the end). The chart on Figure 18 indicates at a load of about 2.25 kips, deflections peaked at 0.004 inches on the north end and 0.014 inches on the south end. The test was stopped as soon as the gages spiked, concluding it would take close to 3 kips to move the panel.

Grout Installation

Once the panels were installed, the next step was to grout the shear pockets and haunches. Up to this point the panels were bearing solely on the leveling bolts. The SRC did release all the bolts, where the panels were bearing solely on the forming material and the concrete beams below, however this was a temporary condition to observe whether the haunch forming material would compress and then rebound. The haunch material did rebound successfully, with some areas needing to be sealed due to this extreme case. For the pilot project the bolts would never be fully released; they would support the slab until grouting.

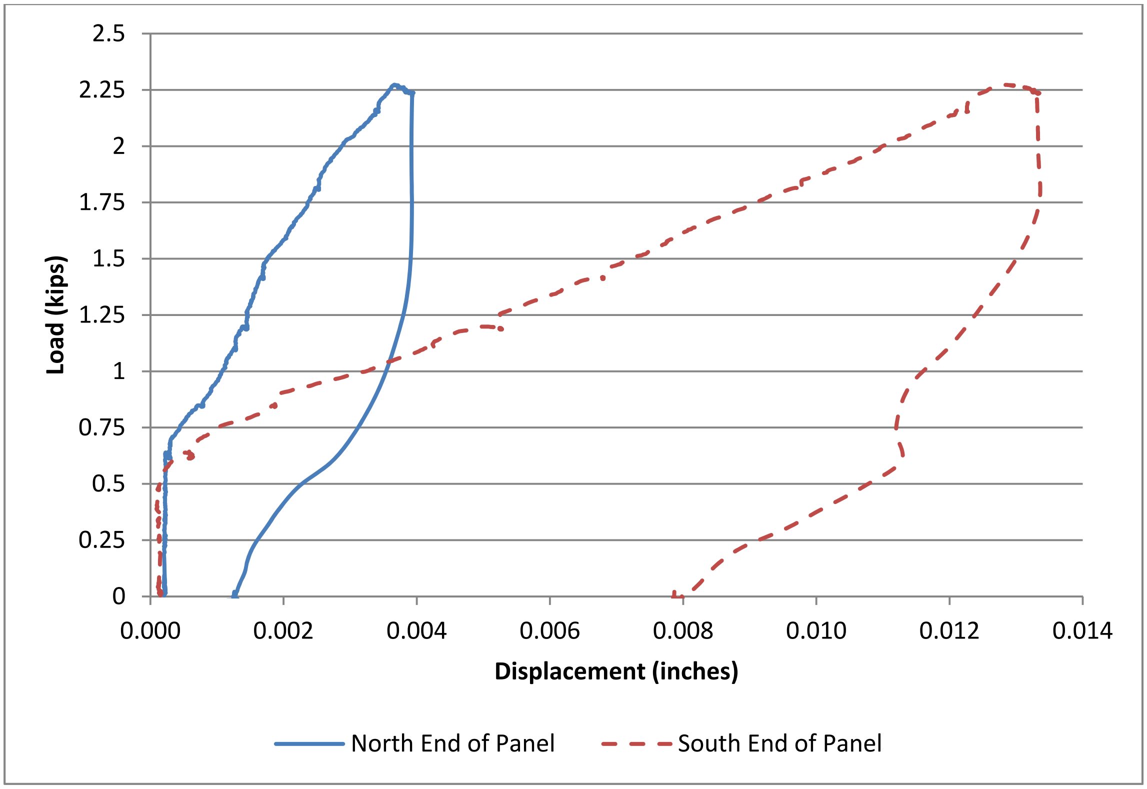


Figure 18 - Load-Displacement Graph for Horizontal Push Test

The grouting was done in two days, July 2 and 3, 2012. Two and a half beams were grouted on Monday and the final two and a half beams were grouted on Tuesday. The shear pockets were cleaned and standing water was removed prior to the grouting operation. The first day grouting commenced around 1 pm, and the ambient temperature, recorded at Tallahassee Regional Airport, was 97 degrees F. The grout used was Masterflow 928 by BASF, the same grout used in the Push-out Test. The grouting started at the South end. The shear pockets and haunch areas were dry prior to grout placement. The procedure described earlier was followed, grouting the lower shear pocket until the higher pocket began to fill with grout. Once the lower pocket was about half full, the hose was moved to the higher pocket. Finally the grout flow was stopped once the grout reached the top of the panel. Grout tubes placed at both the high and low end of the panel were observed, to verify that the grout was reaching the extreme ends. Wet burlap was placed over the shear pockets a few hours later to allow for curing. See Figures 19 and 20.

The second day, the midday high was also 97 degrees F, however the grouting started at around 10 am. The final five North end shear pockets and haunches were grouted. Two of the haunch areas were pre-wet using a water hose with a light spray prior to pouring the grout. The pre-wet shear pocket locations as well as the final measured haunch heights for all panels on all beam lines is shown in the Grout Placement and Coring Locations Sheet found in Appendix A. All other haunches on the second day of grouting were left dry to compare the bond once the grout and panel interface were ready to be cored. The procedure for both days was the same.

However on the second day, the grout did not reach the grout tubes. The temperature at the time of grout placement for both days exceeded the manufacturer's recommendations.

Figure 19 - Day 1 Grout Installation

Figure 20 - Grouted shear pockets; Day 1 on left and Day 2 on right

Transverse Closure Joint

The final step was to cast the closure joint. The twelve inch joint had the No. 4 hoop bars projecting from the keyway side of both panels. The haunch material had to be removed from the top of the beams at the closure joint locations, prior to pouring the concrete. Due to rebar congestion and the adhesive used, removing the haunch material was found to be challenging.

Per the FDOT Standard Specifications, when pouring fresh concrete to hardened concrete, a Type A epoxy resin must be applied to the hardened surface. The epoxy chosen was Concresive Liquid LPL. Painting the epoxy on the concrete surface with rebar projecting every three inches was time consuming. In addition, because the epoxy must remain tacky during pouring, it was found to be labor intensive. To investigate the benefit of using the epoxy resin, it was decided that the North half of the joint would have an epoxy resin, and the South half would be left saturated surface dry (SSD), using the center beam line as the stopping point for each option.

The four No. 5 bars were threaded prior to placing the epoxy to conserve time. Once the bars were placed, and the keyways were painted or saturated, the concrete with Shrinkage Reducing Admixture (SRA) was then poured. Additional information on the concrete with SRA is presented in Part II. See figures below.

Figure 21 - Epoxy (left) and SSD (right) Keyways

Figure 22 - Transverse Closure Joint

Figure 23 - Completed Full Depth Precast Panel Mock-Up

H. Results

Additional cylinder breaks were performed on August 29, 2012. Precast Deck Panel 2's cylinders resulted in an average concrete compressive strength of 7322 psi after 131 days and Panel 1's cylinders resulted in an average 7265 psi after 153 days.

Day 1 - Installation of Precast Deck Panel 2

The results acquired from the strain gages were compiled in three categories: Lifting, Leveling and Return Lift. The Lifting data was graphed per sequence, and the results show that all of the gages at the lifting devices (Gages 4, 8, 12, 14, 18, 22, 26 and 30) recorded strains that were all in tension during lifting. After Sequence 1, Gages 4 and 30 were damaged and no longer recorded good results. The other six gages recorded strains, all with similar behavior during lifting, indicating good distribution of stresses at all of the lifting devices. Figure 24 shows results for Sequence 2 when two of the lifting devices were loosened with additional shackles. The gages at those two loose lifting devices were Gages 12 and 14, the two bottom lines on the graph in Figure 24. Figure 25 is a graph of the third sequence data during lifting. Again it shows similar behavior between all lifting locations, and all strain values recorded were positive indicating tension. All other sequences resulted in similar behavior.

As far as Leveling none of the gages placed by the leveling bolts generated good results. As mentioned earlier in Part G of this report, there were several factors that may have contributed to the gages not recording accurate data including the amount of time between instrumentation and testing, as well as the temperature and weather conditions. However, from visual inspection for all three parts of the testing, there were no cracks.

The torque data recorded for all of the readings during leveling were tabulated. The torque data was combined with the strain readings for each bolt, to develop a relationship between the two. The objective was to verify the loads (strains) were evenly distributed to all of the beams when the torque readings were all within 20% of each other. However, torquing the bolts within 20% was not achieved. In addition, as mentioned above, the strain gages were not recording good results. The only relationship generated out of the tables was for the bolts that were loose and needed to be hand-tightened. Prior to initial torquing; the torque readings for those bolts were very low between 5 and 10 foot pounds. In addition the bolts left intentionally loose to not make contact (Sequence 3 and 4), had to be hand tightened and again resulted in very low initial torque readings.

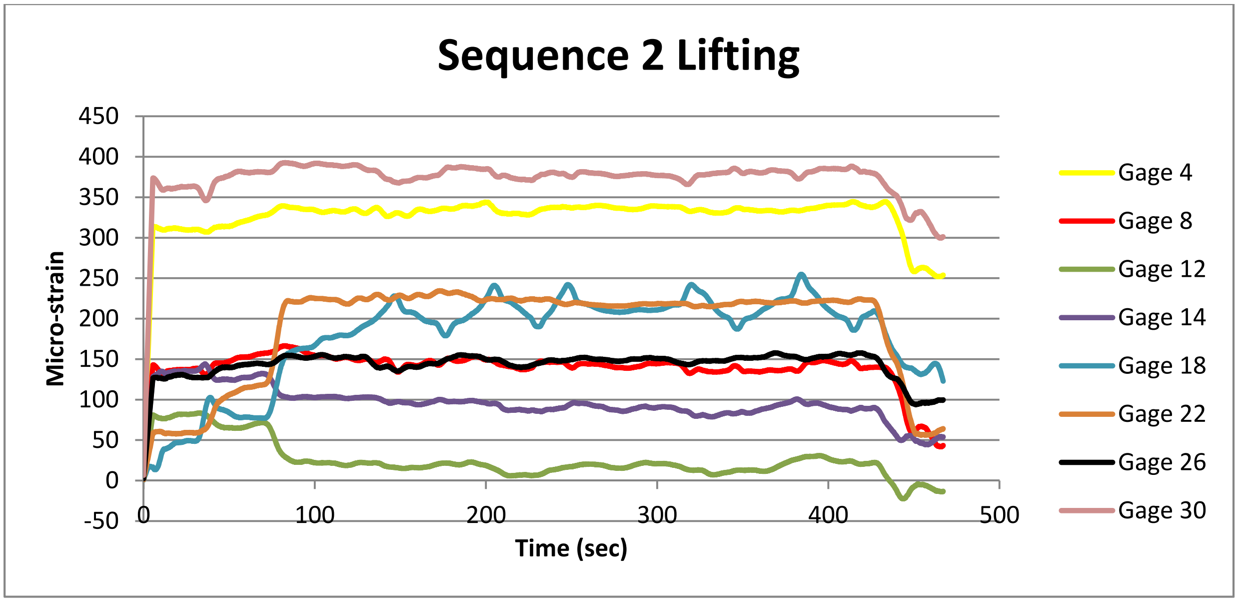


Figure 24 - Day 1, Sequence 2 Lifting Strain vs. Time Graph

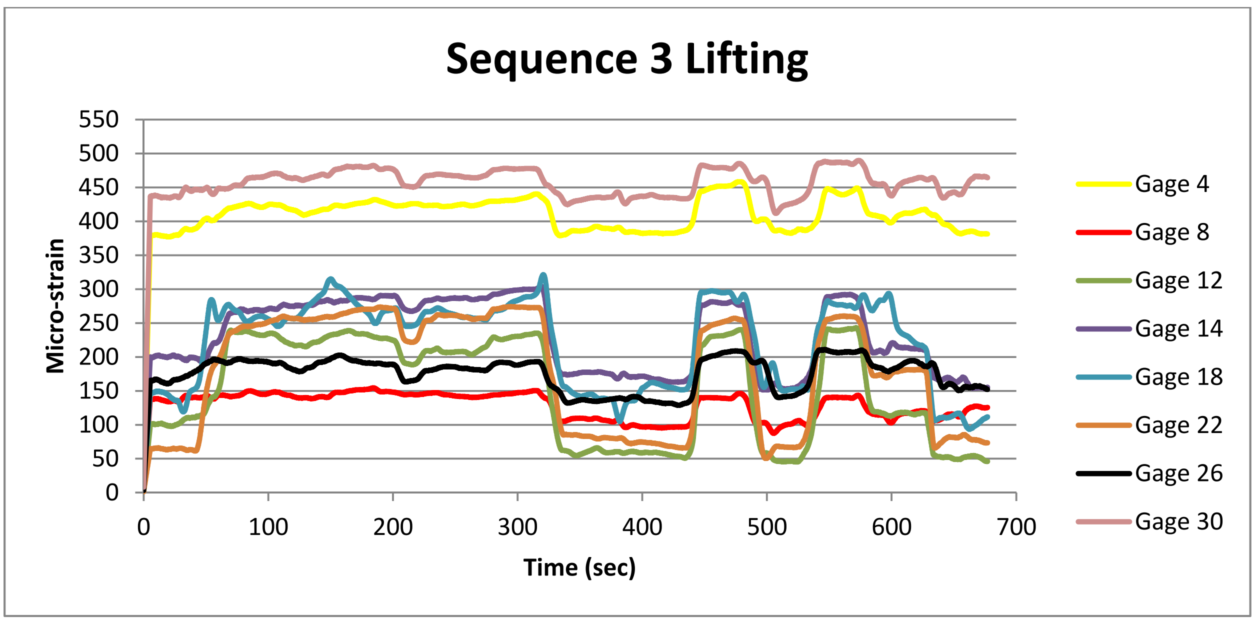


Figure 25 - Day 1, Sequence 3 Lifting Strain vs. Time Graph

Day 2 - Installation of Precast Deck Panel 1

The strain gages on Day 2 did provide more useful results. During Lifting, the finite element model created during design (see Figure 4b), calculated strains ranging from 41 to 56 micro-strain. The graphs in Figure 26 for Sequence 2 and Figure 27 for Sequence 3, show more than half of the strain gages recording strains in that range. After graphing all five sequences, Gages 18 and 22 were not working and should be ignored in the graphs. For Figure 26, all of the strains were in tension during lifting, except for the strains recorded from Gages 12 and 14, the gages next to the loose cables. Notice these two locations were in compression (negative strain). Overall the data and the graphs show that the load is being distributed to the all of the eight cables (or six cables for Sequence 2).

The Leveling data was not as clear and consistent and did not match the predicted values. The predicted value at each bolt location was about 30 micro-strain, assuming all bolts in contact. During testing, snap shots of the strain reading were taken before, during and after torquing the bolts. The same thing was done after Sequence 2 of Day 1, but the gages did not provide any useful data. From the data collected on Day 2, tables were created to again establish a relationship between torque and strain. One of the main patterns observed from the table for each torque reading was that all of the torque readings greater than the average torque resulted in positive strain values or tension. In other words, a bolt with a higher than average torque value was supporting load. The table also indicates that the maximum number of bolts in contact at one time was six bolts (where the torque was higher than the average and the strains were positive indicating a tension reading). However on average only four bolts were in contact at one time.

Similar to Day 1, the Day 2 results indicated the bolts that needed to be hand-tightened prior to initial torquing, had very low torque readings between 5 and 10 foot pounds. This included the bolts left intentionally loose to not make contact (Sequence 3 and 4).

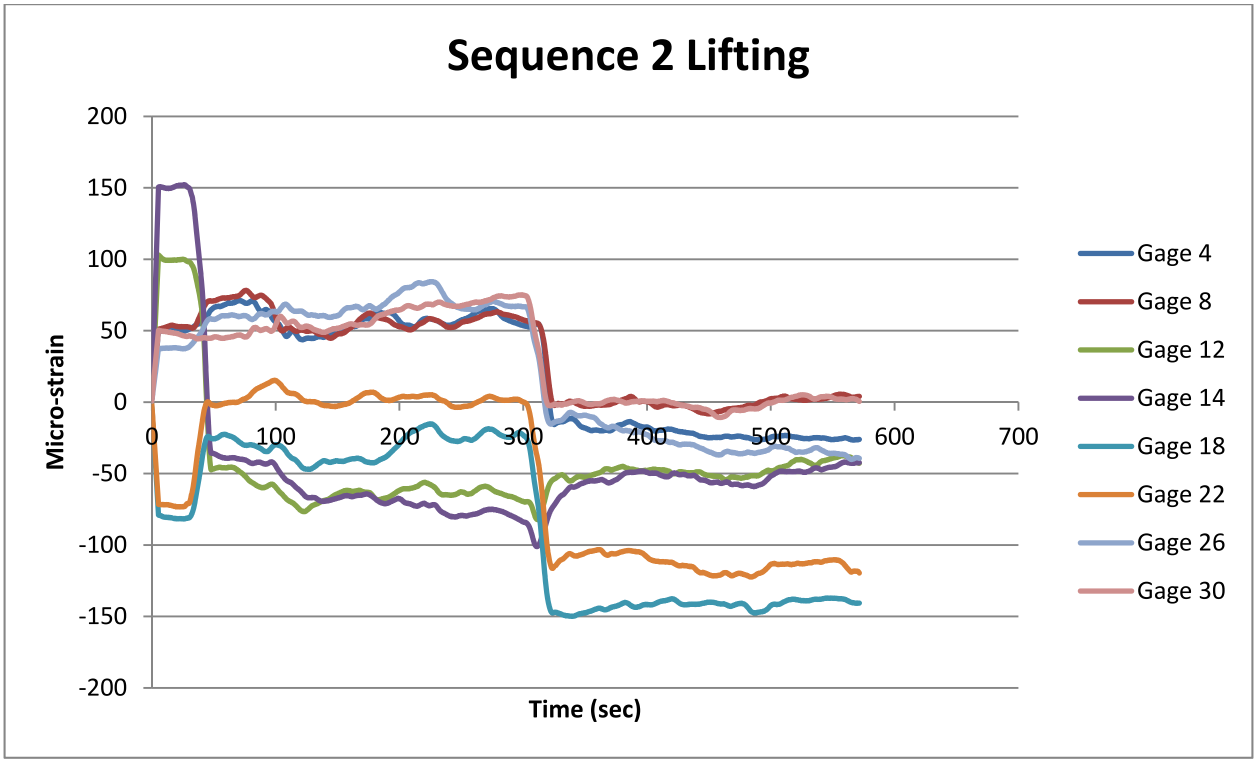


Figure 26 - Day 2, Sequence 2 Lifting Strain vs. Time Graph

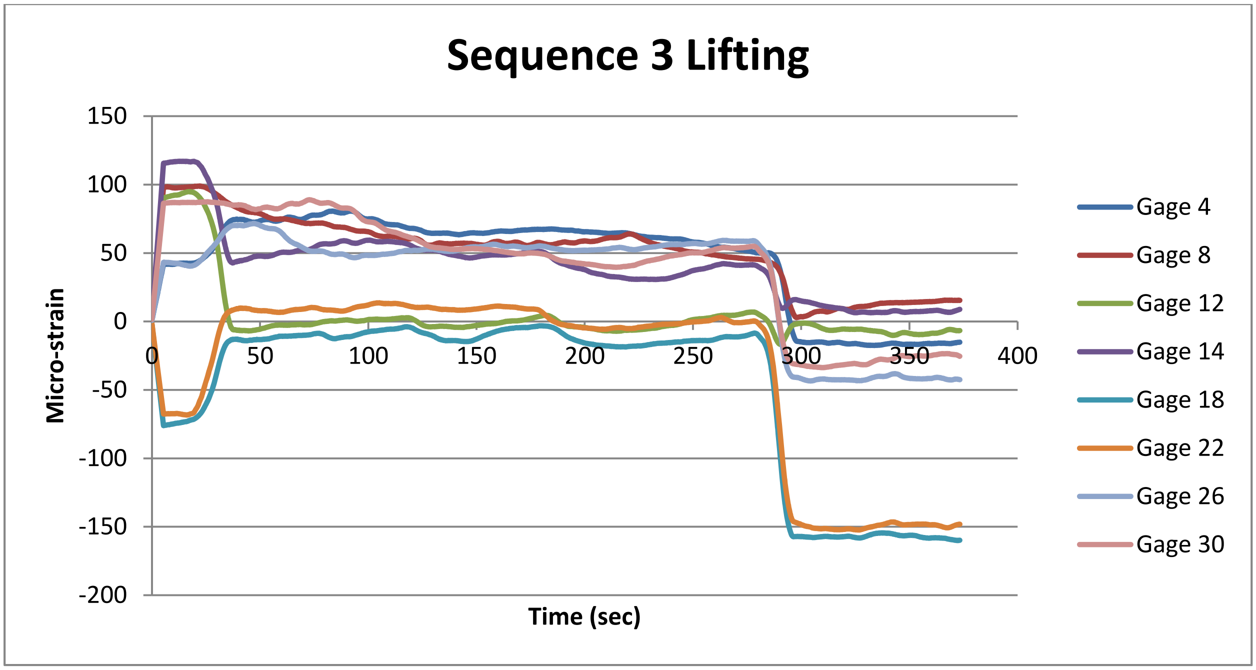


Figure 27 - Day 2, Sequence 2 Lifting Strain vs. Time Graph

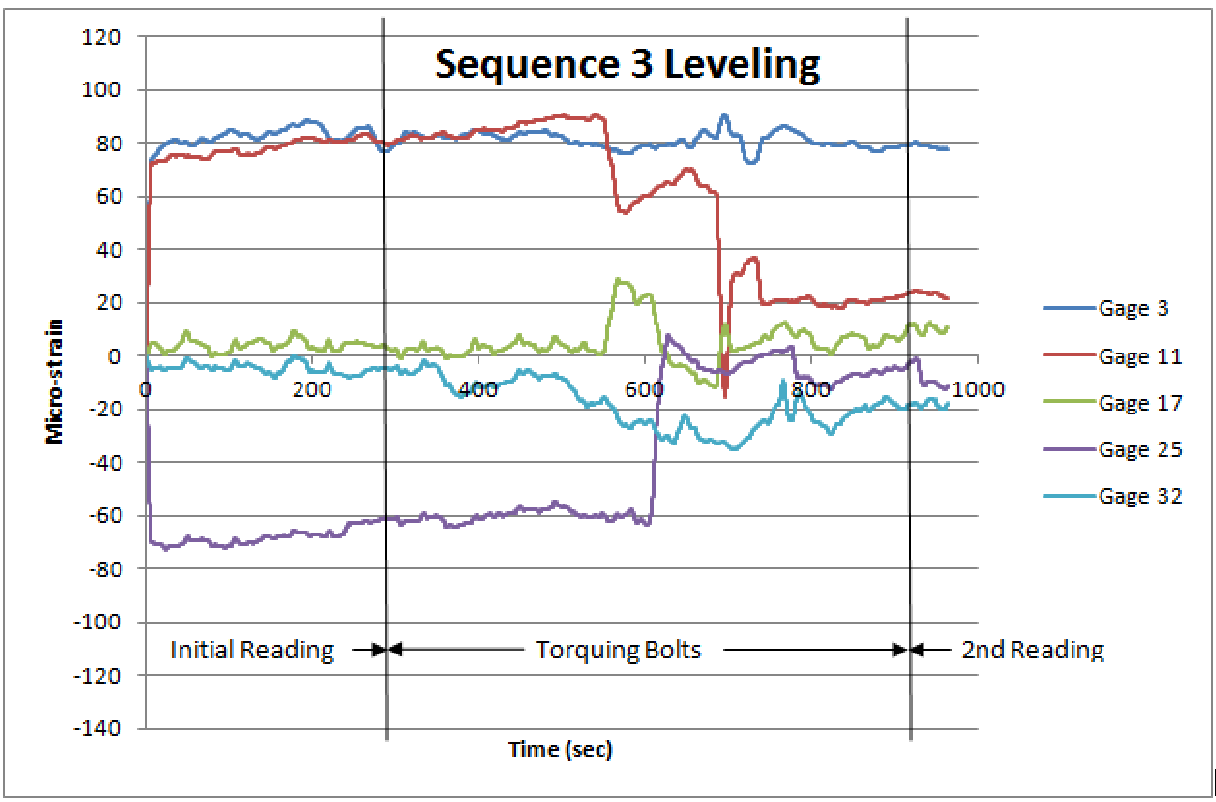


Figure 28 - Sequence 3 Leveling West, Strain vs. Time Graph

All of the Leveling data was graphed for each sequence. A vertical line was drawn at the time of initial torque reading and at the second torque reading, as shown in Figure 28. During the initial torque reading the strain values were far apart and showed no real pattern. However once all of the bolts were torqued to 30 foot pounds, the strain values on the second vertical line were closer together. Although torquing the bolts within 20% of each other was never achievable, this procedure of recording the torques, taking the average and torquing the bolts to that average, did redistribute the stresses. Finally, through visual inspection there was no cracking due to lifting or leveling.

All of the strain graphs for both panels during Lifting and for Panel 1 during Leveling are provided in Appendix C.

Shear Pockets, Haunches and Transverse Closure Joint

The haunch forming material was removed to inspect the haunch grout. There were two areas on Beam Line 1 where the grout did not flow to. They were both at the Northeast end of each panel along Beam Line 1, where the actual haunch dimensions were 1.6875 inch and 1 inch for Panels 1 and 2, respectively. These were not the smallest haunch dimensions, Panel 2 on Beam Line 3 measured 0.875 inch, and was fully grouted. In Appendix A, the Grouting Placement and Coring Locations drawing shows the grouting area as well as where the voids were found and the dimension of those voids. Through inspection, the other four beam lines resulted in fully grouted pockets and haunches.

There were 11 four inch diameter cores taken at the panel-shear pocket interface and at the panel between the shear pockets. The cores taken at the panel between the shear pockets was to observe the bond between the panel and the haunch. All of the cores went about 12 inches deep. The observations showed that 7 out of the 11 cores had resulted in a good bond between the interfaces. In three of the cores, separation did occur at the interface and some cracking was observed. A full table of the core observations can be found in Appendix C. Pictures of each of the concrete cores are shown in Appendix D.

There were also four cores taken at the panel-closure joint interface. The concrete cores were also four inches in diameter and about 12 inches deep. Two cores were taken at each panel interface that had the epoxy compound, and two were taken at each panel interface on the SSD side. The observations of the cores indicate that the panel and joint had a good bond regardless of which method was used to bond the hardened concrete to the wet concrete. There was some separation at the top taper of the keyways for three out of the four samples. However at the vertical parts of the keyway there was a good bond. From the observations, the slope at the taper of the keyways will be increased in future use, to create a better bond and allow an easier flow of concrete to the inner corners of the keyway.

J. Conclusion

The very first objective was to determine any reinforcement congestion issues. After installing two panels, one with mild reinforcement and the second with welded wire reinforcement, no congestion issues arose. The second objective was to investigate some of the hardware for the panels including lifting inserts and leveling devices. All of the hardware was available and met the minimum requirements to support the panels. Additional reinforcement was required and added for lifting.

The strain readings were not a good indication of load distribution to the beams and the torque was also difficult to relate directly to the strain values recorded. The torque readings at each bolt did not verify distribution but confirmed that only about half the bolts were supporting the panel at one time. Although the torque values were never within 20% of each other, the procedure followed in Day 2 seemed to redistribute the stresses, bringing the strain values closer together.

The final objective was to identify any grouting challenges, as well as investigate the materials and procedures used for grouting. The polyethylene foam compressed and rebounded as the panels' full weight was placed on the forming material. The grout flowed fully into eight of the ten haunches, leaving small voids in two North end haunches (Beam Line 1). Although there was no conclusive evidence to indicate pre-wetting was beneficial, it is recommended to pre-wet the hardened concrete to reduce drawing moisture from the grout. The concrete cores showed that the bond between the precast panels and the grouting material was good. The bond between the closure joint concrete and the precast panels also showed good results. One minor modification was recommended from this testing, to increase the slope at the taper of the keyway to help create a better bond and allow the concrete to flow into the corners.

II. Testing of Transverse Joints – Lapping Hoop Joint

Various joints and reinforcing layouts were considered and investigated to connect the precast deck panels. This included a Lapping Hoop joint, Class C Splice and a Tongue and Groove joint. With further investigation into previous research performed by Federal Highway Administration (FHWA) and National Cooperative Highway Research Program (NCHRP) two joints were selected to be tested. The first joint was introduced in Part I of this report, the Lapping Hoop Joint. The second joint tested was the Tongue and Groove joint. This report will only focus on the Lapping Hoop Joint.

A. Introduction

This was the transverse connection detail selected for over one hundred joints for the pilot project. The SRC performed testing to evaluate the strength of two precast concrete panels with a lapping hoop joint connection. The testing, conducted on December 21, 2011 thru December 27, 2011, involved measuring and recording strain, displacement and load values at a sample rate of 10 Hz. Each joint specimen consisted of two seven feet long by two feet wide by eight inch thick panels joined together by a one foot closure joint. Pictures of the joint were shown throughout Part I of this report. This was a smaller version of the same connection that allowed for testing. The two panels were joined with No. 4 hoop bars that overlap, and then four No. 5 lacing bars were installed within the hoop bars. The detail of the joint is shown in Figure 29. The closure joint was made using concrete with a Shrinkage Reducing Admixture (SRA). There were three of these joint specimens. The three control specimens were 15 feet long by two feet wide by eight inches thick concrete panels. The control panels, representing a conventional continuously poured deck, were tested and the data was used to compare with the capacity of the panels with a joint. Detailed drawings of both panel types are shown in Appendix A, Sheets EX-5 and EX-6.

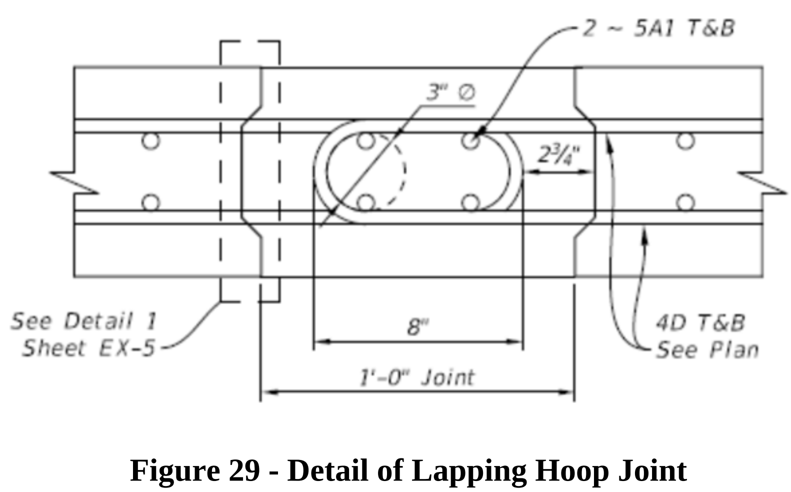


Figure 29 - Detail of Lapping Hoop Joint

B. Materials and Design

All of the panels used FDOT Concrete Class II Bridge Deck mix. The reinforcement included four No. 4 bars spaced at six inches on the bottom mat and two No. 4 bars spaced at 12 inches on the top mat in the longitudinal (long) direction. In the transverse (short) direction, No. 5 bars at six inch spacing were placed on both top and bottom mat. Both the control panels and the joint panels had the same reinforcement with the exception of four No. 4 hoop bars (4D) added at each end of the smaller panels to be joined. The panels were poured on November 3, 2011.

Per the deck design described in Part I, the required area of reinforcement was 0.57 square inches per foot, which was used for the 12 inch closure pour. Two No. 5 bars were provided (top and bottom), for a total area of steel of 0.62 square inches. The four No. 5 lacing bars required a threaded nut to be tack welded at each end of the bar to help develop the bars, similar to what was done in the NCHRP web173 report. Although adding the threaded nuts did not fully develop the bars, it provided some resistance in the joint.

The joint concrete was FDOT Class II Bridge Deck mix with Eucon's SRA. Concrete producer, A Materials Group, provided the concrete; however, they did not have an approved mix design incorporating SRA. They added the SRA to the approved Class II Bridge Deck mix as specified by the product sheet (2% by cementitious weight). The epoxy bonding agent, Concresive Liquid LPL was placed on the keyways prior to placing the joint concrete. The epoxy was applied using a brush. This was the same application process used in the full scale panels discussed in Part I of this report. The joints were cast on November 23, 2011. The forming and casting of the panels and the joints are shown in Figures 30 through 33.

Figure 30 – Forming Panel Specimens with No Joint

Figure 31 - Forming Panel Specimens for Lapping Hoop Joint

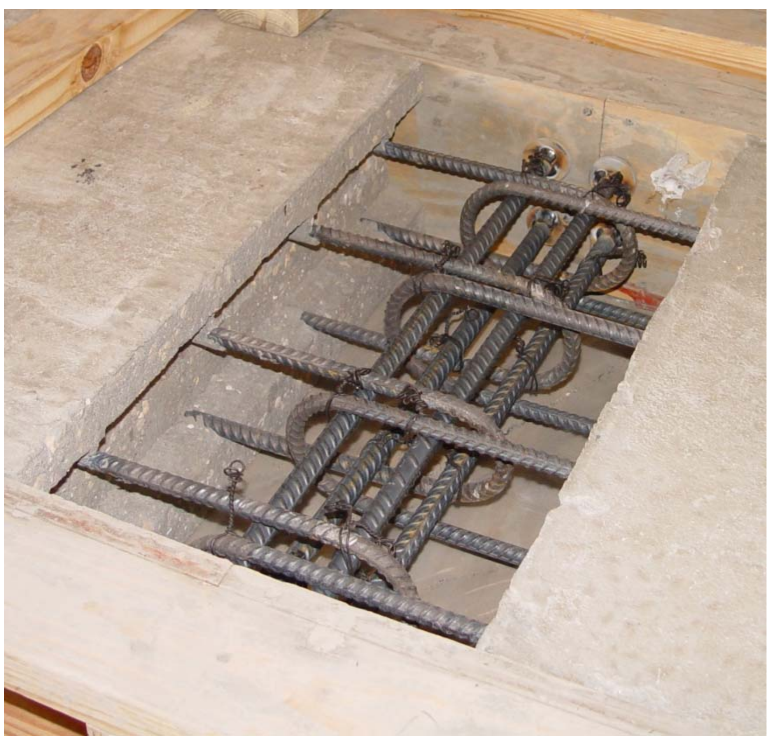


Figure 32 – Lapping Hoop Joint

Figure 33 – Joining the Panels using the Lapping Hoop Joint

C. Test Setup and Testing

The centerline of the supports was one foot from each end of the 15 foot panels, resulting in a 13 foot span. A single point load was applied to a spreader beam consisting of a steel I-beam that provided the two point loads applied to the panels. The centerline of each load was five feet from each end of the panel. This configuration provided zero shear at mid-span where the joint is located for the joint specimens. Nine strain gages were placed on the top of the panels towards the center of the span as shown in the drawings provided in Appendix A labeled Instrumentation Layout, Precast Joint Specimens. Eight displacement gages were placed along the length of the panel as shown in that same sheet. The gages were labeled from North to South and East to West. The panel thickness varied from 8 inches to 8.25 inches. The transverse spacing of the displacement gages D4 and D5, at mid-span, varied between 19.5 inches and 19.875 inches. The top of the detail sheet illustrates how the load was applied. Figure 34 shows an illustration of the load setup and a picture of the setup the day of testing.

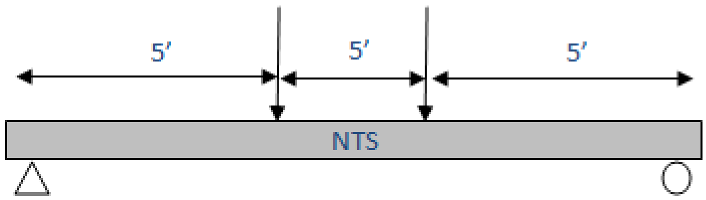


Figure 34 – Load Testing Setup

Once the instrumentation was complete and the panels were ready for testing, the load was applied until failure. The load was held at around 4 kips and 8 kips to observe and mark cracks. Initial cracking occurred around 3.5 to 4 kips with the yielding of the steel around 12 kips. The initial cracking in the joint specimens occurred at both interfaces of the joint concrete and panel concrete. The initial cracking for the control specimens typically occurred within the proximity of one of the load points. The cracking within the control specimens was more evenly distributed at the higher loads than the joint specimens.

D. Test Results

Concrete cylinders of the concrete mixes were tested for both the precast panels and the closure joints resulting in an average concrete compressive strength of 7.7 ksi and 6.5 ksi, respectively. These values were the average of (3) 4 inch by 8 inch cylinders tested for each mix type the first day of testing (December 21, 2011). The two different mixes were again tested on January 31, 2012 and the panel mix resulted in a concrete strength of 8.4 ksi and the joint mix had a concrete strength of 7.8 ksi.

As a result of the testing, the following graphs were created from the data collected, load versus strain and load versus displacement. In Figures 35 and 36, the strain values for strain gages S1, S4 and S7 were averaged. Similarly S2, S5 and S8 were averaged and S3, S6 and S9 were averaged. Strain gages S2, S5, and S8 were located in the mid-span of the panel (dashed curves in Figures 35 and 36). The other six strain gages were offset one foot from either side of the mid-span. The control panels experienced a compressive failure offset about two feet from the mid-span (or about six to eight inches away from the point the load was applied), as the concrete approached compressive strains between 1200 and 1500 micro-strain at all gage locations. The joint panels failed at the mid-span. In Figure 36, the strain values recorded from the gages located at the mid-span exceed a strain of 1500 micro-strain, while the gages that were offset recorded strains under 1000 micro-strain. This differs from the control panels, where the strains stay consistent throughout the panel. Note the strain values in Figure 35 are all very close (with the exception of Control 3 gages S3, S6 and S9, which were no longer working at this point of the testing). In Figure 36 the strain values at mid-span are close to the strain values from the offset gages up to about 12 to 14 kips, where the strain values at mid-span begin to increase more rapidly.

All panels failed at a load of about 18 kips, resulting in a moment of about 600 kip-in (50 kip-ft). These moments include the dead load moment as well as the moment due to the applied load (includes the spreader beam). The theoretical moment capacity for the control panels were calculated per AASHTO LRFD 5.7.3.2 using the concrete compressive strength determined during testing. According to these calculations the maximum moment in pure bending for the panels is 22.33 kip-ft. Resistance factors and load factors were not used. These calculations can be found in Appendix C. As shown in Table 1, the performance ratio for the joint panels is slightly higher than that of the control panels, and both are significantly higher than the calculated moment.

	Joint Panels	**Control Panels**
Theoretical Moment Capacity	22.33 kip-ft	
Actual Moment Capacity	51 kip-ft	50 kip-ft
Ratio (Actual/Theoretical)	2.28	2.24

Table 1

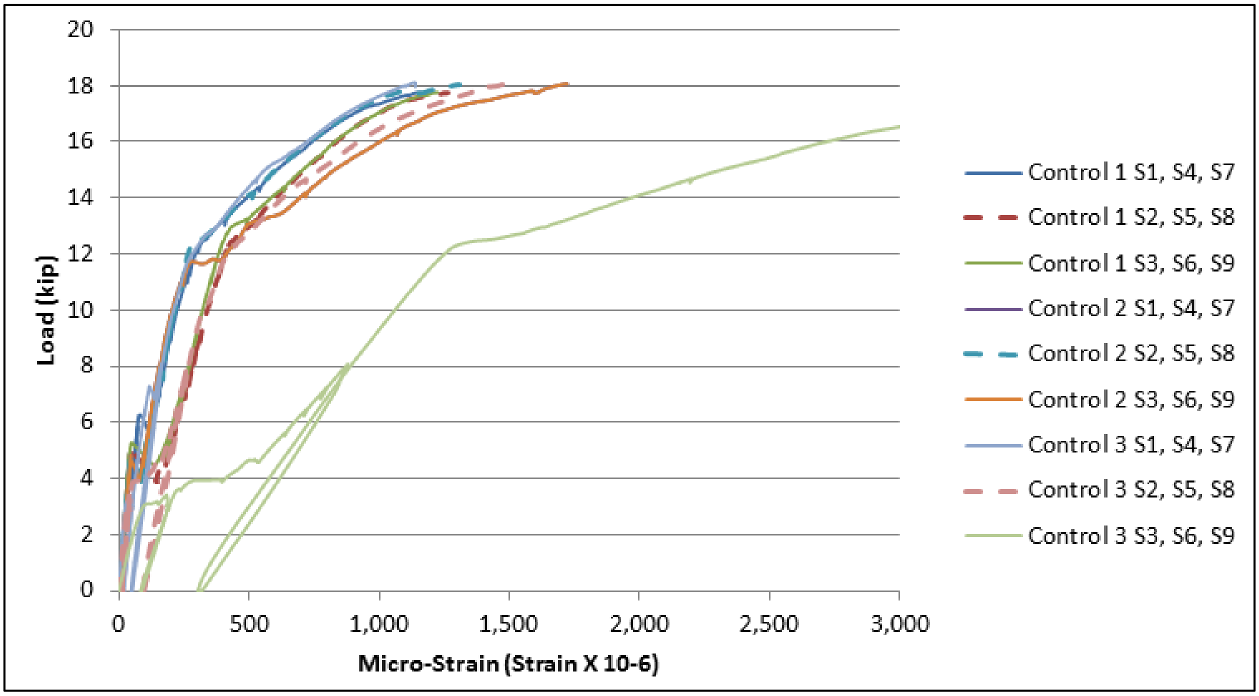


Figure 35 – Load versus Strain Diagram for Control Panels

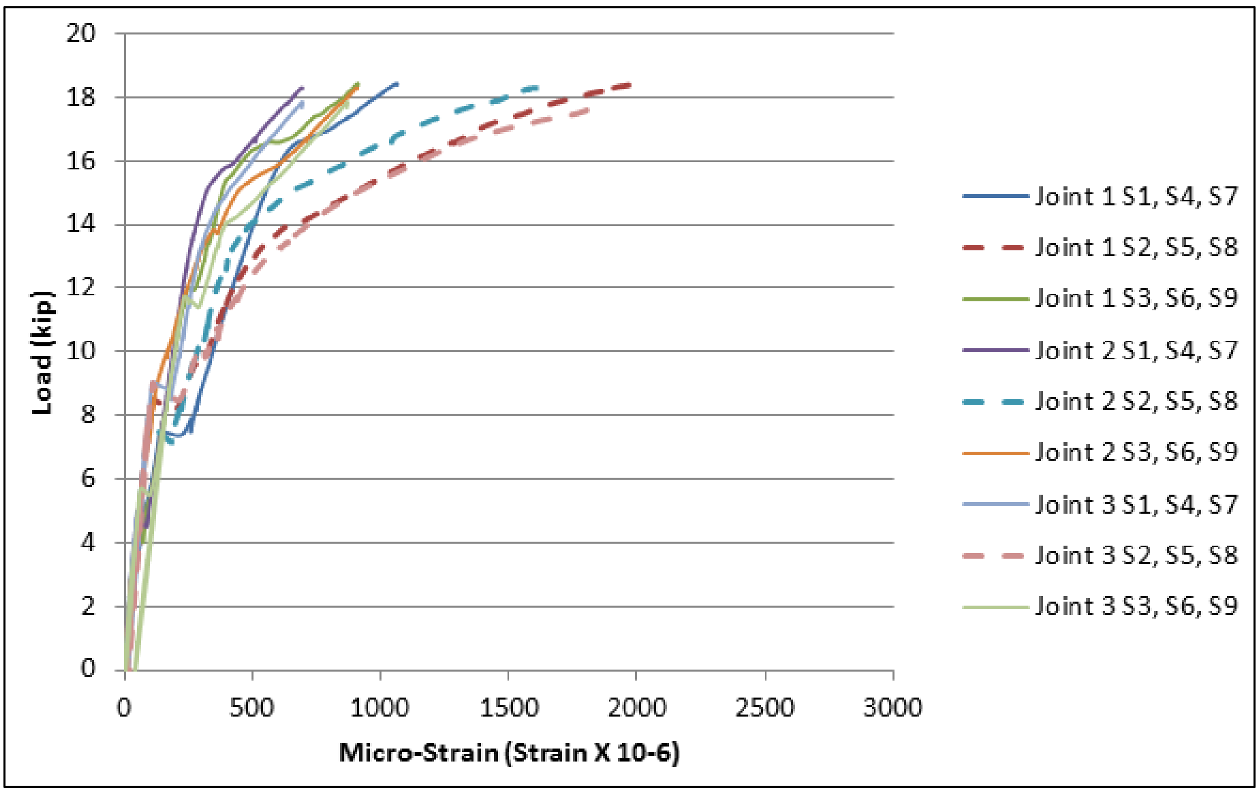


Figure 36 – Load versus Strain Diagram for Joint Panels

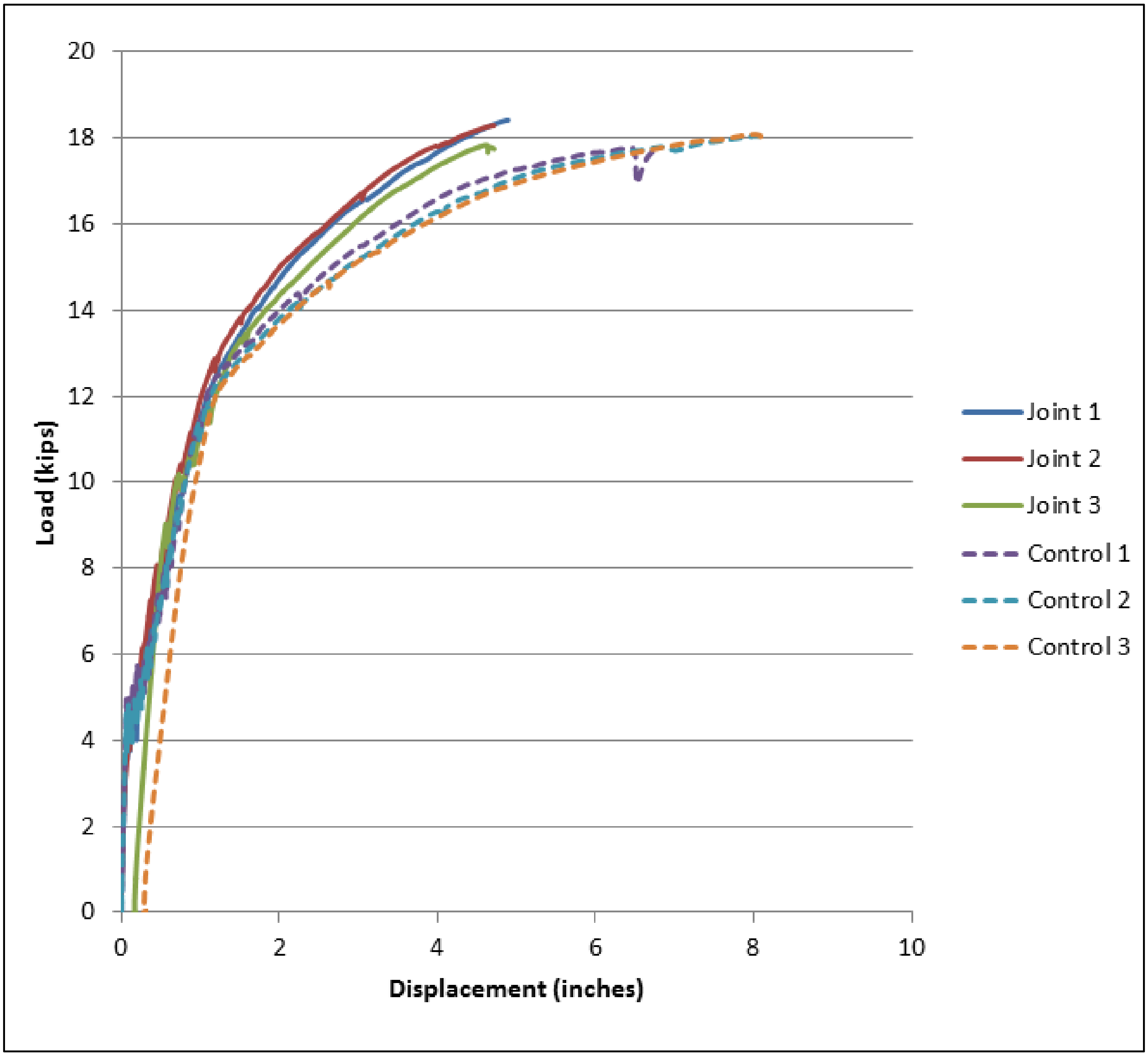


Figure 37 – Load versus Displacement Diagram at Mid-span

Figure 37, Load versus Displacement Diagram at Mid-span, compares the deflected behavior of the joint panels and the control panels. The results used to plot Figure 37 were taken as the average of displacement gages D4 and D5 located at the center of the span. The curve(s) indicate that the control panels did experience a larger displacement, indicating a slight increase in ductility, than the joint panels with equal force. However as the curves approach yielding, the difference in displacement is very small.

Various cracks were monitored when the 4 kip and 8 kip loads were applied to the third joint and control specimens. For the joint specimen the cracks maintained a 0.003 - 0.004 inch crack width after the 4 kip load was removed. At the 8 kip load the crack width was 0.015 inches on the North side and closed to 0.006 - 0.007 inches after the load was removed. The South side had a crack width of 0.01 inches at the 8 kip load and closed to 0.004 - 0.005 inches after the load was removed. The control specimen maintained a 0.002 inch crack width at 3.2 kips and closed to 0.001 inches or less when the load was removed. At the 8 kip load the South crack width was 0.008 inches and closed to 0.002 - 0.003 inches. The North crack width was 0.013 inches and closed to 0.005 - 0.006 inches for the 8 kip load. Figures 38 and 39 show the typical cracking patterns for the control and joint specimens.

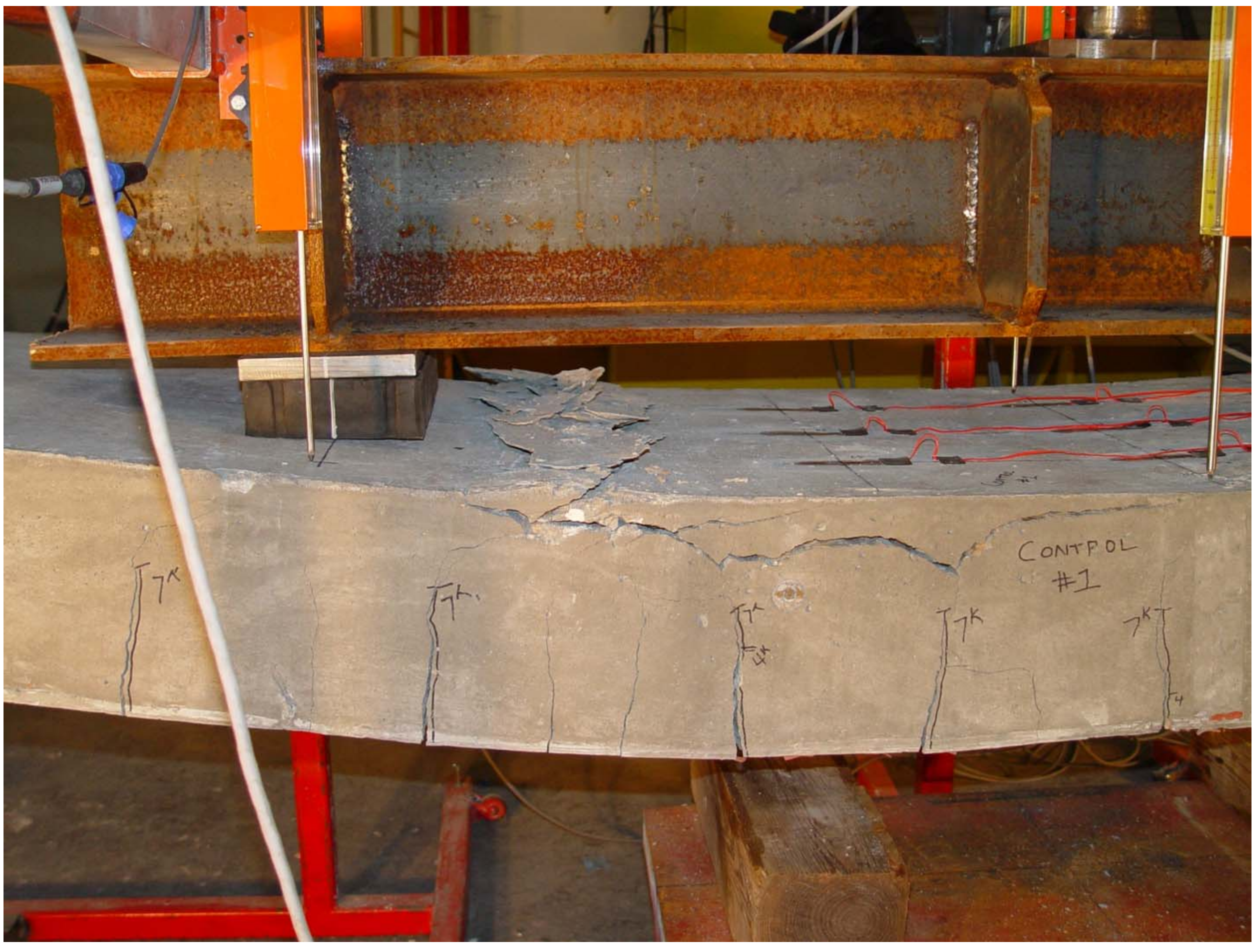


Figure 38 - Failure of Control Specimen #1

Figure 39 - Failure of Joint Specimens

E. Conclusion

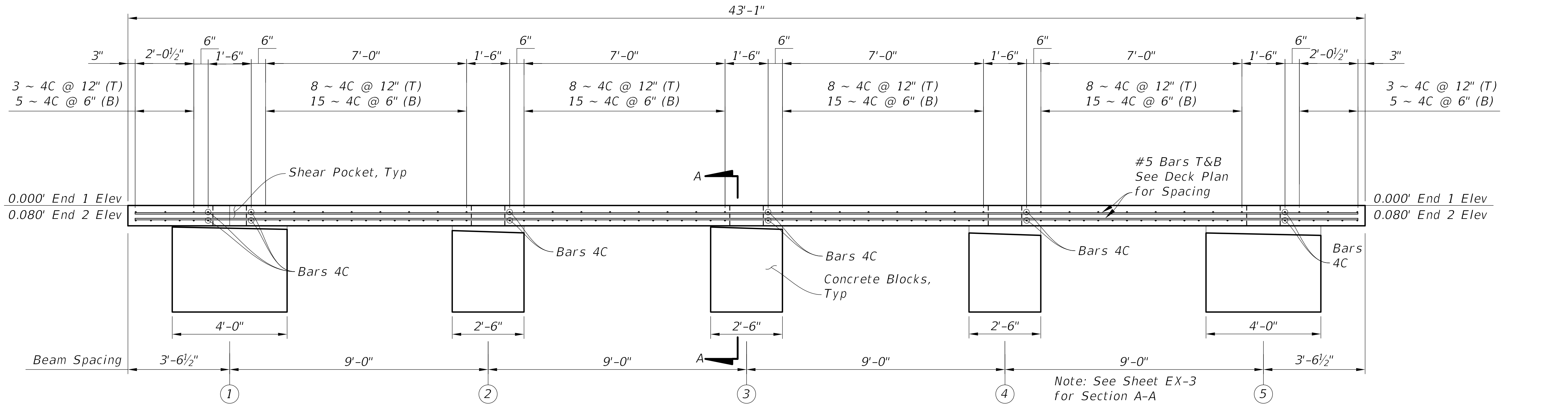
The test results indicate that the use of precast panels with a 12 inch closure joint using the lapping hoops can withstand loads equivalent to a continuous panel. The panels resulted in a moment capacity two times what was calculated during design. It also resulted in a slightly greater ratio, actual capacity to theoretical capacity, than that of the control panels tested.

The failure for all specimens was due to concrete compression well after steel yielding. The control specimens exhibited a greater ductility than the joint specimens. The joint specimens on average withstood a slightly higher load. The failure occurred in the joint of the joint specimens and at one of the load points on the control specimens.

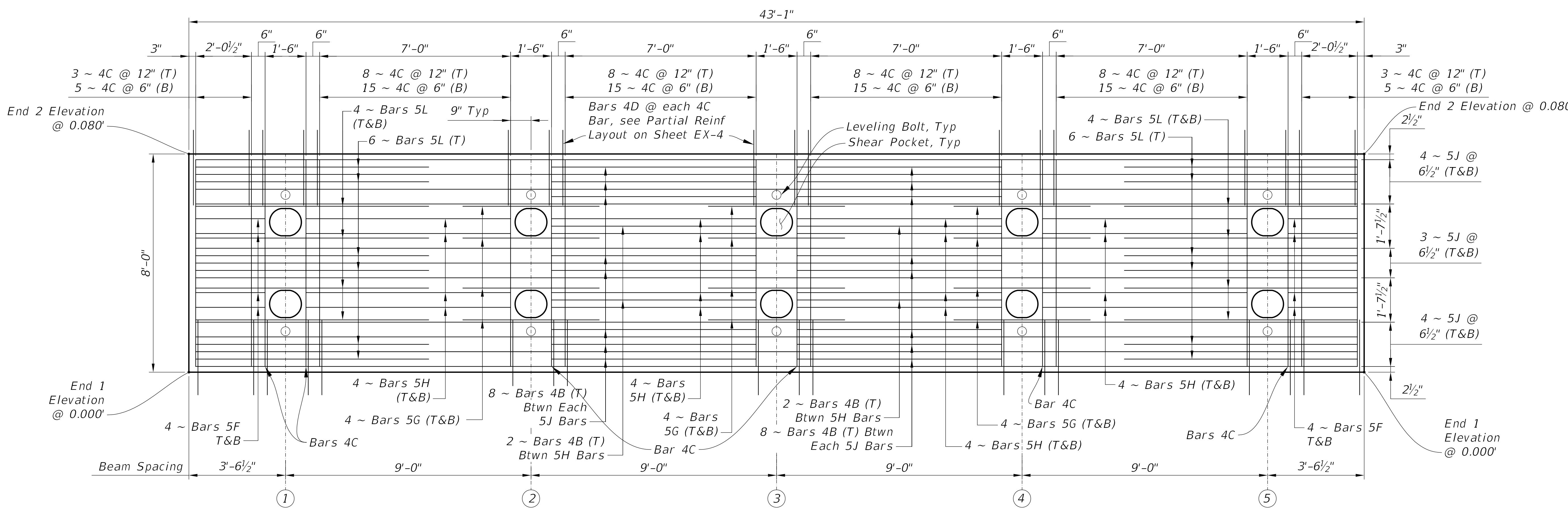
III. Appendix

A. Precast Deck Panel Drawings

Test 1 - Precast Full Depth Deck (Sheets EX-1 thru EX-4)	*Part I*
Test 2 - Precast Deck Joint (Sheets EX-5 thru EX-6)	*Part II*
Precast Deck Reinforcement Layout	*Part I*
Strain Gage Layout	*Part I*
Grout Placement and Coring Locations	*Part I*
Instrumentation Layout	*Part II*



Precast Deck Elevation



Precast Deck Plan

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

DRAWN BY: VA 06/11
CHECKED BY: GEH 06/11
DESIGNED BY: VA 06/11
CHECKED BY:

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID.

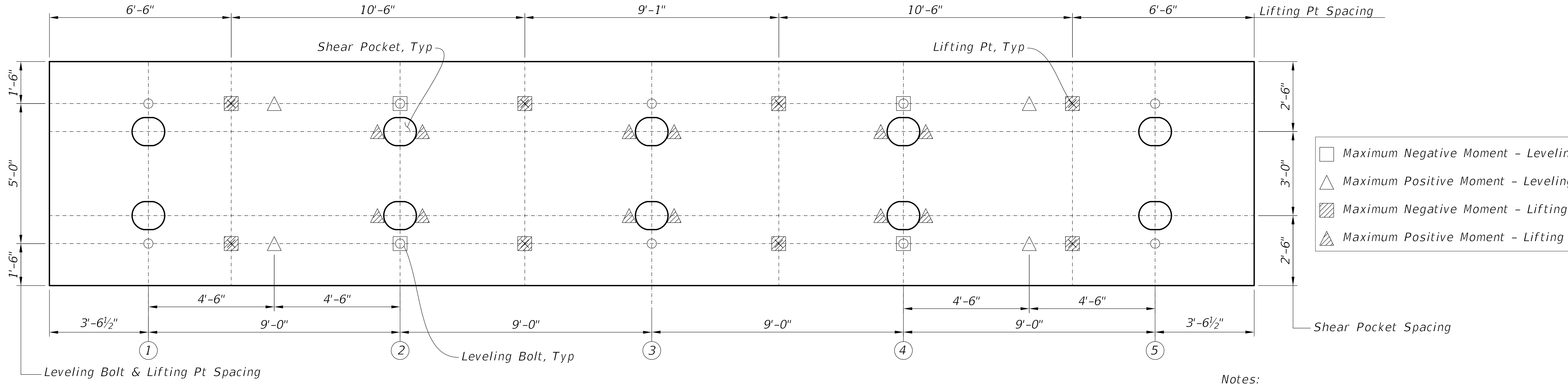
SHEET TITLE: Test 1 - Precast Full Depth Deck - Plan & Elevation

REF. DWG. NO.

PROJECT NAME: PRECAST SUPERSTRUCTURE

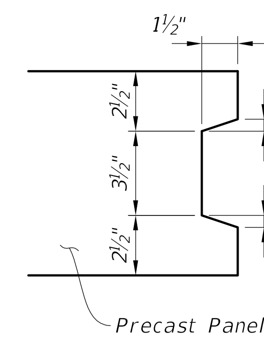
SHEET NO. EX-1

Abalo, Vickie 10/15/2012 10:27:46 AM C:\d\projects\standards\structures\current\precast_bridges\B1SuperstDet02.DWG



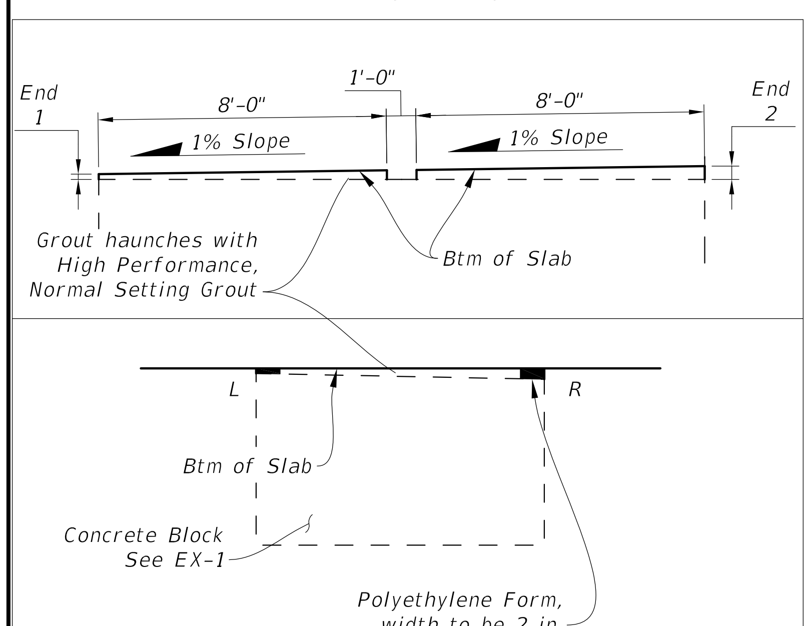
Lifting, Leveling & Shear Pocket Layout

Notes:
1. Rigging for lifting shall be assembled such that each lifting point shall not be subjected to any shear. The panel weight is approximately equally distributed to each lifting device.
2. Prior to releasing rigging verify that each leveling bolt is in contact with the beam.
3. When installing bolts, add lubricant as necessary to achieve free turning of bolt by hand.

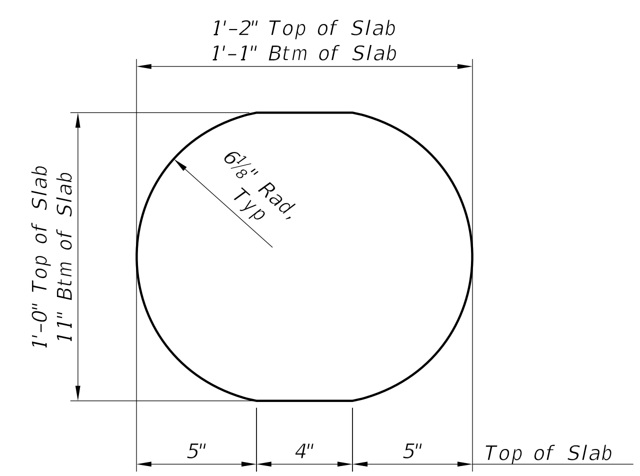


Keyway Detail

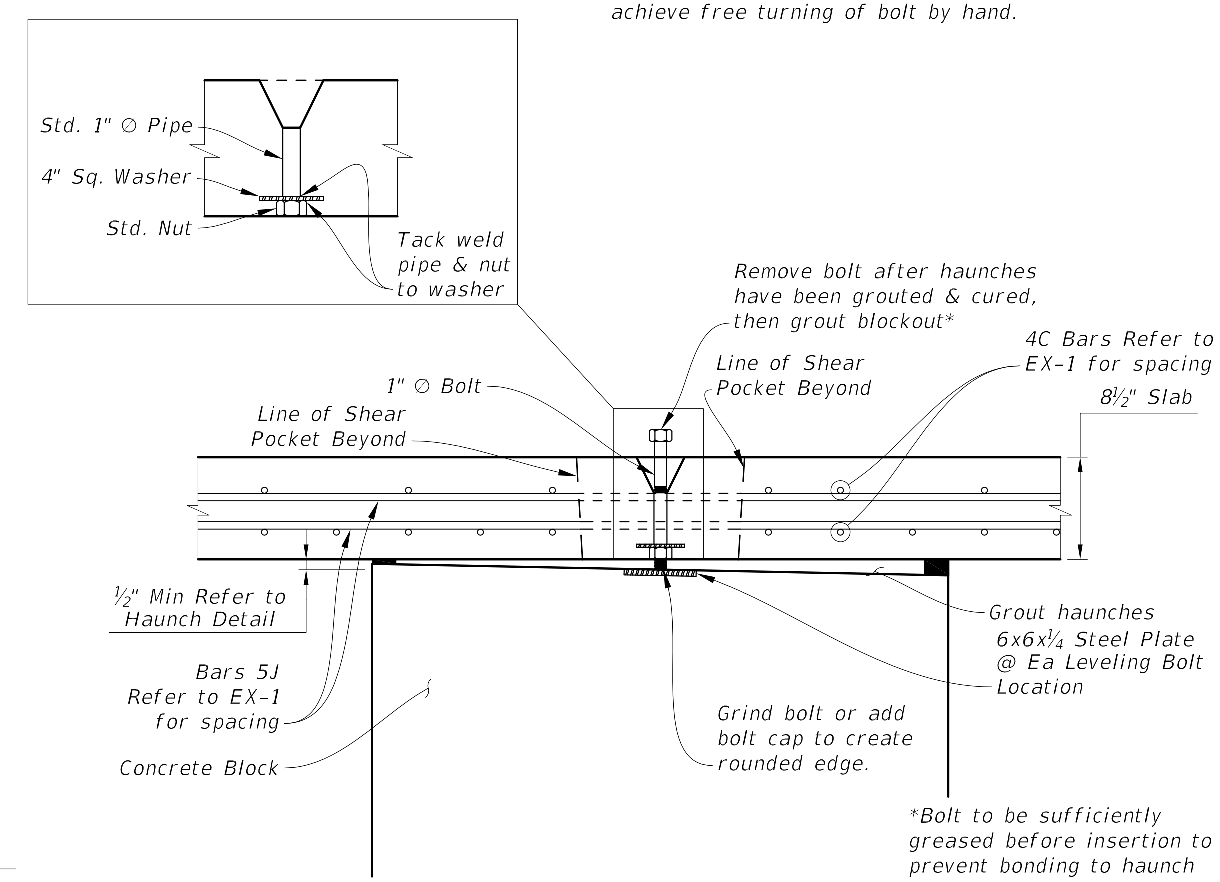
Buildup heights are variable & assume a 2% cross slope.
See attached Revision Sheet for grout heights



Haunch Detail



Shear Pocket Detail



*Bolt to be sufficiently greased before insertion to prevent bonding to haunch grout and to facilitate removal after grout reaches appropriate strength.

Leveling Bolt Detail

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CHECKED BY: GEH 06/11			
DESIGNED BY: VA 06/11	ROAD NO.	COUNTY	FINANCIAL PROJECT ID.
CHECKED BY:			

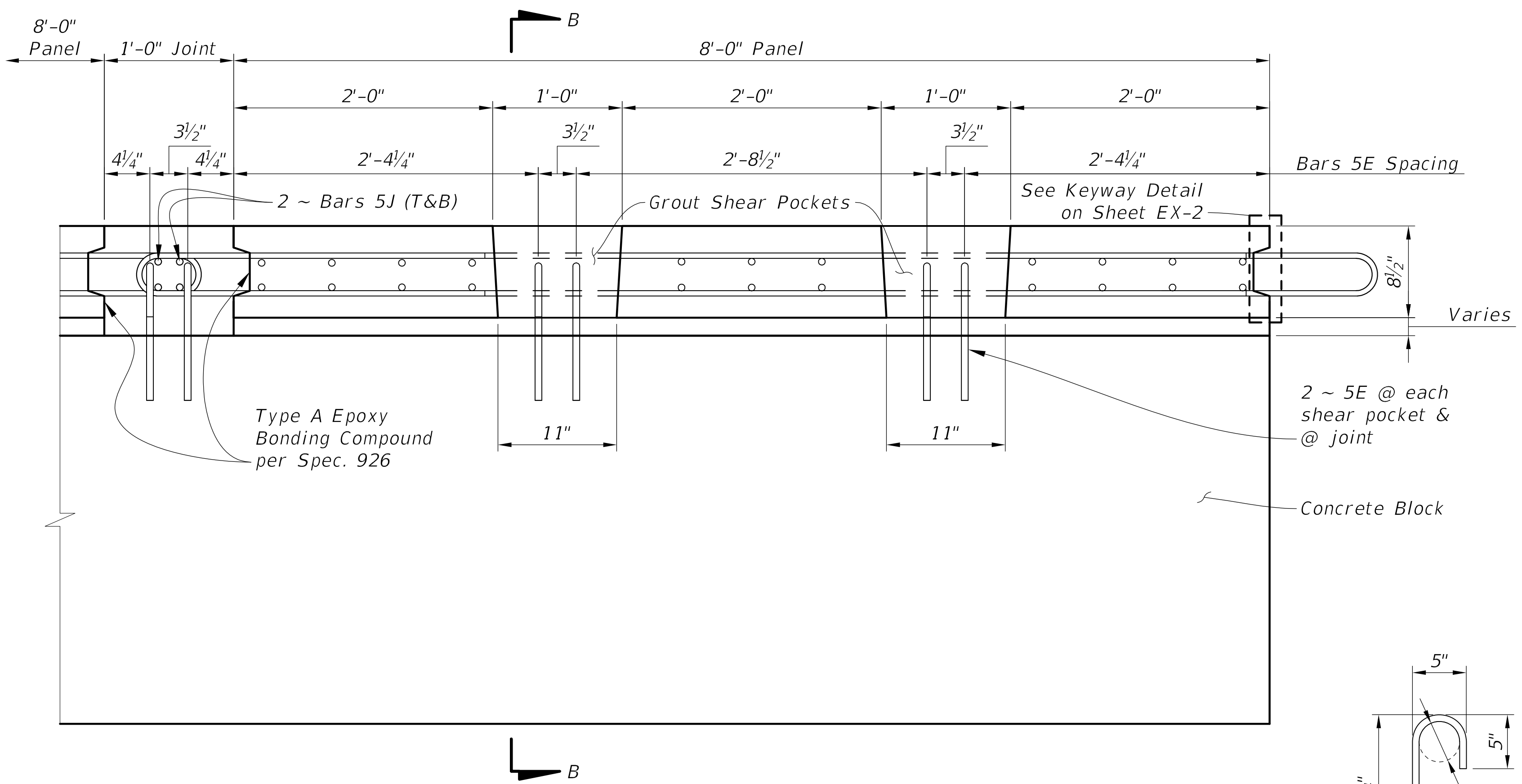
SHEET TITLE: Test 1 - Precast Full Depth Deck - Details

REF. DWG. NO.

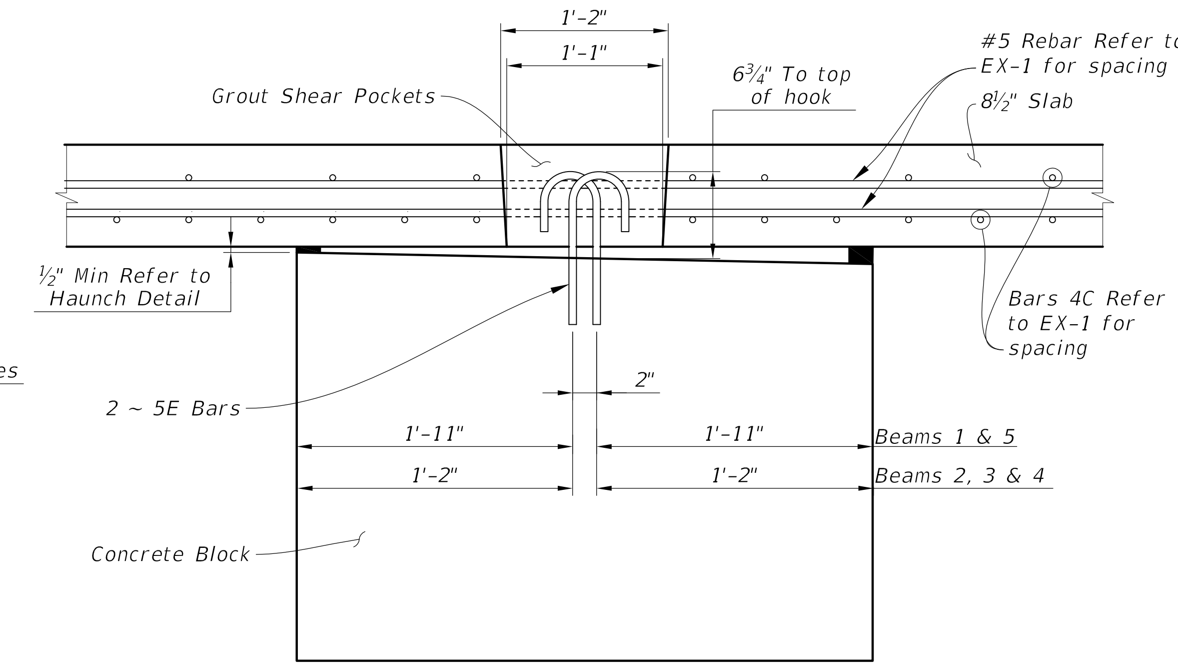
PROJECT NAME: PRECAST SUPERSTRUCTURE

SHEET NO. EX-2

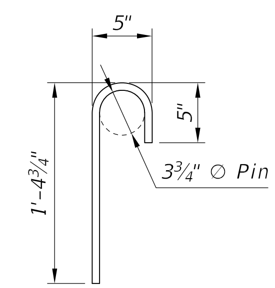
Abalo, Vickie 10/15/2012 10:28:17 AM C:\d\projects\standards\structures\current\precast_bridges\B1SuperstDet02.DWG



Section A-A

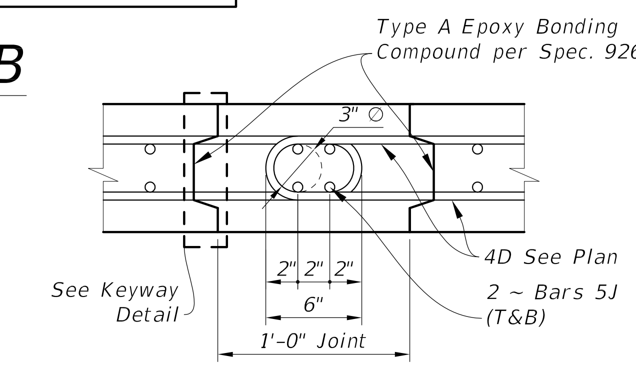


Section B-B

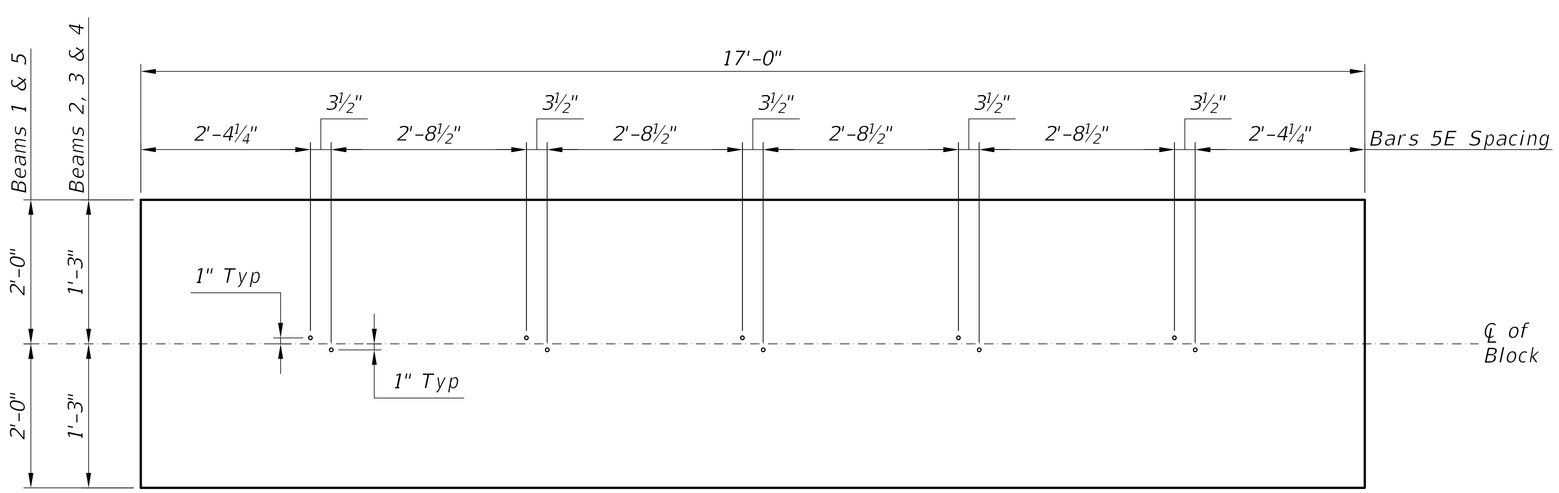


Mark	Size	No. Reqd.	Length
E	5	50	2'-1"

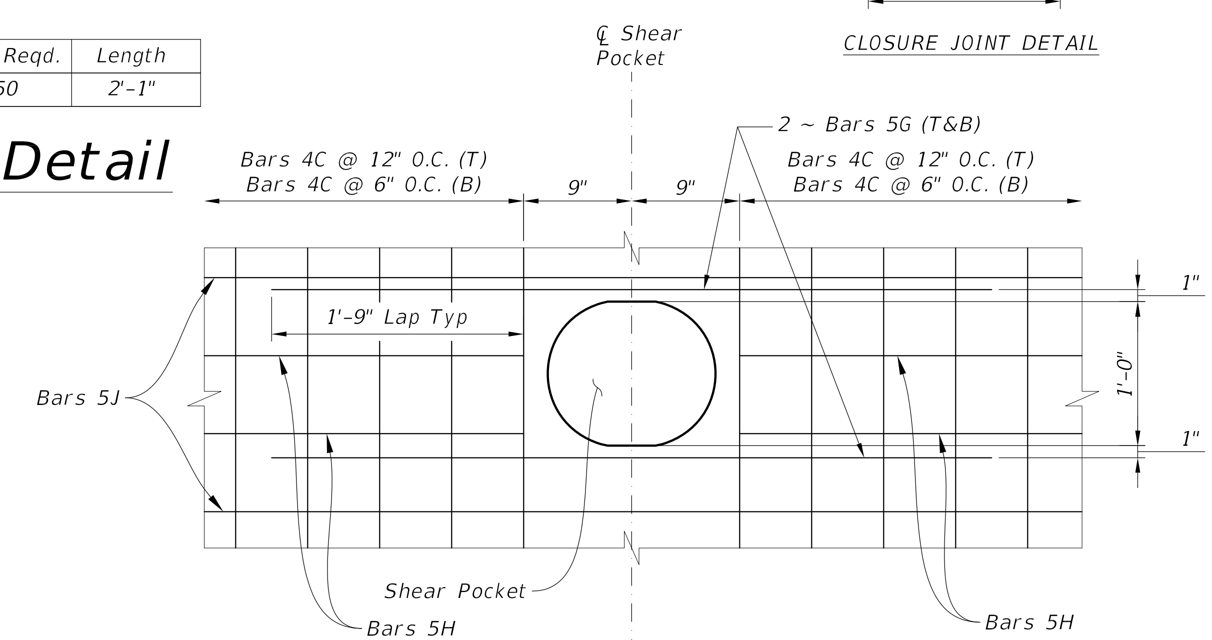
Bar 5E Detail



CLOSURE JOINT DETAIL



Typical Concrete Block Plan



Reinforcement at Shear Pockets @ Interior Beams

REVISIONS					
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ROAD NO.	COUNTY	FINANCIAL PROJECT ID.

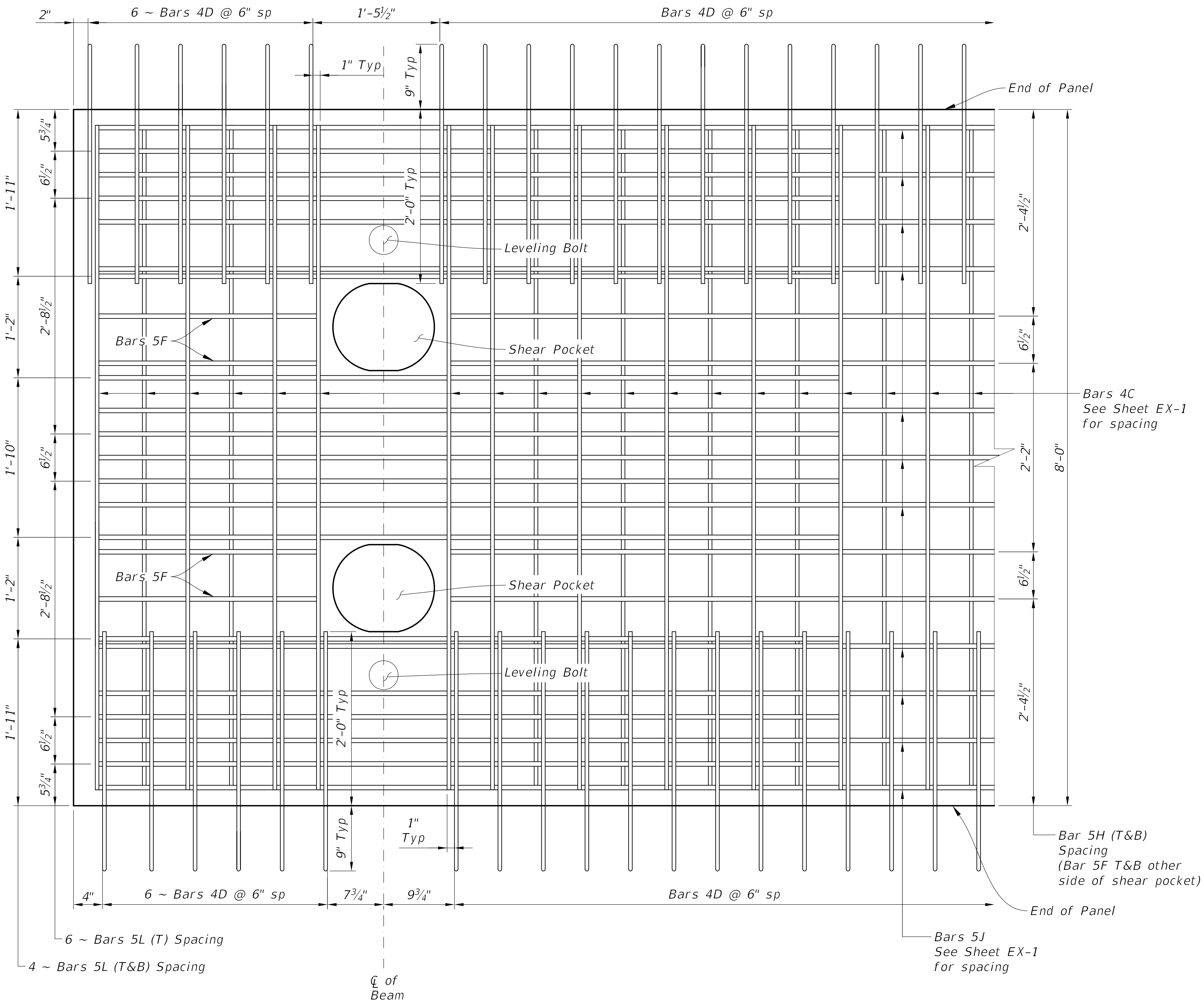
SHEET TITLE: Test 1 - Precast Full Depth Deck - Shear Pocket Details

REF. DWG. NO.

PROJECT NAME: PRECAST SUPERSTRUCTURE

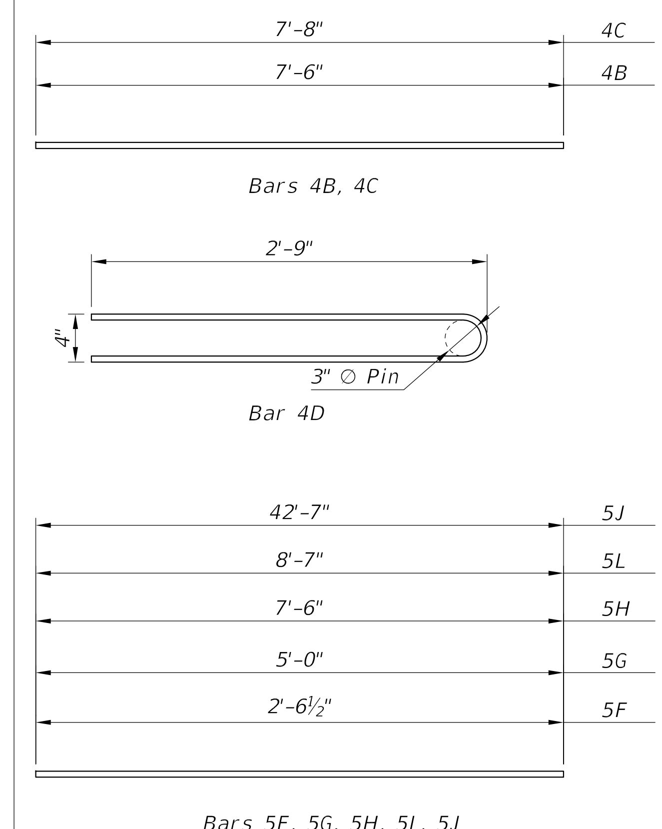
SHEET NO. EX-3

Abalo, Vickie 10/15/2012 10:28:45 AM C:\d\projects\standards\structures\current\precast_bridges\B1SuperstDet02.DWG



Precast Deck Partial Reinforcement Layout

Bill of Reinforcing Steel (Per Panel)			
Mark	Size	No. Reqd.	Length
A	Not Used		
B	4	12	7'-6"
C	4	120	7'-8"
D	4	152	5'-9"
F	5	16	2'-6½"
G	5	24	5'-0"
H	5	32	7'-6"
J	5	26	42'-7"
K	Not Used		
L	5	28	8'-7"



Reinforcing Bar List

REVISIONS					
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STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID.

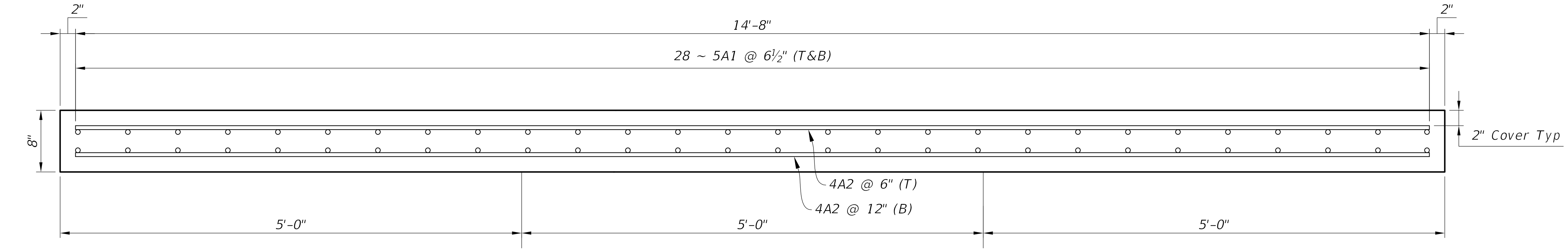
SHEET TITLE:
Test 1 - Precast Full Depth Deck - Reinforcing Details

PROJECT NAME:
PRECAST SUPERSTRUCTURE

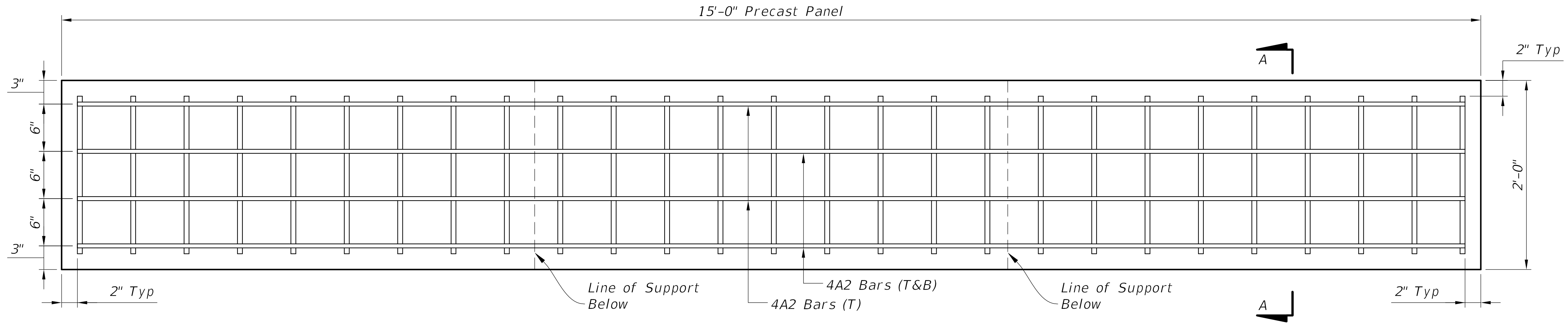
REF. DWG. NO.

SHEET NO.
EX-4

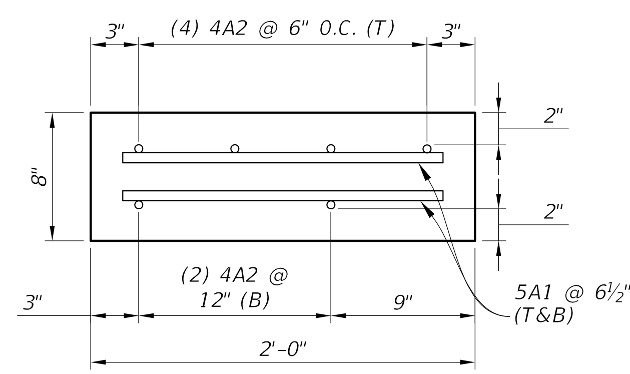
Abalo, Vickie 10/15/2012 10:29:05 AM C:\d\projects\standards\structures\current\precast_bridges\B1SuperstDet02.DWG



Elevation

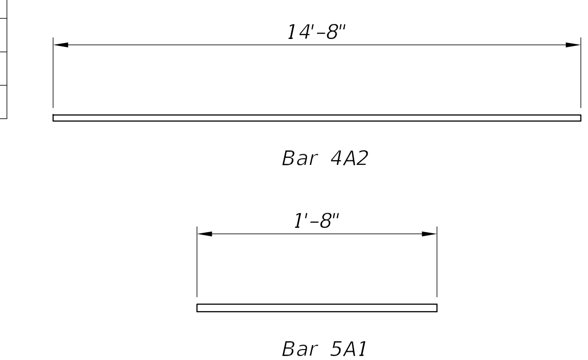


Plan



Section A-A

Bill of Reinforcing Steel Per Test			
Mark	Size	No. Reqd.	Length
A1	5	56	1'-8"
A2	4	6	14'-8"



Reinforcing Bar List

REVISIONS					
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ROAD NO.	COUNTY	FINANCIAL PROJECT ID.

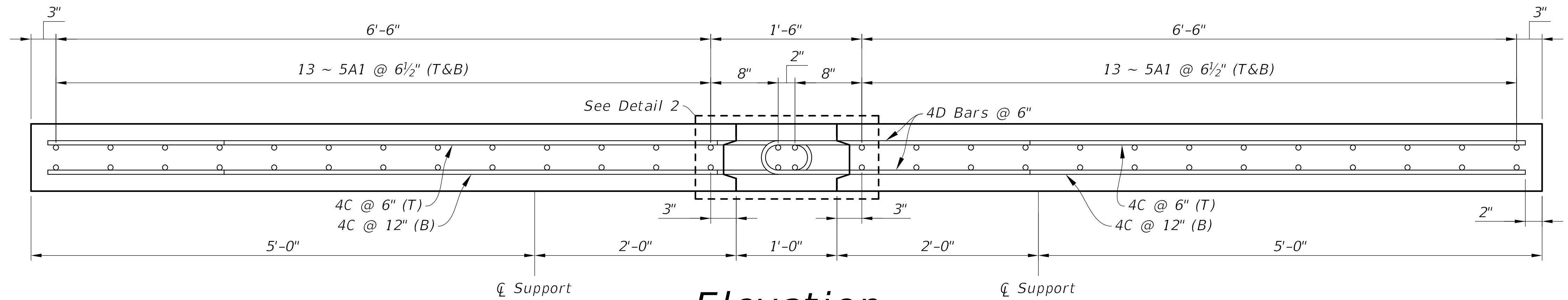
SHEET TITLE: Test 2 - Precast Deck Joint - Continuous Reinforcement (No Joint)

REF. DWG. NO.

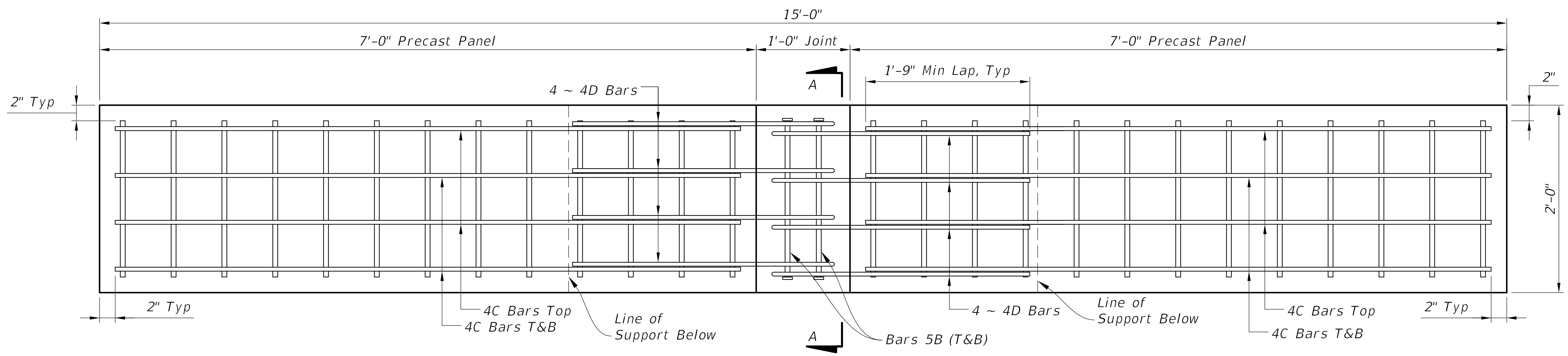
PROJECT NAME: PRECAST SUPERSTRUCTURE

SHEET NO. EX-5

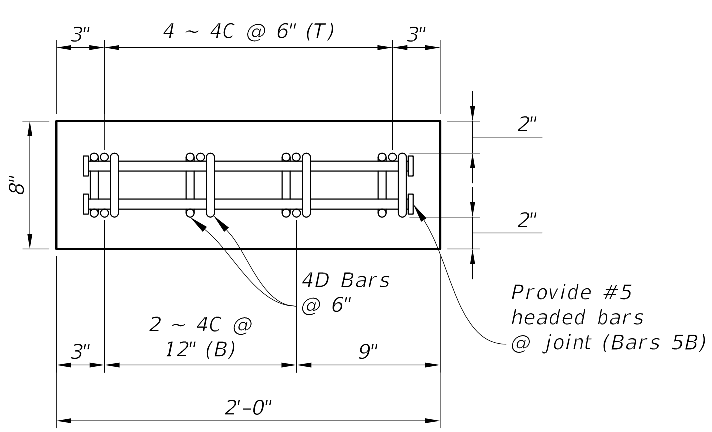
Abalo, Vickie 10/15/2012 10:29:28 AM C:\d\projects\standards\structures\current\precast_bridges\B1SuperstDet02.DWG



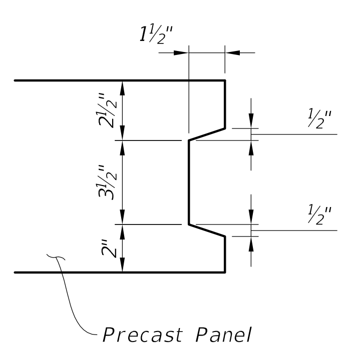
Elevation



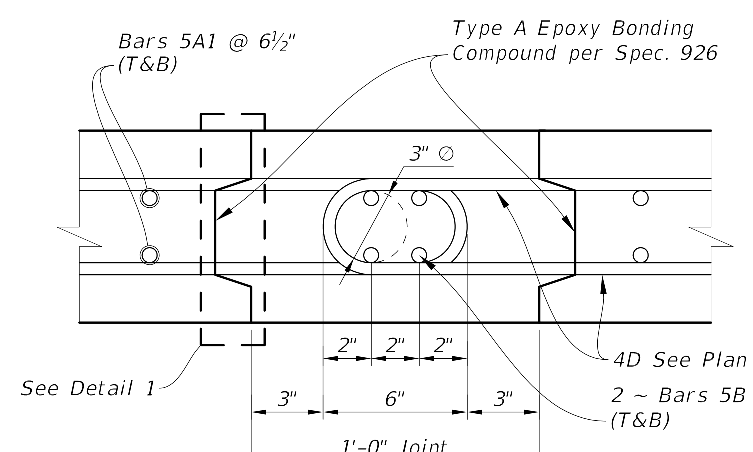
Plan



View A-A



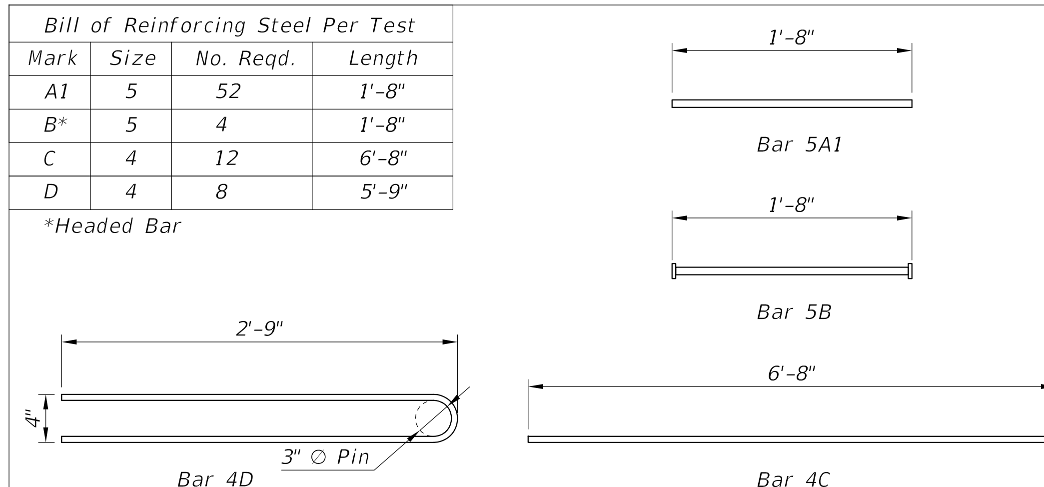
Detail 1



Detail 2

Bill of Reinforcing Steel Per Test			
Mark	Size	No. Reqd.	Length
A1	5	52	1'-8"
B*	5	4	1'-8"
C	4	12	6'-8"
D	4	8	5'-9"

*Headed Bar



Reinforcing Bar List

REVISIONS					
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ROAD NO.	COUNTY	FINANCIAL PROJECT ID.

SHEET TITLE: Test 2 - Precast Deck Joint - Lapping Hoop Joint

PROJECT NAME: PRECAST SUPERSTRUCTURE

REF. DWG. NO.

SHEET NO. EX-6

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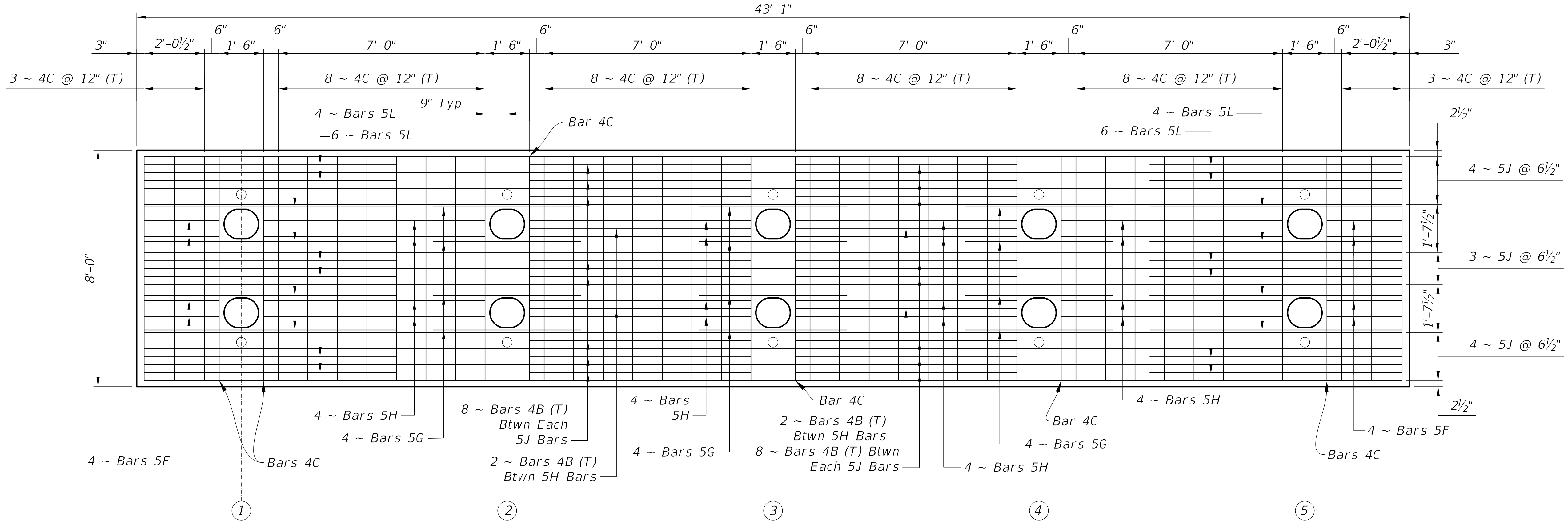
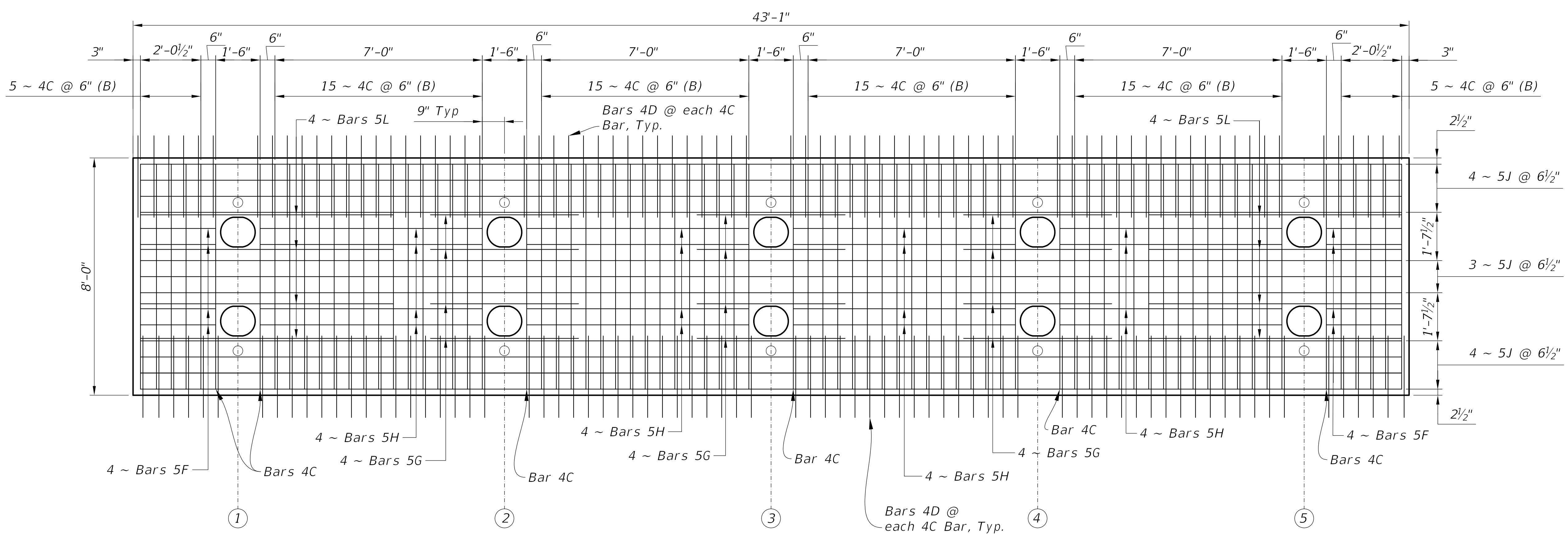
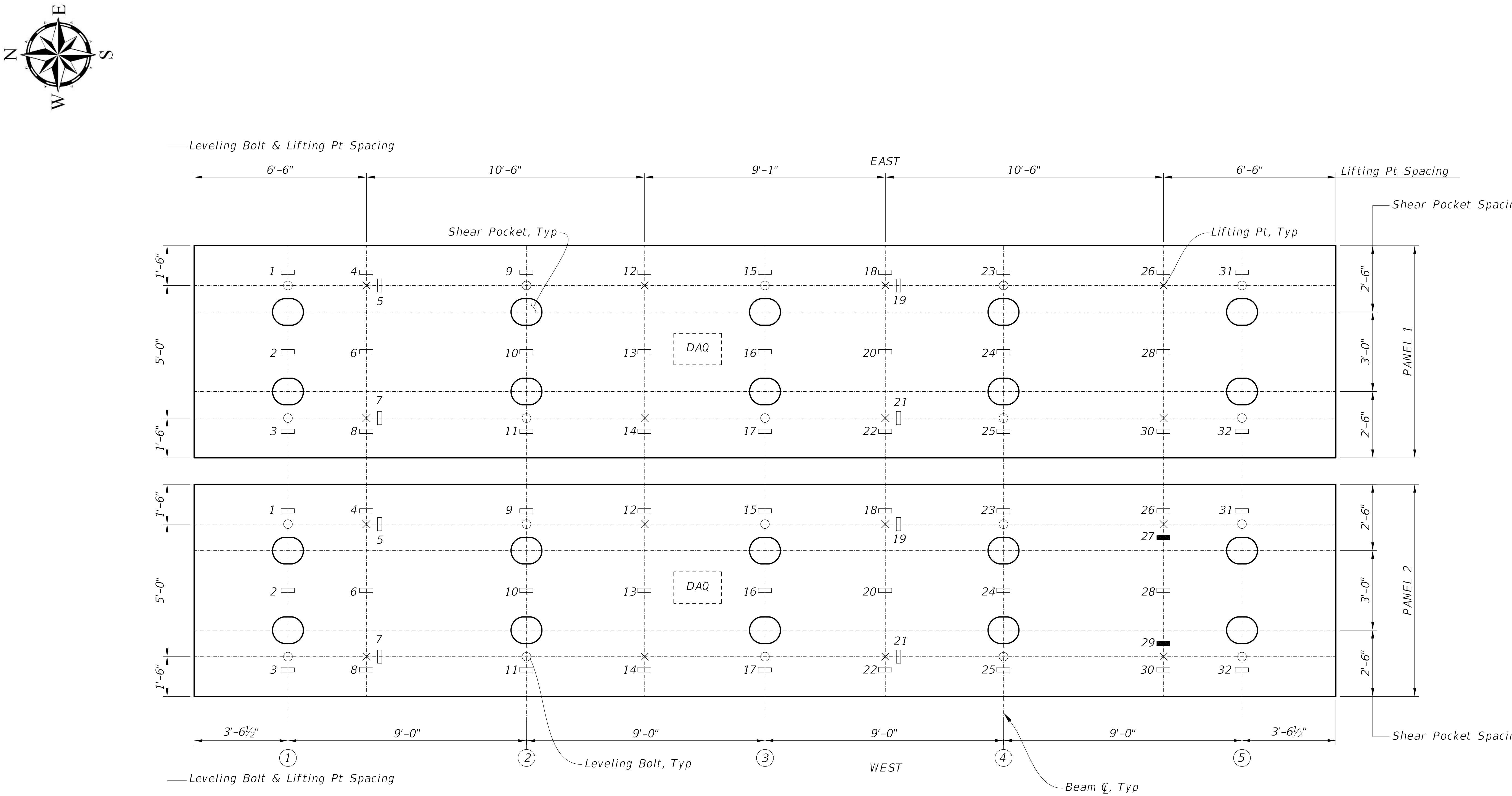


TOP MAT
(Bars 4D not shown for clarity)



BOTTOM MAT

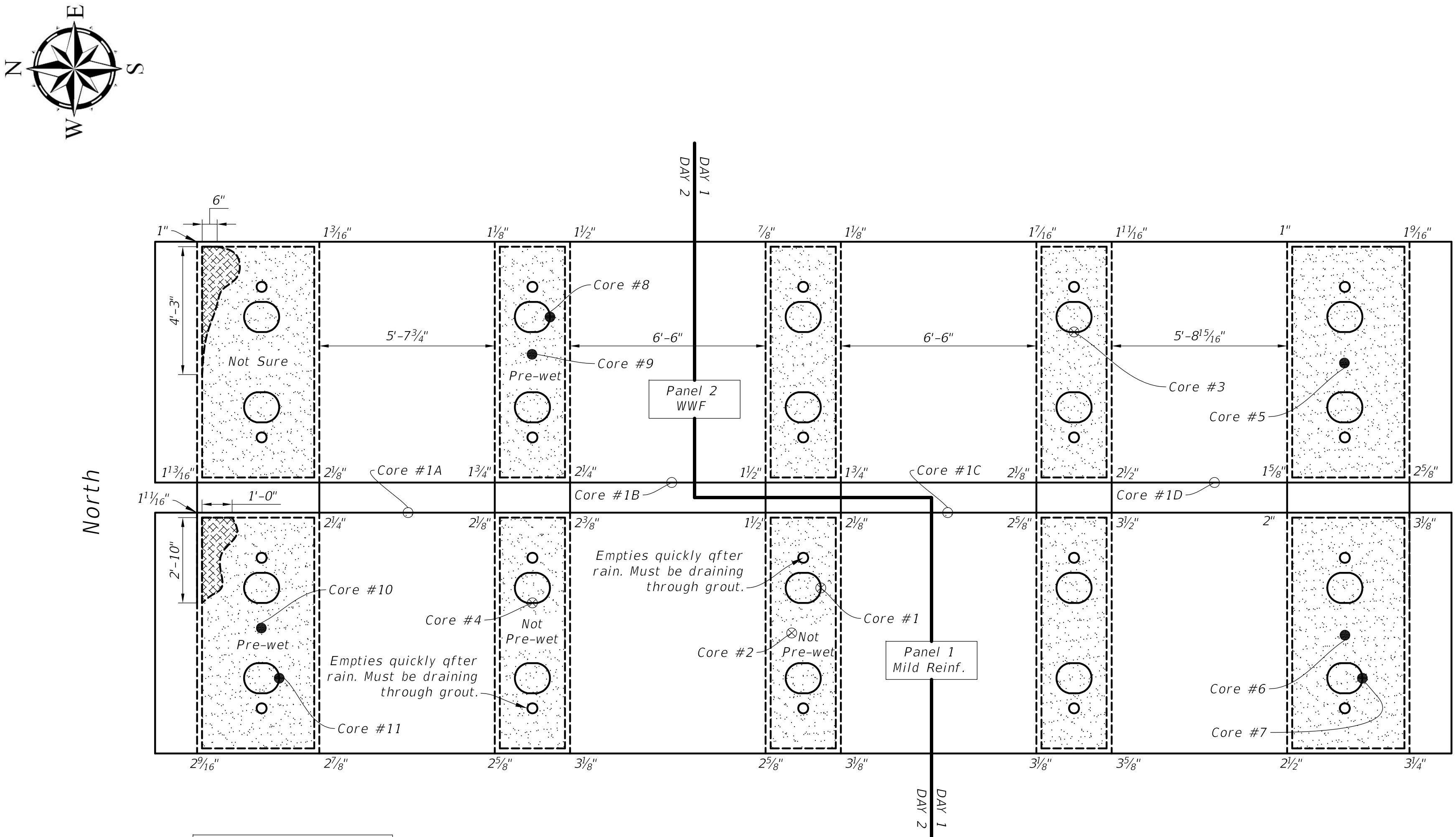
Precast Deck Reinforcement Layout

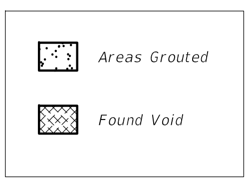


Strain Gage

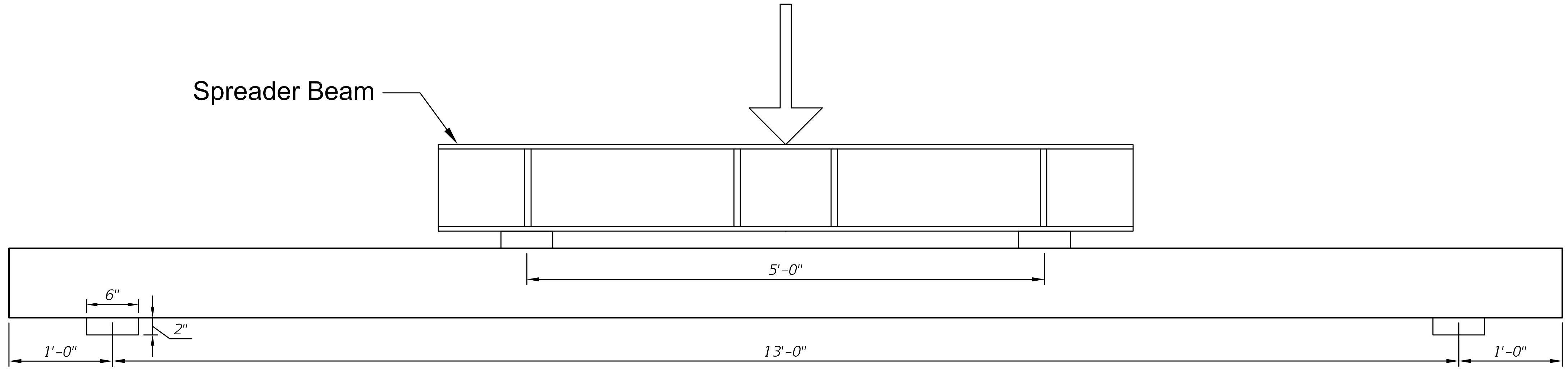
Embedded Gages on Reinforcement

STRAIN GAGE LAYOUT ON PRECAST DECK PANELS

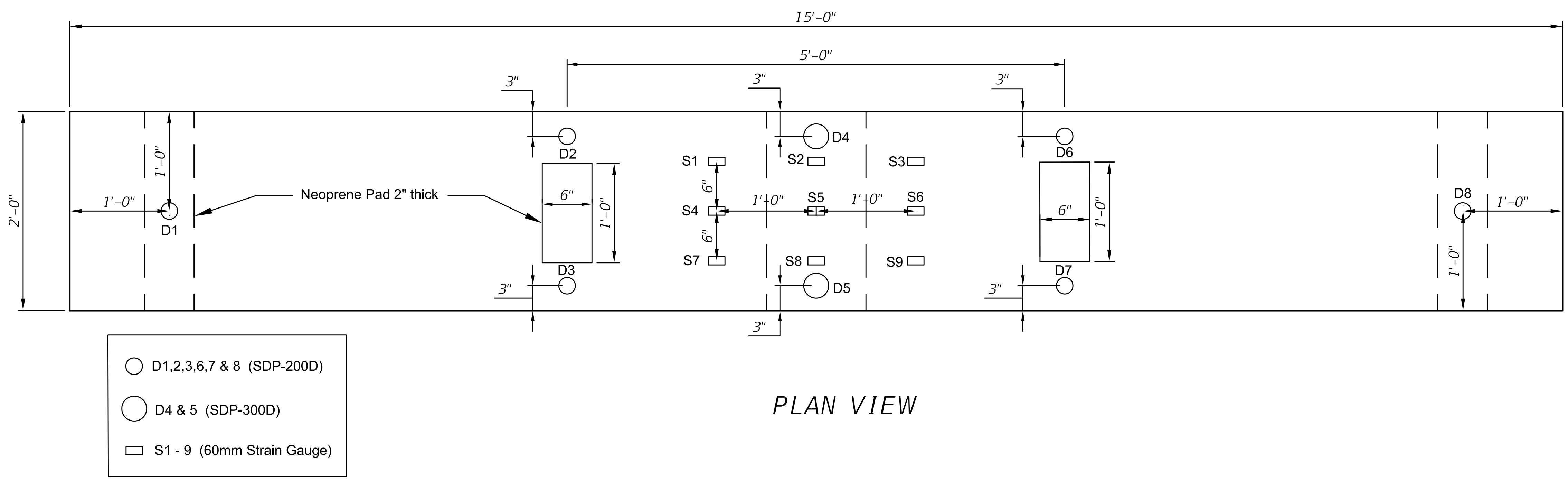




Grout Placement and Coring Locations



ELEVATION VIEW



PLAN VIEW

REVISIONS					
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DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID.

SHEET TITLE: Instrumentation Layout

PROJECT NAME: PRECAST Joint Specimens

REF. DWG. NO.

SHEET NO. 1

Abalo, Vickie 11/20/2012 9:15:44 AM S:\Public\Standards\PrecastStructures\Testing\Small Specimen Test Layout2.dwg

B. Manufacturer's Specifications

Haunch Material - Polyethylene Foam Sheet	*Part I*
Lifting Device - Dayton Superior P-94-S Fleet-Lift S-Anchor	*Part I*
Grout Material - BASF Masterflow 928	*Part I*
Small Specimen Closure Pour Concrete Mix Design - Eucon SRA	*Part II*
Small Specimen Epoxy Compound - BASF Concresive Liquid LPL	*Part II*

Foam Sheet, 1.8 lb. Poly, Blue, 2 x24x54 In

Foam Sheet, Closed Cell, Material 1.8 lb. Polyethylene, Thickness 2 In, Width 24 In, Length 54 In, Finish Coarse Cell, Color Blue, Backing Type Plain, Temp. Range -20 to 150 F, Thickness Tolerance +/-0.250 In, Width Tolerance +/-0.250 In, Length Tolerance +/-0.250 In, Tensile Strength 32 PSI, Firmness @ 25% Deflection 9 PSI, Stretch Limit 80%, Density 1.8 lbs./cu.ft.

Grainger Item #	5GDK4
Price (ea.)	**$62.40**
Brand	APPROVED VENDOR
Mfr. Model #	5GDK4
Ship Qty.	1
Sell Qty. (Will-Call)	1
Ship Weight (lbs.)	5.8
Usually Ships**	1-3 Days
Catalog Page No.	3197
Country of Origin **(Country of Origin is subject to change.)**	USA

Enlarge Image

Qty. 3

Add to Order | Add to Personal List | Compare Alternates

Price shown may not reflect your price. Sign in or register.

Tech Specs | Additional Information | Compliance & Restrictions | MSDS | Required Accessories | Optional Accessories | Alternate Products | Repair Parts

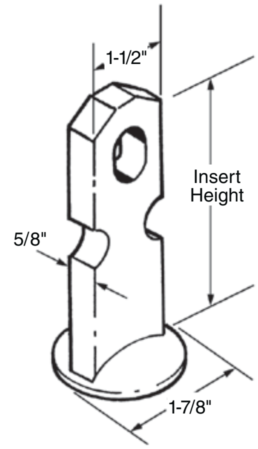
Item	Foam Sheet
Type	Closed Cell
Material	1.8 lb. Polyethylene
Thickness	2"
Width	24"
Length	54"
Finish	Coarse Cell
Color	Blue
Backing Type	Plain
Temp. Range (F)	-20 to 150
Thickness Tolerance	+/-0.250"
Width Tolerance	+/-0.250"
Length Tolerance	+/-0.250"
Tensile Strength (PSI)	32
Firmness @ 25% Deflection (PSI)	9
Stretch Limit	0.8
Density	1.8 lbs./cu. ft.
Package Quantity	1

P-94-S Fleet-Lift S-Anchor 6-Ton

The Dayton Superior P-94-S Fleet-Lift S-Anchor is a high-strength, hot forged anchor that can be "wet set" or used with the optional T-41 Plastic base in face-lift applications. When the P-94-S anchor is used with the Fleet-Lift high capacity ring clutches, safe working loads up to 12,000 pounds can be realized. See the chart below for appropriate anchor lengths and concrete compressive strengths. P-94-S anchors are available in plain and hot-dipped galvanized.

To Order:
Specify: (1) quantity, (2) name, (3) product code, (4) finish

Example:
400, P-94-S Fleet-Lift S-Anchors, FL505 with hot-dipped galvanized finish.



P-94-S Fleet-Lift S-Anchor

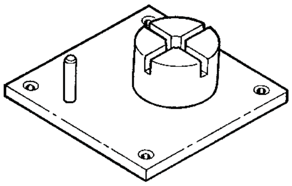
P-94-S Fleet Lift S-Anchor						
Ton x Length	Fleet Code	Product Code	Tension Safe Working Load @ 4:1 SF	Shear Safe Working Load @ 4:1 SF	Minimum Edge Distance	Minimum Corner Distance
6-ton x 3-1/8" Long	FL501	122782	3,930 lbs.	3,930 lbs.	15"	24"
6-ton x 3-7/8" Long	FL502	122784	5,000 lbs.	5,000 lbs.	18"	24"
6-ton x 4-1/8" Long	FL503	122785	5,370 lbs.	5,370 lbs.	18"	27"
6-ton x 4-7/8" Long	FL505	122788	6,560 lbs.	6,560 lbs.	20"	28"
6-ton x 5-1/8" Long	FL506	122789	6,970 lbs.	6,970 lbs.	21"	30"
6-ton x 5-7/8" Long	FL509	122792	8,250 lbs.	8,250 lbs.	24"	34"
6-ton x 6-1/8" Long	FL510	122793	8,700 lbs.	8,700 lbs.	24"	36"
6-ton x 6-7/8" Long	FL513	122796	10,070 lbs.	10,070 lbs.	26"	38"
6-ton x 7-7/8" Long	FL515	122800	12,000 lbs.	12,000 lbs.	30"	42"

Safe working load is based on an approximate factor of safety of 4 to 1 in 3,500 psi normal weight concrete
Use FL062 Recess Member and FL002S Ring Clutch (both 4-6T) with these anchors.

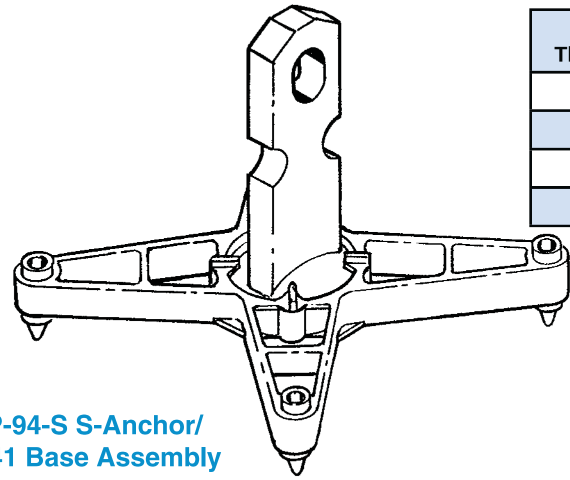
Optional T-41 Plastic Base

The T-41 Plastic Base is available to facilitate setting the P-94-S anchor in back-strip applications. The insert and base assembly may be ordered pre-assembled from the factory, or the plastic base can be ordered separately for field or shop installation. The insert and base can be assembled by hand, with a simple assembly fixture available from Miamisburg. The P-99-D Disposable Recess Member (part #FL0675) or T-90 Two-Part Recess Member (part #122024) can be used in this application.

Panel Thickness	P-94-S Length	P-94-S Part #
5"	3-7/8"	FL502
6"	4-7/8"	FL505
7"	5-7/8"	FL509
8"	6-7/8"	FL513



Assembly Fixture



P-94-S S-Anchor/
T-41 Base Assembly

BASF

The Chemical Company

PRODUCT DATA

3 03 62 13 **Non-Metallic Non-Shink Grouting**

MASTERFLOW® 928

High-precision mineral-aggregate grout with extended working time

Description

Masterflow® 928 grout is a hydraulic cement-based mineral-aggregate grout with an extended working time. It is ideally suited for grouting machines or plates requiring precision load-bearing support. It can be placed from fluid to damp pack over a temperature range of 45 to 90° F (7 to 32° C). Masterflow® 928 grout meets the requirements of ASTM C1107 and US Army Corps of Engineers CRD C621 (ASTM C1107-91a, Grades B and C), at a fluid consistency over a 30-minute working time and ANSI/NSF 61 approved Suitable for use with potable water.

Yield

One 55 lb (25 kg) bag of Masterflow® 928 grout mixed with approximately 10.5 lbs (4.8 kg) or 1.26 gallons (4.8 L) of water, yields approximately 0.50 ft^3 (0.014 m^3) of grout.

The water requirement may vary due to mixing efficiency, temperature, and other variables.

Packaging

55 lb (25 kg) multi-wall paper bags

3,300 lb (1,500 kg) bulk bags

Shelf Life

1 year when properly stored

Storage

Store in unopened bags in clean, dry conditions.

Features	Benefits
• Extended working time	Ensures sufficient time for placement
• Can be mixed at a wide range of consistencies	Ensures proper placement under a variety of conditions
• Freeze/thaw resistant	Suitable for exterior applications
• Hardens free of bleeding, segregation, or settlement shrinkage	Provides a maximum effective bearing area for optimum load transfer
• Contains high-quality, well-graded quartz aggregate	Provides optimum strength and workability
• Sulfate resistant	For marine, wastewater, and other sulfate-containing environments
• ANSI / NSF 61 approved	Suitable for use with potable water

Where to Use

INDUSTRIES

- Power generation
- Pulp and paper mills
- Steel and cement mills
- Stamping and machining
- Water and waste treatment
- General construction

APPLICATIONS

- Where a nonshrink grout is required for maximum effective bearing area for optimum load transfer
- Where high one-day and later-age compressive strengths are required
- Applications requiring a pumpable grout
- Compressors and generators
- Pump bases and drive motors
- Tank bases
- Conveyors
- Grouting anchor bolts, rebar and dowel rods
- Nonshrink grouting of precast well panels, beams, columns, curtain walls, concrete systems and other structural and non-structural building components
- Repairing concrete, including grouting voids and rock pockets

LOCATIONS

- Interior or exterior
- Marine applications
- Freeze/thaw environments

How to Apply

Surface Preparation

1. Steel surfaces must be free of dirt, oil, grease, or other contaminants.

2. The surface to be grouted must be clean, SSD, strong, and roughened to a CSP of 5 – 9 following ICRI Guideline 03732 to permit proper bond. For freshly placed concrete, consider using Liquid Surface Etchant (see Form No. 1020198) to achieve the required surface profile.

3. When dynamic, shear or tensile forces are anticipated, concrete surfaces should be chipped with a “chisel-point” hammer, to a roughness of (plus or minus) 3/8" (10 mm). Verify the absence of bruising following ICRI Guideline 03732.

4. Concrete surfaces should be saturated (ponded) with clean water for 24 hours just before grouting.

5. All freestanding water must be removed from the foundation and bolt holes immediately before grouting.

Protection and Repair

Technical Data

Composition

Masterflow® 928 is a hydraulic cement-based mineral-aggregate grout.

Compliances

- ASTM C1107 US Army Corps of Engineers CRD C621 (ASTM C1107-93a, Grades B and C), requirements at a fluid consistency over a temperature range of 45 to 90° F (7 to 32° C)
- City of Los Angeles Research Report Number RR 23137
- ANSI / NSF 61 for use with potable water

Test Data

PROPERTY	RESULTS			TEST METHODS
Compressive strengths, psi (MPa)				ASTM C 942, according to ASTM C 1107
	Plastic[1]	**Consistency Flowable**[2]	**Fluid**[3]	
1 day	4,500 (31)	4,000 (28)	3,500 (24)	
3 days	6,000 (41)	5,000 (34)	4,500 (31)	
7 days	7,500 (52)	6,700 (46)	6,500 (45)	
28 days	9,000 (62)	8,000 (55)	7,500 (52)	
Volume change*				ASTM C 1090
	% Change	**% Requirement of ASTM C 1107**		
1 day	> 0	0.0 – 0.30		
3 days	0.04	0.0 – 0.30		
14 days	0.05	0.0 – 0.30		
28 days	0.06	0.0 – 0.30		
Setting time, hr:min				ASTM C 191
	Plastic[1]	**Consistency Flowable**[2]	**Fluid**[3]	
Initial set	2:30	3:00	4:30	
Final set	4:00	5:00	6:00	
Flexural strength,* psi (MPa)				ASTM C 78
3 days	1,000 (6.9)			
7 days	1,050 (7.2)			
28 days	1,150 (7.9)			
Modulus of elasticity,* psi (MPa)				ASTM C 469, modified
3 days	2.82×10^6 (1.94×10^4)			
7 days	3.02×10^6 (2.08×10^4)			
28 days	3.24×10^6 (2.23×10^4)			
Coefficient of thermal expansion,* in/in/° F (mm/mm/° C)	6.5×10^{-6} (11.7×10^{-6})			ASTM C 531
Split tensile and tensile strength,* psi (MPa)				ASTM C 496 (splitting tensile) ASTM C 190 (tensile)
	Splitting Tensile	**Tensile**		
3 days	575 (4.0)	490 (3.4)		
7 days	630 (4.3)	500 (3.4)		
28 days	675 (4.7)	500 (3.4)		
Punching shear strength,* psi (MPa), 3 by 3 by 11" (76 by 76 by 279 mm) beam				BASF Method
3 days	2,200 (15.2)			
7 days	2,260 (15.6)			
28 days	2,650 (18.3)			
Resistance to rapid freezing and thawing	300 Cycles RDF 99%			ASTM C 666, Procedure A

[1]100 – 125% flow on flow table per ASTM C 230

[2]125 – 145% flow on flow table per ASTM C 230

[3]25 to 30 seconds through flow cone per ASTM C 939

*Test conducted at a fluid consistency

Test results are averages obtained under laboratory conditions. Expect reasonable variations.

Test Data, continued

PROPERTY	RESULTS	TEST METHODS
Ultimate tensile strength and bond stress		ASTM E 488, tests*

Diameter in (mm)	Depth in (mm)	Tensile strength lbs (kg)	Bond stress psi (MPa)
5/8 (15.9)	4 (101.6)	23,500 (10,575)	2,991 (20.3)
3/4 (19.1)	5 (127.0)	30,900 (13,905)	2,623 (18.1)
1 (25.4)	6.75 (171.5)	65,500 (29,475)	3,090 (21.3)

*Average of 5 tests in 4,000 psi (27.6 MPa) concrete using 125 ksi threaded rod in 2" (51mm) diameter, damp, core-drilled holes.

Notes:

1. Grout was mixed to a fluid consistency.
2. Recommended design stress: 2,275 psi (15.7 MPa).
3. Refer to the “Adhesive and Grouted Fastener Capacity Design Guidelines” for more detailed information.
4. Tensile tests with headed fasteners were governed by concrete failure.

Jobsite Testing

If strength tests must be made at the jobsite, use 2" (51 mm) metal cube molds as specified by ASTM C 942 and ASTM C 1107. DO NOT use cylinder molds. Control field and laboratory tests on the basis of desired placement consistency rather than strictly on water content.

6. Anchor bolt holes must be grouted and sufficiently set before the major portion of the grout is placed.

7. Shade the foundation from sunlight 24 hours before and 24 hours after grouting.

Forming

1. Forms should be liquid tight and nonabsorbent. Seal forms with putty, sealant, caulk, polyurethane foam.

2. Moderately sized equipment should utilize a head form sloped at 45 degrees to enhance the grout placement. A moveable head box may provide additional head at minimum cost.

3. Side and end forms should be a minimum 1" (25 mm) distant horizontally from the object grouted to permit expulsion of air and any remaining saturation water as the grout is placed.

4. Leave a minimum of 2" between the bearing plate and the form to allow for ease of placement.

5. Use sufficient bracing to prevent the grout from leaking or moving.

6. Eliminate large, nonsupported grout areas wherever possible.

7. Extend forms a minimum of 1" (25 mm) higher than the bottom of the equipment being grouted.

8. Expansion joints may be necessary for both indoor and outdoor installation. Consult your local BASF field representative for suggestions and recommendations.

Temperature

1. For precision grouting, store and mix grout to produce the desired mixed-grout temperature. If bagged material is hot, use cold water, and if bagged material is cold, use warm water to achieve a mixed-product temperature as close to 70° F (21°C) as possible.

Recommended Temperature Guidelines for Precision Grouting

	MINIMUM ˚F (˚C)	PREFERRED ˚F (˚C)	MAXIMUM ˚F (˚C)
Foundation and plates	45 (7)	50 – 80 (10 – 27)	90 (32)
Mixing water	45 (7)	50 – 80 (10 – 27)	90 (32)
Grout at mixed and placed temp	45 (7)	50 – 80 (10 – 27)	90 (32)

2. If temperature extremes are anticipated or special placement procedures are planned, contact your local BASF representative for assistance.

3. When grouting at minimum temperatures, see that the foundation, plate, and grout temperatures do not fall below 40° F (7° C) until after final set. Protect the grout from freezing (32° F or 0° C) until it has attained a compressive strength of 3,000 psi (21 MPa).

Mixing

1. Add the minimum potable or ASTM C1602-compliant water to the mixer, then slowly add the Masterflow 928, while mixing.

2. Masterflow 928 water requirements depend on the desired consistency, mixing efficiency, material and ambient temperature conditions. Begin with the minimum water listed (See table below), and gradually add additional water while mixing until the desired placement consistency is reached.

Consistency	Test Method	Results	Minimum Water per Bag
Plastic	ASTM C230	100-125%	6.0 lbs
Flowable	ASTM C230	125-145%	8.0 lbs
Fluid	ASTM C939	25-30 sec	9.0 lbs

3. Do not use water in an amount or at a temperature that will produce an ASTM C939 initial flow of less than 25 seconds, or cause mixed grout to bleed or segregate.

4. Moderately sized batches of grout are best mixed in one or more clean mortar mixers. For large batches, use ready-mix trucks and 3,300 lb (1,500 kg) bags for maximum efficiency and economy.

5. Mix grout a minimum of 5 minutes after all material and water is in the mixer. Use mechanical mixer only.

6. Do not mix more grout than can be placed in approximately 30 minutes.

7. Transport by wheelbarrow or buckets or pump to the equipment being grouted. Minimize the transporting distance.

8. Do not retemper grout by adding water and remixing after it stiffens.

9. DO NOT VIBRATE GROUT TO FACILITATE PLACEMENT.

10. For aggregate extension guidelines, refer to Appendix MB-10: Guide to Cementitious Grouting.

Application

1. Always place grout from only one side of the equipment to prevent air or water entrapment beneath the equipment. Place Masterflow® 928 in a continuous pour. Discard grout that becomes unworkable. Make sure that the material fills the entire space being grouted and that it remains in contact with plate throughout the grouting process.

2. Immediately after placement, trim the surfaces with a trowel and cover the exposed grout with clean wet rags (not burlap). Keep rags moist until grout surface is ready for finishing or until final set.

3. The grout should offer stiff resistance to penetration with a pointed mason's trowel before the grout forms are removed or excessive grout is cut back. After removing the damp rags, immediately coat with a recommended curing compound com-pliant with ASTM C 309 or preferably ASTM C 1315.

4. Do not vibrate grout. Use steel straps inserted under the plate to help move the grout.

5. Consult your BASF representative before placing lifts more than 6" (152 mm) in depth.

Curing

Cure all exposed grout with an approved membrane curing compound compliant with ASTM C 309 or preferably ASTM C 1315. Apply curing compound immediately after the wet rags are removed to minimize potential moisture loss.

For Best Performance

- For guidelines on specific anchor-bolt applications, contact BASF Technical Service.
- Do not add plasticizers, accelerators, retarders, or other additives unless advised in writing by BASF Technical Service.
- The water requirement may vary with mixing efficiency, temperature, and other variables.
- Hold a pre-job conference with your local representative to plan the installation. Hold conferences as early as possible before the installation of equipment, sole plates, or rail mounts. Conferences are important for applying the recommendations in this product data sheet to a given project, and they help ensure a placement of highest quality and lowest cost.
- The ambient and initial temperature of the grout should be in the range of 45 to 90° F (7 to 32° C) for both mixing and placing. Ideally the amount of mixing water used should be that which is necessary to achieve a 25 – 30 second flow according to ASTM C 939 (CRD C 611). For placement outside of the 45 to 90° F (7 to 32° C) range, contact your local BASF representative.
- For pours greater than 6" (152 mm) deep, consult your local BASF representative for special precautions and installation procedures.
- Use Embeco® 885 grout for dynamic load-bearing support and similar application conditions as Masterflow® 928.
- Use Masterflow® 816, Masterflow® 1205, or Masterflow® 1341 post-tensioning cable grouts when the grout will be in contact with steel stressed over 80,000 psi (552 MPa).
- Masterflow® 928 is not intended for use as a floor topping or in large areas with exposed shoulders around baseplates. Where grout has exposed shoulders, occasional hairline cracks may occur. Cracks may also occur near sharp corners of the baseplate and at anchor bolts. These superficial cracks are usually caused by temperature and moisture changes that affect the grout at exposed shoulders at a faster rate than the grout beneath the baseplate. They do not affect the structural, nonshrink, or vertical support provided by the grout if the foundation-preparation, placing, and curing procedures are properly carried out.
- The minimum placement depth is 1" (25 mm).
- Make certain the most current versions of product data sheet and MSDS are being used; call Customer Service (1-800-433-9517) to verify the most current version.
- Proper application is the responsibility of the user. Field visits by BASF personnel are for the purpose of making technical recommendations only and not for supervising or providing quality control on the jobsite.

Health and Safety

MASTERFLOW® 928

WARNING!

Masterflow® 928 contains silica, crystalline quartz; portland cement; limestone; calcium oxide; gypsum; silica, amorphous; magnesium oxide.

Risks

Product is alkaline on contact with water and may cause injury to skin or eyes. Ingestion or inhalation of dust may cause irritation. Contains small amount of free respirable quartz which has been listed as a suspected human carcinogen by NTP and IARC. Repeated or prolonged overexposure to free respirable quartz may cause silicosis or other serious and delayed lung injury.

Precautions

Avoid contact with skin, eyes and clothing. Prevent inhalation of dust. Wash thoroughly after handling. Keep container closed when not in use. DO NOT take internally. Use only with adequate ventilation. Use impervious gloves, eye protection and if the TLV is exceeded or used in a poorly ventilated area, use NIOSH/MSHA approved respiratory protection in accordance with applicable Federal, state and local regulations.

First Aid

In case of eye contact, flush thoroughly with water for at least 15 minutes. In case of skin contact, wash affected areas with soap and water. If irritation persists, SEEK MEDICAL ATTENTION. Remove and wash contaminated clothing. If inhalation causes physical discomfort, remove to fresh air. If discomfort persists or any breathing difficulty occurs or if swallowed, SEEK IMMEDIATE MEDICAL ATTENTION.

Waste Disposal Method

This product when discarded or disposed of, is not listed as a hazardous waste in federal regulations. Dispose of in a landfill in accordance with local regulations.

For additional information on personal protective equipment, first aid, and emergency procedures, refer to the product Material Safety Data Sheet (MSDS) on the job site or contact the company at the address or phone numbers given below.

Proposition 65

This product contains material listed by the State of California as known to cause cancer, birth defects or other reproductive harm.

VOC Content

0 g/L or 0 lbs/gal less water and exempt solvents.

For medical emergencies only,
call ChemTrec (1-800-424-9300).

BASF Construction Chemicals, LLC – Building Systems

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Shakopee, MN, 55379

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Customer Service 800-433-9517
Technical Service 800-243-6739

BASF Construction Chemicals, LLC
Registered to ISO 9001:2000
Certificate No. A14263

Form No. 1019303 6/09
Printed on recycled paper including 10% post-consumer fiber.

CONCRETE MIX DESIGN

Class: II DECK | Mix Design Number: 03-1668 | Minimum Strength: 4500 psi

..fective Date: 12/21/2009 | Hot Weather? Yes | Issuer's Name: R.D. Hagen
Status: APPROVED | Project #:
Producer: A Materials Group | Plant #:

Source of Materials

Product / Product Name	Quantity	Producer / Plant #	QPL # / Spec:	SSD	FM	Geological Type
Cement: / Type II Cement	510 LB	SUWANNEE AMERICAN CEMENT - BRANFORD / CMT29	AASHTO M 85 - Type II	3.15		
Fly Ash: / Class F Fly Ash	127 LB	HEADWATERS-GASTON / FA31	ASTM C 618 - Class F	2.27		
Coarse Aggregate: / # 67 Stone	1705 LB	A GROUP CABBAGE GROVE MINE / 38627		2.50		Limestone
Fine Aggregate: / Silica Sand	1240 LB	CROWDER EXCAVATING & LAND CL, INC / 50471		2.64	2.40	Silica Sand
Air Ent Admixture: / AEA92 S	3.0 OZ	EUCLID CHEMICAL CO.	S924-0023 / AASHTO M 154 - AEA			
Type D Admixture: / Eucon WR	70.0 OZ	EUCLID CHEMICAL CO.	S924-0307 / AASHTO M 194 - Type D			
Water: / Water for Concrete	33.50 GA					
Water: / Water for Concrete	279.0 LB					

Specification Limits

..nperature	Less than or equal to 100	degree F
Slump	1.50 to 4.50	inches
Compressive Strength	Greater than or equal to 4500	avgpsi
Air Content	1.00 to 6.00	percent
W/C Ratio	Less than or equal to 0.44	LB per LB

Aggregate Correction Factor: 1.0

Producer Data

W/C Ratio	0.44	LB per LB
Theoretical Yield	27.00	CF
Temperature	97	degree F
Slump	3.50	inches
Density	143.2	LB per CF
Chloride Content	0.357	LB per CY
Air Content	3.00	percent
28 DAY	5980	avgpsi

Comments:

Mix Designer: First Name Last Name

EUCON SRA

SHRINKAGE REDUCING ADMIXTURE

Euclid Concrete Admixtures

DESCRIPTION

EUCON SRA is a ready to use liquid admixture specifically designed to reduce drying shrinkage and the potential for subsequent cracking in concrete and mortar.

EUCON SRA acts directly on the shrinkage causing mechanism at the time of the cement hydration while reducing capillary tension of pore water in concrete. This action substantially reduces the drying shrinkage. EUCON SRA does not contain added chlorides and will not promote the corrosion of steel.

PRIMARY APPLICATIONS

- Foundations
- Floors
- Silos
- Concrete pipes
- Water tanks
- Interior/Exterior
- Arenas
- Walls
- Piers
- Water purification plants
- Swimming pools
- Underground construction
- Watertight construction
- Artificial skating rinks

Note: *Do not use EUCON SRA if these structures are subjected to de-icing salts and/or salt water.*

FEATURES/BENEFITS

- Typically, shrinkage may be reduced up to 50%, depending on the cement used.
- Reduces cracking caused by shrinkage.
- Increases the life of the structure.
- Decreases expensive maintenance costs and increases the durability of the structure.

TECHNICAL INFORMATION

Specific Gravity 0.95
pH 6-7.5
Color .... Clear
Flash Point .... 220°F (104.4°C)

EUCON SRA is compatible with all cementitious materials that meet current standards and the full range of EUCLID CHEMICAL admixtures.

Consult your local Euclid Chemical representative for additional information. Trial batches are highly recommended.

PACKAGING

EUCON SRA is available in 55 gal (208 L) drums and 5 gal (18.9 L) pails.

SHELF LIFE

2 years in original, unopened container

SPECIALTY

EUCON SRA

Master Format #: 03 3000 03 4000 03 7000

The Euclid Chemical Company
19218 Redwood Rd. • Cleveland, OH 44110
Phone: [216] 531-9222 • Toll-free: [800] 321-7628 • Fax: [216] 531-9596
www.euclidchemical.com

An RPM Company

Directions for Use

Add EUCON SRA after all admixtures have been introduced into the mix. It is also recommended to allow enough mixing time before the addition of water and the superplasticizer to ensure concrete homogeneity. When an air entraining agent is used, it must be introduced with the first 50% of water and aggregates (before the introduction of the cement.) This will allow the air void system to develop before the addition of superplasticizer and EUCON SRA.

Since EUCON SRA decreases the surface tension, the dosage of the air entraining agent should be decreased in relation to concrete without EUCON SRA, to ensure proper air content. EUCON SRA has a plasticizing effect on plastic concrete, therefore, it is recommended to decrease the volume of mixing water by an equivalent amount.

To reduce shrinkage, a dosage of 1% to 2% by weight of cementitious should be used. In most cases, the recommended dosage needed to optimize the effect of EUCON SRA is 2% by weight of cementitious.

NOTE: *EUCON SRA may have a plasticizing effect on plastic concrete.*

Precautions/Limitations

Note: Because EUCON SRA decreases the surface tension of water, it may increase the air content in the mix. For this reason, the dosage rate of air entraining agents must be decreased. In the case of concrete having a water/cement ratio of 0.43 containing silica fume, the relative durability factor was 98% assuring the resistance of the concrete to freezing and thawing cycles.

- All results obtained to date, indicate that even if the concrete has good resistance to freeze and thawing cycles or a good air void system, the de-icing salts resistance according to the ASTM C 672 Standard may not be achieved.
- EUCON SRA can be used in concrete that will be subjected to freezing and thawing cycles without the presence of de-icing salts and/or salt water. Do not use in concrete that can be subjected to de-icing salts and/or salt water.
- EUCON SRA may reduce the compressive strength up to 15% depending on the concrete mix design.
- Do not allow to freeze.
- In all cases, consult the Material Safety Data Sheet before use.

Security/Handling

EUCON SRA has a flash point of 220°F (104°C). EUCON SRA must be handled with care and must not be subjected to excessive heat or in the presence of an open flame or sparks. For more information, please consult the Material Safety Data Sheet.

BASF

The Chemical Company

PRODUCT DATA

3 03 01 00 **Maintenance of Concrete**

CONCRESIVE® LIQUID LPL

Concrete bonding adhesive with long pot life

Description

Concresive® Liquid LPL is a two-component 100% solids liquid epoxy bonding adhesive. It is designed for application in warm environments or applications requiring a long working time.

Features	Benefits
• Creamy high-build liquid	Single application
• Very long working time	Facilitates proper placement; ideal for warm environments
• Moisture insensitive	Bonds to damp concrete surfaces
• May be extended with properly graded sand	More economical applications

Yield

Smooth surfaces:
100 ft²/gallon (2.4 m²/L)

Rough surfaces:
50 – 75 ft²/gallon (1.2 – 1.8 m²/L)

Coverage rates are approximate. Actual coverage rate will depend on texture and porosity of concrete and application method employed.

Packaging

1 gallon (3.8 L) units

3 gallon (11.4 L) units

Shelf Life

2 years when properly stored

Storage

Store in sealed containers at temperatures between 50 and 90° F (10 and 32° C) in a clean, dry area.

Where to Use

APPLICATION

- Bonding fresh concrete to existing concrete
- Grouting bolts, dowels, and rebar into concrete, stone, and masonry
- Filling joints and voids in masonry
- Bonding concrete to dissimilar materials like steel and wood
- Coating rebar

LOCATION

- Interior or exterior

How to Apply

Surface Preparation

CONCRETE

1. Substrate may be dry or damp, although dry surfaces product optimum results. New concrete must be fully cured (28 days minimum).

2. Remove grease, wax, oil contaminants, and curing compounds by scrubbing with an industrial-grade detergent or a degreasing compound.

3. Follow with mechanical cleaning (refer to ASTM D 4258). Remove weak, contaminated, or deteriorated concrete by shotblasting, bushhammering, gritblasting, scarifying, or other suitable mechanical means. Follow mechanical cleaning with vacuum cleaning (refer to ASTM D 4259).

STEEL

Remove dirt, grease, and oil with a suitable industrial-grade cleaning-and-degreasing compound (SSPC-SP-1). Remove rust and mill scale by gritblasting. Blast steel to white metal. Follow gritblasting with vacuuming or oil-free dry-air blast (refer to SSPC-SP-10 or NACE-2).

Mixing

1. The mix ratio is 2 (Parts A) to 1 (Part B). Mix only the amount of material usable before the pot life expires. Thoroughly stir each component before mixing.

2. Measure (ratio) each component carefully and then add Part B (hardener) to Part A (resin).

3. Mix Parts A and B using a low-speed drill (600 rpm) and mixing paddle (e.g., a Jiffy mixer). Carefully scrape the sides and bottom of the container while mixing. Keep the paddle below the surface of the material to avoid entrapping air. Proper mixing will take at least 3 – 5 minutes. Well-mixed material will be free of streaks or lumps.

Protection and Repair

Technical Data

Composition

Concresive® Liquid LPL is a two-component 100% solids liquid epoxy.

Compliances

- ASTM C 881, Type II, Grade 2, Class C

Typical Properties

COMPONENT	PART A (Resin)	PART B (Hardener)
Form	Liquid	Liquid
Color	White	Black
Mixing ratio, by volume	2	1
Mixed color	Dark gray	

PROPERTY	VALUE		
	50° F (10° C)	**77° F (25° C)**	**105° F (41° C)**
Pot life			
1 qt (946 ml)	4.5 hrs	75 min	30 min
1 gal (3.8 L)	3.9 hrs	70 min	25 min
5 gal (18.9 L)	2.5 hrs	60 min	20 min
Viscosity, cps			
Resin	66,000	12,000	9,000
Hardener	1,150	350	110
Mixed	63,000	9,000	8,500
Thin film, open time	4 hrs	2 hrs	40 min
Thin film, days, full cure	14	7	3

Test Data[1]

PROPERTY	RESULTS	TEST METHODS
Tensile strength, psi (MPa)	4,400 (30.4)	ASTM D 638
Elongation at break, %	1.49	ASTM D 638
Compressive yield strength, psi (MPa)	8,300 (57.3)	ASTM D 695
Compressive modulus, psi (MPa)	3.5×10^5 (2.4×10^3)	ASTM D 695
Heat deflection temperature, ° F (° C)	127 (53)	ASTM D 648
Slant shear strength, psi (MPa)	5,000 (34.5)	AASHTO T-237
Bond strength, damp-to-damp concrete	100% concrete failure	AASHTO T-237
Bond strength at 14 days, psi (MPa)	1,800 (12.4)	ASTM C 882
Flexural bond strength, psi (MPa)	570 (3.9)	ASTM C 293

[1]Test temperature 77° F (25° C), cured 7 days.

Properties listed are typical and may be used as a guide for determining suitability for particular applications.

Application

GENERAL BONDING

Although this product will adhere to damp surfaces, dry surfaces produce the best results. When the surface is wet, remove free water by air blast or squeegee. Apply the bonding agent with a brush, paint roller, squeegee, conventional sprayer, or airless sprayer. The minimum bondline thickness should be 15 mils.

BONDING FRESH CONCRETE TO EXISTING CONCRETE

1. The new concrete being bonded should be a relatively low-slump mix.
2. When bonding concrete containing latex polymer admixtures, check compatibility either by installing a test patch and performing a pull-off test or by conducting a laboratory slant shear test (AASHTO T-237).
3. Apply the bonding agent as described in the General Bonding section above. Lightweight concrete may require a second coat if the first coat penetrates. Place fresh concrete within the open time or while the bonding agent is still tacky. Be careful when applying the fresh concrete not to damage the bonding layer.
4. For highly irregular surfaces sand may be used to extend this material. For proper application techniques refer to Appendix MB-17: Surface Preparation for Adhesives.

BOLT AND REBAR GROUTING

1. Holes may be cut by either rotary-percussion drilling, followed by air blow-out with oil-free compressed air, or diamond core boring, followed by water flush. The hole must be free of water before grouting. Where holes will be precast into the concrete, cast them undersized and drill them to fit.
2. The optimum hole size is 1/4" (6 mm) larger than the bar's; larger annular spaces are less desirable.
3. Pour a measured amount of bonding agent into the hole. Insert the bar, displacing the bonding agent, then secure the bar in the center of the hole. Remove excess bonding agent from around the hole before it hardens. Use pressure grouting for holes deeper than 2 ft (0.6 m).

Clean Up

Clean all tools and equipment immediately with xylene or mineral spirits. Cured material must be removed mechanically.

For Best Performance

- Precondition all components to 70° F for 24 hours before using.
- Application temperature range is 50 to 105° F (10 to 41° C).
- Do not add solvents or water to epoxy components.
- Make certain the most current versions of product data sheet and MSDS are being used; call Customer Service (1-800-433-9517) to verify the most current versions.
- Proper application is the responsibility of the user. Field visits by BASF personnel are for the purpose of making technical recommendations only and not for supervising or providing quality control on the jobsite.

Health and Safety

CONCRESIVE® LIQUID LPL PART A

Caution

Contains epoxy resin, 0-cresyl glycidyl ether.

Risks

May cause skin, eye and respiratory irritation. May cause dermatitis and allergic responses. Potential skin and/or respiratory sensitizer. Ingestion may cause irritation.

Precautions

Use only with adequate ventilation. Avoid contact with skin, eyes and clothing. Keep container closed when not in use. Wash thoroughly after handling. DO NOT take internally. Use impervious gloves, eye protection and if the TLV is exceeded or used in a poorly ventilated area, use NIOSH/MSHA approved respiratory protection in accordance with applicable Federal, state and local regulations.

First Aid

In case of eye contact, flush thoroughly with water for at least 15 minutes. In case of skin contact, wash affected areas with soap and water. If irritation persists, SEEK MEDICAL ATTENTION. Remove and wash contaminated clothing. If inhalation causes physical discomfort, remove to fresh air. If discomfort persists or any breathing difficulty occurs or if swallowed, SEEK IMMEDIATE MEDICAL ATTENTION.

For additional information on personal protective equipment, first aid, and emergency procedures, refer to the product Material Safety Data Sheet (MSDS) on the job site or contact the company at the address or phone numbers given below.

Proposition 65

This product contains materials listed by the state of California as known to cause cancer, birth defects, or reproductive harm.

VOC Content

0 g/L or 0 lbs/gal less water and exempt solvents when components are mixed and applied per Manufacturer's instructions.

CONCRESIVE® LIQUID LPL PART B

Danger-Corrosive:

Contains: Tall oil fatty acids, reaction products with tetraethylene pentamine; Tetraethylene pentamine; 2,4,6-Tris((dimethylamino)methyl)phenol.

Risks

Contact with skin or eyes may cause burns. Ingestion may cause irritation and burns of mouth, throat and stomach. Inhalation of vapors may cause irritation. May cause dermatitis and allergic responses. Potential skin and/or respiratory sensitizer. Repeated or prolonged contact with skin may cause sensitization. INTENTIONAL MISUSE BY DELIBERATELY INHALING THE CONTENTS MAY BE HARMFUL OR FATAL. Refer to Material Safety Data Sheet (MSDS) for effects of repeated overexposure.

Precautions

DO NOT get in eyes, on skin or clothing. Wash thoroughly after handling. Keep container closed. DO NOT take internally. Use only with adequate ventilation. DO NOT breathe vapors. Use impervious gloves, eye protection and if the TLV is exceeded or used in a poorly ventilated area, use NIOSH/MSHA approved respiratory protection in accordance with applicable Federal, state and local regulations.

First Aid

In case of eye contact, flush thoroughly with water for at least 15 minutes. In case of skin contact, wash affected areas with soap and water. If irritation persists, SEEK MEDICAL ATTENTION. Remove and wash contaminated clothing. If inhalation causes physical discomfort, remove to fresh air. If discomfort persists or any breathing difficulty occurs or if swallowed, SEEK IMMEDIATE MEDICAL ATTENTION.

For additional information on personal protective equipment, first aid, and emergency procedures, refer to the product Material Safety Data Sheet (MSDS) on the job site or contact the company at the address or phone numbers given below.

Proposition 65

This product does not contain materials listed by the state of California as known to cause cancer, birth defects, or reproductive harm.

VOC Content

0 g/L or 0 lbs/gal less water and exempt solvents when components are mixed and applied per BASF instructions.

**For medical emergencies only,
call ChemTrec (1-800-424-9300)**

BASF Construction Chemicals, LLC – Building Systems

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BASF Construction Chemicals, LLC
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C. Supplemental Calculations & Charts

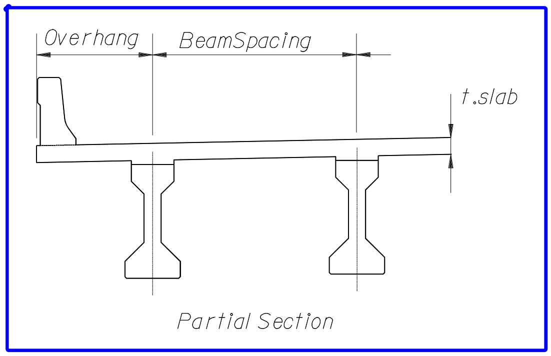
A. Input Variables

Bridge design span length....................... $L_{span} := 110ft$

Number of beams................................. $N_{beams} := 5$

Beam Spacing...................................... $Beam_{Spacing} := 9ft$

Average Buildup................................... $h_{buildup} := 1in$



Beam top flange width........................... $b_{tf} := 48in$

Beam web width.................................. $b_w := 7in$

Thickness of deck slab........................... $t_{slab} := 8in$

Milling surface thickness........................ $t_{mill} := 0.5in$

Deck overhang..................................... $Overhang := 3.5ft$

Dynamic Load Allowance....................... $IM := 1.33$

Bridge skew... $Skew := 0deg$

Weight of reinf concrete........................ $\gamma_{conc} := 150pcf$

Barrier dead load.................................. $w_{barrier.ea} := 420plf$

Modulus of elasticity for reinforcing steel... $E_s := 29000ksi$

Yield strength of reinforcing steel............ $f_y := 60ksi$

Minimum 28-day compressive strength of concrete components........................ $f_{c.slab} := 4.5ksi$

Correction factor for Florida lime rock coarse aggregate..... $K_1 := 0.9$

Unit Weight of Florida lime rock concrete (kcf)............... $w_{c.limerock} := .145\dfrac{kip}{ft^3}$

Modulus of elasticity for slab............ $E_{c.slab} := 33000K_1 \cdot \left(\dfrac{w_{c.limerock}}{\dfrac{kip}{ft^3}}\right)^{1.5} \cdot \sqrt{f_{c.slab} \cdot ksi}$

$E_{c.slab} = 3479 \cdot ksi$

Concrete cover for top steel.................... $cover_{deck.top} := 2.5in$

Concrete cover for bottom steel............... $cover_{deck.bot} := 2in$

B. Approximate Methods of Analysis - Decks [LRFD 4.6.2]

HL-93 Live Load Design Moments - Deck Slab Design Table [LRFD Appendix A4]

Table A4-1 in Appendix A4 will be used to determine the live load design moments.

Location of Negative Live Load Design Moment [LRFD 4.6.2.1.6]

The negative live load design moment is taken at a distance from the supports....... $Loc_{negative} := \min\left(\frac{1}{3} \cdot b_{tf}, 15 \cdot in\right)$

$Loc_{negative} = 15.0 \cdot in$

Positive Live Load Design Moment........... $M_{LL.pos} := 6.29 \cdot ft \cdot kip$

Negative Live Load Design Moment.......... $M_{LL.neg} := (15in - 12 \cdot in) \cdot \left[\frac{(3.31 \cdot ft \cdot kip - 3.71 \cdot ft \cdot kip)}{(18 \cdot in - 12 \cdot in)}\right] + 3.71 \cdot (ft \cdot kip)$

$M_{LL.neg} = 3.51 \cdot ft \cdot kip$

*(**Note**: Interpolated value)*

Dead Load Design Moments

Design width of deck slab....................... $b_{slab} := 1ft$

"DC" loads include the dead load of structural components and non-structural attachments

Self-weight of deck slab.................... $w_{slab} := \left[\left(t_{slab} + t_{mill}\right) \cdot b_{slab}\right] \cdot \gamma_{conc}$

$w_{slab} = 0.106 \cdot \frac{kip}{ft}$

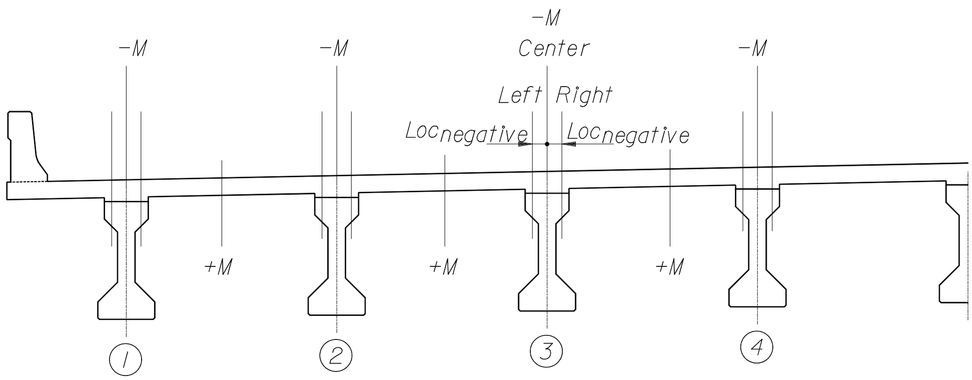
Weight of traffic barriers.................. $P_{barrier} := w_{barrier.ea} \cdot b_{slab}$

$P_{barrier} = 0.42 \cdot kip$

"DW" loads include the dead load of a future wearing surface and utilities $\rho_{fws} := 0$

Weight of Future Wearing Surface..... $w_{fws} := \rho_{fws} \cdot b_{slab}$

$w_{fws} = 0 \cdot klf$



Negative and Positive Moment Locations

Dead load moments determined using LUSAS:

	Design Moments for DC Loads			
Beam /	Positive	Negative Moment (k-ft)		
Span	Moment (k-ft)	Center	Left	Right
1	0.04	-2.03	-1.21	-1.36
2	0.48	-0.28	-0.06	0.11
3	0.48	-0.84	-0.32	-0.32

The governing negative design moment for DC loads occurs at beam 1. However, this moment is due to the overhang, which typically has more negative moment steel requirements than the interior regions of the deck. Since the overhang is designed separately, the overhang moments are not considered here. For the interior regions, the positive moment in Span 2 and the negative moment to the right/left of beam 3 govern.

Positive moment.............................. $M_{DC.pos} := 0.48 \cdot kip \cdot ft$

Negative moment............................ $M_{DC.neg} := 0.32 \cdot kip \cdot ft$

Positive moment.............................. $M_{DW.pos} := 0.054 \cdot kip \cdot ft$

Negative moment............................. $M_{DW.neg} := 0.095 \cdot kip \cdot ft$

(15 psf FWS)

Limit State Moments

The service and strength limit states are used to design the section:

STRENGTH I - Basic load combination relating to the normal vehicular use of the bridge without wind.

$WA, FR = 0$ For superstructure design, water load / stream pressure and friction forces are not applicable.

$TU, CR, SH, FR = 0$ Uniform temperature, creep, shrinkage are generally ignored.

$Strength1 = 1.25 \cdot DC + 1.50 \cdot DW + 1.75 \cdot LL$

SERVICE I - Load combination relating to the normal operational use of the bridge with a 55 MPH wind and all loads taken at their nominal values.

$BR, WS, WL = 0$ For superstructure design, braking forces, wind on structure and wind on live load are not applicable.

$Service1 = 1.0 \cdot DC + 1.0 \cdot DW + 1.0 \cdot LL$

Strength I Limit State

Positive Strength I Moment..................... $M_{strengthI.pos} := 1.25M_{DC.pos} + 1.50 \cdot M_{DW.pos} + 1.75 \cdot M_{LL.pos}$

$M_{strengthI.pos} = 11.7 \cdot kip \cdot ft$

Negative Strength I Moment................... $M_{strengthI.neg} := 1.25M_{DC.neg} + 1.50 \cdot M_{DW.neg} + 1.75 \cdot M_{LL.neg}$

$M_{strengthI.neg} = 6.7 \cdot kip \cdot ft$

Service I Limit State

Positive Service I Moment...................... $M_{serviceI.pos} := M_{DC.pos} + M_{DW.pos} + M_{LL.pos}$

$M_{serviceI.pos} = 6.8 \cdot kip \cdot ft$

Negative Service I Moment..................... $M_{serviceI.neg} := M_{DC.neg} + M_{DW.neg} + M_{LL.neg}$

$M_{serviceI.neg} = 3.9 \cdot kip \cdot ft$

C. Moment Design

Positive Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

Factored resistance

$$M_r = \phi \cdot M_n$$

Nominal flexural resistance

$$M_n = A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) - A'_s \cdot f'_y \cdot \left(d'_s - \frac{a}{2}\right) + 0.85 \cdot f'_c \cdot (b - b_w) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Simplifying the nominal flexural resistance

$$M_n = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) \quad \text{where} \quad a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$

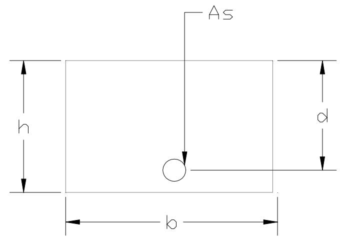
Using the variables defined above........... $M_r = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_s - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right]$

where $M_r := M_{strengthI.pos}$

$\phi := 0.90$

$t_{slab} = 8 \cdot in$ $t_{slab} = h$

$b := b_{slab}$



$b = 12 \cdot in$

Initial assumption for area of steel required

Size of bar..................................... bar := "5"

Proposed bar spacing...................... $spacing_{pos} := 6.5 \cdot in$

Bar area.. $A_{bar} = 0.310 \cdot in^2$

Bar diameter.................................. $dia = 0.625 \cdot in$

Area of steel provided per foot of slab...... $A_{s.pos} := \frac{A_{bar} \cdot 1ft}{spacing_{pos}}$

$A_{s.pos} = 0.57 \cdot in^2$

Assume longitudinal steel to be #4 bars with diameter of 0.5 in.

Distance from extreme compressive fiber to centroid of reinforcing steel.................... $d_{s.pos} := t_{slab} - cover_{deck.bot} - 0.5in - \frac{dia}{2}$

$d_{s.pos} = 5.2 \cdot in$

Solve the quadratic equation for the area of steel required....................................... Given $M_r = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_{s.pos} - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right]$

$A_{s.reqd.pos} := Find(A_{s.pos})$

Reinforcing steel required....................... $A_{s.reqd.pos} = 0.54 \cdot in^2$

The area of steel provided, $A_{s.pos} = 0.57 \cdot in^2$, is greater than the area of steel required, $A_{s.reqd.pos} = 0.54 \cdot in^2$.

Negative Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

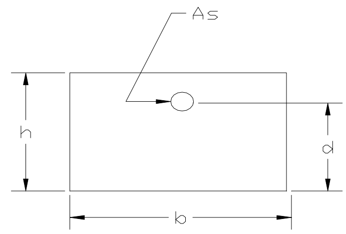
Variables: $M_r := M_{strengthI.neg}$

$\phi = 0.9$

$t_{slab} = 8 \cdot in$

$t_{slab} = h$

$b := b_{slab}$



$b = 12 \cdot in$

Initial assumption for area of steel required

Size of bar...................................... bar = "5"

Proposed bar spacing....................... $spacing_{neg} := 6.5in$

Bar area... $A_{bar} = 0.310 \cdot in^2$

Bar diameter................................... $dia = 0.625 \cdot in$

Area of steel provided per foot of slab...... $A_{s.neg} := \dfrac{A_{bar} \cdot 1ft}{spacing_{neg}}$

$A_{s.neg} = 0.57 \cdot in^2$

Assume longitudinal steel to be #4 bars with diameter of 0.5 in.

Distance from extreme compressive fiber to centroid of reinforcing steel................. $d_{s.neg} := t_{slab} + t_{mill} - cover_{deck.top} - 0.5in - \dfrac{dia}{2}$

$d_{s.neg} = 5.2 \cdot in$

Solve the quadratic equation for the area of steel required..

Given $M_r = \phi \cdot A_{s.neg} \cdot f_y \cdot \left[d_{s.neg} - \dfrac{1}{2} \cdot \left(\dfrac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$

$A_{s.reqd.neg} := Find(A_{s.neg})$

Reinforcing steel required........................ $A_{s.reqd.neg} = 0.30 \cdot in^2$

The area of steel provided, $A_{s.neg} = 0.57 \cdot in^2$, is greater than the area of steel required, $A_{s.reqd.neg} = 0.30 \cdot in^2$.

Crack Control by Distribution Reinforcement [LRFD 5.7.3.4]

The maximum spacing of the mild steel reinforcement for control of cracking at the service limit state shall satisfy.........

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

where

$$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)}$$

Exposure factor for Class 1 exposure condition.......................................

$\gamma_e := 1.00$ **[SDG 3.10]**

Overall thickness or depth of the component.............................

$t_{slab} = 8 \cdot in$

Positive Moment

Distance from extreme tension fiber to center of closest bar..............................

$$d_c := cover_{deck.bot} + 0.5in + \frac{dia}{2}$$

$d_c = 2.813 \cdot in$

$$\beta_s := 1 + \frac{d_c}{0.7\left(t_{slab} - d_c\right)} \qquad \beta_s = 1.775$$

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. This process is iterative, so an initial assumption of the neutral axis must be made.

Guess $x := 1.8in$

Given $$\frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_{c.slab}} \cdot A_{s.pos} \cdot \left(d_{s.pos} - x\right) \qquad x_{na.pos} := Find(x)$$

$x_{na.pos} = 1.672 \cdot in$

Tensile force in the reinforcing steel due to service limit state moment......................

$$T_s := \frac{M_{serviceI.pos}}{d_{s.pos} - \frac{x_{na.pos}}{3}}$$

$T_s = 17.7 \cdot kip$

Actual stress in the reinforcing steel due to service limit state moment......................

$$f_{s.actual} := \frac{T_s}{A_{s.pos}}$$

$f_{s.actual} = 30.9 \cdot ksi$

Required reinforcement spacing.......... $s_{required} := \dfrac{700 \cdot \gamma_e \cdot \dfrac{kip}{in}}{\beta_s \cdot f_{s.actual}} - 2 \cdot d_c$

$s_{required} = 7.1 \cdot in$

Provided reinforcement spacing.......... $spacing_{pos} = 6.5 \cdot in$

The required spacing of mild steel reinforcement in the layer closest to the tension face shall not be less than the reinforcement spacing provided due to the service limit state moment.

$$LRFD_{5.7.3.4} := \begin{cases} \text{"OK, crack control for +M is satisfied"} & \text{if } s_{required} \geq spacing_{pos} \\ \text{"NG, crack control for +M not satisfied, provide more reinforcement"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.4} =$ "OK, crack control for +M is satisfied"

Negative Moment

Distance from extreme tension fiber to center of closest bar .... $d_c := cover_{deck.top} - t_{mill} + 0.5in + \dfrac{dia}{2}$

$d_c = 2.813 \cdot in$

$\beta_s := 1 + \dfrac{d_c}{0.7(t_{slab} - d_c)}$ $\beta_s = 1.775$

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. This process is iterative, so an initial assumption of the neutral axis must be made.

Guess $x := 1.8 \cdot in$

Given $\dfrac{1}{2} \cdot b \cdot x^2 = \dfrac{E_s}{E_{c.slab}} \cdot A_{s.neg} \cdot (d_{s.neg} - x)$ $x_{na.neg} := Find(x)$

$x_{na.neg} = 1.7 \cdot in$

Tensile force in the reinforcing steel due to service limit state moment...................... $T_s := \dfrac{M_{serviceI.neg}}{d_{s.neg} - \dfrac{x_{na.neg}}{3}}$

$T_s = 10.2 \cdot kip$

Actual stress in the reinforcing steel due to service limit state moment................... $f_{s.actual} := \dfrac{T_s}{A_{s.neg}}$

$f_{s.actual} = 17.8 \cdot ksi$

Required reinforcement spacing........... $s_{required} := \dfrac{700 \cdot \gamma_e \cdot \dfrac{kip}{in}}{\beta_s \cdot f_{s.actual}} - 2 \cdot d_c$

$s_{required} = 16.6 \cdot in$

Provided reinforcement spacing........... $spacing_{neg} = 6.5 \cdot in$

The required spacing of mild steel reinforcement in the layer closest to the tension face shall not be less than the reinforcement spacing provided due to the service limit state moment.

$$LRFD_{5.7.3.4} := \begin{cases} \text{"OK, crack control for -M is satisfied"} & \text{if } s_{required} \geq spacing_{neg} \\ \text{"NG, crack control for -M not satisfied, provide more reinforcement"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.4} =$ "OK, crack control for -M is satisfied"

Limits for Reinforcement [LRFD 5.7.3.3]

Minimum Reinforcement [5.7.3.3.2]

The minimum reinforcement requirements ensure the moment capacity provided is at least 1.2 times greater than the cracking moment.

Modulus of Rupture.............................. $f_r := 0.24 \cdot \sqrt{f_{c.slab} \cdot ksi}$ [SDG 1.4.1.B]

$f_r = 509.1 \cdot psi$

Distance from the extreme tensile fiber to the neutral axis of the composite section... $y := \dfrac{t_{slab}}{2}$

$y = 4.0 \cdot in$

Moment of inertia for the section............ $I_{slab} := \dfrac{1}{12} \cdot b \cdot t_{slab}^{\,3}$

$I_{slab} = 512.0 \cdot in^4$

Section modulus.................................... $S := \dfrac{I_{slab}}{y}$

$S = 128.0 \cdot in^3$

Cracking moment.................................. $M_{cr} := f_r \cdot S$

$M_{cr} = 5.4 \cdot kip \cdot ft$

Positive Moment

Minimum reinforcement required............

$$A_{min.pos} := \frac{\dfrac{1.2 \cdot M_{cr}}{\phi}}{f_y \cdot \left[d_{s.pos} - \dfrac{1}{2}\left(\dfrac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]}$$

$A_{min.pos} = 0.30 \cdot in^2$

Required area of steel for minimum reinforcement should not be less than $A_{s.reqd} \cdot 133\%$ or A_{min} ....

$$A_{s.req.pos} := \min\left(A_{s.reqd.pos} \cdot 133 \cdot \%, A_{min.pos}\right)$$

$A_{s.req.pos} = 0.30 \cdot in^2$

Maximum bar spacing for mimimum reinforcement......................................

$$spacing_{max.pos} := \frac{b}{\left(\dfrac{A_{s.req.pos}}{A_{bar}}\right)}$$

$spacing_{max.pos} = 12.4 \cdot in$

Negative Moment

Minimum reinforcement required............

$$A_{min.neg} := \frac{\dfrac{1.2 \cdot M_{cr}}{\phi}}{f_y \cdot \left[d_{s.neg} - \dfrac{1}{2}\left(\dfrac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]}$$

$A_{min.neg} = 0.30 \cdot in^2$

Required area of steel for minimum reinforcement should not be less than $A_{s.reqd} \cdot 133\%$ or A_{min} ....

$$A_{s.req.neg} := \min\left(A_{s.reqd.neg} \cdot 133 \cdot \%, A_{min.neg}\right)$$

$A_{s.req.neg} = 0.30 \cdot in^2$

Maximum bar spacing for mimimum reinforcement......................................

$$spacing_{max.neg} := \frac{b}{\left(\dfrac{A_{s.req.neg}}{A_{bar}}\right)}$$

$spacing_{max.neg} = 12.4 \cdot in$

The bar spacing should be less than the maximum bar spacing for minimum reinforcement

$$LRFD_{5.7.3.3.2} := \begin{array}{|l} \text{"OK, minimum reinforcement requirements are satisfied"} \quad \text{if } spacing_{pos} \le spacing_{max.pos} \wedge spacing_{neg} \\ \text{"NG, section is under-reinforced, so redesign!"} \quad \text{otherwise} \end{array}$$

$LRFD_{5.7.3.3.2} =$ "OK, minimum reinforcement requirements are satisfied"

Shrinkage and Temperature Reinforcement [LRFD 5.10.8]

Shrinkage and temperature reinforcement provided

Size of bar ("4" "5" "6")........ $bar_{st} := "4"$

Bar spacing...................................... $bar_{spa.st} := 12 \cdot in$

Bar area.. $A_{bar.st} = 0.20 \cdot in^2$

Bar diameter.................................... $dia = 0.500 \cdot in$

Gross area of section............................. $A_g := b_{slab} \cdot t_{slab}$

$A_g = 96.0 \cdot in^2$

Area of shrinkage and temperature reinforcement provided........................ $A_{bar.st} := \dfrac{A_{bar.st}}{bar_{spa.st}}$

$A_{bar.st} = 0.2 \cdot \dfrac{in^2}{ft}$

Check area of steel...............................

$$A_{ST.Check1} := \text{if}\left[A_{bar.st} \ge \frac{1.30 \cdot \frac{kip}{in \cdot ft} \cdot A_g}{2 \cdot (b_{slab} + t_{slab}) f_y}, \text{"OK"}, \text{"Not OK"}\right]$$

$$A_{ST.Check2} := \text{if}\left(0.11 \frac{in^2}{ft} < A_{bar.st} < 0.60 \frac{in^2}{ft}, \text{"OK"}, \text{"Not OK"}\right)$$

$$A_{ST.Check} := \text{if}\left(A_{ST.Check1} = \text{"OK"} \wedge A_{ST.Check2} = \text{"OK"}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Maximum spacing for shrinkage and temperature reinforcement...................... $spacing_{ST} := \min(bar_{spa.st}, 3 \cdot t_{slab}, 12 \cdot in)$ **[SDG 4.2.11]**

$spacing_{ST} = 12.0 \cdot in$

The bar spacing should be less than the maximum spacing for shrinkage and temperature reinforcement

$$LRFD_{5.7.10.8} := \begin{cases} \text{"OK, minimum shrinkage and temperature requirements"} & \text{if } bar_{spa.st} \le spacing_{ST} \wedge A_{ST.Check} = \text{"OK} \\ \text{"NG, minimum shrinkage and temperature requirements"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.10.8}$ = "OK, minimum shrinkage and temperature requirements"

Distribution of Reinforcement [LRFD 9.7.3.2]

This reinforcement is placed in the bottom of the deck slab as a percentage of the primary reinforcement. Distribution reinforcement provided:

Size of bar ("4" "5" "6")........ $bar_{dist} := "4"$

Bar spacing.................................... $bar_{spa.dist} := 6 \cdot in$

Bar area.. $A_{bar.dist} = 0.20 \cdot in^2$

Bar diameter.................................. $dia = 0.500 \cdot in$

The effective span length (LRFD 9.7.2.3) is the distance between the flange tips plus the flange overhang.............................. $Slab_{eff.Length} := Beam_{Spacing} - b_w$

$Slab_{eff.Length} = 8.417\ ft$

The area for secondary reinforcement should not exceed 67% of the area for primary reinforcement.......................... $\%A_{steel} := \min\left(\dfrac{220}{\sqrt{\dfrac{Slab_{eff.Length}}{ft}}}\%, 67\%\right)$

$\%A_{steel} = 0.67$

Required area for secondary reinforcement...................................... $A_{s.DistR} := A_{s.pos} \cdot \%A_{steel}$

$A_{s.DistR} = 0.38 \cdot in^2$

Maximum spacing for secondary reinforcement...................................... $MaxSpacing_{DistR} := \dfrac{b}{\left(\dfrac{A_{s.DistR}}{A_{bar.dist}}\right)}$

$MaxSpacing_{DistR} = 6.3 \cdot in$

The bar spacing should not exceed the maximum spacing for secondary reinforcement

$$LRFD_{9.7.3.2} := \begin{cases} \text{"OK, distribution reinforcement requirements"} & \text{if } bar_{spa.dist} \le MaxSpacing_{DistR} \\ \text{"NG, distribution reinforcement requirements"} & \text{otherwise} \end{cases}$$

$LRFD_{9.7.3.2} =$ "OK, distribution reinforcement requirements"

Note: Over the Intermediate Piers or Bents, supplemental longitudinal reinforcement in the top of the slab should be provided per **SDG 4.2.6**. Per **SDG 4.2.6.B**, provide No. 5 bars between the continuous, longitudinal reinforcing bars.

Bar size $bar_{dist} = "4"$

Top spacing $bar_{spa.piers} := \dfrac{bar_{spa.st}}{2}$

$bar_{spa.piers} = 6 \cdot in$

Deck Overhang Design

A. Input Variables

Thickness of slab.................................. $t_{slab} := 8in$

Milling surface thickness......................... $t_{mill} = 0.5 \cdot in$

Design width of overhang........................ $b_{overhang} := 1ft$

B. Deck Overhang Reinforcement

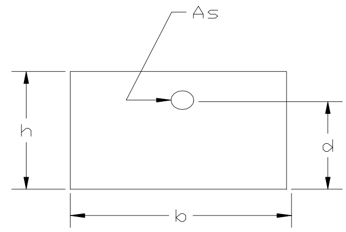
Negative Moment Region - Reinforcement Requirements [SDG 4.2.4B]

Reinforcement required for the extreme event limit states

$A_{s.TL4} := 0.8in^2$ per foot of overhang slab

$\phi = 0.9$

$t_{slab} = h$



$b := b_{overhang}$

Initial assumption for area of steel required

Size of bar..................................... bar = "5"

Proposed bar spacing....................... $spacing_{OH} := 3.25 \cdot in$

Bar area... $A_{bar} = 0.310 \cdot in^2$

Bar diameter................................... $dia = 0.625 \cdot in$

Area of steel provided per foot of slab...... $A_{s.overhang} := \dfrac{A_{bar} \cdot 1ft}{spacing_{OH}}$

$A_{s.overhang} = 1.14 \cdot in^2$

Check minimum reinforcement requirements

$$SDG_{4.2.4.B} := \begin{cases} \text{"OK, reinforcement requirements"} & \text{if } A_{s.TL4} \le A_{s.overhang} \\ \text{"NG, reinforcement requirements"} & \text{otherwise} \end{cases}$$

$SDG_{4.2.4.B} = \text{"OK, reinforcement requirements"}$

Summary of Reinforcement Provided

Transverse reinforcing

Bar size $bar = "5"$

Top spacing $spacing_{neg} = 6.5 \cdot in$

Bottom spacing $spacing_{pos} = 6.5 \cdot in$

Shrinkage and temperature reinforcing

Bar size $bar_{st} = "4"$

Bottom spacing $bar_{spa.st} = 12.0 \cdot in$

$LRFD_{5.7.10.8}$ = "OK, minimum shrinkage and temperature requirements"

Longitudinal Distribution reinforcing (bottom)

Bar size $bar_{dist} = "4"$

Bottom spacing $bar_{spa.dist} = 6.0 \cdot in$

$LRFD_{9.7.3.2}$ = "OK, distribution reinforcement requirements"

Transverse reinforcing at overhang

Bar size $bar = "5"$

Top spacing $spacing_{OH} = 3.25 \cdot in$

From: Brennan, Adam
Sent: Wednesday, August 29, 2012 4:12 PM
To: Abalo, Vickie
Subject: Cylinder Breaks

Precast Deck Panel # 2   Cast 4/20/2012

Weight (lb)	Length (in)	Diameter (in)	psi	lbs

Cylinder Breaks   days   19   5/9/2012 - 11:30 AM

27.84	11 15/16	6	6245	176,570
27.64	11 25/32	6	6204	175410

Cylinder Breaks   days   131   8/29/2012 - 2:00 PM

27.2	11 13/16	6	7396	209,120
27.28	11 7/8	6	7397	209,140
27.36	12 1/16	6	7172	202,790
		Avg.	7322	

Precast Deck Panel # 1   Cast 3/29/2012

Cylinder Breaks   days   153   8/29/2012 - 3:30 PM

27.5	12 1/16	6	6938	196,170
27.2	11 15/16	6	7195	203,420
27.24	11 15/16	6	7663	216,670
		Avg	7265	

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Tallahassee, FL 32310
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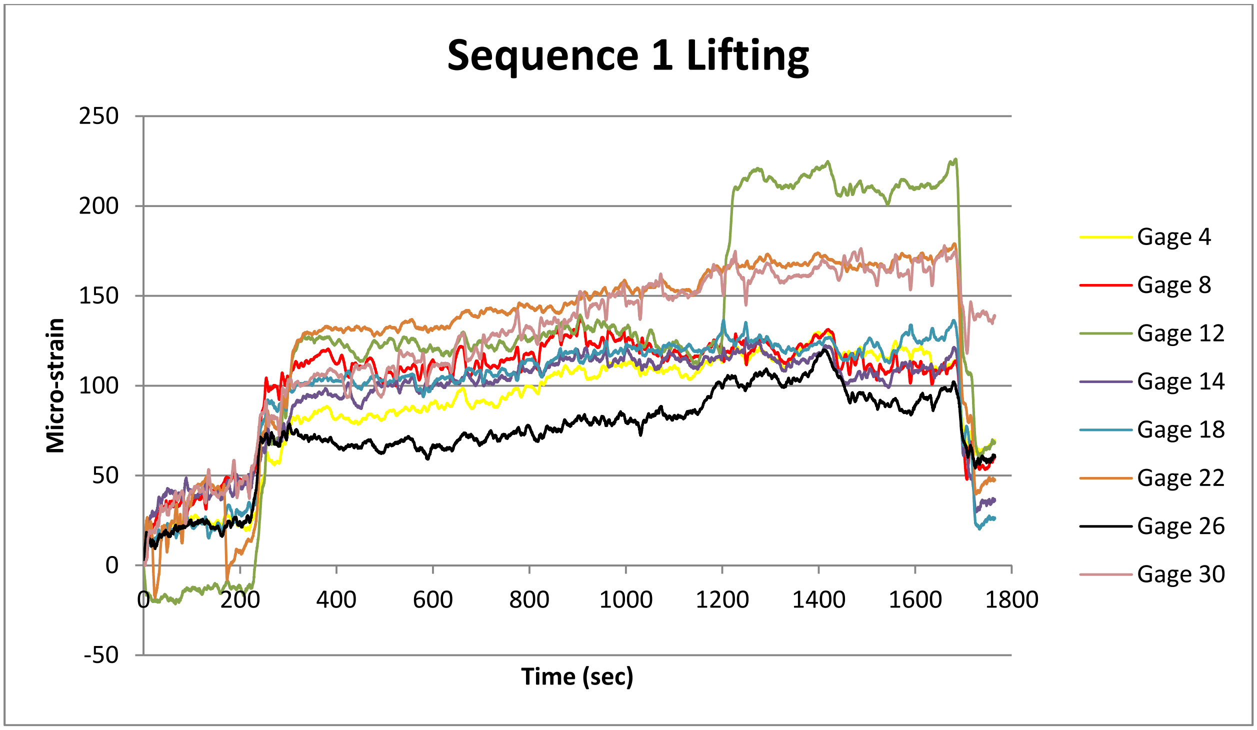
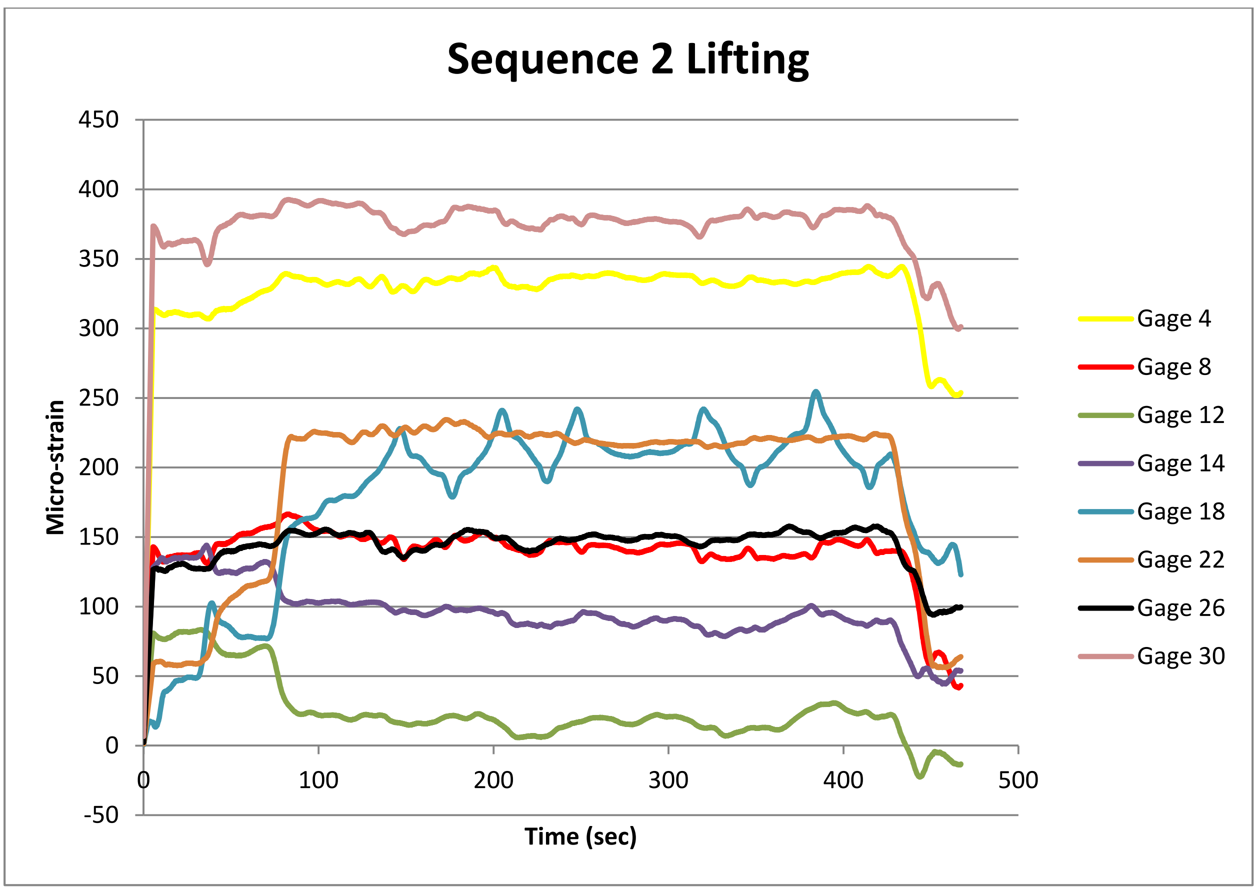
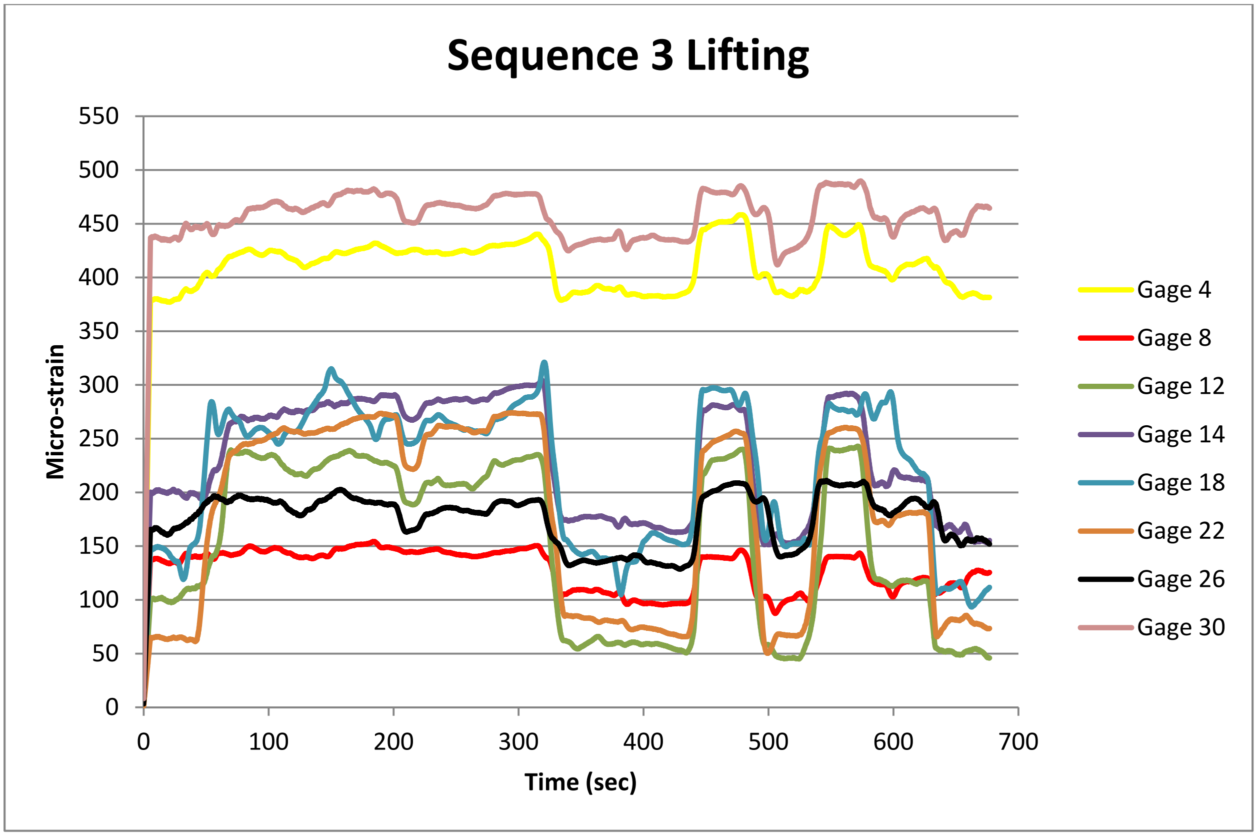
Fax: (850) 921-7101

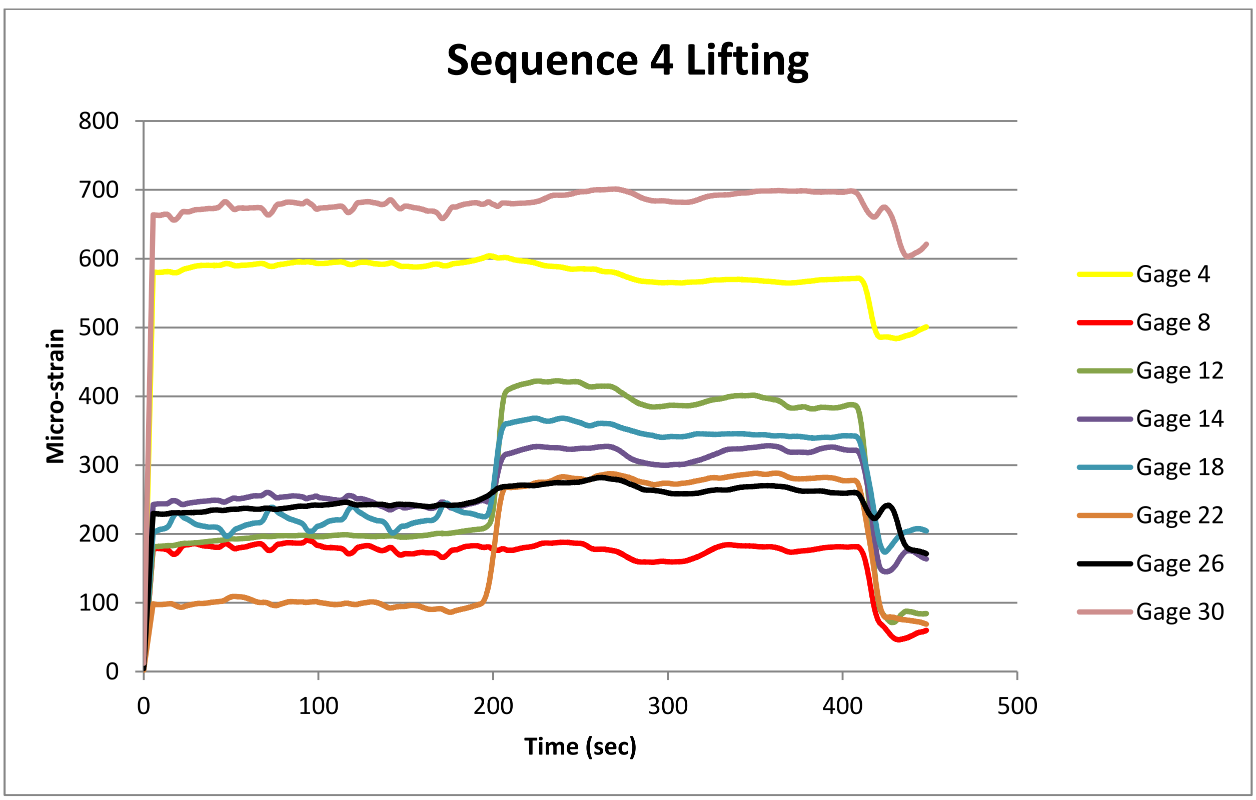
Day 1, Panel 2 Lifting



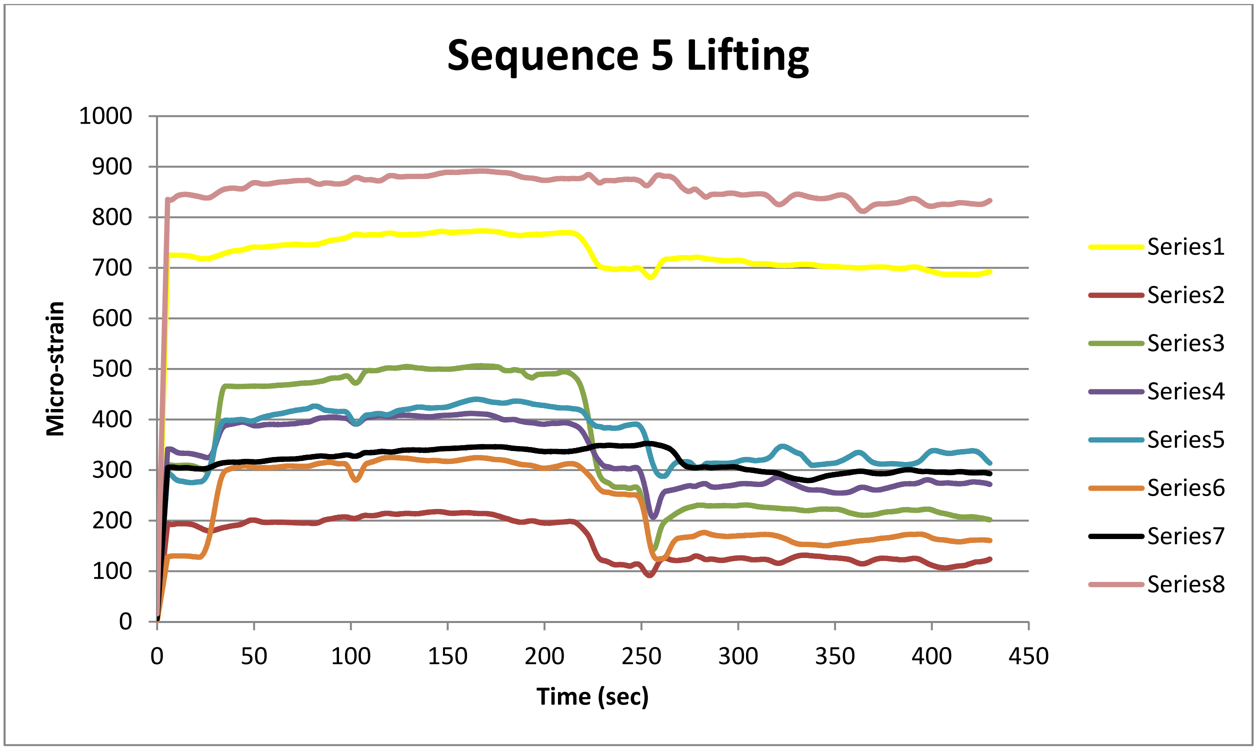


Day 1, Panel 2 Lifting

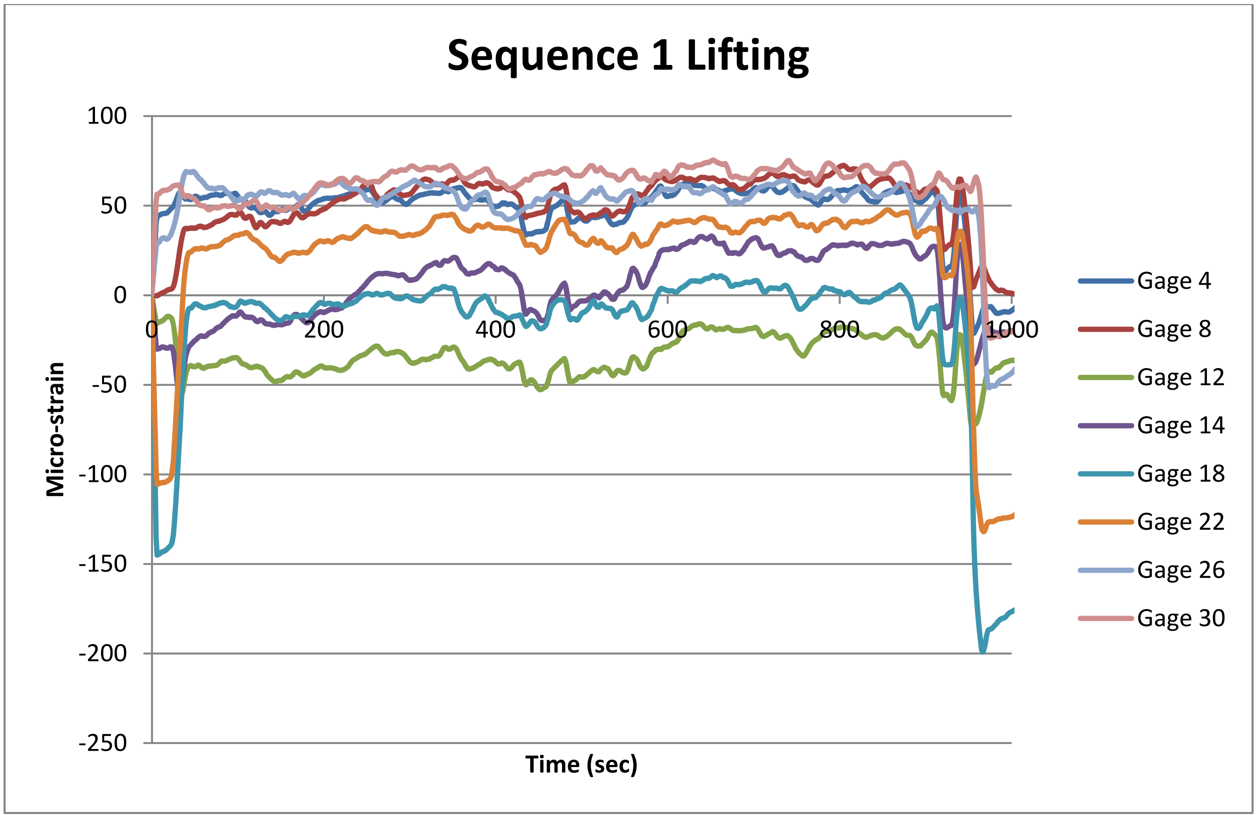


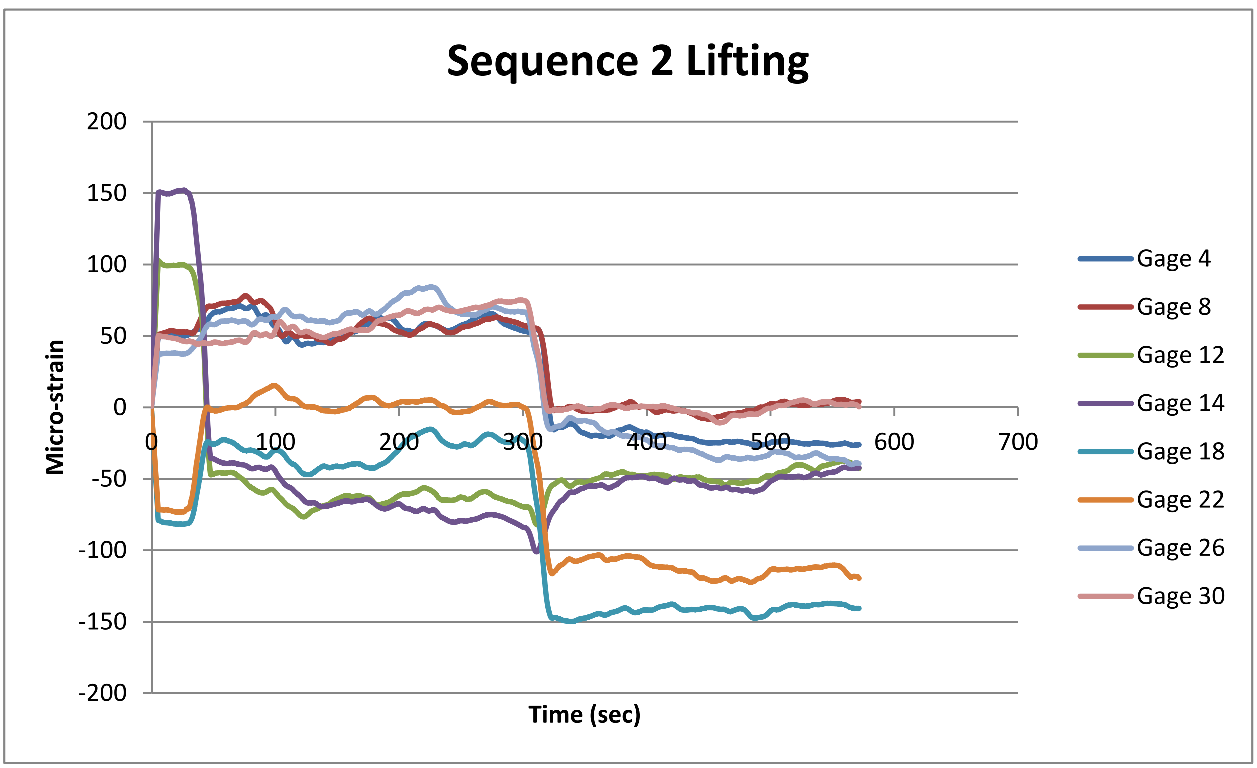


Day 1, Panel 2 Lifting

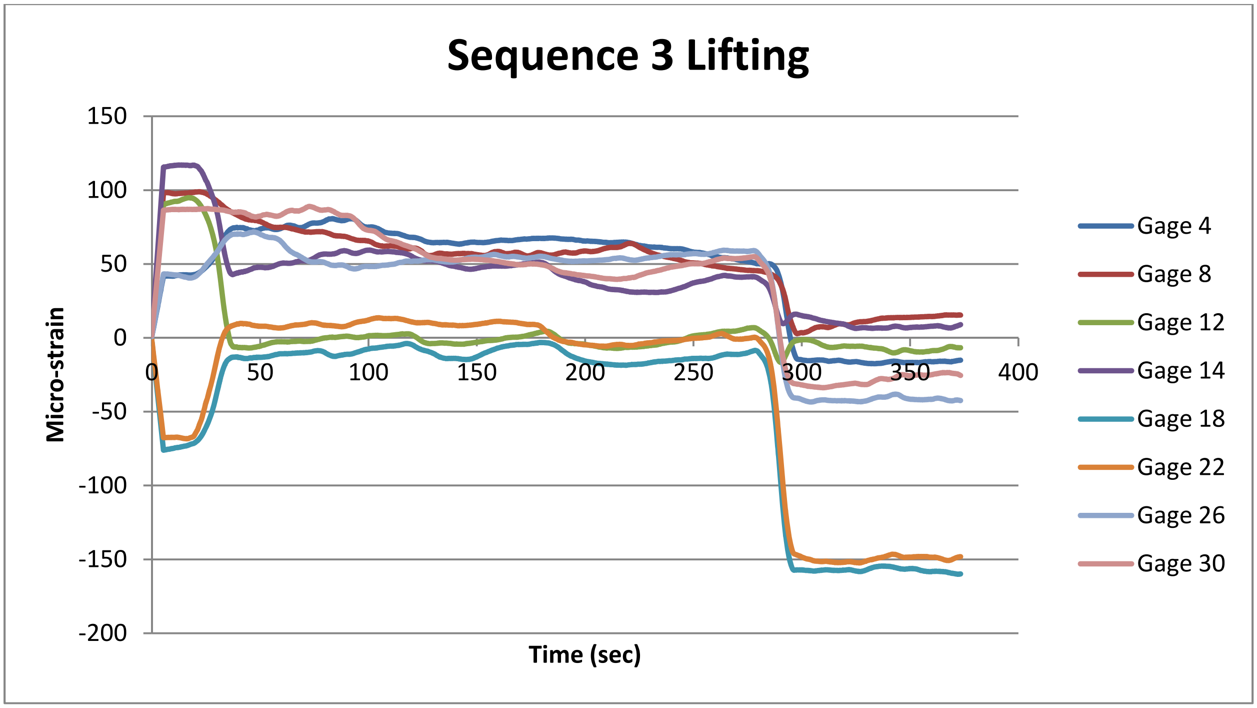


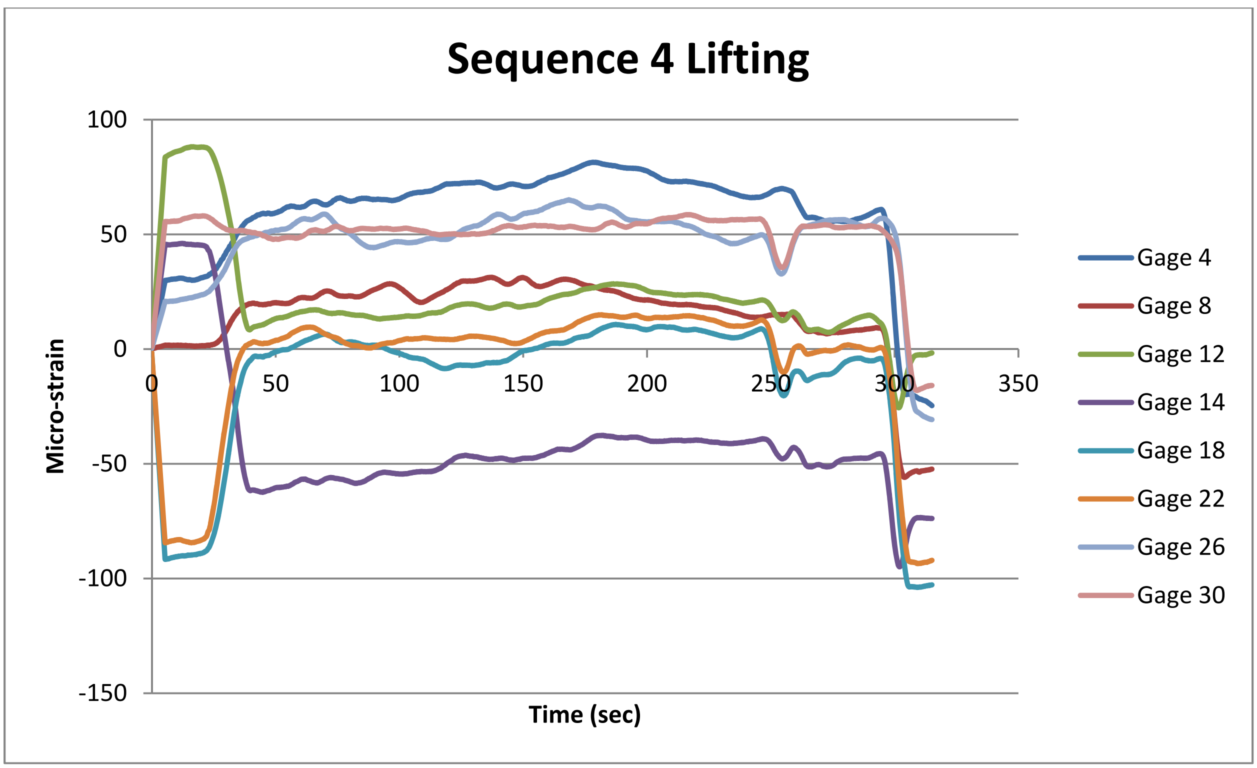
Day 2, Panel 1 Lifting



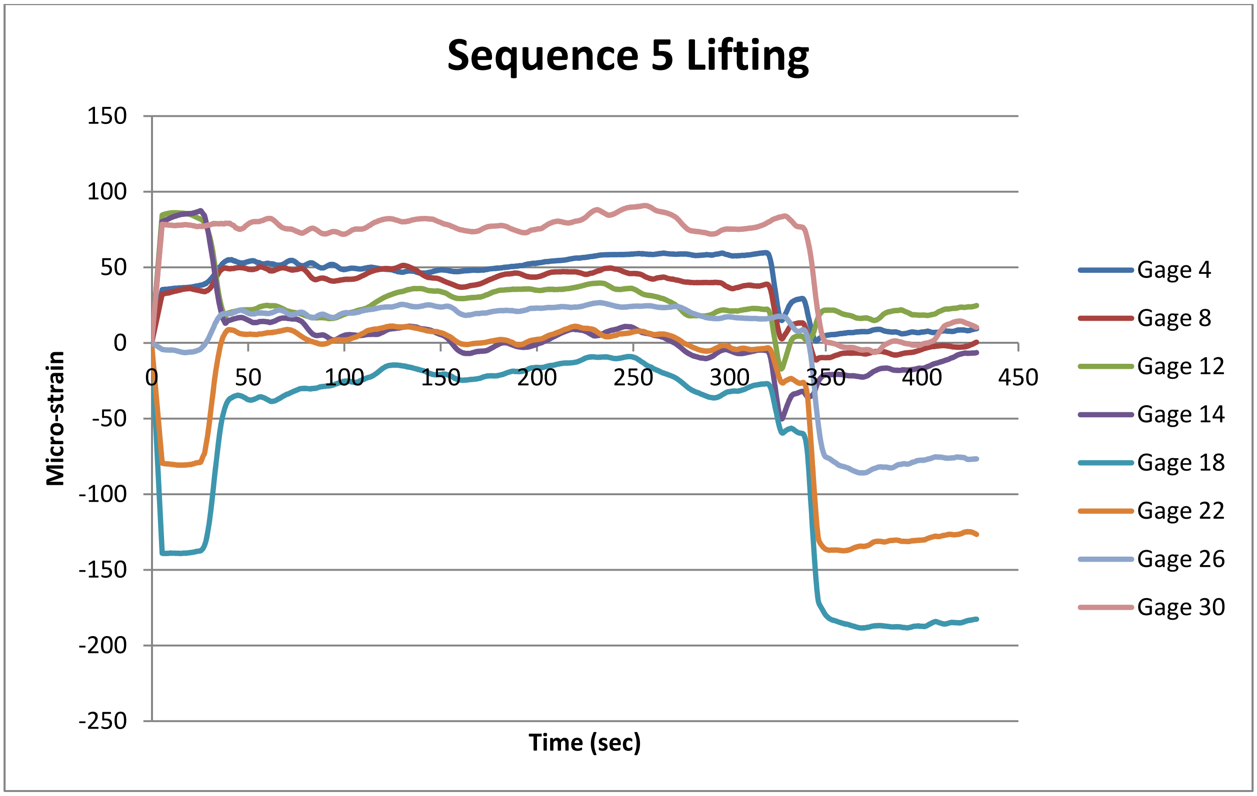


Day 2, Panel 1 Lifting



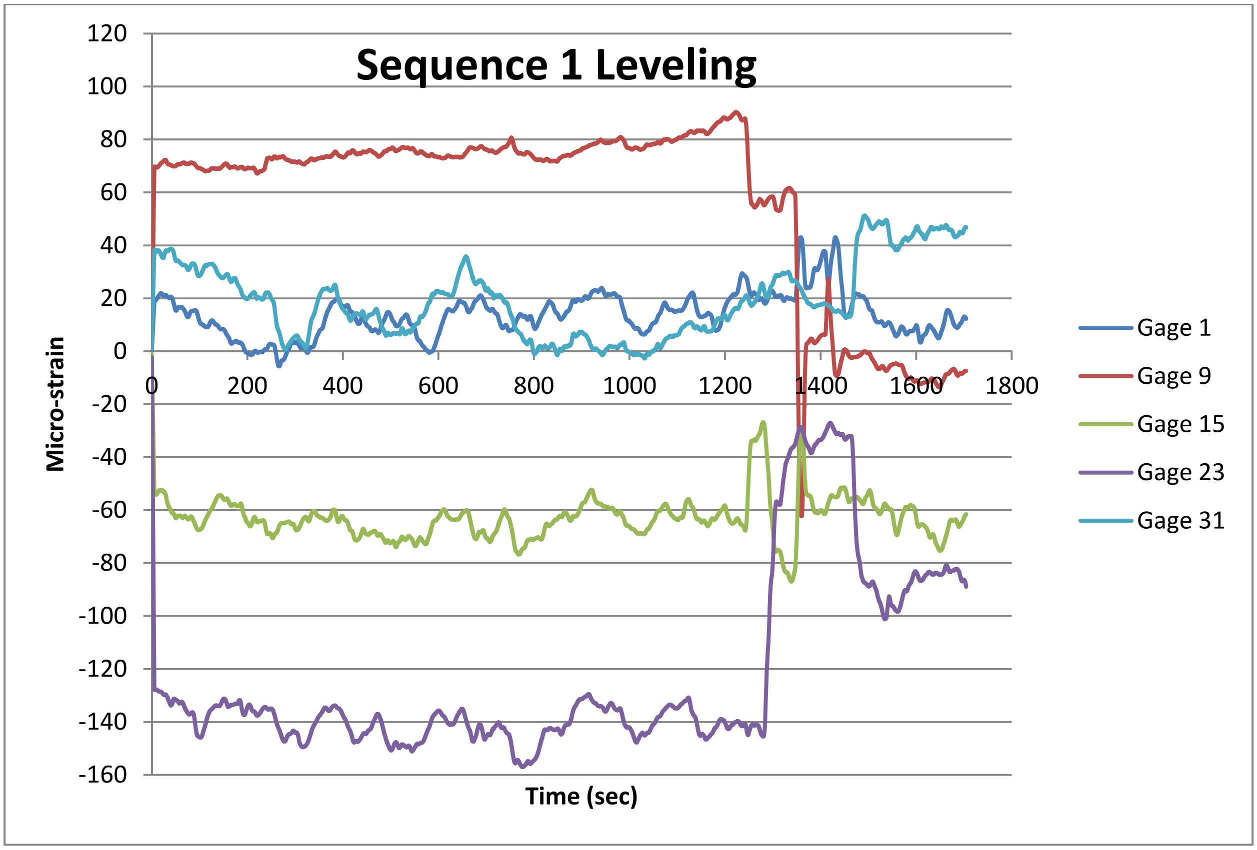


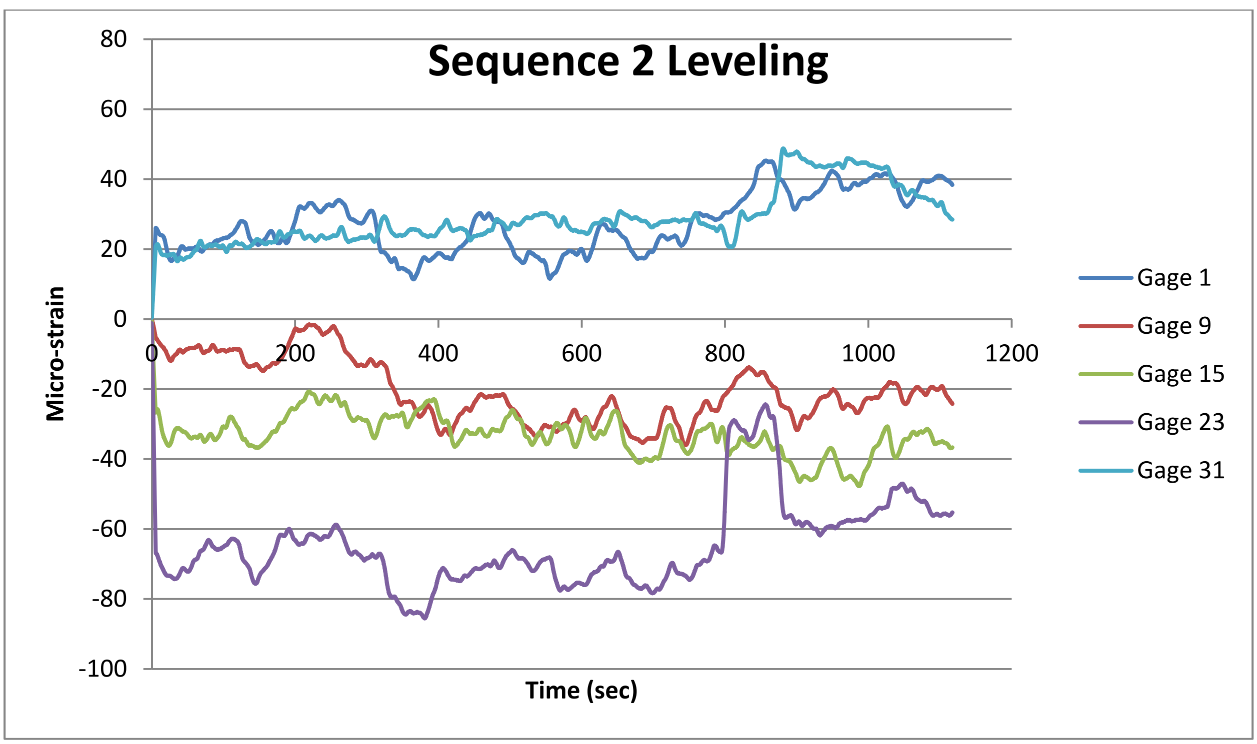
Day 2, Panel 1 Lifting



Transverse - East
Leveling

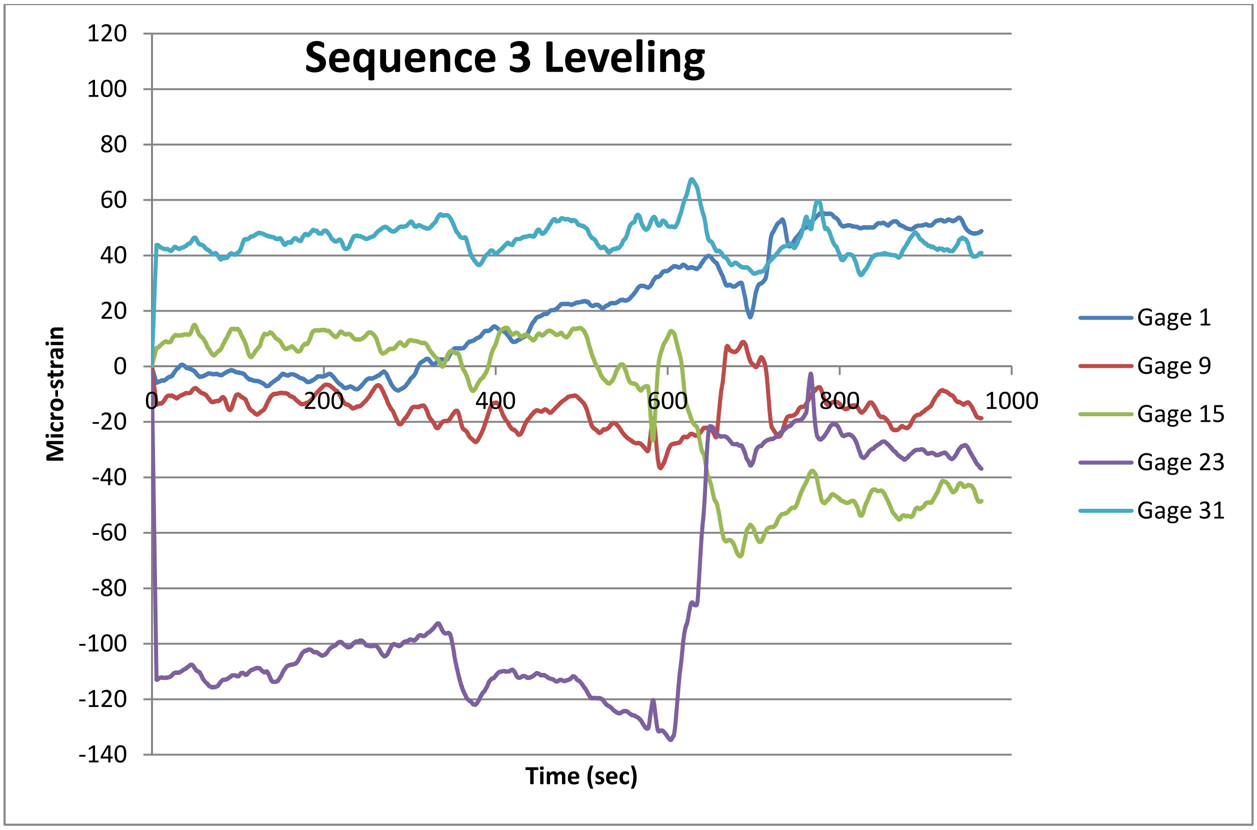
Day 2, Panel 1

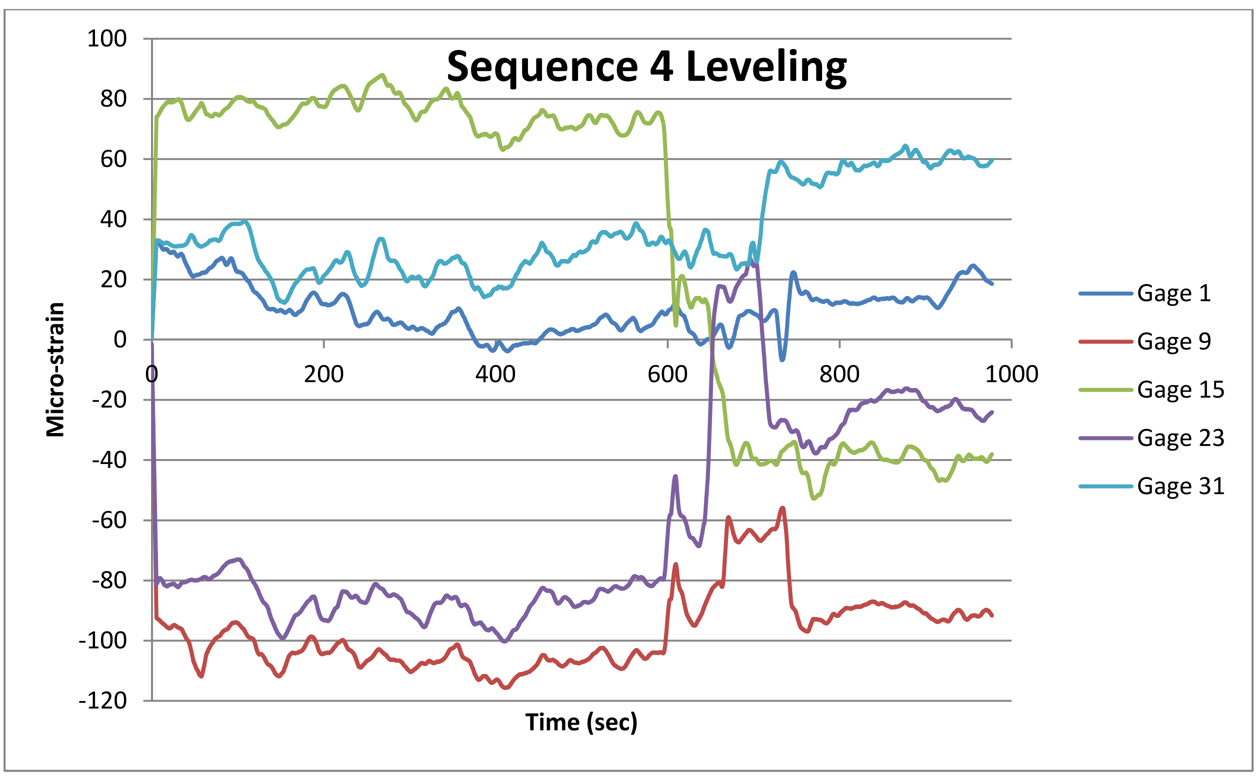




Transverse - East
Leveling

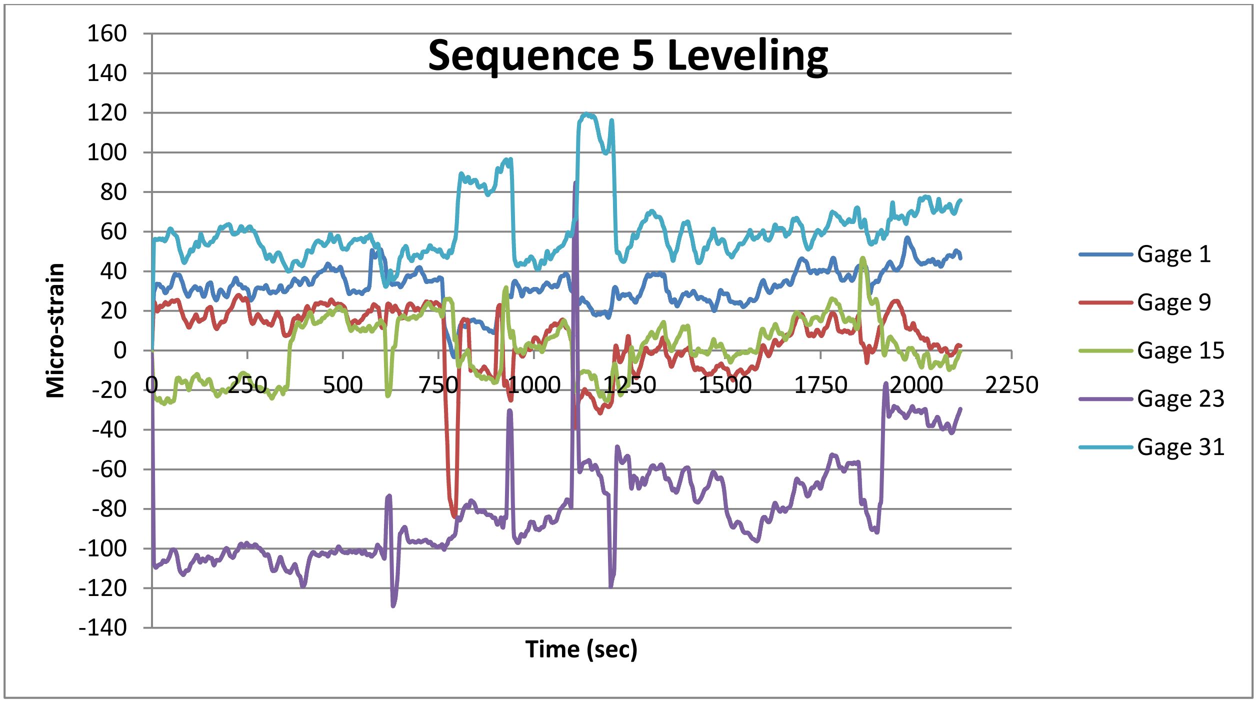
Day 2, Panel 1





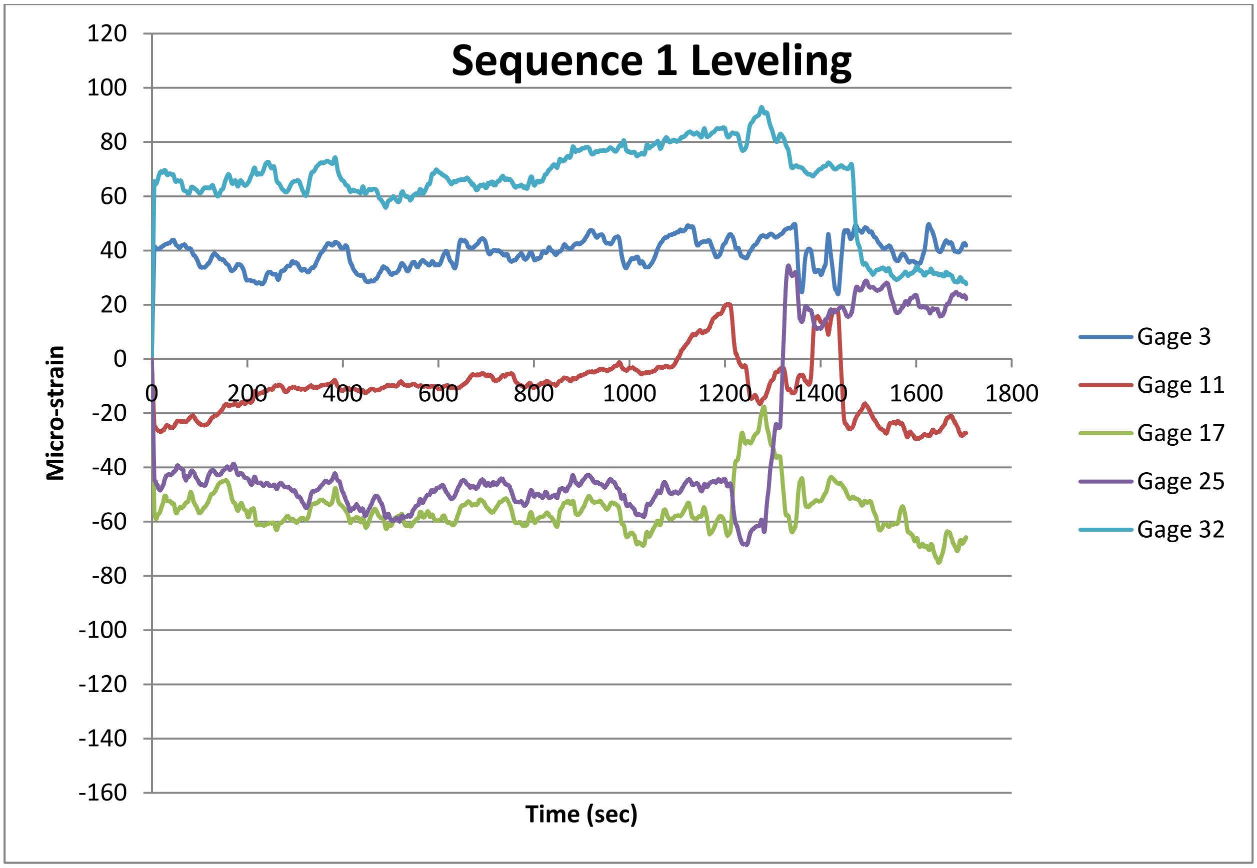
Transverse - East
Leveling

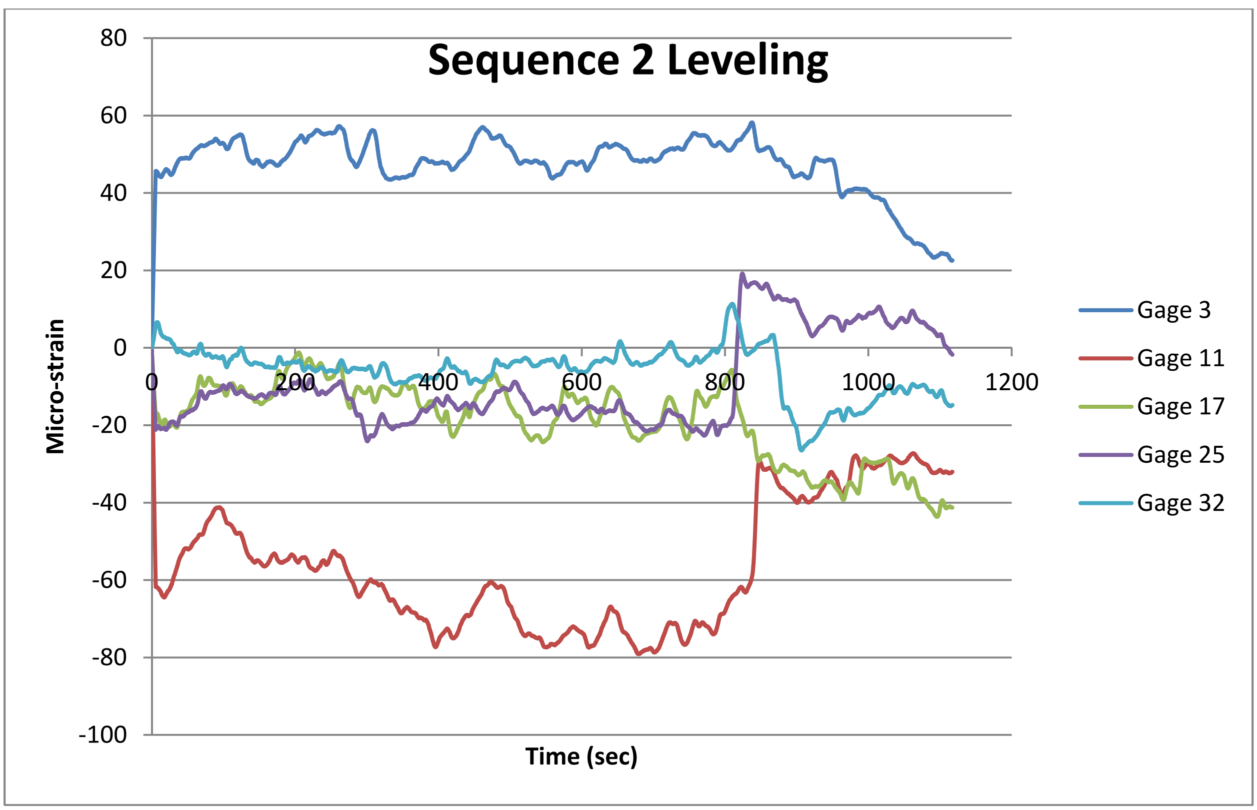
Day 2, Panel 1



Transverse - West Leveling

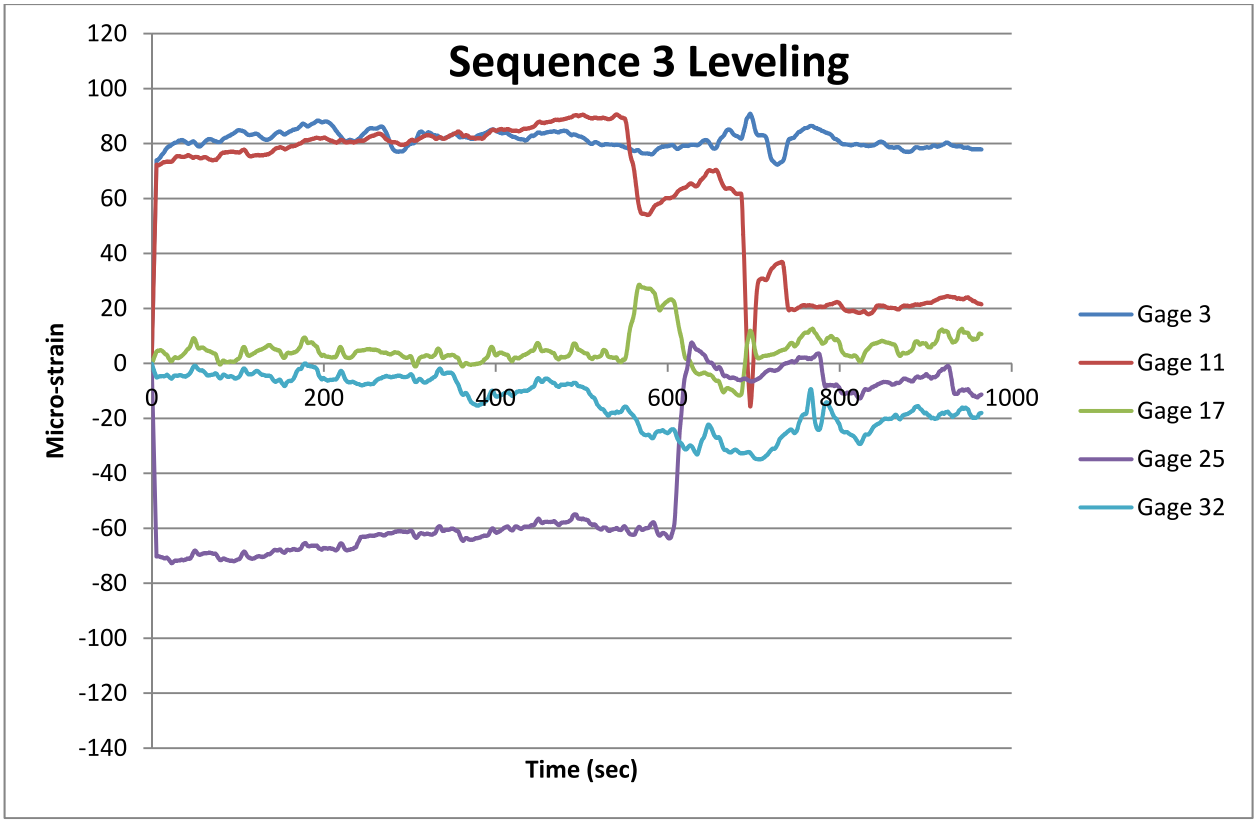
Day 2, Panel 1

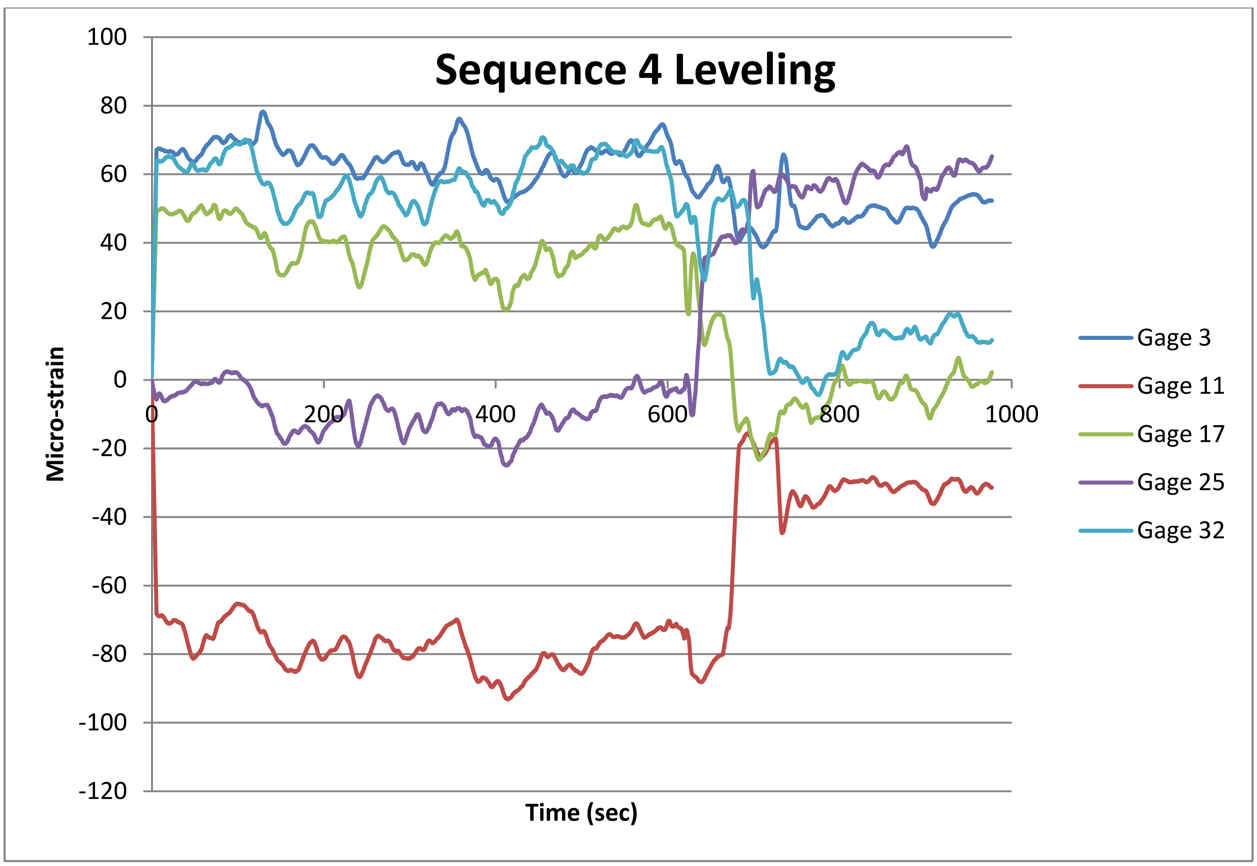




Transverse - West
Leveling

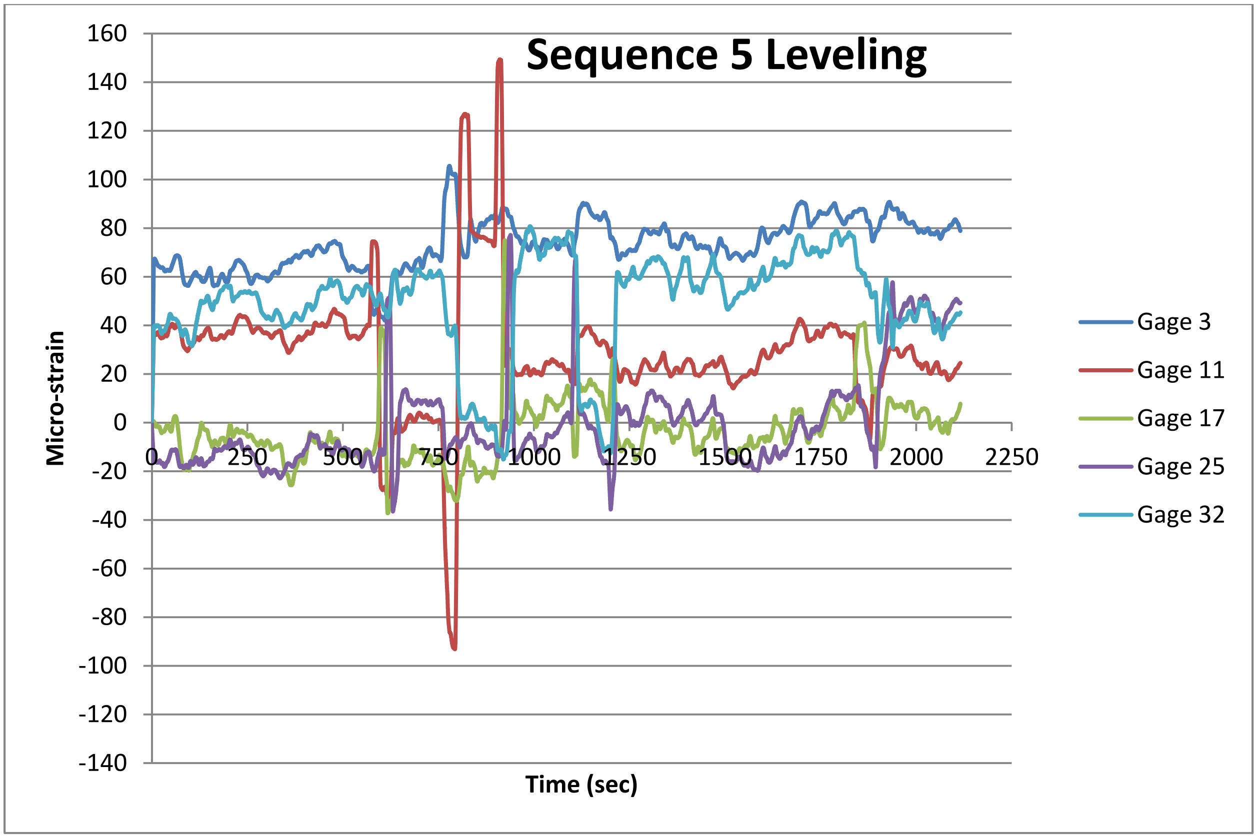
Day 2, Panel 1





Transverse - West
Leveling

Day 2, Panel 1



Day 2 - Panel 1

Leveling

Sequence 1 - Torque Reading

	Initial Reading				Torqued to		2nd Reading			
Timestamp	10:56 PM						11:19 PM			
	East		West		East	West	East		West	
Beam 1	10	1.514	50	33.502	35	35	20	9.893	35	41.919
Beam 2	147	71.53	32	-9.354	35	35	30	-6.467	25	-22.043
Beam 3	35	-65.615	5	-54.4	35	35	30	-63.841	5	-65.803
Beam 4	5	-145.062	12	-50.581	35	35	5	-83.549	50	24.339
Beam 5	5	12.579	105	69.439	35	35	35	44.336	35	32.089
Avg. Torque	40.6						27			

Theorectical strain		
	East	West
Beam 1	27.2	27.2
Beam 2	29.06	29.06
Beam 3	28.9	28.9
Beam 4	29.09	29.09
Beam 5	27.09	27.09

Sequence 2 - Torque Reading

	Initial Reading				Torqued to		2nd Reading			
Timestamp	11:47 PM						11:57 PM			
	East		West		East	West	East		West	
Beam 1	45	27.508	50	55.599	30	30	30	39.205	27	23.027
Beam 2	35	-22.219	5	-59.985	30	30	25	-18.931	25	-30.582
Beam 3	40	-32.869	30	-5.68	30	30	20	-32.392	5	-41.542
Beam 4	10	-69.035	10	-11.172	30	30	5	-54.658	30	3.526
Beam 5	15	28.25	75	-4.781	30	30	30	34.011	40	-9.786
Avg. Torque	31.5						23.7			

Sequence 3 - Torque Reading

	Initial Reading				Torqued to		2nd Reading			
Timestamp	12:24 PM						12:35 PM			
	East		West		East	West	East		West	
Beam 1	32	-1.456	40	85.913	30	30	20	52.192	30	81.098
Beam 2	2	-8.683	110	84.285	30	30	25	-8.14	30	25.011
Beam 3	80	5.987	5	3.376	30	30	5	-41.603	25	11.723
Beam 4	2	-104.342	5	-63.204	30	30	35	-31.842	10	-1.257
Beam 5	60	50.582	55	-5.476	30	30	30	42.133	30	-18.629
Avg. Torque	39.1						24			

Sequence 4 - Torque Reading

	Initial Reading				Torqued to		2nd Reading			
Timestamp	1:33 PM						1:43 PM			
	East		West		East	West	East		West	
Beam 1	42	3.317	60	61.916	30	30	30	24.524	30	53.686
Beam 2	5	-105.9	5	-78.944	30	30	10	-96.25	32	-32.285
Beam 3	105	75.211	30	32.322	30	30	12	-38.414	15	-3.065
Beam 4	5	-96.773	5	-15.688	30	30	10	-25.302	40	63.437
Beam 5	5	18.78	90	45.128	30	30	37	61.205	17	12.929
Avg. Torque	35.2						23.3			

Sequence 5 - Torque Reading

	Initial Reading				Torqued to		2nd Reading			
Timestamp	2:12 PM						2:45 PM			
	East		West		East	West	East		West	
Beam 1	40	27.638	40	57.772	30	30	30	49.802	30	81.834
Beam 2	35	16.658	80	35.925	30	30	10	-2.622	40	17.648
Beam 3	5	-17.664	5	-3.383	30	30	5	-8959	7	-0.145
Beam 4	5	-105.102	5	-17.466	30	30	15	-41.526	20	46.265
Beam 5	10	53.894	105	48.04	30	30	30	73.546	35	40.571
Avg. Torque	33						22.2			

Loose, needed hand-tightening prior to torqueing.

Coring of Shear Pockets, Haunches, and Closure Joint

Core #	Location	Observation
1	Beam Line 3W Shear Pocket/Panel Interface	Large cracks and separation towards top. Bond at deck/haunch interface
2	Beam Line 3W Panel/Haunch/Beam Interface	Deck and haunch completely separated. Not a good bond
3	Beam Line 4E Shear Pocket/Panel Interface	Good Interface
4	Beam Line 2W Shear Pocket/Panel Interface	Separation; no cracks
5	Beam Line 5E Panel/Haunch/Beam Interface	Very good bond
6	Beam Line 5W Panel/Haunch/Beam Interface	Good bond
7	Beam Line 5W Shear Pocket/Panel Interface	Good bond
8	Beam Line 2E Shear Pocket/Panel Interface	Good bond; crack first inch
9	Beam Line 2E Panel/Haunch/Beam Interface	Very good bond
10	Beam Line 1W Panel/Haunch/Beam Interface	Appeared to have a good bond
11	Beam Line 1W Shear Pocket/Panel Interface	Crack at shear pocket interface. Crack in haunch.
1A	Panel 1 - Joint Interface Epoxy Side	Some separation top part of keyway at taper part. Good bond rest of the way down.
1B	Panel 2 - Joint Interface Epoxy Side	Same as 1A
1C	Panel 1 - Joint Interface SSD Side	Separation at top taper. Otherwise good bond.
1D	Panel 2 - Joint Interface SSD Side	Good bond

Calculations for Small Scale Panel Testing

Concrete Unit Weight	$\gamma_c := .145 \frac{kip}{ft^3}$
Reinforcement Yield Strength	$f_y := 60ksi$
Panel Width	$b := 24in$
Panel Depth	$t := 8in$
Total Panel Length	$L := 15ft$
Unsupported Length	$L_{clear} := 13ft$
Distance from edge of panel to point load	$a := 5ft$
Area of steel (longitudinal, bottom)	$A_s := 4 \cdot 0.2in^2 = 0.8 \cdot in^2$
Distance from extreme compressive fiber to centroid of reinforcing steel	$d_s := 5.75in$
Weight of steel spreader beam	$P_{steel} := 425lbf$
Panel self weight	$w_{sw} := \gamma_c \cdot b \cdot t = 0.193 \cdot klf$
Concrete compressive strength the day of testing	$f'_{c.joint} := 6.5ksi$
	$f'_{c.control} := 7.7ksi$

Applicable equations

Moment before loading

$$M_{before\_loading} := \frac{w_{sw} \cdot L_{clear}^2}{8} \ldots + \frac{P_{steel}}{2} \cdot a = 61.76 \cdot kip \cdot in$$

Equation for moment after load (P) is applied

$$M_{total} = M_{before\_loading} + \frac{P}{2} \cdot a$$

where

$$\frac{a}{2} = 30 \cdot in$$

Theoretical Value

$$M_{r.joint} := A_s \cdot f_y \cdot \left[d_s - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f'_{c.joint} \cdot b} \right) \right] = 22.28 \cdot kip \cdot ft$$

$$M_{r.control} := A_s \cdot f_y \cdot \left[d_s - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f'_{c.control} \cdot b} \right) \right] = 22.39 \cdot kip \cdot ft$$

$$M_{r.avg} := \frac{M_{r.joint} + M_{r.control}}{2} = 22.33 \cdot kip \cdot ft$$

D. Pictures

Concrete Cores *Part I*

Part I - Panel Coring at Shear Pockets, Haunches and Closure Joints

Core 1 **Core 2** **Core 3**

Core 4 **Core 5** **Core 6**

Core 7 **Core 8** **Core 9**

Part I - Panel Coring at Shear Pockets, Haunches and Closure Joints

Core 10

Core 11

Core 1A

Core 1B

Core 1C

Core 1D