Gilford Rapid Bridge Deck Replacement
Project Summary

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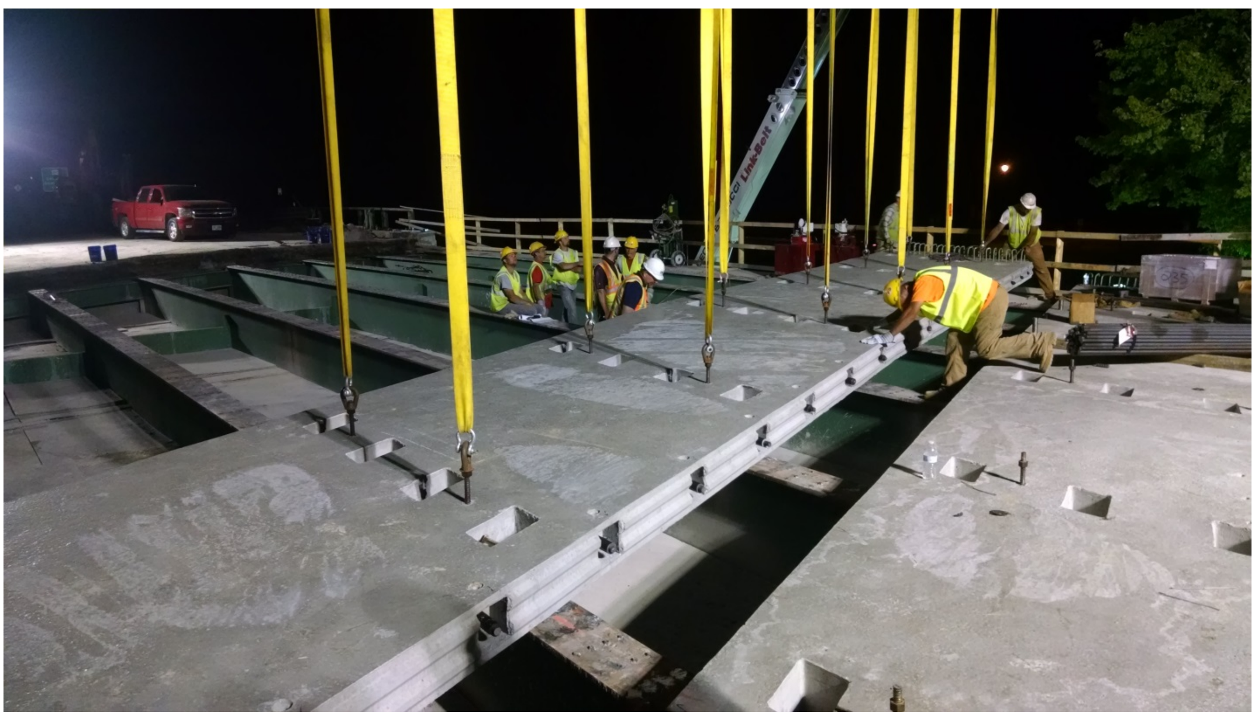

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The US 3/NH 11 Bypass over the NH 11A bridge in Gilford, NH rapid bridge deck replacement incorporated several techniques to minimize traffic congestion and bridge closure time during bridge deck rehabilitation. Many of the innovations integrated into the project were researched and laboratory tested by UNH faculty and students with involvement of the NHDOT. The research focused on replacing existing concrete bridge decks with precast concrete panels to minimize on-site construction time and extend service life at a higher level of performance. This system primarily focused on bridges where the deck had reached the end of its useable life but the structural steel had not. For this project, Accelerated Bridge Construction (ABC) used full width precast/prestressed deck panels fabricated to match the bridge skew and with the required deck cross-slopes, post-tensioned in the bridge longitudinal direction to create a monolithic deck with no closure pours between panels, and erected on adjustable leveling screws pre-set prior to placement. This project illustrates the constructability of these ABC details and provides field experience to improve the system on future projects.

The stimulus for this project comes from the current interest in ABC designs to meet the demands imposed by the continuing deteriorated condition of our nation's transportation infrastructure and congestion constraints. Limiting the closure time of critical bridges from months to days is often the only feasible method to complete projects with prohibitive detours, or unacceptable delays to the travelling public and emergency services response times. These accelerated projects have typically required the use of prefabricated bridge units (PBU's) or precast segments to limit the use of cast-in-place (CIP) structural elements and their required on-site curing times. However, most precast systems in use today require the use of CIP closure pours between prefabricated components. These closure pours are labor intensive to install and require the use of rapid setting concrete or grouts, particularly on bridge deck sections. The research team began evaluating improved methods of joining precast deck segments in 2006 with research on tongue-and-groove joint configurations reinforced by post-tensioning transverse to the joint. Improved bearing across the mating surfaces was achieved by using a two-step post-tensioning procedure. The first step is the application of a high-performance polymer sealant on both faces of the joint and post-tensioning to a magnitude only sufficient to squeeze excess sealant from the joint. After the polymer has achieved sufficient compressive strength, the full post-tensioning load is applied across the joint. By uniformly spreading and curing the polymer throughout the joint, the polymer provides improved bearing between the deck segments allowing much higher final post-tensioning stress to be developed across the joint without spalling or crushing at the high spots of initial contact. Following final post-tensioning in the laboratory, these sealed tongue-and-groove joints proved to be of greater strength than the concrete deck segments themselves.

Figure 1 - Polymer forced out of a tested joint following initial joint stressing.

Figure 2 - Tested joint showing failure behind the sealed joint.

Using this joint design, the team was able to develop a full-width deck panel concept that can be set across all girders on a bridge to eliminate the time, cost, and possible long-term performance losses associated with smaller panels and CIP joints. The goal was to develop a rapidly-erectable deck system consisting of high-quality precast elements held in uniform compression by prestressing the panels transversely and post-tensioning longitudinally (in the direction of traffic), resulting in a deck with superior durability as compared to a traditional cast-in-place deck and erected in a fraction of the time.

Full width deck panels have been used but typically consisted of a panel with a level bottom and the cross-slope profile cast on the top surface, set on girders with top flanges at the same elevation. These panels are thicker at the centerline and are significantly heavier than a traditional uniform depth deck. These types of deck panels are limited to use on narrow bridges as increasing their width increases the dead load of the deck. Casting a constant-thickness panel that follows the deck cross-slope makes the dead load similar to that of a traditional deck. This allows for panels to be precast as long as the width of the bridge deck they are replacing, at a width of 8'-10' to maintain the ability to transport them by truck, and with a constant panel thickness of 9". These full-width panels require an adjustable method of bearing on each girder to maintain even distribution of dead load as well as to maintain the deck grade. Pockets must also be cast into these panels to accommodate shear studs welded to the steel girders following erection as well as ducts through the center of the panels to accommodate post-tensioning bars. The length and thin aspect ratio of these panels requires multiple support or blocking points during storage and transportation and spreader beams with multiple rigging points to prevent cracking during lifting and handling. Research was conducted to develop solutions to these design challenges.

As the developmental research progressed, NHDOT proposed demonstrating the system by replacing the deck of the US 3/NH 11 Bypass over NH 11A bridge. Only the deck required replacement as the existing steel girders were in good condition. The Gilford bridge consisted of a single 76' span carrying two lanes of US 3/NH 11 Bypass, plus shoulders, over two lanes of NH 11A. The superstructure consists of seven W36 girders carrying an 8 inch concrete deck paved with 2¼' of asphalt. The bridge is skewed at an angle of 23 degrees relative to the centerline of US 3. The preliminary concept involved the use of nine panels with a width of 8'-4" and length of 52'-6" along the skew. Preliminary analysis showed that the deck depth would likely have to be increased to 9 inches to accommodate the prestressing and post-tensioning elements anticipated in the panels as well as mild reinforcement.

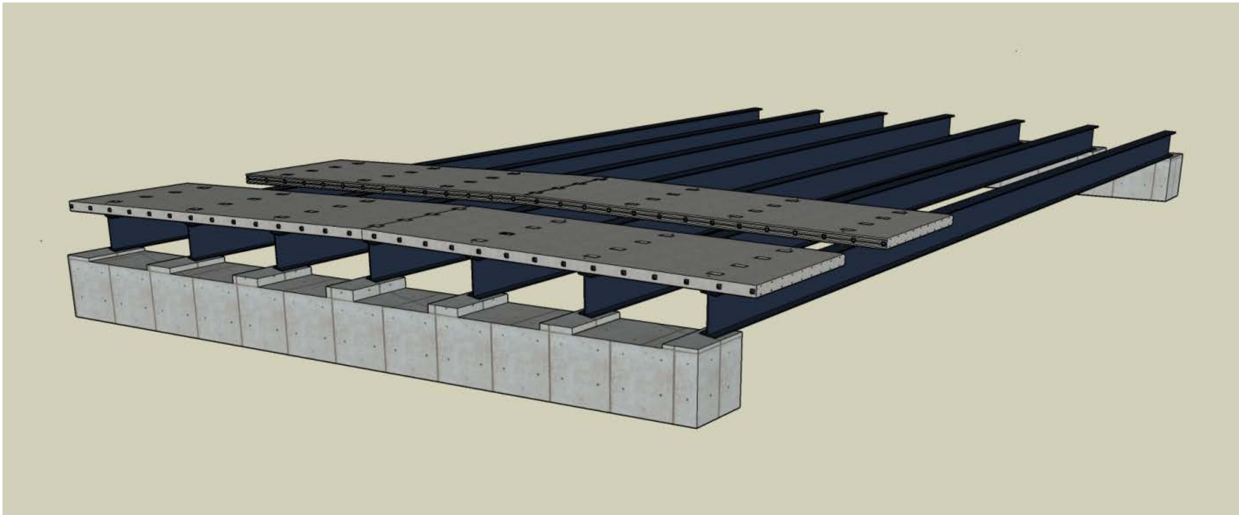

Figure 3 - Gilford deck replacement concept.

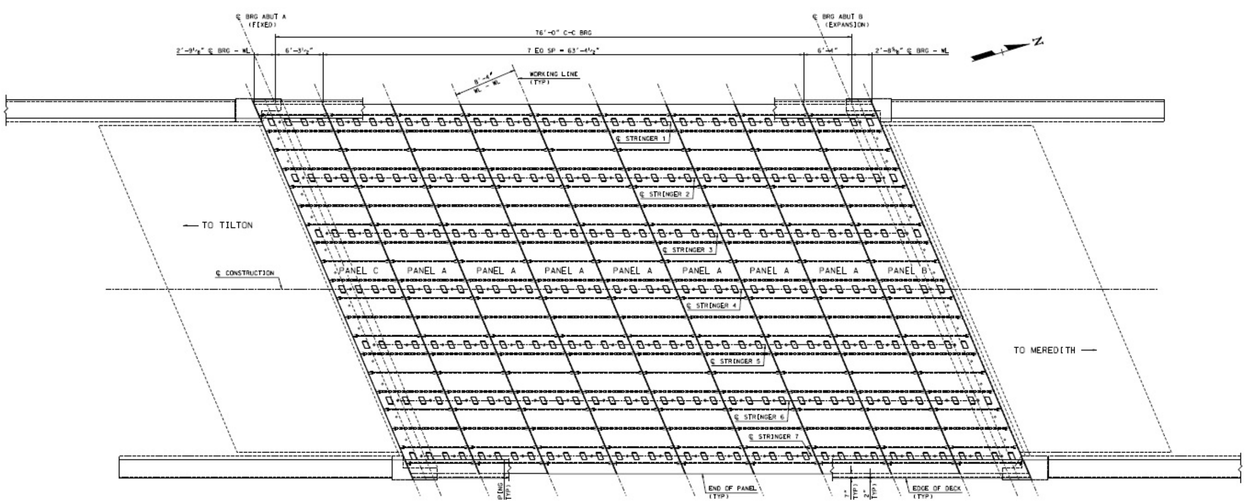

Figure 4 - Deck panel layout.

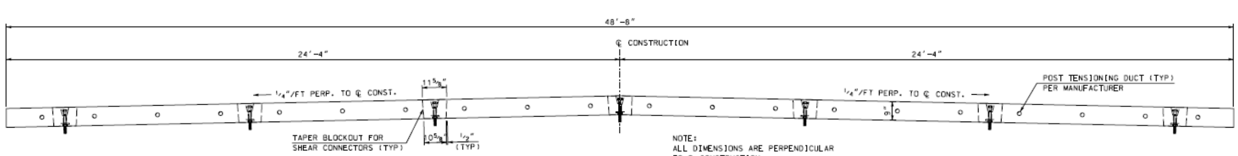

Figure 5 - Typical panel cross-section.

The team conducted research over several years to evaluate the panel design constraints and demonstrate proposed solutions. The primary question was whether a crowned deck panel could be prestressed without developing moment around the peak at the center. After verifying with hand-calculations that prestressing tendons that remained concentric about the panel centroid were not likely to develop such a moment, a 1/8th scale model was fabricated. Prestressing was accomplished with galvanized aircraft cable instrumented to monitor stressing tension. Blockouts were made in the model panel to represent shear connector pockets and wire mesh was used to simulate traditional reinforcement. After casting, the prestressing cables were released from the bed and the model monitored for cracking, particularly at the panel apex. No cracking has been observed through the writing of this paper; however, the ends of the model have cambered up approximately ¼” in 4 feet. This is in the opposite direction to the cross slope cast in the panel. Further investigation showed the placement of the prestressing strands was not concentric to the panel centroid with an eccentricity of approximately 1/32” due to the difficulty in maintaining tolerances on a small-scale model. The scale model appeared to verify that such a panel could be prestressed successfully.

Figure 6 – Panel scale model showing prestressing cables.

With the panel concept initially tested on a small scale, research focus was shifted to developing a method of supporting the panels during transportation. Over-the-road transportation was considered in which panels of up to 60’ long would be subjected to dynamic loading and torsional loading when transported on a traditional flexible trailer. A frame was designed to rigidly support the panel by spanning the length of the trailer carrying each panel on blocking supports. A spreader beam was designed to attach slings to each lifting/leveling screw cast into a panel and allows the panel to be lifted and handled without developing excessive tensile stresses. This proved to work well as no cracks developed in any of the nine panels.

As planning for the deck replacement project continued, the problem of traffic control around the closed bridge and road underneath the bridge was discussed. The team proposed the concept of turning the four-ramp bridge interchange into an improvised traffic “circle” by adding paved cross-overs between the ramps and US 3/NH 11 roadway to allow vehicles to progress around the closed interchange while having the option of continuing to travel in any direction desired.

The team performed an initial assessment of the crossover grades and transitions to ensure all truck traffic could pass through the temporary traffic “circle”. Due to the extensive temporary change to the traffic pattern around the bridge, it was determined that police officers or flaggers should be positioned at all times at each entry point to direct drivers on navigation of the closure.

Figure 7 - Proposed traffic flow around the work zone (center).

The final research topic was the development of a way to pre-set threaded leveling screws heights prior to panel erection to provide a correct final roadway profile and specified distribution of load to each girder with a minimum of field adjustment. Challenges addressed included the fact that girders under an existing bridge would experience an unknown amount of rebound towards their original cambered condition. The leveling screws bear on the rebounded girder profiles, which continuously change as panels are placed, and must provide a correct top of deck profile when all deck panels are installed. This constant change in grade as panels are added to the bridge makes grade checks during erection difficult. The proposed solution was to develop software that would give leveling screw lengths for each panel based on a survey of the girder profiles following the existing deck demolition.

Development of the software included several structural models in SAP2000 to characterize the behavior of the Gilford bridge after demolition, while the deck dead load was distributed on the leveling screws alone during erection and during full composite behavior after deck completion. Information from this modeling was used to develop an Excel spreadsheet that would calculate the required length of each leveling screw to create the required profile. To maintain design dead load distribution, the leveling screws are extended up through the top of the panel through greased sleeves and topped with a welded nut. This allows the bearing of each leveling screw to be checked with a torque wrench. A staggered pattern working from the center girder out was recommended to check the torque on each leveling screw prior to post-tensioning. These leveling screws were also intended to be the panel rigging points by threading an eye bolt onto the top of each protruding leveling screw.

To validate the software results, a scale model consisting of four 4 foot wide slab sections 16 feet long placed at a 22 degree skew on four 27' long steel girders was constructed in the lab at UNH. The beams were not cambered, but grade and rebound variations were simulated by placing shims under the ends of each beam prior to each erection test. The elevation at the ends and quarter-points on each beam were surveyed and the results entered into a version of the Excel software modified to analyze the lab model. It was found that the four panels could be rapidly set to an accurate grade using the calculated leveling screw lengths at any combination of random shims under the ends of the beams. The lab tests showed a grade tolerance within 1/8". The final lab demonstration included the application of the polymer and post-tensioning of one joint and was filmed. Contractors that bid on the Gilford bridge deck replacement project were shown the demonstration video as part of the bid package.

Figure 8 - Full-scale constructability test at UNH.

The project began on June 3rd, 2016 with the closure of US 3/NH 11 Bypass and the demolition of the existing deck. US 3/NH 11 was closed just before 8 p.m. and traffic diverted to the temporary traffic "circle". The contractor began by removing temporary Jersey barriers that had been placed along the existing fog lines on the bridge. Demolition of the granite curbs and bridge rails had been previously completed behind these barriers. Shielding sufficient to support the weight of demolition debris had also been previously placed between the girders. Crews below the bridge began by drilling up through the deck next to the girder flanges to accurately locate them from the top side. String lines were painted longitudinally at 1.25" offsets from the flange edges towards the beam centerlines. These lines would denote longitudinal close to full-depth sawcuts. Transverse stringlines were painted to define a grid of slabs supported by 1.25" along the edge of each girder. The contractor's engineer had determined that this overlap was sufficient to support the weight of a 30 ton excavator and other smaller equipment operating on the sawcut deck. The copings were also sawcut to facilitate their removal in roughly 6' sections. The sawcutting began at 10 p.m. and was completed by 12:30 a.m. with up to seven saws operating simultaneously.

Figure 9 - Sawcutting in progress showing slab layout.

Figure 10 - Segmental coping demolition.

Deck demolition began on the north end of the deck with removal of coping sections by crane and hydraulic hammer demolition of the angled segments of deck left between each row of rectangular sawcut slabs and the skewed deck end. When removed, this created clearance for an excavator to peel up each slab with a slab bucket and place it whole into the bed of a dump truck. After the removal of several slabs sections, crews began chipping the haunch concrete remaining over each girder engaged by the existing shear connectors. This concrete was initially broken up with a hydraulic hammer on a rubber tired backhoe and final removal was accomplished with chipping guns. Demolished concrete was consolidated on the shielding and loaded into scale pans to be removed from the bridge by a 30 ton hydraulic truck crane. Two scale pans were cycled removing roughly 1/3CY of demolition debris at a rate of 9-10 minutes each cycle. As haunch concrete was removed, the shear connectors were torch-cut from the flanges. The location of each new panel leveling screw was laid out and the flanges were ground smooth in these locations. Demolition was completed with all girders prepped for panels by 5 p.m. Saturday June 4th. Deck demolition began at 12:30 a.m. with an approximate total duration of 16 hours and 30 minutes.

Figure 11 - North deck end demolition.

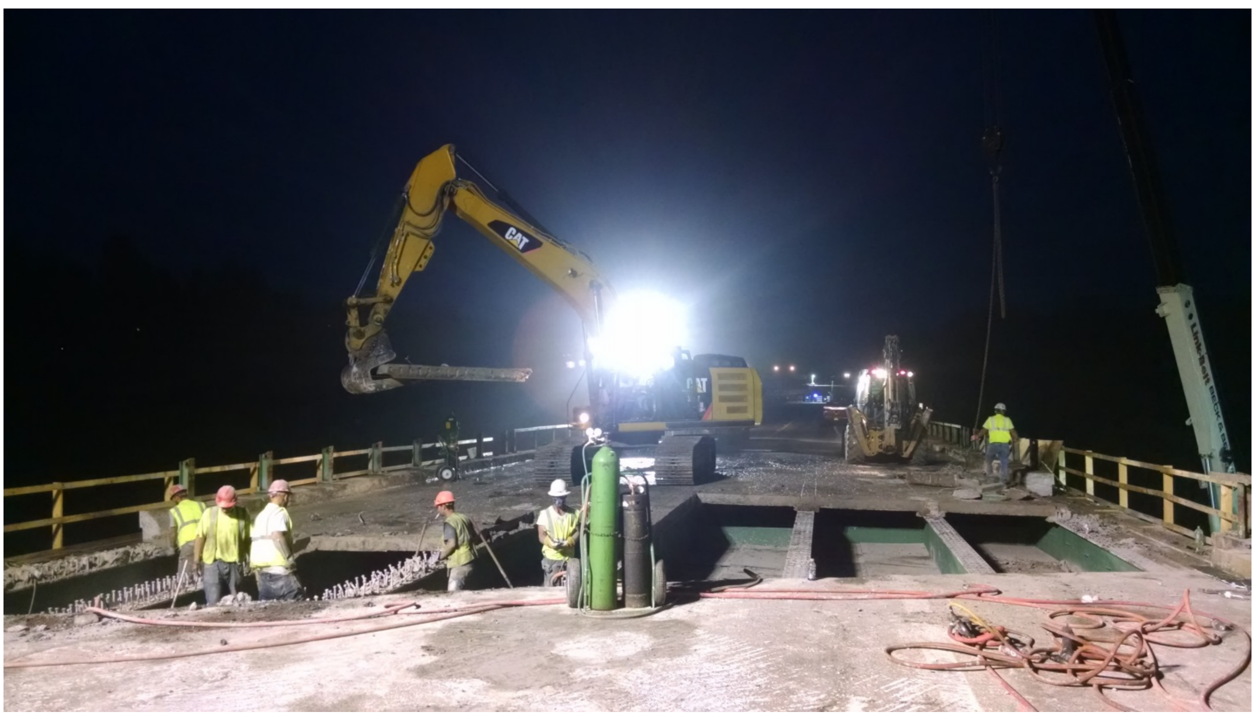

Figure 12 - Slab removal with haunch concrete demolition and shear connector removal in progress.

Figure 13 - Deck removal nearing halfway point.

Figure 14 - Leveling screw location preparation.

The deck panels were staged approximately one mile south of the bridge. When inspected prior to erection, the panels were found to have no cracking resulting from handling and transportation. A bed of H-piles were placed on a gravel pad with appropriate blocking to support the panels. Sufficient clearance was available under the panels to set the leveling screw heights prior to load-out.

Figure 15 - Panels staged on blocking prior to erection.

Figure 16 - Loading out Panel 1.

Figure 17 - Panel on shipping frame (bottom) with lifting frame (top) prior to erection.

At 6:30pm, the first panel was loaded onto the designed trucking frame by a 150 ton crawler crane and transported to the bridge location where a 210 ton truck crane was set up on the NH 11A underpass to erect the panels. The first panel was placed at 7:45pm on the north end of the bridge.

Figure 18 - Setting Panel 1 at 7:45 p.m.

After setting in position, the leveling screws were checked with a torque wrench to ensure that each leveling screw was bearing on a girder with the appropriate amount of panel dead load. It was noted that several screws appeared to have excessive bearing while others were not making contact, which was adjusted with the torque wrench. Overall panel grade appeared to be accurate when surveyed. Post-tensioning bars were inserted into the ducts and the panel was post-tensioned to itself. The second panel was lifted at 9:50pm and set on the girders to allow placement of the joint sealing polymer.

The polymer components were stored on site inside insulated boxes partially filled with cold water to prolong their storage life and to reduce their pot temperature when mixed to provide maximum working time. After verifying that the panels were ready to be drifted together, the full crew was mobilized to mix and apply the polymer. Mixing was conducted in gallon plastic pails with drill mixers. The manufacturer advised that the polymer be mixed and applied across the entire joint within 9 minutes to ensure workability. In practice, the polymer was mixed and applied within 8-12 minutes with the longer periods showing increasing difficulty in consistent application as the polymer viscosity rapidly increased. After polymer application the panels were drifted together to as tight as possible with a separation of roughly 3/4 of an inch at the capacity of two 2-ton chain jacks. Partial post-tensioning to 10% of the final load was performed to mate the panels immediately after contact was made, working from the center out with two stressing crews. It was noted that contact sufficient to force excess polymer out of the joint was only apparent over a portion of the total joint length. The crew poured excess liquid polymer into the remaining gaps. Final post-tensioning began 45 minutes after initial post-tensioning was completed.

Figure 19 - Polymer component cold storage on site.

Figure 20 - Applying mixed polymer along Joint 1.

Figure 21 - Typical polymer application by hand.

The third panel was set at 12:30 a.m. and the procedure was repeated. Due to crew experience and a greater quantity of applied polymer, a much larger portion of the joint showed excess polymer being squeezed out after partial post-tensioning indicating improved joint bearing. At this point it was noted that the third panel had drifted to the east by roughly 3/4" after final post-tensioning. The fourth panel was set at 2:45 a.m. and efforts were made to adjust the panel location west and put the deck back on centerline. The crew had difficulty in making the post-tensioning bar couplings and required the panel position to be adjusted by the crane several times to complete them. It was also noted around 3:00 a.m. that striations were appearing in the previously completed joint polymer surfaces indicating lateral deflection due to the post-tensioning load. The fourth panel was mated and initially post-tensioned after a lateral position correction was successfully completed. Due to the previous joint deflections, it was decided to wait two hours before final post-tensioning to allow the polymer to cure longer. Around 5am, alignment marks were placed between panels 3 and 4 to monitor joint deflection as the joint was post-tensioned. A deflection of roughly 1/8" was noted after post-tensioning was completed.

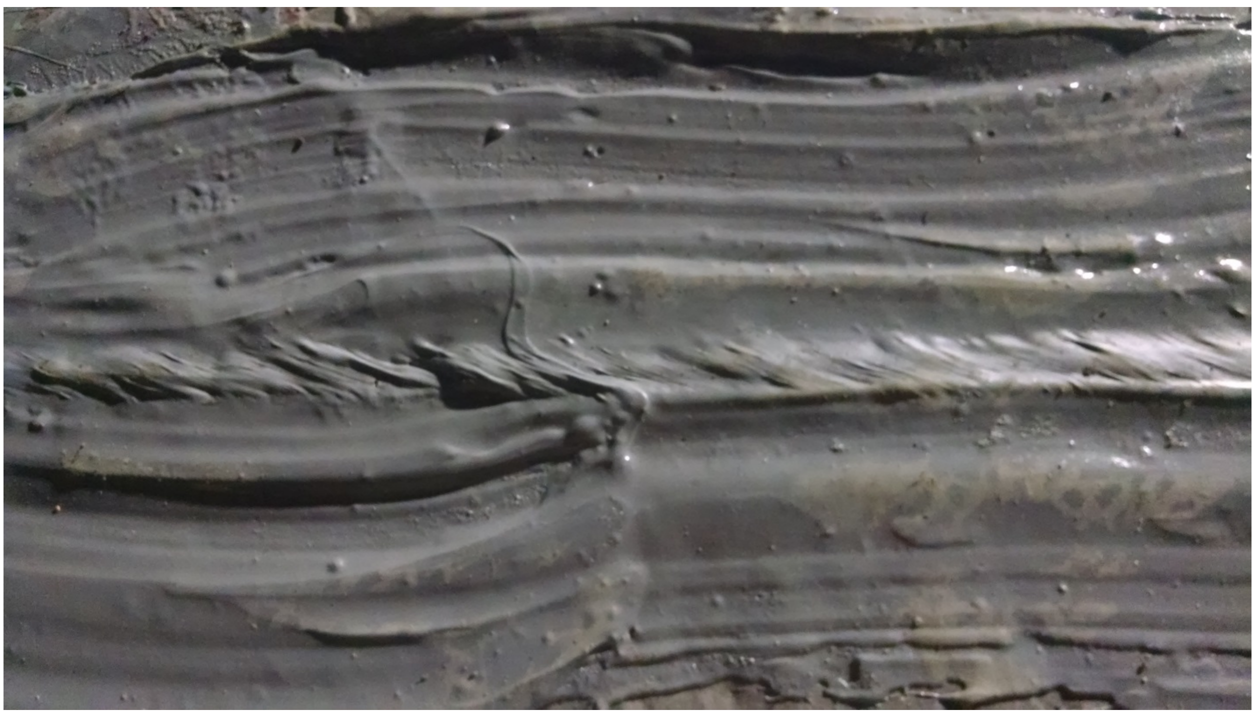

Figure 22 - Joint 1 polymer striation.

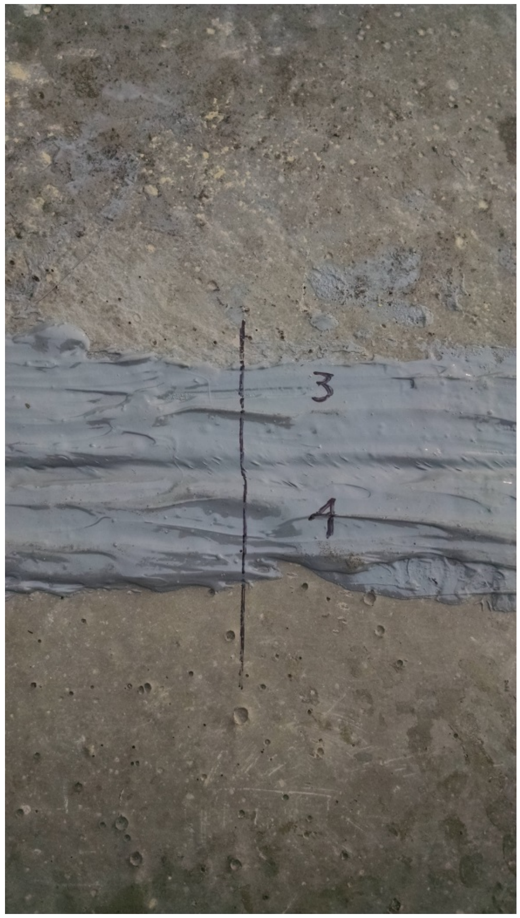

Figure 23 - Panels 3 and 4 joint deflection after post-tensioning (alignment line drawn prior to post-tensioning).

The fifth panel was placed at 5:45 a.m. Sunday, June 5th. The chain jacks used to drift the panel into position were placed at a roughly 45 degree angle to the centerline of the bridge to provide lateral resistance against slippage along the joint while drifting the panels together and initial post-tensioning. This proved to be successful in continuing correction of the panel centerline alignment.

Shear connector installation in the first panel pockets began at 8 a.m. Alignment marks were made between panels 4 and 5 just prior to final post-tensioning and showed the same roughly 1/8" deflection after post-tensioning was completed 1.5 hours after initial post-tensioning. The sixth panel was placed at 8:45am and mated at 10am. The delay was partly due to the commencement of light rain and the placement of plastic sheeting across the joint prior to polymer application as the polymer could not be applied to a wet surface. Due to the forecast for heavy rain for most of the day, the decision was made to stop panel installation. Over the remainder of June 5th, panels 7 and 8 were staged on the girders and end diaphragm concrete form construction began.

On the morning of June 6th, delamination, separation, and severe striation of the polymer was observed on most of the previously completed joints. The joint between panels 3 and 4, which had experienced the longest polymer curing time prior to final post-tensioning, was in the best visual condition, but the deflection between alignment marks had grown to nearly ½".

Figure 24 - Polymer delamination due to continued deflection.

A basic grade check was performed between the staged panel 8 and the existing south approach roadway and was found to be nearly level; the leveling screws on panel 8 having been previously set with the UNH software. Panel 7 was mated with Panel 8 around 9:45 a.m. and the chain jacks were again used to compensate for lateral slippage. Alignment marks were made prior to initial post-tensioning and showed nearly ¼" of eastward deflection after initial post-tensioning. Panel 8 was mated at 12:15 p.m. After final post-tensioning of panel 8, the deflection between panels 6 and 7 grew to nearly 3/4". After final post-tensioning, the deflection was roughly 1 inch from the panel alignment when initially mated. At this time, significant failure of several previous joints was observed as lateral striations in the polymer broke into gaps of up to 1 inch.

Figure 25 - Apparent joint failure with large surface deformation and polymer break-up.

Panel 9 was hoisted to the bridge at 12:30 p.m. A section of the panel 6 surface was found to have spalled at the joint with panel 7 due to apparent uneven bearing between the panels and a resulting stress concentration. At 3:20 p.m., the joint between panels 3 and 4 was observed to have apparently failed with a total deflection of over 1 inch.

Figure 26 - Apparent failure of the Panels 3 and 4 joint (3:20pm 6/6/16).

Panel 9 was mated with panel 8 at 4:30 p.m. completing the deck and with joint deflection behavior similar to the previous panels during initial post-tensioning. It was decided not to finish post-tensioning of panel 9 and prevent renewed mobilization at the rest of the failed joints. The longitudinal post-tensioning stress was apparently relieved with each joint failure and a method to prevent panel lateral slippage was needed prior to final post-tensioning of the deck. Several methods were discussed including the use of cast-in-place reinforced shear keys, but it was ultimately decided to abandon post-tensioning and grout the shear connector pockets and haunches to tie the superstructure together and allow the bridge to behave as a typical composite structure.

On the afternoon of June 7th, and prior to grouting, a "bang" was reported followed by noticeable movement of previously welded shear connector groups on the east side of the bridge to positions against the north side of their pockets. After evaluation of the residual post-tensioning force, which was determined to have been likely completely relieved, it was determined that the deflection was likely due to differential thermal expansion. The deflected shear connector groups were observed only on the northern end of the bridge, which is the expansion bearing end, and on the side of the bridge in sunlight for most of the day. Additionally, the deflections were in the range expected for the temperature fluctuations the bridge experienced over the previous 24 hours.

Figure 27 - Shear connector group after apparent girder expansion prior to grouting.

The shear connector pockets, post-tensioning ducts, and haunches were grouted on June 9th. By June 10th, additional sealant had been placed over the joint slippage locations and the deck was sandblasted and opened to traffic with barriers providing crash protection prior to bridge rail installation. Over the following month, the parapet and bridge rail was installed, the temporary traffic “circle” was removed, and the project area was paved.

Figure 28 - Deck following shear connector and haunch grouting.

Figure 29 – Sandblasted (l) versus non-sandblasted (r) deck surface.

Traffic control during the project duration performed well. The only period of significant backup of US 3 traffic was during the evening of Friday, June 3rd when a large event concluded north of Laconia. Police officers directing traffic said that most drivers seemed to understand the new pattern and there were few incidents of drivers attempting to enter the closure. There were also no significant issues with large vehicles navigating the cross-over points. The concept appeared to be successful and could be applied to similar types of interchanges in the future.

Figure 30 - Crossover between an off-ramp (l) and US 3 (r) with traffic flow to the left.

This project demonstrated the success of numerous innovations. The full-width deck panels, which included the roadway skew, showed that a thin concrete panel with a crown could be successfully prestressed, transported with an adequate frame support, and erected without producing cracking in the panels. The software developed to compute the leveling screw lengths resulted in quick and accurate panel placement with minimal field adjustment. Finally, the traffic management concept of turning a conventional interchange into a traffic "circle" performed well without significant impacts to the traveling public.

To evaluate the cause of the poor joint performance, the testing performed in the UNH laboratory was reviewed in detail. The full scale laboratory test, discussed earlier in this paper, was performed on four 16 foot long, 4 foot wide, and 9 inch thick panels supported on four steel girders. The first panel was installed and post-tensioned to itself using three 1" steel post-tensioning bars pulled to 57,000 pounds each. The second panel was set and polymer was applied to the joint mating surfaces and the panels were drifted together with a crane. The post-tensioning across this joint was at a 23 degree skew to the joint's transverse axis as intended to

be applied on the Gilford bridge. The panels fabricated in the lab did not meet project fabrication tolerance of +/- 1/4” inch, which resulted in a joint gap of up to ¾” at one end of the joint when mated. The amount of polymer that was applied was sufficient to fill the joint for a length of approximately 10 feet out of 16 feet. To close the end of the joint not fully mated, a post-tensioning rod was stressed causing slippage between the panels as the polymer was not sufficiently cured to resist the shear along the joint induced by the post-tensioning.   No further testing was done on this full scale model. When dismantled, this joint did not appear to show any of the cosmetic damage or failure that was observed in the field during the deck replacement. However, the laboratory joint was not fully post-tensioned within hours of erection, which is when the failures were observed in the field. It is apparent that further polymer studies need to be conducted to determine curing time, creep, and elastic behavior of these joints at all stages of erection.
The behavior of the skewed joints indicates that post-tensioning loads should only be applied perpendicular to a joint unless shear keys or other mechanical means are provided to resist the lateral reactions. The liquid and semi-liquid polymer appeared to provide a lubricating effect when the panels were initially mated and continued to creep after elastic deformation when final post-tensioning was applied. This appears to show that polymer joint sealants should not be required to resist loads other than pure compression across the joint.

The team continues to evaluate the lessons learned on this project to advance the state-of-the-art of Accelerated Bridge Construction in New Hampshire.