MONITORING PRECAST DECK PANELS FOR LOAD AND ENVIRONMENTAL EFFECTS

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ABSTRACT

As part of Strategic Initiative #9, Building Bridges Smarter, Faster Cheaper, the Ohio Department of Transportation has a pilot project which uses precast, prestressed concrete deck panels. The bridge is a single 170 foot span, steel plate girder bridge. The 41 foot by 10 foot 3 inch wide deck panels are post-tensioned in the transverse direction (which is perpendicular to the bridge girders) in the plant and are then longitudinally post-tensioned together after being erected. After post-tensioning, shear studs are welded to the top of the beams at pockets cast in the panels. The panels are secured to the beams by grouting these pockets. Vibrating wire strain gages were placed in the grouted shear keys between some of the panels. These gages were monitored during the lateral post-tensioning process (October 2004). Data shows that the expected level of stress was obtained in all the joints. The gages have been monitored for environmental effects since construction and the results show effects of creep, shrinkage and temperature. In March 2005, a load test was performed. The joints in the bridge were sound and uncracked and the load test results indicated that the bridge was behaving as a composite structure. This paper details the construction of the bridge and gives preliminary results of the environmental monitoring and the load tests.

Keywords: accelerated construction, bridge, deck, monitoring, panels, precast

INTRODUCTION

Ohio has the 10th largest highway network, the fifth highest volume of traffic, the fourth largest interstate network, the fourth largest amount of freight shipments and the second largest inventory of bridges in the United States. Due to the shear volume of truck and automobile traffic, major delays along Ohio highways because of construction can have significant economic consequences. The Ohio Department of Transportation (ODOT) instituted Strategic Initiative #9 (SI-9) – "Build Bridges Smarter, Faster, Cheaper" in an effort to reduce delays on Ohio bridges. This initiative sought to reduce the time bridges are closed or restricted due to construction ("down time"), even if the overall project time increased.

ODOT has considered many way of reducing down time, including innovative technologies and contracting methods. One technology which ODOT finds promising is the use of prefabricated elements[1]. As part of the SI-9 project, a survey of contractors was conducted. It was noted that forming, casting and curing a concrete deck slab consumed quite a bit of time. ODOT decided to try precast/post-tensioned, full depth deck panels as a means of reducing the time needed to place a concrete deck on a structure.

DESCRIPTION OF THE BRIDGE AND PANELS

The bridge in this project is HAN-75-15.99 over I-75 in Findlay, Ohio. This bridge is a 170 foot single span steel plate girder bridge. The deck is comprised of 16 precast, prestressed concrete panels, each 10 foot, 3 inches long and 41 feet wide, placed transversely on the girders (Figures 1 and 2).

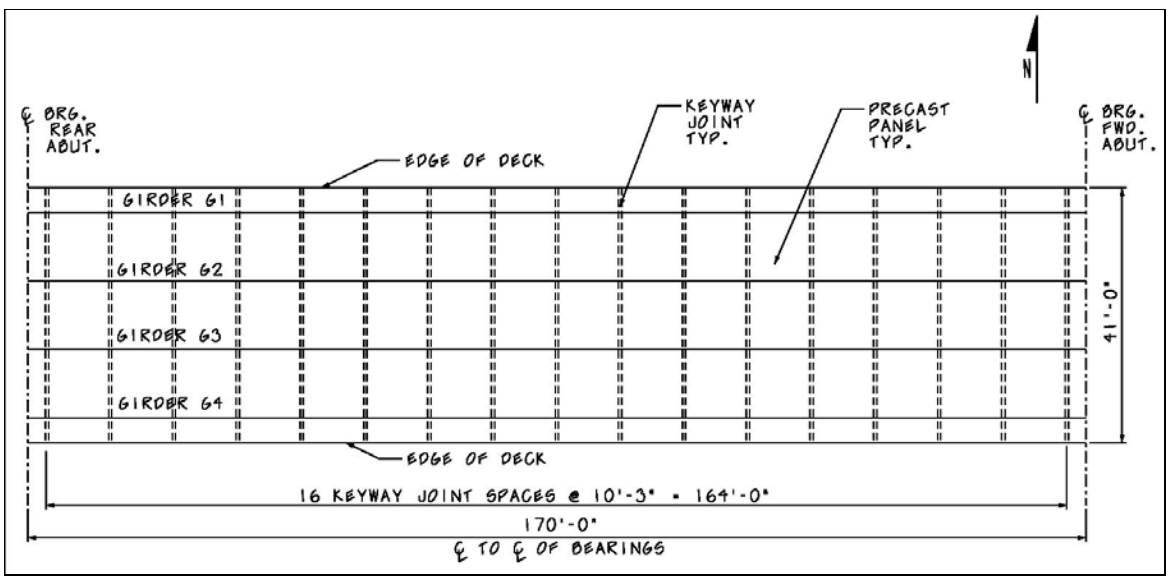

Figure 1 Panel Layout

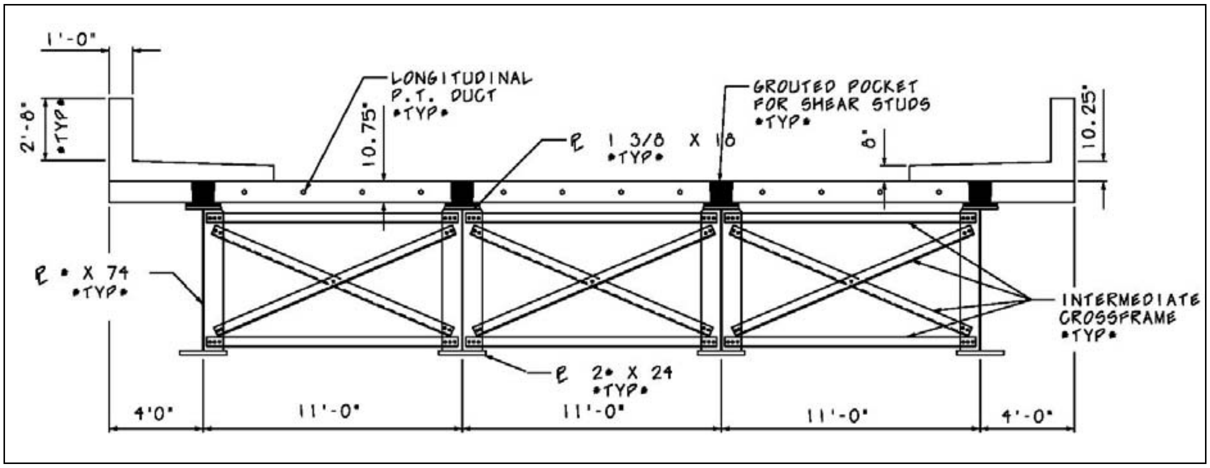

Figure 2 Typical Transverse Section (Transverse post tensioning omitted for clarity)

The panels were fabricated at the plant and shipped to the site where they were placed side by side on the girders. Shear key joints between the panels were grouted and the panels were post-tensioned longitudinally to the direction of the bridge (transverse to the panels). After post-tensioning, the panels were attached to the girders using shear studs in grouted pockets.

DETAILS OF THE PANELS

Details of the concrete deck panels are shown in Figures 3 and 4. The deck panels have four post-tensioning ducts that run longitudinal to the panel (transverse to the bridge). These ducts contain four 0.6 inch diameter uncoated seven wire strands which were tensioned and grouted before the panels left the fabricator's yard (Figure 5). The concrete was to be at least 5500 psi at this time. The maximum jacking force per tendon was specified 45k/strand or 180 kips total for the 4 strand tendon. For two tendons in one panel, a friction test was performed by placing a load cell at the deadhead (unjacked end). The friction loss was found to be approximately 6-9%. Seating loss, as measured by the change in elongation during the seating process, was 2-4%.

There are 12 transverse (longitudinal to the bridge) post tensioning ducts in each panel. After assembly of the panels on the bridge, four 0.6 inch diameter uncoated seven wire strands were pushed into these ducts and tensioned. The tensioning procedure is explained in a later section No friction test was performed on the strands longitudinal to the bridge (transverse to the panels).

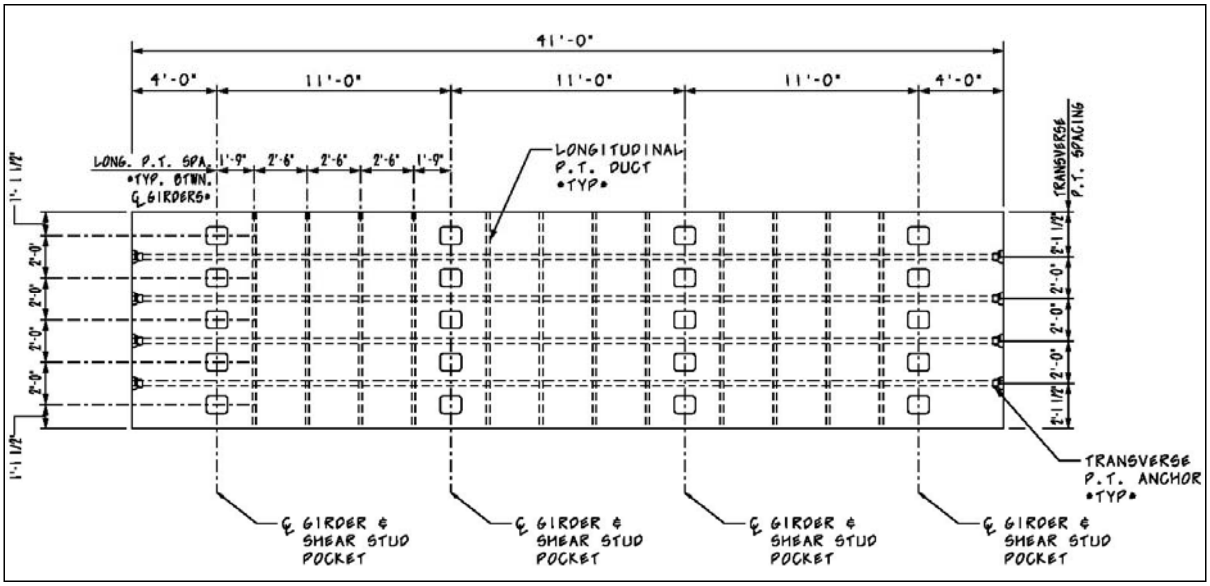

Figure 3 Typical Precast Concrete Panel (Mild steel omitted for clarity)

Figure 4 Typical Panel Before Casting

Figure 5 Typical Longitudinal Post-Tensioning Duct

To tie the panels to the steel plate girders there are 5 shear stud pockets with 9 studs per pocket per panel (Figures 3, 4 and 6).

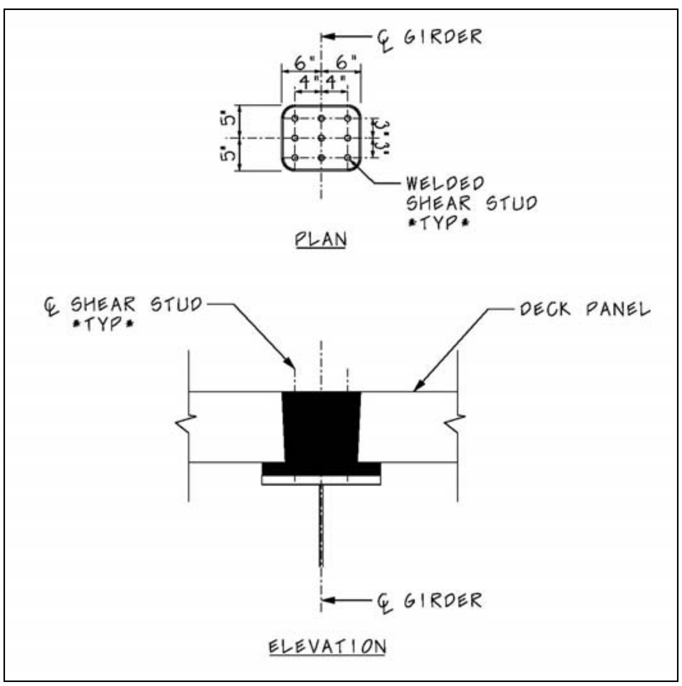

Figure 6 Shear Stud Pocket Details

It was specified that each panel be at least 45 days old at erection to allow the panels creep and shrink. This helps to minimize loss of longitudinal (to the bridge) prestressing forces. The top surface of the panels would later be ground to profile, there is no overlay on this bridge. Prior to shipping the panels to the project site, the entire deck was assembled in the yard. This was to verify that all the deck panel units were constructed in compliance with all the plan requirements. Blocking and bedding strips were used to simulate the support of the deck panels on the beam top flanges. Although the deck panels were checked for proper fit-up and alignment, during placement of the panels on the girders the contractor had to go back twice and switch the panels because of differential camber.

CONSTRUCTION SEQUENCE

The previous bridge was a 3 span continuous steel beam bridge carrying 2 lanes of traffic. Because the detour was short, the bridge was closed for construction. The expected closure was 120 days starting in July 2004. Once the bridge was closed to traffic the superstructure and abutments were demolished and removed. The existing piers remained until the new bridge was erected.

After the MSE abutment walls and wing walls were constructed, the 78 inch deep steel plate girders were shipped and set into place. The girders were placed at night while I-75 was closed intermittently using the abandoned piers as a launch point. This eliminated one splice which in turn decreased erection time. Once the girders and cross frames were in place, the precast concrete deck panels were placed on the girders (Figure 7).

Figure 7 Placing the Precast Panels on the Steel Plate Girders

As with the girders, I-75 was closed intermittently to place the panels. This operation took two nights to complete. Styrofoam shims were placed on top of the girders. Leveling bolts were used to achieve the required profile grade, although the deck was to be ground to final grade. Figure 8 shows the bridge after the completion of the panel placement.

Figure 8 Top of Bridge After the Panel Placement Before Grouting the Joints

Before the grout was poured in the joints the post-tensioning ducts between panels needed to be connected. This was also the time that the vibrating wire strain gauges were placed in the joints, (see Experimental Program). It took two days to grout all of the joints due to rainy weather conditions. Once the joints were grouted, the post tensioning strands were installed into the longitudinal ducts (Figure 9).

Figure 9 Post Tensioning at the Forward Abutment

The bridge was post-tensioned from both ends when the shear keyway grout, anchorage blisters, and deck panels achieved a minimum of 6,000 psi compressive strength. Three days after post tensioning the longitudinal ducts were grouted and the Nelson studs were installed in the deck panel pockets. Four days after this the diaphragms, shear stud pockets, and the void between the top of the girder and the bottom the deck panels were grouted. At the point the bridge is completely tied together. The approach slabs were then constructed.

This would allow the deck grinder to be used on the bridge. The deck was ground to profile, a minimum of ¼ inch in the longitudinal direction between the faces of the sidewalk curbs. The rest of the items, pouring sidewalk and parapet, earthwork, fence installation, and sealing the entire concrete deck and approach slabs between faces of sidewalk curbs were then completed.

EXPERIMENTAL PROGRAM

One of the common problems in adjacent structures is the leaking of the shear keys. This causes premature deterioration of the structure. Post-tensioning compresses the keys so that cracking and leaking is avoided or minimized. To this end, the experimental program had 3 goals:

1) To visually monitor the joints for signs of cracking/leakage.
2) To use instruments in the joints to try to determine:
    a. Initial post-tensioning stresses in the joints;
    b. Loss of post-tensioning force over time;
    c. Thermal stresses.
3) To periodically load test the bridge to assess the composite action of the precast panels.

It would have been desirable to instrument the panels, but the panels had already been cast by the time the decision to instrument the bridge was made. As a result, only the steel girders and the panel joints could be instrumented.

Vibrating wire strain gauges were placed in the joints between the precast deck panels (Figure 10 and 11) prior to grouting the joints. Once the grout was set the plastic holder pieces were removed and the remaining wire cut. The holes were then plugged with grout.

Figure 10 Vibrating Wire Gauge Placed in the Joint

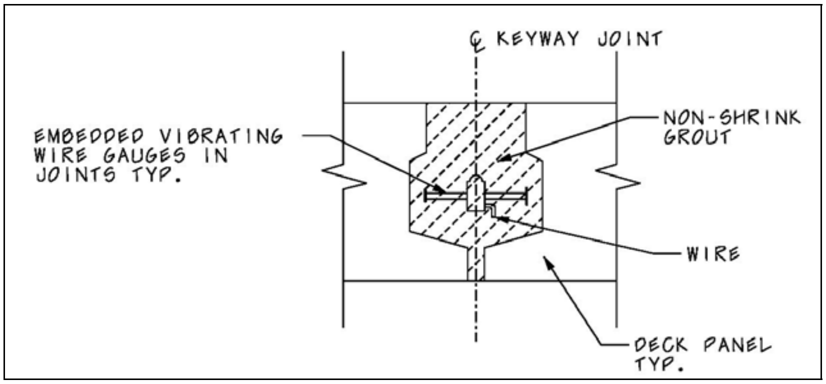

Figure 11 Keyway Joint Details

Three vibrating wire gauges were placed in the midspan joint to determine the maximum stress when load is applied to the bridge. Three were placed in each quarter point joint. At one quarter point, vibrating wire gauges were placed on the top and bottom flanges of the center two girders. The quarter point was the point furthest away from the abutment that could be safely reached from underneath. Finally, three vibrating wire gauges were placed in each end joint. These gauges would see very minimal changes in strain due to loading. This would also be the location to obtain the truest measure of the post-tensioning stress. This is a total of 15 gauges in the joints between the panels and four gauges on the steel girders (Figure 12).

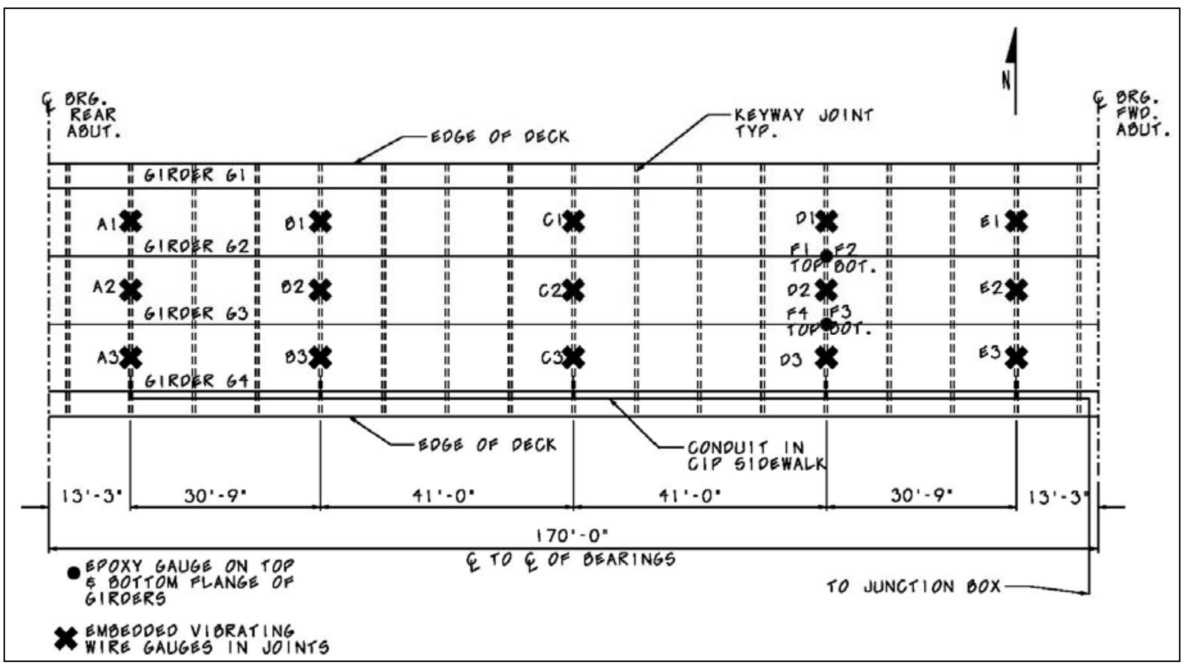

Figure 12 Vibrating Wire Gauge Layout

All of the cables were threaded through PVC pipe to the sidewalk. Another pipe was poured into the sidewalk and turned out at the southeast side of the abutment. The CR10X data acquisition system was then attached to an existing pole at the end of the southeast wing wall.

GROUTING

The grouting of the transverse joints took place from west to east over two days due to rainy weather conditions. In the first day two of the joints that contained vibrating wire gauges were grouted and set overnight while the rest were poured on the next day. The grout used was a non shrink grout which will expand once it is hardened. This and the fact that the panels were free to move will explain why the joints were showing a tensile strain (Figure 13).

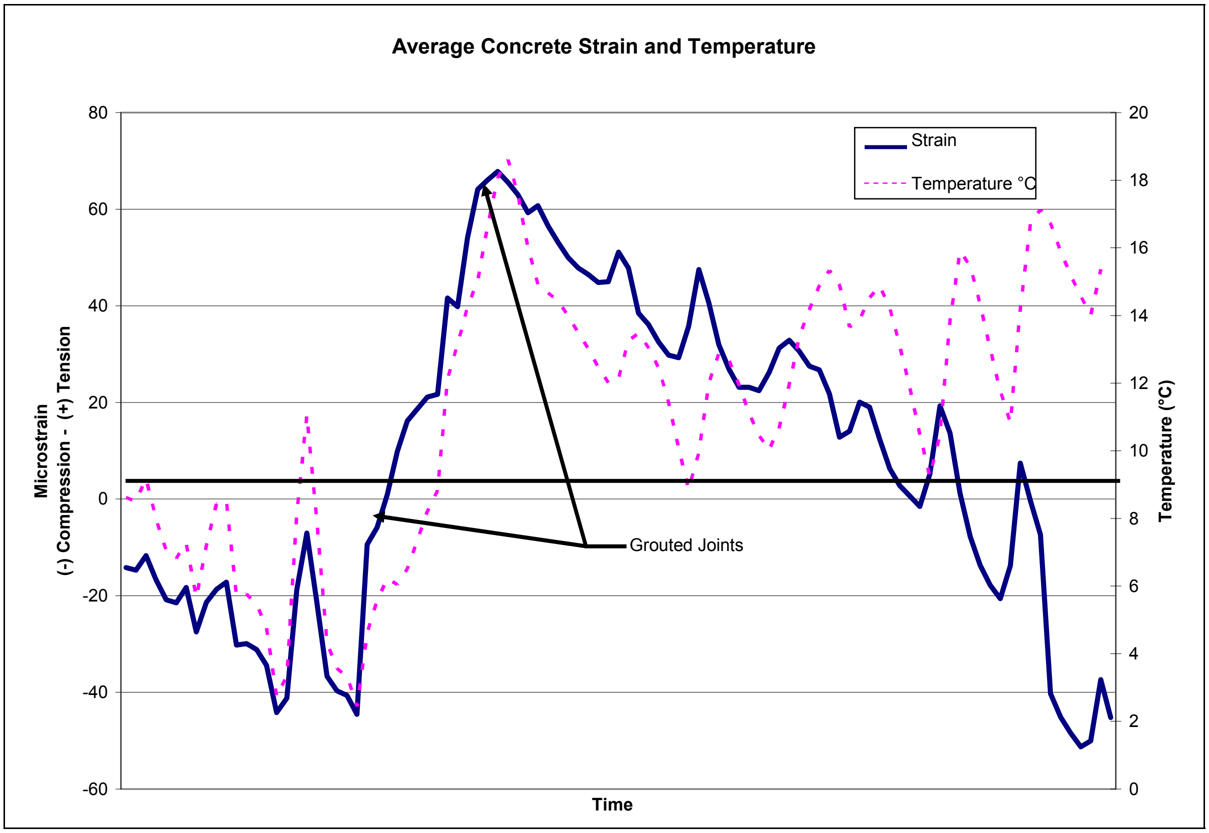

Figure 13 Average Strain and Temperature During and After Grouting the Joints

The vibrating wire gages measure both strain and temperature. There were large variations in the responses of the joints. For example, Joint A (Figure 12) was the first joint poured and it showed almost no heat of hydration and no increase in strain. Joint B showed an increase 12 degrees C and 55 microstrain. The variations in temperature and tensile strain developed could be caused by the way the gout was mixed. The grout comes in 50 pound bags and is mixed a few bags at a time. There were variations in the way the grout was mixed from batch to batch as the contractor tried to find the right consistency. This causes variations in the grout properties. Mixing grout consistently has been a problem with other projects[2].

POST-TENSIONING

The post-tensioning started nine days after the joints between the panels were grouted. During those nine days the forward ends of the girders were jacked up to readjust the bearing devices. As the panels were not yet tied to the girders, this did not appear to cause any changes in the panel behavior. During this time, the grouting valves and post-tensioning strands were installed in the longitudinal ducts, and a friction test was performed. There are twelve ducts in each panel and it took two days to stress them all. Figure 14 shows the longitudinal tendon stressing sequence.

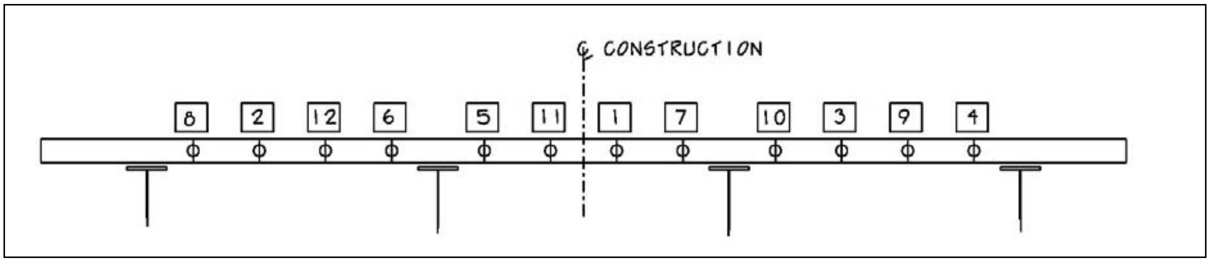

Figure 14 Longitudinal Tendon Stressing Sequence

Table 1 shows details of the strains which occurred when the center, far north and far south tendons were tensioned. Table 2 shows the final strains developed in the joints.

Table 1. Change in Microstrain in Each Gauge Due to Post-Tensioning

Gauge	Location of Tendon (Tendon Stressed)		
	North (4)	Center (1)	South (6)
A1 – North	27.9	3.6	1.2
A2- Center	5.3	33.1	7.9
A3- South	2.7	8.5	26.5
B1 – North	21.8	7.1	0.7
B2- Center	14	11.6	8.4
B3 - South	4.7	12.8	17.3
C1- North	18.3	7.7	0.7
C2 – Center	13.2	8.5	12.7
C3 - South	1	11	19.4

***Note that joint D is similar to B and joint E is similar to A*

Table 2. Total Strain Developed by Post-Tensioning

Gauge	A1	A2	A3	B1	B2	B3	C1	C2	C3	D1	D2	D3	E1	E2	E3
Before PT	2560	2634	2691	2605	2619	2690	2458	2503	2692	2503	2584	2611	2531	2640	2532
After PT	2415	2461	2544	2481	2482	2549	2348	2332	2551	2378	2471	2474	2395	2485	2434
Diff.	145	170	147	124	136	141	110	172	141	125	113	137	136	154	98
	West End			West Quarter Point			Center			East Quarter Point			East End		
Avg.	154			134			141			125			129		
Avg.	137														

Currently there is no information regarding the elastic modulus of the grout used. The testing for the material properties of the grout is in progress. Using an approximate value of 3,000,000 psi the average total strain developed by the post-tensioning is consistent with the engineer's estimate of 400 psi.

12 ducts * 180 kips = 2160 kips
2160 kips / (41' * 10.75" * 12 in/ft) = 0.408 ksi = 408 psi
408 psi / 3,000,000 psi = 136 μ strain (Compares with the total average in Table 2.)

Table 2 shows differences in the strains both within a joint and between joints. The differences in strain could be attributed to many factors. The gauges could have shifted after placement before grouting so they wouldn't be in a line longitudinally. The material properties of each joint might vary (i.e. elastic moduli of cement based materials commonly vary ± 10%). There may also be a variation in stress in the joints caused by friction between the panels and the girders, friction losses in the tendons and variations in stress from the tensioning sequence. However, in spite of these minor differences, the tensioning was successful and the joint stresses are consistent with the engineer's expectations.

After post-tensioning, the panels were monitored for environmental effects. It was noted that the relationship between temperature and strain is reversed. When the temperature increases the strain decreases, signifying compression. The panels want to expand due to the increase in temperature but are restrained so they cannot move. This causes compression in the joints. Conversely, temperature decreases cause an increase in strain as the panels try to contract but are restrained.

GROUT THE SHEAR STUD POCKETS

The last step to tie the superstructure together is to grout the shear stud pockets and void between the top of the girder and the bottom of the deck panel. This occurred 18 days after the joints were grouted and 9 days after the panels were longitudinally post-tensioned. The strains in the joints didn't significantly change after shear stud pockets were grouted (Figure 15).

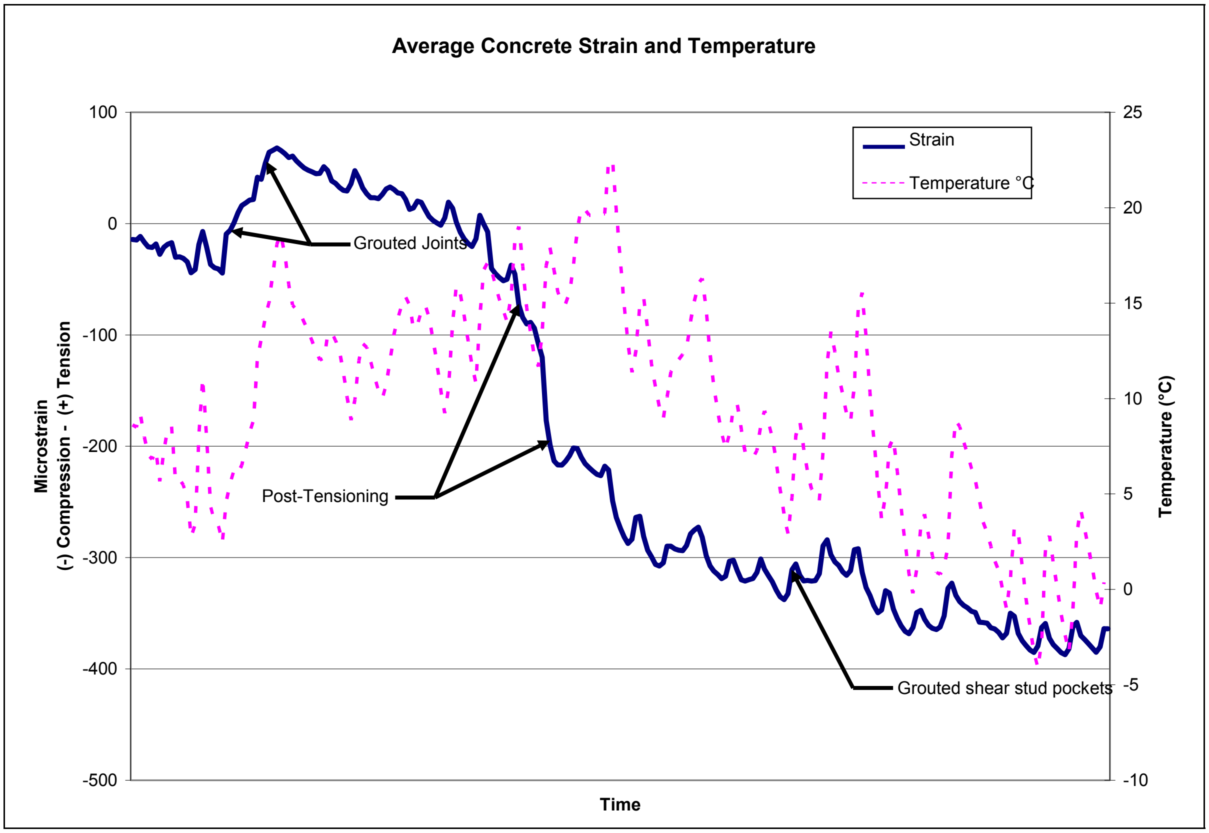

Figure 15 Average Concrete Strain and Temperature – One Month

CONTINUED MONITORING

The panel joints are being monitored for environmental effects. After 8 months, there were no cracks in the joints. This is significant as ODOT and other states have seen cracking in cast-in-place desks within 8 months[3,4,5]. Figure 16 shows typical results for the first 8 months. There are 2 breaks in the data. At approximately one month after the post-tensioning the wires to this box were severed by the contractor digging a trench. This damage was repaired a month later when the CR10X was moved to a new post in front of the southeast wing wall. In this process a gauge was damaged and no longer works. The second gap occurred to a problem with the data system which caused a loss of data. However, even with the gaps the trends are still clear.

Over the first 2 ½ months, there is a steady decrease in strain. This is not necessarily compression, but is probably a contraction associated with creep and shrinkage in the panels and the joints. After 2 ½ months, the strains level out and it appears that there is only daily temperature effects. The final data set shows a decrease in strain with an increase in average temperature; again indicating that the strain changes are due to temperature. Currently, the creep, shrinkage and elastic properties of the grout are being measured. Analysis of the temperature response is in progress.

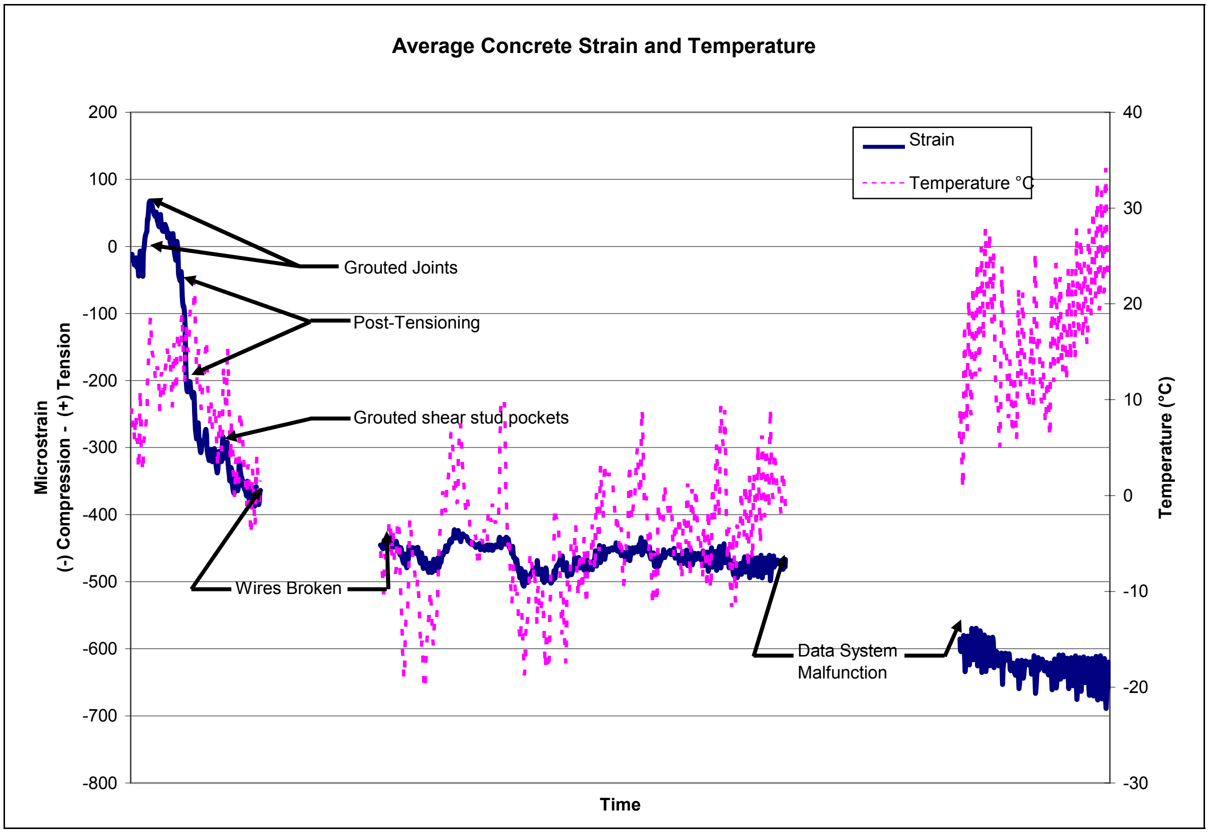

Figure 16 Average Concrete Strain and Temperature – Eight Months

LOAD TEST

Approximately 5 months after the structure was post-tensioned a load test was performed. Four loaded trucks were placed on the bridge over the joints to determine the response and check for composite action. The trucks were not heavy enough to obtain a large response. The case of the four trucks placed over the center joint, two side by side then the pair back to back, yielded the greatest response (Figure 17).

Again using 3,000,000 psi for the modulus of elasticity the calculated strain at the center of the deck is approximately 16 microstrain. While the maximum strain (at the center gauge) is 36 microstrain (Figure 18), the average for the joint is 23 microstrain, which is consistent with the calculated value considering the small values of strain obtained. When the loads are placed at the quarter points, the calculated strain at the center joint is 8 microstrain. The measured average is 12 microstrain. Again, this is reasonable considering the small values of strain. The data suggests that the panels are behaving as part of composite system, as designed. Another load test is scheduled for late July 2005 and heavier trucks will be used.

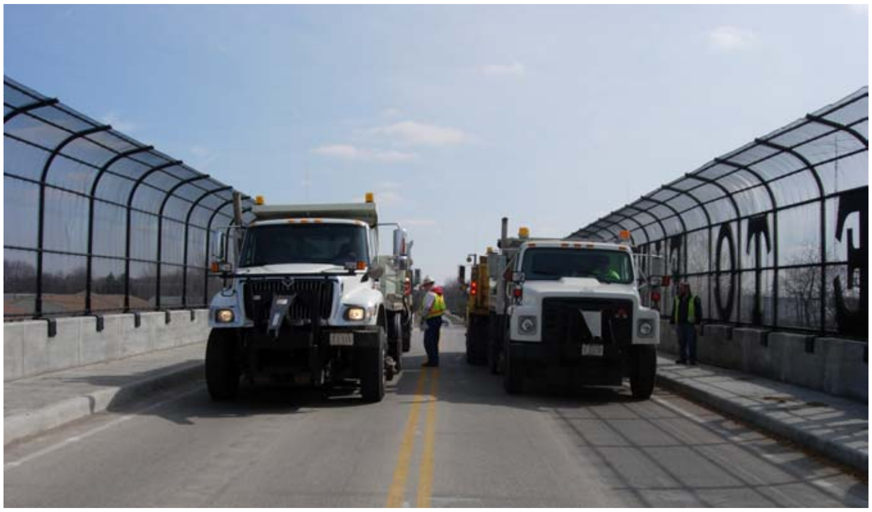

Figure 17 Truck Configuration During Load Test

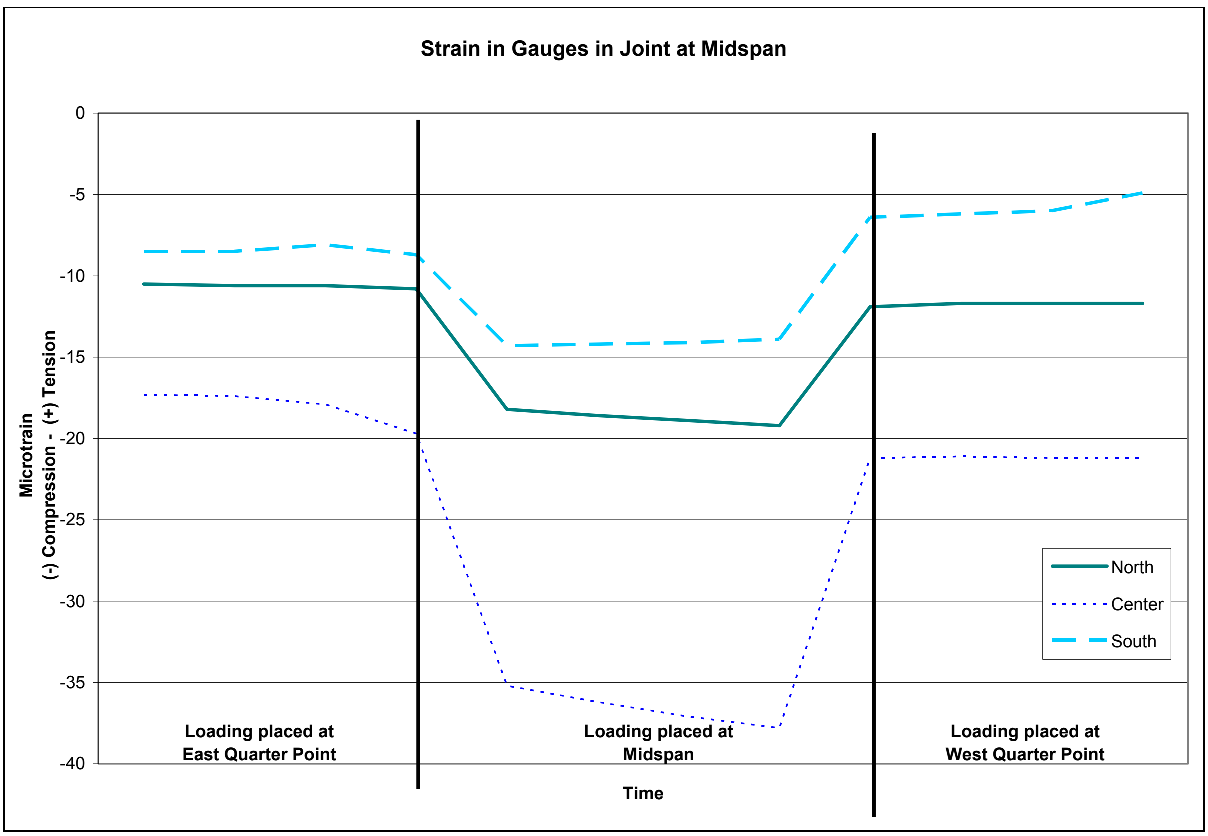

Figure 18 Strain in Gauges at the Midspan During Load Test

POST CONSTRUCTION MEETING

A post construction meeting was held on February 14, 2005 to discuss the positive and negative aspects of this bridge. Overall the project was a success as the bridge was opened to traffic within 120 days of closure. Everyone involved in the project agreed that the communication system in place was the key to this type of construction. Open lines of communication were crucial for transporting the girders and panels to the job site. All of the affiliated agencies, including the Highway Patrol, were contacted in advance about transporting after dark. This was significant as problems had been encountered with transporting large precast members after dark in previous projects.

The project also showed the importance of clear and consistent specifications. The contract documents specified 2 companies whose products could be used for the post-tensioning ducts. The engineer, wishing to avoid leaks during the grouting operation, specified that a leak test was to be performed by pressurizing the PT ducts to 100 psi using air. However, the specified products did not have fitting capable of sustaining this pressure. This became a major issue which needed to be resolved between the engineer, the contractor and ODOT.

During the post-construction meeting, some items were identified which could be improved if this design was used in the future. The design and specifications on this project lead itself to a linear construction sequence after grouting the joints. For example, the grinding of the panels had to be done after the approach slabs were placed but before the sidewalks were placed. It was suggested that an overlay on the deck, instead of grinding it, would allow for a less linear schedule. Although actually grinding the panels was very straight forward and didn't pose a problem, overlaying the deck would allow more than one task to be completed at once. Cast the sidewalk and parapet on the panels at the plant may also speed construction, but such panels would be more difficult to ship.

The precast concrete panels were much easier to install than forming and pouring a deck the conventional way. The cambers of the panels were within the specifications but while erecting the panels the contractor had to go back twice to shift them around. This was expensive but not difficult.

Future panels could be pretensioned as opposed to post-tensioned. For this particular project, the panels were post-tensioned to avoid special bed preparation. It was recommended that the panels be match cast so that no keyway is needed.

CONCLUSIONS

Overall this project and method of construction was successful. Based on the preliminary data, the following conclusions can be drawn:

1) The bridge was completed on time under a very aggressive schedule. Although the there was not a time savings, with some changes to the design to eliminate the construction schedule linearity precast concrete panels for bridge decks is a viable alternative to conventional construction.
2) The post-tensioning of the deck panels was successful. Data shows that the required stress was achieved in all of the joints. Eight months after grouting the

joints between the panels the deck shows no signs of cracking. Cast-in-place decks often display transverse, full depth cracking by this time.

3) Data shows a contraction in the joints over the first 2 ½ months, probably due to creep and shrinkage of the panels and the joints. After this time, strain changes appear to be temperature induced.
4) Load testing shows that the panels are behaving as designed, as part of a composite system.

FUTURE WORK

As part of an ongoing project, this bridge will continue to be monitored for time dependant effects. Creep, shrinkage, and temperature effects of the panels and the grout joints will be examined. Material properties of the grout will be investigated. Periodic load tests will be performed on the bridge and compared to each other to determine if there is any deterioration of the structure. Along with the load tests visual inspections of the bridge superstructure will occur to compare with similar conventional bridges.

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