

*New Hampshire*  
**DOT**  
Research Record



**Precast Concrete Deck Panel  
Performance On Long Span, High Traffic  
Volume Bridges**

**Final Report**

Prepared by the New Hampshire Department of Transportation, in cooperation with the University of New Hampshire and the U.S. Department of Transportation, Federal Highway Administration

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16. Abstract  <p>The NHDOT prohibited the use of partial depth precast deck panels on its long span, high traffic volume bridges until it could investigate if the precast slabs and the concrete overpour were acting in a composite manner. The NHDOT also wanted to ensure that there was adequate transfer of traffic loads between the panels without causing reflective cracking. The purpose of this project was to investigate these two issues.</p> <p>The NHDOT reconstructed the deck of the I-393 bridge over the Merrimack River in Concord using precast concrete slabs and a concrete overpour in 2002-2003. Cores were taken from this bridge deck to evaluate the composite behavior of the slabs and overpour and cracking was evaluated. Cores were taken before the introduction of traffic and then after the deck had been exposed to traffic for several months. Cracking was evaluated before and after traffic.</p> <p>The cores taken were subjected to tensile and shear strength testing across the panel/overpour interface and were found to perform satisfactorily. The tests provided conclusive evidence that there was excellent bond between the deck slabs and the overpour and that the strength of the bond was actually improving slightly over time. An inspection of the deck before and after traffic loading revealed that the panels were adequately transferring traffic loading without causing reflective cracking. Based on these results the NHDOT will permit the use of this type of deck construction on long span, high traffic volume bridges.</p>			
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## **Executive Summary**

In the early 1990's the New Hampshire Department of Transportation (NHDOT) began using partial depth precast deck panel construction with a concrete overpour as a means to speed up bridge deck construction and reconstruction. By the late 1990's, the use of this technique was becoming frequent and problems with deck cracking were becoming evident. The use of the panels was temporarily halted and the NHDOT developed guidelines and standards to better regulate their use.

One result of the guidelines and standards was the restriction of the use of the panels on long span, multi-span bridges until the behavior of the composite bridge deck (partial depth precast deck panel with concrete overpour) could be investigated. This research project was initiated to address that issue. Specifically, the research focused on investigating if the strength of the bond between the precast panels and concrete overpour was sufficient to allow the panels and the overpour to act compositely, and also if the concrete overpour adequately transferred traffic loads across the interface between panels without causing reflective cracking up through the concrete overpour. The I-393 bridge over the Merrimack River in Concord, a long span, multi-span bridge with high traffic volume, was selected as the test site for this research. .

The results of the research showed that the bond between the precast deck panel sections and the concrete overpour was excellent under high traffic volume loadings. The results indicated that the strength of the bond was actually improving over time. An inspection of the deck before and after traffic loading revealed that the panels were adequately transferring traffic loading without causing reflective cracking. Based on these results, the Department has modified its standard design practice to include the use of partial depth precast deck panel construction as a "Contractor's Option" on all single and multi-span bridges regardless of the traffic volume.

## **Introduction**

A common refrain heard in the AASHTO bridge community is the slogan "Get In, Get Out, and Stay Out". This slogan is intended to bring attention to a national initiative to reduce construction time and at the same time maintain or improve the quality of construction. NHDOT has actively pursued different methods, materials and technologies in an effort to address the goals of this initiative. One of the promising options that has been implemented, in a limited fashion in the past, is the use of partial depth precast deck panels.

The use of stay-in-place partial depth concrete deck panels in New Hampshire began in the early 1990's. The NHDOT first started using these panels somewhat cautiously on shorter span, low traffic volume overpasses. These early installations were successful and with increased frequency Contractors began requesting to substitute the panels (with a concrete overpour) for concrete decks designed and detailed as cast-in-place. At the time, the Department did not have an implementation plan or otherwise provide clear guidance on where use of the panels was acceptable. By the late 1990's, the number of requests for deck panel substitutions had increased substantially. Without clear



guidelines, the deck panels were allowed in many locations with various traffic conditions and span configurations. The increased use revealed a number of weaknesses in the specifications, and the quality of the construction became a concern as significant cracking in the panels became more commonplace.

Subsequently, the Department temporarily called for a halt to the use of the panels until the specifications could be revised to address the concrete cracking concerns and a carefully thought out implementation plan could be developed. New guidelines were developed in the form of a list of criteria that had to be met in order to use the deck panels. These guidelines are included in Appendix E to this report. They emphasized a step-by-step implementation process and described the necessary research to support each expanded use of the panels. A specification (special provision) that allowed the Contractor the option of using the cast-in-place (CIP) details or set-in-place (SIP) deck panel details was included in the contract only on those jobs where the DOT would permit their use. There were two situations where the new guidelines restricted deck panel use: 1) high volume roadways and 2) long span, multi-span structures with high Average Daily Truck Traffic (ADTT).

### **Project Objective**

The first restriction noted above was addressed by monitoring previous installations as well as a new experimental installation on a shorter multi-span structure on I-93 with high ADTT. The objective of the research project featured in this report was to investigate the performance of partial depth precast concrete deck panels with a cast-in-place concrete overpour when used on a multi-span structure with span(s) exceeding 175 feet. A bridge meeting this criterion was scheduled to undergo a deck replacement in 2002, and was selected as the test bridge for this project. The research studied whether the precast panels act in a composite manner with the concrete overpour. In addition, the research investigated whether the traffic loads are adequately transferred across the interface between panels without causing reflective cracking up through the overpour.

To determine whether the panels and overpour act together in a composite manner, the strength of the bond between the panels and overpour was tested in shear and tension in both the negative and positive moment regions of the bridge deck. Cores for testing were taken from the composite deck before and after traffic loading. Phase 1 cores were taken after the completion of the deck section at the conclusion of Construction Phase 1 (See Fig. 2), before the application of temporary pavement and traffic. Phase 2 cores were taken 10 months later from the same areas of the deck so that the effects of traffic loading on the deck could be investigated.

To conform to the specification issued by NHDOT Bridge Design, the tensile strength of the composite deck section needed to be  $4-6\sqrt{f'_c}$ . This strength range reflects the recommendations of the AASHTO Load Factor Design Standard Specifications for Bridge Design.

The specification issued by NHDOT Bridge Design did not contain a required range of shear strength for the composite deck section. The researchers determined that it would

be reasonable to expect the bond strength to be equal to that of monolithic concrete thereby ensuring composite action of the deck section. A shear strength range of  $2-4\sqrt{f'c}$  was determined to be acceptable based upon what is recommended for monolithic concrete by ACI and AASHTO.

To determine if reflective cracking was present, the composite deck was checked for cracks before the application of traffic and again after 10 months of traffic loading.

### Bridge Details

The bridge used to conduct the research carries I-393 over the Merrimack River (Bridge Number 154/123) in Concord, New Hampshire (see Figure 1). It has 3 spans totaling 520'-0" in length. The total width of the structure is 92 feet and it carries 4 lanes of traffic with an average daily traffic count (ADT) of 38,000, including significant truck traffic (ADTT>2000). The existing bridge was constructed in 1978 and bridge inspections indicated significant deck delaminations necessitating the deck replacement work.

The span arrangement for the structure is 160'-200'-160'. The existing superstructure was comprised of a composite section utilizing variable depth plate girders and an 8" thick reinforced concrete deck (See right side of Figure 2). The existing bridge had a maximum live load + impact (LL+I) deflection of 2.6 inches under HS-25 loading. The negative moment regions of the bridge did not include shear studs.

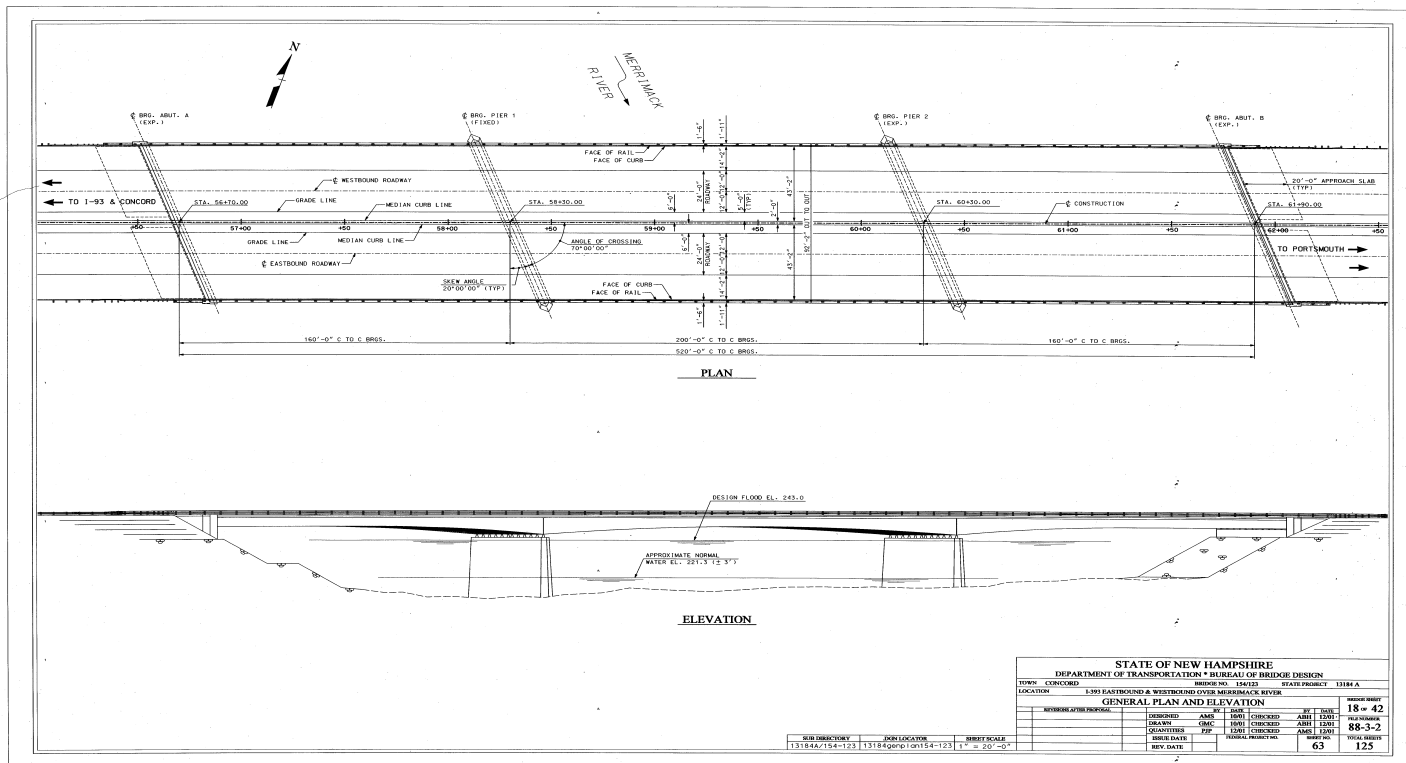
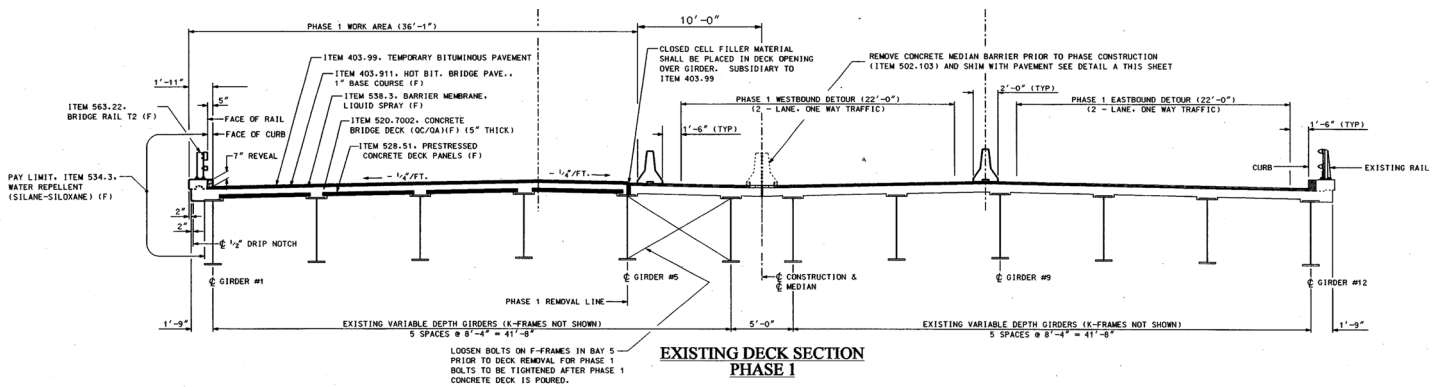


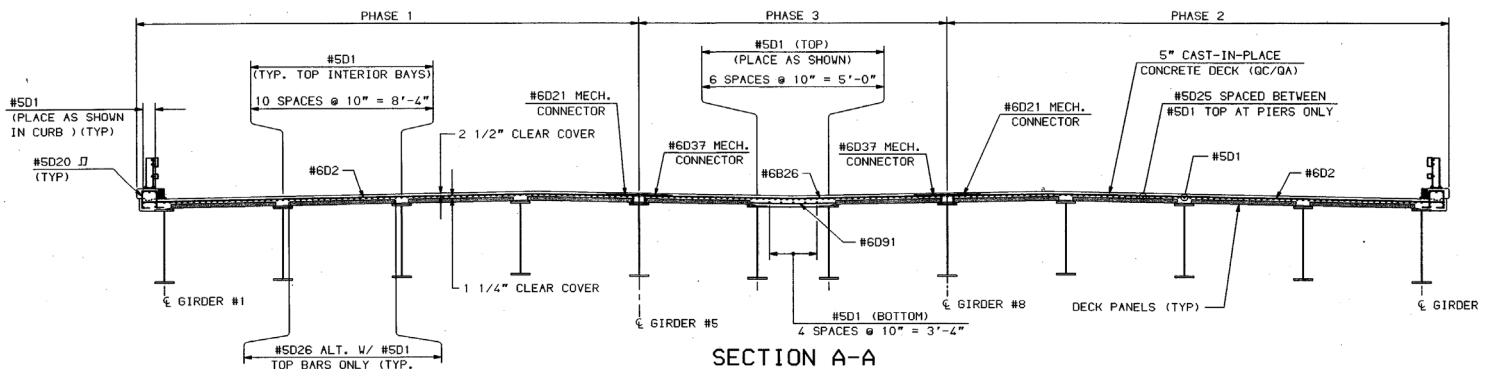
Figure 1 – Bridge Layout



**Figure 2 – Construction Phase 1**

(Left side labeled “Phase 1 Work Area” is location where Phase 1 and Phase 2 cores were taken and where traffic was routed during construction of the remainder of the bridge deck.)

The deck replacement was completed using 3.5-inch thick prestressed precast deck panels with a 5-inch reinforced concrete overpour (See Figure 3). The new deck design live load was HS-25 and the 1996 AASHTO Standard Specifications with interims were used for design. The corresponding maximum LL+I deflection is 2.6 inches. Shear studs were added to the negative moment regions. The total amount of top mat reinforcement used at pier locations was #5’s at 5”. The amount added to address AASHTO Section 10.38.4.3 equaled 2/3 of the total amount required. The additional bottom mat reinforcement required by the code was not added to the top mat.



**Figure 3 – New Composite Deck Section**

**Testing Program – General**

The testing program consisted of 4 different tests on cores taken from both the deck and a control slab. These tests were as follows:

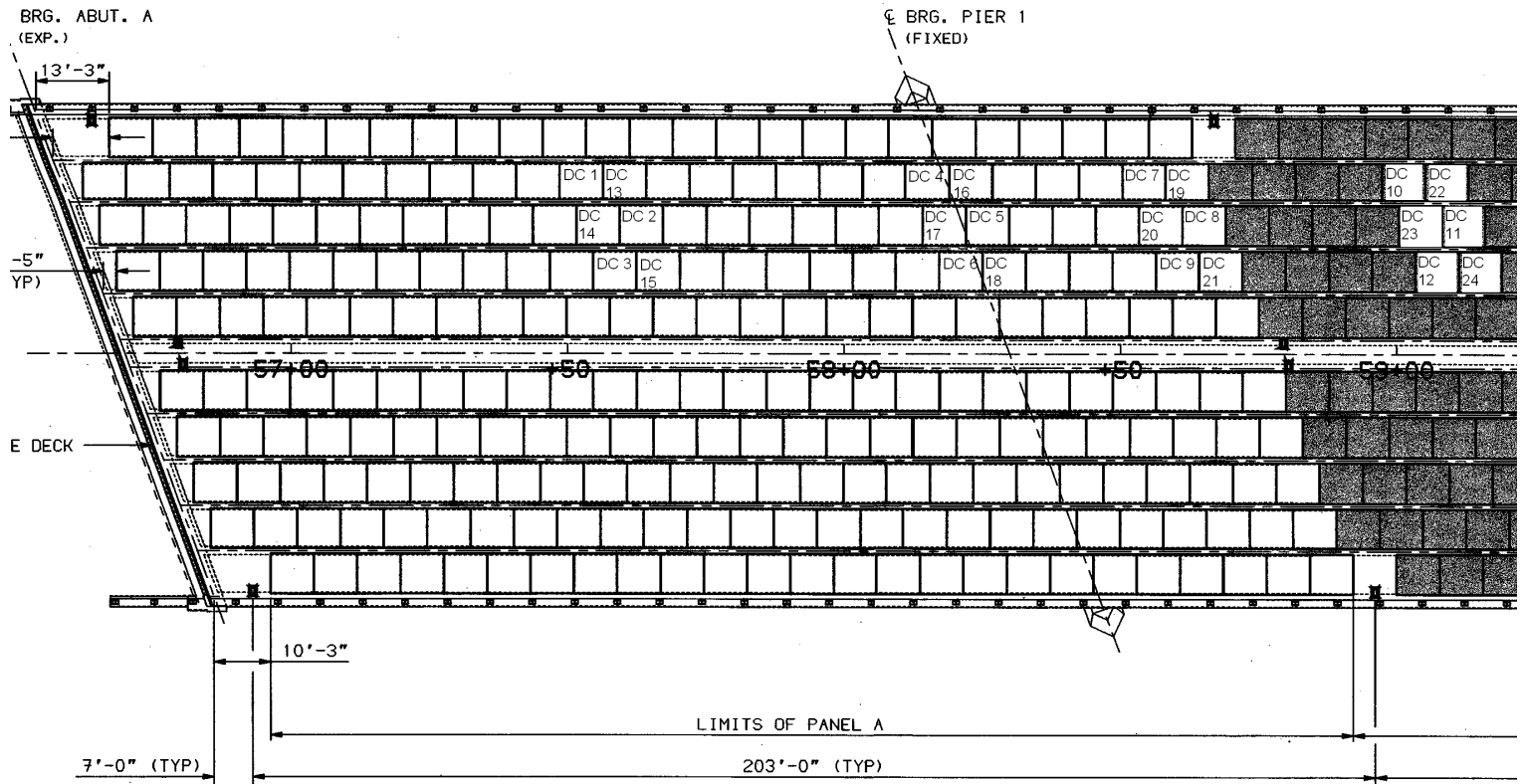
- 1) Parallel Shear – shearing at the panel/overpour interface in the direction parallel to the finish marks on the concrete surface (Appendix A)
- 2) Perpendicular Shear – shearing at the panel/overpour interface in the direction perpendicular to the finish marks on the concrete surface (Appendix A)
- 3) Compressive Strength – traditional test
- 4) Straight Tension – attempting to separate overpour and the deck panel by pulling from each end of core (Appendix C)

Narratives and details of these tests can be found in the Appendices to this report.

A control slab was constructed using the same mixes and design as the bridge deck. This slab was not subjected to loading. Cores were taken from this slab for Phase 1 and for Phase 2 testing at the same time cores were taken from the bridge deck. The cores taken from the control slab during Phase 1 were numbered CC-1 through CC-25. The cores taken from the control slab during Phase 2 were numbered CC-26 through CC-34.

A total of 24 cores were taken from the new bridge deck (See Figure 4). Core locations were set at four specific areas, near the inflection points of Span 1 and Span 2, centered over Pier 1, and near mid-span of Span 2. Phase 1 tested cores DC-1 through DC-12. Phase 2 tested cores DC-13 through DC-24. It was agreed that these locations would be the areas most likely impacted the greatest by live loads. Six cores were taken at each of these locations, three cores per each testing phase. For example, three concrete cores through the overpour and precast panels were taken in the negative moment region over Pier 1 at the completion of Construction Phase 1 (see Fig. 2) prior to any traffic loading. The cores were tested to determine the strength of the bond between panel and overpour. A visual inspection of the topside of the deck surface was conducted and the location of all visible cracks were mapped for the area from the centerline of Span 1 to the centerline of Span 2.

Cored holes were temporarily patched, and the Construction Phase 1 deck was paved (using temporary pavement and no membrane) and put into service for approximately nine months while the remainder of the deck was reconstructed. At the completion of Construction Phase 3, but before the median barrier was installed; the pavement on the test section was removed so that Phase 2 testing could be done. This portion of the deck was cleaned and once again thoroughly inspected for cracks. All visible cracks were mapped for the same area as in Phase 1 testing and an evaluation comparing these results with the Phase 1 inspection/mapping was conducted. Phase 2 cores were taken at this time. Bond strengths from the Phase 2 cores were evaluated and compared to Phase 1 values. All cored holes had the cored surfaces roughened, keyed, and patched using expansive concrete. Appendix D to this report contains pictures and details of the coring process. This portion of the deck was subsequently membraned and final pavement was placed.



**Figure 4 – Deck Core Locations**

**Testing Program - Procedures**

Physical testing for the project was performed on both the plastic and hardened forms of the cast-in-place and precast concrete portions of the deck system. Standard testing and inspection was performed on the panels at the precast production facility by a consultant firm contracted to perform materials testing for the Bureau of Materials & Research. Routine tests performed on the precast concrete included temperature, slump, air content, unit weight, and fabrication of strength specimens. The forms and pieces also underwent pre-placement and post-placement inspections. Production of the panels used for the project occurred over the period of April 22<sup>nd</sup> through July 1<sup>st</sup>, 2002. Placement of the overpour concrete took place at night, August 15<sup>th</sup> to 16<sup>th</sup>, 2002. In addition to the standard testing, prisms were fabricated to measure the length change of the overpour concrete mixture. Details on the mix specifications for the precast deck panels, overpour concrete, and bridge deck grout used to attach the panels to the girders can be found in Appendix F.

Four-inch diameter core samples were obtained and tested from the composite deck and the control slab. Cores were tested in tension, shear, and compression. Phase 1 deck and control slab cores were obtained September 12<sup>th</sup>, 2002. Phase 2 deck and control slab cores were taken July 17<sup>th</sup> and July 22<sup>nd</sup>, 2003 respectively.

## **Coring Procedures**

Coring of the bridge deck and the control slab was performed in accordance with procedures outlined in AASHTO Test Method T-24, *Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*. Cores were obtained by advancing a four-inch nominal inside-diameter diamond core bit through the cast-in-place overpour and the precast panel below. Core sites were located in such a way as to avoid hitting any of the strands in the panels. In addition, prior to placement of the overpour, the reinforcing steel was spread apart slightly to facilitate obtaining cores without contacting any steel with the core bit. The location of the reinforcing was verified prior to coring with a handheld pachometer. None of the pre-stressing strands were hit while coring. On a few occasions, reinforcing steel was contacted, but the cores passed through the reinforcement without incident. The direction of the broomed finish markings on the panel surface was marked on the top of each core prior to coring for reference during testing.

In Phase 1, the core bit was typically advanced to within approximately 1/8" of the deck thickness. Next, the core was dislodged by lightly tapping the top with a hammer. Planking on the bottom flange of the bridge beams allowed access to the underside of the deck in Phase 1. The cores were secured from the underside of the deck by allowing them to drop one inch into a waiting receptacle. The cores were then pushed back up through the core hole to a person on the top of the deck and immediately labeled. The resulting holes in the bridge deck from the Phase 1 coring were covered with 12" x 12" x 1/4" steel plates. The plates were adhered to the deck using a one-part polyurethane construction adhesive. The adhesive held the plates in place during temporary paving operations and throughout the winter months. The plates were removed when the temporary pavement was removed in the spring of 2003. The holes were inspected and then patched using a qualified non-shrink grout, following the Phase 2 coring and deck inspection operations.

Phase 2 coring operations were similar to Phase 1, except that access to the underside of the deck was gained by use of a man-lift secured to a barge on the river. From the perspective of the laborers beneath the deck, it was easier to advance the core bit completely through the deck, rather than stopping short and tapping the cores loose. All cores were secured without incident.

## **Compressive Strength**

Compressive strength testing was performed as part of routine acceptance and quality assurance procedures on the precast panels, the cast-in-place overpour, and the deck grout placed around the panels. Fabrication and compressive strength testing of cylinders was done in accordance with the procedures outlined in AASHTO Test Methods T-23, *Making and Curing Concrete Specimens in the Field* and T-22, *Compressive Strength of Cylindrical Concrete Specimens*. In addition, compressive strength tests were performed on selected cores from both Phase 1 and Phase 2 on the cast-in-place overpour portion. Strength tests were not performed on the precast panel portions of the cores because the length to diameter ratio was outside allowable limits as specified in the test method.

Cores were obtained with the procedures outlined in AASHTO Test Method T-24, *Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*. Testing of the cores also followed the AASHTO Test Method with the exception that the cores were not immersed in saturated limewater prior to testing.

<b>Material Tested</b>	<b>Age (days)</b>	<b>Average Strength (psi)</b>
Overpour Cylinders	28	5120
Panel Cylinders	28	8830
Grout Cylinders	28	8420
Phase 1 Deck Cores (Overpour Only)	33	5270
Phase 1 Control Cores (Overpour Only)	32	5440
Phase 2 Deck Cores (Overpour Only)	342	6850
Phase 2 Control Cores (Overpour Only)	349	6250

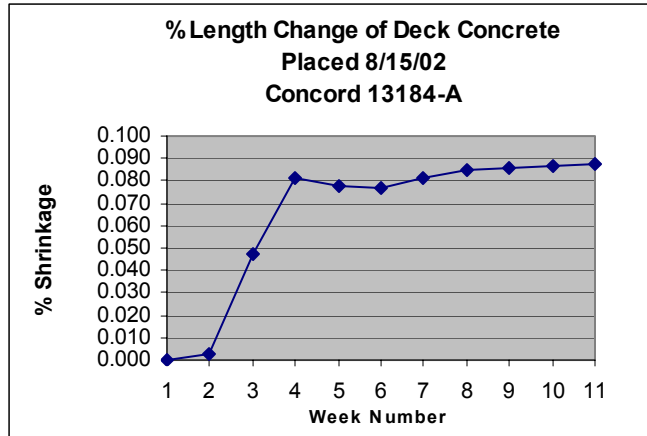
**Table 1 - Compressive Strength Results**

### **Length Change**

Four 3" x 3" prisms were fabricated for measuring the length change of the concrete placed for the overpour of the Construction Phase 1 portion of the deck. The concrete sample was taken during sampling of Sublot 1 of the placement. The concrete was transported to the Materials & Research laboratory where fabrication took place. The specimens were molded at approximately 9:00 p.m. and were placed directly into moist storage following fabrication. Consolidation of the specimens was accomplished by rodding in accordance with the requirements of AASHTO T-126.

The prisms were removed from the molds at the age of 4 days, 12 hours. An initial measurement was taken on each of the four beams using the length comparator. First, the comparator was set to the length of the reference bar. Following that, each beam was placed in the comparator and the difference in the length of each beam to the length of the reference bar was recorded. The beams were immediately returned to moist storage.

The prisms were removed from moist storage seven days after fabrication. The length change was measured and recorded following the same procedure as described previously. Following length measurements, the beams were placed into air storage in a regulated drying chamber. The drying chamber was programmed to maintain 50% relative humidity and a temperature of 23°C. Length change of the beams was checked once a week (Fridays at 9:00 a.m.), for a period of ten weeks. At the end of the 10 weeks, the prisms had an average length change of 0.087% shrinkage from their initial length. Table 2 shows the average change in length over time. A narrative of the shrinkage monitoring can be found in Appendix B to this report.



**Table 2 –Average Length Change vs. Time**

**Shear Testing**

Selected cores were tested for shear strength both parallel and perpendicular to the broomed finish marks on the surface of the precast panel. Upon trying to load the first Phase 1 core into the testing device, it became apparent that the diameter of the core was slightly larger than the shear device could accommodate. The core would not fit properly into the clamps of the device. It appeared that, through use, the walls of the core bit had become worn on the inside resulting in the cores having a larger diameter than would fit into the apparatus. This discovery was not made until the time of testing the cores, because the difference in diameter between molded cylinders (that fit) and cores was not anticipated. After consultation with members of the Technical Advisory Group, it was decided it would be best to trim the cores lengthwise to make them fit (see Appendix A, Fig. A-1). A jig was fabricated to go onto the wet saw used for trimming the cores so that the cuts would be consistent. The surface area was determined, and the strength in psi was calculated based on the adjusted (reduced) area. After trimming, the cores were clamped into the shear device so that the overpour/panel interface was between the two plates sliding past each other (see Appendix A, Fig. A-4), forcing the failure to occur at the interface of the two layers to evaluate the bond strength. The cores were loaded at the rate of approximately 10 lbs/sec until failure. Cores obtained for Phase 2 did not require trimming to fit into the shear-testing device because the core bit used during this phase was relatively new and produced cores that fit snugly into the clamps without modification. Failure of all cores tested occurred at the interface between the panel and the overpour concrete. Table 3 shows the average shear strengths of the control and deck cores for each phase.

<b>Direction of force relative to panel finish</b>		<b>Control Cores</b>	<b>Deck Cores</b>
Phase 1	Parallel	316 psi	528 psi
	Perpendicular	383 psi	475 psi
Phase 2	Parallel	454 psi	579 psi
	Perpendicular	416 psi	554 psi

**Table 3 – Average Shear Strength Values**



## **Tensile Testing**

Tensile testing consisted of bonding a 4" square by ¼" inch thick steel plate to each end of the core with a two-part epoxy. To prepare the cores for tensile testing, it was necessary to trim a thin section from each end of the core to remove laitance and give a flat plane for adhering the cores to the testing plates. Once the epoxy had cured (24 hours), steel rods were screwed into a coupling welded to each plate. The rods were clamped into the jaws of the testing machine and plumbed. The cores were loaded at a constant rate of approximately 10 pounds per second until failure

In Phase 1, 50% of the cores tested for tensile strength failed at the interface between the panel and overpour, and 50% failed in the overpour portion of the core, near the testing plate. All of the Phase 2 cores failed in the overpour concrete. Table 4 shows the average tensile strengths of the cores tested.

	<b>Control Cores</b>	<b>Deck Cores</b>
Phase 1	236 psi	263 psi
Phase 2	314 psi	337 psi

**Table 4 – Average Tensile Strength Values**

## **Summary of Results**

The following describes how the results of the tests compared with the target values for tensile and shear strength of the bond between panel and overpour. The compressive strength,  $f'_c$ , used below in the equations to derive the target values for the tensile and shear strength was 4900 psi. 4900 psi represents the lower bound of the actual compressive strengths measured in the overpour portion of the core samples taken from the bridge deck in Phase 1. The strength was recorded at 66 days of age. The mix design for the concrete overpour targeted a value of 4000 psi, and the average strength of the deck samples at 66 days was 5270 psi. Using the lower bound of the actual measurements, 4900 psi, was deemed to be a reasonable compromise when selecting a compressive strength to derive the target values of shear and tensile stress.

The tensile strength testing target value was based on an  $f'_c$  of 4900 psi. Using 4900 psi, the equation  $4-6\sqrt{f'_c} = \text{desired strength}$  gives a range of 280 - 420 psi. The average tensile stresses achieved in the tests were as follows:

$$\begin{aligned}\text{Average Tension: Phase 1} &= 263 \text{ psi} \\ \text{Phase 2} &= 337 \text{ psi}\end{aligned}$$

In Phase 1 testing, the failure was observed to occur at the interface between overpour and panel on half of the cores and in the overpour on half of the cores. In Phase 2 testing, the failure occurred in the overpour on all of the cores. It should be noted that the Phase 2 bond strengths between the overpour and the precast panel are stronger than 337 psi as

this number represents the stress at which the overpour concrete failed. The tensile strengths of the Phase 1 cores were close to the targeted range of strength, and the Phase 2 cores were firmly within the strength range. This shows that the strength of the bond increased with age and that the deck section was acting compositely.

The shear strength testing target value was based on an  $f'c$  of 4900 psi. Using 4900 psi, the equation  $2-4\sqrt{f'c} = \text{desired strength}$  gives a range of 140-280 psi. The average shear stresses achieved in the tests were as follows:

Average Shear: Phase 1 = 502 psi  
Phase 2 = 567 psi

These values are well above the target range and show that the strength of the bond increased over time.

Visual deck inspections were conducted during both Phases 1 & 2. The detailed inspections revealed no visible cracks. The Phase 1 core holes were also re-inspected during Phase 2 to evaluate whether there had been any movement or separation at the interface between the overpour and the deck panels. There were no indications of any separation or movement between the two surfaces.

### **Conclusions**

This trial installation of partial depth concrete deck panels on high volume roadways and long span, multi-span bridges, illustrated that deck panels can be used successfully in these applications. The test results provided conclusive evidence that there was excellent bond between the deck panels and the overpour, and that this bond was unaffected by the repeated truck loadings the deck was subjected to in actual service over nine months. The results actually indicated that the bond between the deck slabs and overpour was improving with time despite the traffic loading. Based on the results of this research, the Department has determined that it can confidently move forward allowing the use of partial depth precast deck panels on long span, multi-span bridges subjected to high traffic volumes. The Department will continue to periodically monitor the various bridges with deck panel installations throughout the State, as well as periodically conduct detailed inspections of the research installations.

## References

AASHTO T 22, *Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO T 23, *Standard Method of Test for Making and Curing Concrete Test Specimens in the Field*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO T 24, *Standard Method of Test for Obtaining Drilled Cores and Sawed Beams of Concrete*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO T 126, *Standard Method of Test for Making and Curing Concrete Test Specimens in the Laboratory*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO T 160, *Standard Method of Test for Length Change of Hardened Hydraulic Cement Mortar and Concrete*, American Association of State Highway and Transportation Officials, Washington, D.C.

*AASHTO Standard Specifications for Bridge Design 16<sup>th</sup> ed.*, American Association of State Highway and Transportation Officials, Washington, D.C. 1996

## Appendix A

LAST UPDATED: 8/6/03

### SPR Project # 13733-D Shear Strength Testing Log

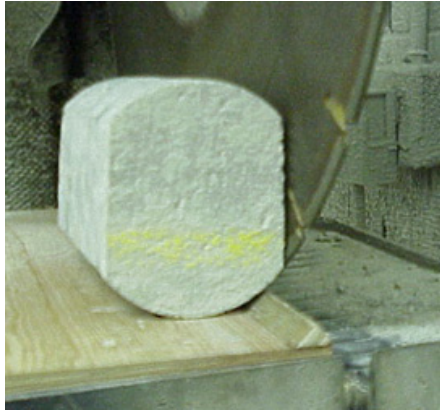
The six cores (CC Nos. 1-6) obtained from the control slab on 8/26/02 were tested for shear strength in the Materials & Research Laboratory. As directed by the TAG, three cores were tested parallel to the surface finish marks on the precast panel and three were tested perpendicular to the surface finish marks. Upon trying to load the first core into the testing device, it became apparent that the diameter of the core was slightly larger than the shear device could accommodate and it would not fit properly into the clamps of the device. This discovery was not made until the time of testing the cores, because the difference in diameter between molded cylinders (that fit) and cores was not anticipated. Evidently, through use, the walls of the core bit had become worn on the inside resulting in the cores having a larger diameter than the shear device was built to accommodate. After consultation with members of the TAG, it was decided it would be best to trim the cores lengthwise to make them fit (Fig. A-1). A jig was fabricated to go onto the wet saw used for trimming the cores to make the cuts consistent. The surface area was determined and the strength in psi was calculated based on the adjusted (reduced) area.

After trimming, the cores were clamped into the shear device so that the overpour/panel interface was between the two plates sliding past each other, forcing the failure to occur at the interface of the two layers (Figs. A-4-7). The cores were loaded at the rate of approximately 10 lbs/sec until failure (Fig. A-11). The average test results were 275 psi in the parallel direction and 125 psi in the perpendicular direction.

#### 9/3/02

Six cores (CC Nos. 11-16) were tested for shear strength. As before, three cores were tested parallel to the surface finish marks on the precast panel and three were tested perpendicular to the surface finish marks. These cores were obtained in response to the discovery of an assembly error in the shear-testing device that applied unwanted tensile forces to the cores when tested. The cores were trimmed and tested as before, with loading at a rate of approx. 10 lbs/sec until failure. The average test results were 341 psi in the parallel direction and 415 psi in the perpendicular direction. These values show an increase of strength in the parallel direction of 66 psi or 24%. In the perpendicular direction, there was an increase of 290 psi or 232%.

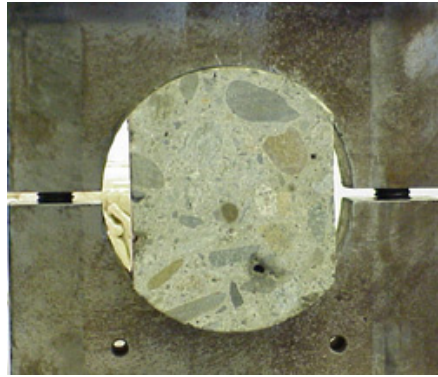
It seems proper assembly of the shear device makes a significant difference in the results. This is especially true in the perpendicular direction. What is not known is how much of a factor the additional age had on the bond. Future testing may indicate if age plays a role in bond strength.



**Figure A-1 - Trimming a core to fit**



**Figure A-2 - Trimmed deck cores ready for testing**



**Figure A-3 - Trimmed core now fits into shear device**



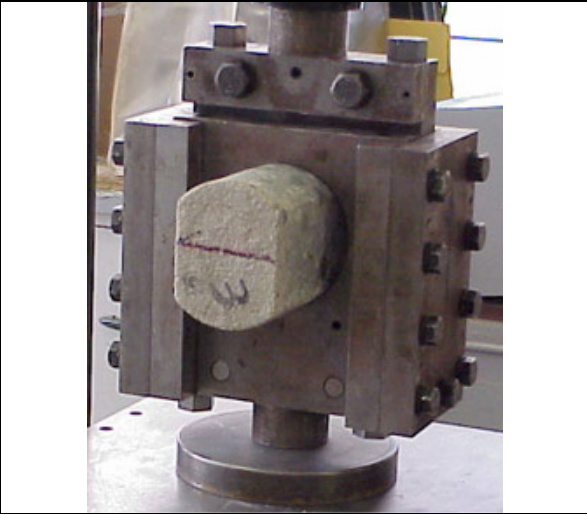
**Figure A-4 - Lining up interface in device**



**Figure A-5 - Core is marked for orientation**



**Figure A-6-Core and shear device are assembled**



**Figure A-7 - Core loaded in compression machine**



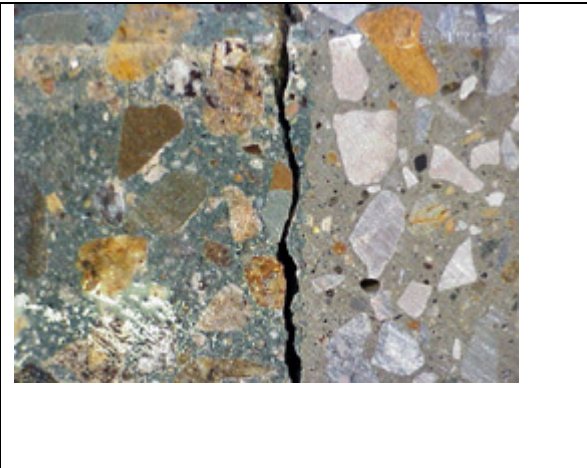
**Figure A-8 - Typical display while loading**



**Figure A-9 - Typical display at failure**



**Figure A-10 - Overpour on left, PCP on right prior to failure**



**Figure A-11 - Same core after testing, failure at interface**

### 9/17/02

Shear testing was performed on cores taken from the control slab on 9/12/02. Core Nos. CC-20, 21 and 22 were tested parallel to the surface finish marks on the precast panel. The average strength for these cores was 316 psi. Core Nos. CC-23, 24 and 25 were tested perpendicular to the surface finish marks on the precast panel. The average strength for these cores was 383 psi.

### 9/18/02

Shear testing was performed on cores taken from the bridge deck on 9/12/02. Core Nos. DC-2, 6, 7 and 12 were tested parallel to the surface finish marks on the precast panel. The average strength for these cores was 528 psi. Core Nos. DC-3, 5, 8 and 10 were tested perpendicular to the surface finish marks on the precast panel. The average strength for these cores was 475 psi. Failure of all cores tested occurred at the interface between the panel and the overpour concrete. Shear strength results obtained from the bridge deck cores were higher than the control slab. In the direction parallel with the surface finish marks, the deck cores were 67% higher than the control cores. In the perpendicular direction, the deck core results were 28% higher than the control. Today concluded the Phase 1 shear testing of all cores obtained to date.

## **PHASE 2 TESTING**

### **Note**

Cores obtained for Phase 2 did not require trimming to fit into the shear-testing device. The core bit used to obtain the cores was relatively new and produced cores that fit snugly into the clamps without modification. Surface area was calculated for the Phase 2 cores by taking the average diameter of the cores and applying it to the formula for the area of a circle.

### 7/24/03

Shear testing was performed on cores taken from the bridge deck on 7/17/03. Core Nos. DC- 15, 17, 19 and 22 were tested parallel to the surface finish marks on the precast panel. The average strength for these cores was 579 psi. Core Nos. DC-14, 18, 20 and 24 were tested perpendicular to the surface finish marks on the precast panel. The average strength for these cores was 554 psi. Shear strength results obtained from the bridge deck cores were higher than results obtained during Phase 1 testing. Phase 2 shear strength results increased 9.7 % from Phase 1 in the direction parallel to the roughened panel surface and 16.6% in the direction perpendicular to the panel surface finish marks.

### 7/31/03

Shear testing was performed on cores taken from the control slab on 7/22/03. Core Nos. CC- 26, 27 and 28 were tested parallel to the surface finish marks on the precast panel. Failure of all cores tested occurred at the interface between the panel and the overpour concrete. The average strength for these cores was 454 psi. Core Nos. CC-29, 30, and 31 were tested perpendicular to the surface finish marks on the precast panel. The average strength for these cores was 416 psi. Failure of all cores tested occurred at the interface between the panel and the overpour concrete. The increase in strength from Phase 1 to Phase 2 in the direction parallel to the surface finish marks was 43.7% and 8.6% in the direction perpendicular to the panel surface finish marks. Comparison shows that in the direction parallel with the panel surface finish marks, the Phase 2 deck cores were 27.5% higher than the Phase 2 control cores. In the perpendicular direction, the Phase 2 deck core results were 32% higher than the Phase 2 control cores.

## Shear Strength Results: Control Cores

CC=Control Core  
DC=Deck Core

**Date Placed** 8/16/2002  
**Date Cored** 8/26/2002 CC-10 not tested  
**Date Tested** 8/27/2002  
**Age at Testing** 11 Days

Average Area (in2)= 11.23			Perpendicular to Panel Surface		
Parallel to Panel Surface			SAMPLE I.D. LOAD (lbs) Shear Strength (psi)		
<u>SAMPLE I.D.</u>	<u>LOAD (lbs)</u>	<u>Shear Strength (psi)</u>	<u>SAMPLE I.D.</u>	<u>LOAD (lbs)</u>	<u>Shear Strength (psi)</u>
CC-4	3980	354	CC-1	920	82
CC-5	3340	297	CC-2	1650	147
CC-6	1950	174	CC-3	1630	145
Avg.:	3090	275	Avg.:	1400	125

**Date Placed** 8/16/2002  
**Date Cored** 9/3/2002  
**Date Tested** 9/3/2002  
**Age at Testing** 18 Days

Average Area (in2)= 11.23			Perpendicular to Panel Surface		
Parallel to Panel Surface			SAMPLE I.D. LOAD (lbs) Shear Strength (psi)		
<u>SAMPLE I.D.</u>	<u>LOAD (lbs)</u>	<u>Shear Strength (psi)</u>	<u>SAMPLE I.D.</u>	<u>LOAD (lbs)</u>	<u>Shear Strength (psi)</u>
CC-11	3060	272	CC-14	4860	433
CC-12	4750	423	CC-15	3990	355
CC-13	3690	329	CC-16	5130	457
Avg.:	3833	341	Avg.:	4660	415

### PHASE I

**Date Cored** 9/12/2002  
**Age at Coring** 27 Days  
**Date Tested** 9/17/2002  
**Age at Testing** 32 Days

Average Area (in2)= 11.14			Perpendicular to Panel Surface		
Parallel to Panel Surface			SAMPLE I.D. LOAD (lbs) Shear Strength (psi)		
<u>SAMPLE I.D.</u>	<u>LOAD (lbs)</u>	<u>Shear Strength (psi)</u>	<u>SAMPLE I.D.</u>	<u>LOAD (lbs)</u>	<u>Shear Strength (psi)</u>
CC-20	3940	354	CC-23	4690	421
CC-21	2370	213	CC-24	4500	404
CC-22	4240	381	CC-25	3610	324
Avg.:	3517	316	Avg.:	4267	383

### PHASE II

**Date Placed** 8/16/2002  
**Date Cored** 7/23/2003  
**Age at Coring** 341 Days  
**Date Tested** 7/31/2003  
**Age at Testing** 349 Days

Average Area (in2)= 12.37			Perpendicular to Panel Surface		
Parallel to Panel Surface			SAMPLE I.D. LOAD (lbs) Shear Strength (psi)		
<u>SAMPLE I.D.</u>	<u>LOAD (lbs)</u>	<u>Shear Strength (psi)</u>	<u>SAMPLE I.D.</u>	<u>LOAD (lbs)</u>	<u>Shear Strength (psi)</u>
CC-26	7360	595	CC-29	7980	645
CC-27	4650	376	CC-30	5400	437
CC-28	4820	390	CC-31	2060	167
Avg.:	5610	454	Avg.:	5147	416



## Shear Strength Results: Deck Cores

CC=Control Core  
DC=Deck Core

### PHASE I

**Date Cored**     9/12/2002  
**Age at Coring**   27 Days  
**Date Tested**     9/18/2002  
**Age at Testing**   33 Days

Average Area (in<sup>2</sup>)=     11.00  
 Parallel to Panel Surface

SAMPLE I.D.	LOAD (lbs)	Shear Strength (psi)
DC-2	6620	602
DC-6	5550	505
DC-7	5050	459
DC-12	6030	548
Avg.:	5813	528

Perpendicular to Panel Surface

SAMPLE I.D.	LOAD (lbs)	Shear Strength (psi)
DC-3	6080	553
DC-5	4680	425
DC-8	5020	456
DC-10	5120	465
Avg.:	5225	475

### PHASE II

**Date Placed**     8/16/2002  
**Date Cored**     7/17/2003  
**Age at Coring**   335 Days  
**Date Tested**     7/24/2003  
**Age at Testing**  342 Days

Average Area (in<sup>2</sup>)=     12.37  
 Parallel to Panel Surface

SAMPLE I.D.	LOAD (lbs)	Shear Strength (psi)
DC-15	6070	491
DC-17	6900	558
DC-19	7820	632
DC-22	7880	637
Avg.:	7168	579

Perpendicular to Panel Surface

SAMPLE I.D.	LOAD (lbs)	Shear Strength (psi)
DC-14	6870	555
DC-18	4710	381
DC-20	9000	728
DC-24	6840	553
Avg.:	6855	554

**Appendix B**  
**SPR Project # 13733-D**  
**Shrinkage Testing Log**

**8/15/02**

Four 3"x3" beams were fabricated for the purpose of measuring the length change of the concrete placed for the overpour of Phase 1 portion of the project. The concrete was taken during sampling of Sublot 1 of the placement. The concrete was transported to the Materials & Research laboratory where fabrication took place. The specimens were molded at approximately 9:00 p.m. and were placed directly into moist storage following fabrication. Consolidation of the specimens was accomplished by rodding, in accordance with the requirements of AASHTO T126.

**8/20/02**

Shrinkage beams cast on 8/15/02 were removed from the molds at approximately 9:00 a.m. (age 4 days, 12 hours). An initial measurement was taken on each of the four beams using the length comparator. First, the comparator was set to the length of the reference bar. Following that, each beam was placed in the comparator and the difference in the length of each beam to the length of the reference bar was recorded. The beams were immediately returned to moist storage until the age of seven days.

**8/23/02**

Beams were removed from moist storage at approximately 9:00 a.m., and the length change was measured and recorded following the same procedure as described in the log on 8/15/02. Following the measurements, the beams were placed into air storage in the drying chamber. The drying chamber is regulated to maintain 50% relative humidity and a temperature of 23°C. Length change of the beams will be checked once a week (Fridays at 9:00 a.m.) from this point on, for a period of ten weeks.

**8/30/02**

Beams were removed from the drying chamber at approximately 9:00 a.m., and the length change was measured and recorded following the same procedure as described in the log on 8/15/02. Following the measurements, the beams were returned to air storage in the drying chamber.

**9/6/02**

Beams were removed from the drying chamber at approximately 9:00 a.m., and the length change was measured and recorded following the same procedure as described in the log on 8/15/02. Following the measurements, the beams were returned to air storage in the drying chamber.

**9/13/02**

Beams were removed from the drying chamber at approximately 9:00 a.m., and the length change was measured and recorded following the same procedure as described in the log on 8/15/02. Following the measurements, the beams were returned to air storage in the drying chamber.

**9/20/02**

Beams were removed from the drying chamber at approximately 9:00 a.m., and the length change was measured and recorded following the same procedure as described in the log on 8/15/02. Following the measurements, the beams were returned to air storage in the drying chamber.

**9/27/02**

Beams were removed from the drying chamber at approximately 9:00 a.m., and the length change was measured and recorded following the same procedure as described in the log on 8/15/02. Following the measurements, the beams were returned to air storage in the drying chamber.

**10/4/02**

Beams were removed from the drying chamber at approximately 9:00 a.m., and the length change was measured and recorded following the same procedure as described in the log on 8/15/02. Following the measurements, the beams were returned to air storage in the drying chamber.

## **Appendix C**

### **SPR Project # 13733-D**

### **Tensile Strength Testing Log**

#### **8/27/02**

Three of the cores obtained on 8/26/02 were tested for tensile strength (CC Nos. 7-9), as directed by the Technical Advisory Group for the subject study. To prepare the cores for testing, it was necessary to trim a thin section from each end of the cores to remove laitance and to give a flat plane for adhering the cores to the tensile testing apparatus. A two-part epoxy was used to bond the core to the testing apparatus. The label on the epoxy indicates full strength is achieved after approximately sixteen hours. The cores were prepared immediately following coring and the epoxy was allowed to cure for the time indicated on the package. Preliminary testing of the epoxy was done to insure that adequate bond between the concrete and steel could be achieved, prior to testing these cores. Test data showed that a bond well above the anticipated bond strength of 200 psi for the precast panel and the overpour could be expected.

The prepared cores were placed into the testing machine (Fig. C-1) and loaded at a constant rate of approximately 10 lbs/sec until failure. In all cases the failure occurred at the interface of the precast panel and the overpour. The area was calculated based on the average diameter of the cores. The average tensile strength of the three cores tested by this method was 190 psi.

#### **9/13/02**

Core Nos. CC-17 through CC-19 were taken from the control slab on 9/12/02 and prepared for tensile testing. The ends were trimmed, as on previous cores, to remove laitance and give a flat true plane for adhering the cores to the testing apparatus.

#### **9/17/02**

Core Nos. CC-17 through CC-19 were tested for tensile strength. Testing was performed following the same procedures previously used. Core CC-17 failed in the over pour concrete instead of at the interface (Fig. C-2). This failure occurred close to the testing plate, but was clearly a failure in the concrete not in the epoxy. The strength recorded at the time of failure was 269 psi, slightly lower than on 9/17/02. The core was trimmed again and the epoxy was reapplied to the plate. Core CC-17 will be retested on 9/18/02, following curing of the epoxy. The surface area of the cores was obtained and the strength was calculated by dividing the total load by the surface area. The average strength for these cores was 199 psi (not including the result from CC-17). Compared with 190 psi as reported on 8/27/02, this result represents little change in strength with approximately two weeks of additional age.

#### **9/18/02**

Core CC-17 was retested today and again failed in the over pour concrete. The strength recorded at the time of failure was 237 psi, slightly lower than on 9/17/02. At this point the decision was made to prepare the cores from the bridge deck for testing and to retest this core following completion of testing on the deck cores. Core Nos. DC-1, DC-4, DC-9 and DC-11 were selected for tension testing. The cores were prepared for testing following the same procedures used for the cores from the control slab. These cores will be tested on 9/19/02.

### **9/19/02**

Core Nos. DC-1, DC-4, DC-9 and DC-11 were tested in tension today. However, there was a problem with the epoxy not setting up. This resulted in the epoxy failing on all of the cores on the bottom or precast panel portion of the core. While not certain, it is believed there was still some moisture present in the concrete, which lead to the failure. The cores were trimmed, dried using compressed air and allowed to sit in sunlight for several hours prior to reapplying the epoxy. The cores will be prepared for testing again and will be retested on 9/20/02.

### **9/20/02**

Core Nos. DC-1, DC-4, DC-9 and DC-11 were retested in tension today. However, the epoxy/concrete failed on all of the cores on the top or overpour portion of the core. These cores and core No. CC-17 will again be prepared for testing. The epoxy that has been used up to this point, will be reevaluated, and perhaps changed. The plates were removed from the panel portion of the cores with a chisel. This end was firmly adhered to the plate and the epoxy was completely cured. Removal caused the concrete to split and all of the epoxy remained firmly attached to the plate. This could indicate that the epoxy failures on 9/19/02 are attributable to the epoxy being stressed and weakened from the loading during testing.

### **9/26/02**

Core Nos. DC-1, DC-4, DC-9 and DC-11 were prepared for testing today. A different two-part epoxy was used for bonding the cores to the plates. The product change came because of conversation with the Alan Rawson, Administrator for Materials and Research and other members of the Research staff.

### **9/27/02**

Core Nos. DC-1, DC-4, DC-9 and DC-11 were retested today for tensile strength. No problems were encountered with the epoxy. DC-1 and DC-11 failed in the overpour concrete, close to the testing plate (See Figs. C-2,3). DC-4 and DC-9 failed at the interface between the precast panel and the overpour concrete. Following testing, one set of testing plates was prepared for reuse on core CC-17 (that has twice failed in the overpour concrete). The conventional method of cleaning (scraping with a chisel) was ineffective in removing the epoxy from the plates. The plates were then submerged in trichlorethylene for the weekend, to break down the epoxy.

### **10/1/02**

Core CC-17 was prepared for retesting today. By allowing the plates to soak in solvent, the old epoxy was completely removed. New epoxy was prepared according to the manufacturer's directions and applied to the core. The epoxy has a cure time of approximately 16 hours. Core CC-17 will be retested on 10/2/02.

### **10/2/02**

Core CC-17 was retested for tensile strength today. It failed for the third time in the overpour concrete at 278 psi. This value is slightly higher than the previous strength results of 269 and 237 psi respectively. After three attempts, this core will not be retested again. The only conclusion

would have to be that the bond strength between the overpour and the precast panel is in excess of 278 psi at this location. It is also unlikely that retesting the cores from the deck that failed in the overpour concrete (DC-1 and DC-11) would fail at the interface if retested. Therefore, it could be concluded that the bond strength at these locations also exceeds the tensile strength of the overpour concrete.

## **PHASE II**

### **7/24/03**

Tensile testing was performed on the cores obtained from the bridge deck on 7/17/03. Core Nos. DC-13, 16, 21 and 23 were selected for testing. In each case the failure occurred in the overpour concrete, near the end of the core. The average strength of the cores tested was 228 psi. The strength obtained showed a 15.4% decrease from Phase 1. As all of the failures occurred in the overpour concrete and /or epoxy, these cores will be trimmed and prepared for retesting.

### **7/30/03**

Phase 2 cores from the control slab were tested for tensile strength today. To aid the bond strength of the epoxy, the testing plates were roughened, and 1/8" deep grooves were cut into the ends of the cores in a grid pattern. The purpose of the grooves was to increase the surface area being bonded and give to the epoxy something more to hold onto. This method of preparation seemed effective as all failures occurred within the overpour concrete with no failure of the epoxy bond to either the plate or the core. As was found in Phase 1, the bond strength between the precast panel and the overpour concrete is stronger than the tensile strength of the overpour concrete. No further testing of these cores is planned at this time.

### **8/6/03**

Phase 2 deck cores were retested today. The cores were prepared for retesting in the manner described for the control cores. Again, all of the failure occurred in the overpour portion of the core. Half were tested with the overpour on top and half were tested with the panel portion on top. The overpour concrete failed regardless of how the core was oriented at the time of testing. The average strength obtained was 337 psi, an increase of 28.1% from the deck cores from Phase 1 and 7.0% higher than the Phase 2 control cores. The bond strength of the interface could not be determined, other than it is more than 337 psi, the average tensile strength of the overpour concrete. No further testing is planned at this time.



**Figure C-1 – Core loaded for testing**



**Figure C-2 – Core after failure in overpour**



**Figure C-3 – Showing panel/overpour bond is stronger than the overpour**

# Tensile Strength Results: Control Cores

CC=Control Core  
DC=Deck Core

**Date Placed**      8/16/2002  
**Date Cored**        8/26/2002  
**Age at coring**      11 Days  
**Date Tested**        8/27/2002  
**Age at Testing**     12Days

Average Area (in2)=	12.95	
SAMPLE I.D.	LOAD (lbs)	Tensile Strength (psi)
CC-7	1820	141
CC-8	2720	210
CC-9	2830	219
Avg.:	2457	190

## PHASE I

**Date Placed**      8/16/2002  
**Date Cored**        9/12/2002  
**Age at Coring**     27Days  
**Date Tested**        9/17/2002  
**Age at Testing**     32Days\*\*

Average Area (in2)=	12.95	
SAMPLE I.D.	LOAD (lbs)	Tensile Strength (psi)
CC-17*	3480	269
CC-17*	3070	237
CC-17*	3600	278
CC-18	2980	230
CC-19	2170	168
<b>Avg.w/CC-17:</b>	2955	236
<b>Avg. w/o CC-17:</b>	<b>2575</b>	<b>199</b>

**\*\* Retested 9/18/02 33 days**  
**\*\* Retested 10/2/02 47 days**

\* Failed in overpour, near plate

## PHASE II

**Date Placed**      8/16/2002  
**Date Cored**        7/22/2003  
**Age at coring**     340 days  
**Date Tested**        7/30/2003  
**Age at Testing**     348 days

Average Area (in2)=	12.37	
SAMPLE I.D.	LOAD (lbs)	Tensile Strength (psi)
CC-32	3710	300
CC-33	3940	319
CC-34	4020	325
Avg.:	3890	314

Failed in over pour concrete  
 Failed in over pour concrete  
 Failed in over pour concrete



## Tensile Strength Results: Deck Cores

CC=Control Core

DC=Deck Core

### PHASE I

**Date Placed**     8/16/2002  
**Date Cored:**     9/12/2002  
**Age at Coring:**   27Days  
**Date Tested:**    9/20/2002  
**Age at Testing:**   35 Days

Average Area (in2)=	12.95			
SAMPLE I.D.	LOAD (lbs)	Tensile Strength (psi)	COMMENTS	
DC-1	2430	188	Failed at top (epoxy)	
DC-4	2680	207	Failed at top (epoxy)	
DC-9	2140	165	Failed at top (epoxy)	
DC-11	2680	207	Failed at top (epoxy)	
Avg.:	2483	192		

**Date Placed**     8/16/2002  
**Date Cored:**     9/12/2002  
**Age at Coring:**   27Days  
**Date Retested:**   9/27/2002  
**Age at Retest:**   42 Days

Average Area (in2)=	12.95			
SAMPLE I.D.	LOAD (lbs)	Tensile Strength (psi)	COMMENTS	
Retest DC-1	3610	279	Failed in over pour concrete	
Retest DC-4	3120	241		
Retest DC-9	3340	258		
Retest DC-11	3550	274	Failed in over pour concrete	
Avg.of DC-4&DC-9:	3230	249		
Avg.of DC-1&DC-11:	3580	266		
<b>Avg.: DC-1, 4, 9 &amp;11</b>	<b>3405</b>	<b>263</b>		

### PHASE II

**Date Placed**     8/16/2002  
**Date Cored:**     7/17/2003  
**Age at Coring:**   335 Days  
**Date Tested:**    7/24/2003  
**Age at Testing:**   342 Days

Average Area (in2)=	12.37			
SAMPLE I.D.	LOAD (lbs)	Tensile Strength (psi)	COMMENTS	
DC-13	2710	219	epoxy/concrete failure at overpour end	
DC-16	2700	218	epoxy/concrete failure at overpour end	
DC-21	2580	209	epoxy/concrete failure at overpour end	
DC-23	3300	267	epoxy/concrete failure at overpour end	
Avg.:	2823	228		

**Date Placed**     8/16/2002  
**Date Cored:**     7/17/2003  
**Age at Coring:**   335 Days  
**Date Tested:**    8/6/2003  
**Age at Testing:**   355 Days

Average Area (in2)=	12.37			
SAMPLE I.D.	LOAD (lbs)	Tensile Strength (psi)	COMMENTS	
<b>Retest DC-13</b>	4080	330	concrete failure at overpour end	
<b>Retest DC-16</b>	3730	302	concrete failure at overpour end	
<b>Retest DC-21</b>	4860	393	concrete failure at overpour end	
<b>Retest DC-23</b>	4000	323	concrete failure at overpour end	
Avg.:	4168	337		

## **Appendix D**

### **SPR Project # 13733-D**

### **Coring Log**

**8/26/02**

A total of 10 cores were obtained from the control slab for testing. Three were designated for shear testing perpendicular to the surface finish marks on the precast deck panel. Three cores were designated for shear testing parallel to the surface finish marks on the precast deck panel and three were designated for testing the tensile strength. One core was held in reserve. Cores were obtained by advancing a four-inch nominal inside diameter diamond core bit through the cast in-place PCP topping and the precast panel below. Prior to coring, the strands and reinforcing were located with a handheld pachometer in order to avoid contacting the core bit with the reinforcement. Also, the direction of the surface finish marks on the panel surface was marked on the top of the core location prior to coring for reference during testing.

At the first location, upon breaking through the bottom of the precast panel, the core became lodged inside the bit. Due to the amount of cement paste present and inadequate water to keep the bit flushed of the paste, efforts to retrieve the core from the bit were unsuccessful. After this experience, it was decided to mark the depth of the control slab on the bit and core until just shy of breaking through. This proved to work well and no problems were encountered on subsequent cores. On average, the bit was advanced to within  $\frac{1}{8}$ " of the panel depth. The cores were dislodged from the final thickness of panel by slightly tapping the top of the cores with a 16 oz. hammer. When the cores broke free, they were allowed to drop approximately one inch onto a pine 2x4. The cores were recovered from the slab by reaching underneath and pushing them up through the top.

None of the pre-stressing strands were hit while coring. On two occasions reinforcing steel was contacted, but was passed through without incident as the bars were only slightly grazed by the bit. On the first and last cores, the lifting inserts were hit. This may have contributed to the difficulty in retrieving the first core from the bit. The lifting insert can be observed in the last core, virtually cut in half. The insert was not discovered until the cores were examined at the laboratory prior to testing. This core was placed in wet storage and while interesting to look at, will most likely not be tested.

**9/3/02**

A total of six cores were obtained for shear testing. As before, three were designated for testing parallel and three were designated for testing perpendicular to the surface finish marks on the precast panel. No extra cores were obtained. These cores were obtained in response to the discovery of an assembly error in the shear-testing device that applied unwanted tensile forces to the cores when tested. Eight attempts were required to obtain the cores. Reinforcing steel was contacted on one occasion causing the core to become lodged in the bit. The core was retrieved intact with some effort. However, it was decided not to keep this core due to possible damage to the bond that could not be seen. In another instance, the core suddenly snapped off inside the bit while advancing through the cast-in-place overpour. Nothing could be seen that would indicate a reason for the sudden breakage.

As has become the procedure, the bit was advanced to within  $\frac{1}{8}$ " of the panel depth. The cores were dislodged from the final thickness of panel by slightly tapping the top of the cores with a 16 oz. hammer. When the cores broke free, the cores were allowed to drop approximately one inch onto a pine 2x4. The cores were recovered from the slab by reaching underneath and pushing them up through the top.

### **9/12/02**

Cores for Phase 1 of the project were obtained today (Figs. D-1,2). Twenty-two cores were taken, 12 from the bridge deck, 10 from the control slab (Fig. D-3). The cores from the bridge deck were obtained in similar fashion to the control slab. The core bit was advanced to within approximately  $\frac{1}{8}$ " of the deck thickness, then the core was dislodged by tapping the top with a hammer. The cores were captured from the underside of the deck by allowing them to drop one inch into a waiting receptacle. The cores were then pushed up through the core hole and taken by a person on the top of the deck.

Core Nos. DC-4 and DC-7 were mistakenly allowed to drop onto the planking situated on the bottom flange of the steel girders. The location of these cores was improperly located from the under side of the bridge deck. Core No. DC-4 located over the pier dropped approximately eight feet. Core No. DC-7, located at mid-span east of the pier, fell approximately five feet to the planking. Both cores were successfully retrieved and there was no visible damage to either core.

### **9/19/02**

Phase 2 core locations were marked on the underside of the deck today. Core locations were marked using a permanent marker that should remain visible until the Phase 2 cores are obtained. The core numbers were marked adjacent to the core locations also.

### **9/20/02**

The core holes in the bridge deck from the Phase 2 coring were covered with 12"x 12" x  $\frac{1}{4}$ " steel plates today. The plates were adhered to the deck using a one-part polyurethane construction adhesive. The adhesive should hold the plates firmly in place during temporary paving operations and throughout the winter months. The plates will be removed when the temporary pavement is stripped in the spring of 2003.



**Figure D-1 – Advancing core bit through deck**



**Figure D-2 – Core location as seen from underneath**



**Figure D-3 – Control Slab with core holes**

**10/8/02**

A call was received today at Materials & Research from Dana Carlson, NHDOT Contract Administrator. Dana called to report that some of the steel plates installed at Phase 1 core locations had been dislodged. Ray Wellman, Research Technician, visited the site because of Dana's report. Plates covering the locations of core Nos. DC-1, DC-2 and DC-7 had been loosened, apparently from construction activities on the bridge. The remaining nine core locations were inspected and the plates were found to still be firmly in place. Dana advised Ray that R.S. Audley, General Contractor for the construction project, would reapply the adhesive and plates prior to paving the deck. Ray left a new tube (cartridge) of the adhesive (OSI Premium Polyurethane Construction Adhesive) with Dana for the contractor to use, unless R.S. Audley felt they had something that would work better. Paving of the bridge deck is tentatively scheduled for early in the week of 10/14.

**10/15/02**

Contract Administrator Dana Carlson was contacted today regarding placement of the temporary pavement. Dana said the paving was done Friday 10/11. The plates that came loose from the deck (location Nos. DC-1, DC-2, and DC-7) were reattached by the contractor prior to paving and did not loosen again. However, as reported by Kevin Dunbar, the plate over core location DC-9 came loose when the paver drove over it. Kevin said it was dragged approximately four feet before being seen. The plate was retrieved and placed back over the core hole with adhesive and the pavement was repaired in this location. Kevin also said that there are no noticeable signs of the plates now that the pavement is down and they all seem to be secure.

**7/17/03**

Bridge deck coring for Phase 2 of the project was performed today. Twelve cores were obtained from the bridge deck by advancing the core bit completely through the overpour concrete and precast panels. The cores were captured from the underside of the deck by allowing them to drop one inch into a waiting receptacle. The cores were then pushed up through the core hole to a person on the top of the deck and labeled.

**7/22/03**

Control slab coring for Phase 2 of the project was performed today. Nine cores were obtained from the control slab by advancing the core bit completely through the overpour concrete and precast panels.

## Appendix E

### SPR Project # 13733-D

### Bridge Design Guidelines

*Note: The following excerpt from the NHDOT Bridge Design Manual provided guidance and restrictions on the use of precast deck panels prior to the research documented in this report.*

#### 620.5 Precast Deck Panels

The Department's approach in the use of precast deck panels has been to take a methodical, step-by-step approach (proceeding from lower volume, less critical locations towards more critical bridge installations) in instituting the use of these panels over time. In keeping with this philosophy, the Department continues to take steps towards expanding the number of sites where panels may be used but proceeding in a cautious fashion that allows proper evaluation of past installations prior to proceeding to more difficult or important sites.

The Department will provide for a bid item that allows for either a CIP or SIP panel alternative, only as provided on the plans. (i.e. The Department will make the decision when it is appropriate to allow the SIP panels to be used as an alternate system.) The Deck Panel Detail Standard sheet shall be included in the Contract Plans when the SIP panel alternative is part of the contract. The following criteria should be used to determine when this item may be included in the Contract.

Criteria for when to include an item for CIP or SIP panel alternative:

1. For Low or Medium Truck Volume Bridges: (Current ADTT<400).

With High Truck Volume Bridges: (Current ADTT>400) the Department will proceed as follows:

1. Monitor installations on NH Rte 51 EB over I-95 & at Tilton Exit 20 (US Rte 3 & NH 11 over I-93).
2. Construct experimental installations on:
  - 1) I-93, shorter span for construction in 2002 (bare deck).
  - 2) I-393, long span for construction in 2002.Provide instrumentation and monitoring for these decks and evaluate for 5 years.
2. Maximum Girder Spacing: Steel = 10'-0"; Concrete = 12'-0".
3. Design Deck Thickness: 8 1/2" (3 1/2" + 5") for deck panels; \*8" for CIP deck.
4. Minimum Girder Flange Width: 12" (Need 6" between panels).

5. Bridge Length (Centerline to Centerline of bearings): Simple Span: Maximum 150'. Multi Span: Maximum of 175' for any individual span of the structure (or if ADTT<100, no limit of span length).
6. Straight Girder Bridges Only.
7. Deck cross-slope <.04
8. Minimum Panel Length: 4'-6".

\*Girders should be designed for 1/2" of additional WDLNC. In order to allow for field adjustments between the two options, an additional 1/2" of haunch height should be provided when detailing the CIP option. (This additional 1/2" should not be used as part of the composite section properties).

#### 620.5.1. Design and Detail Requirements

1. Precast prestressed concrete deck panels used, as permanent forms spanning between girders should be designed to act composite with the cast-in-place portion of the deck slab.
2. The concrete compressive strength at release  $f'_{ci}$  should be minimum 27.5 MPa (4000 psi). The 28-day compressive strength  $f'_c$  should be 40 MPa (6000 psi).
3. Tension in the precompressed tensile zone under final service conditions after all losses have occurred should be  $\leq 6\sqrt{f'_c}$ . Compression in the panel at release should be  $\leq 5.2$  MPa (750 +/- psi).
4. Prestressing strands shall be 9.5 mm (3/8 in.) diameter and should extend 100 mm (4 in.) minimum outside the panel edges.
5. Panel length should be set to provide a minimum grout bed width of 50 mm (2 in.). This requires a minimum of 75 mm (3 in.) from the edge of panel to the edge of steel girder flange and 87 mm (3.5 in.) to the edge of a bulb-tee flange, assuming a 25 mm (1 in.) minimum grout dam width.
6. The minimum thickness of the cast-in-place portion of the slab should be 125 mm (5 in.).
7. The minimum thickness of the panel should be 90 mm (3 1/2 in.). The top surface shall receive a broom finish.

#### 620.5.2 Temporary Support System

Two Step System: After setting deck panels, a grout bed is placed in the girder haunch area and allowed to cure prior to placing the remainder of the deck.

1. Temporary supports for precast deck panels shall consist of continuous, high-density expanded polystyrene strips (grout dam) with a minimum compressive strength of 380 Pa (55 psi). If leveling screws are used a 27.2 kg per cubic meter (1.7 pound per cubic foot) polyethylene foam seal shall be used as a grout dam. An approved adhesive should be specified to affix the dam to the girder and the deck panel.
2. If leveling screws are used, temporary bracing between the ends of panels shall be installed as required to prevent transverse panel movement that could lead to loss of bearing on the leveling screws.
3. Deck panels should be specified to be grouted in place prior to placement of the cast-in-place concrete deck. The grout bed should extend for the full width of the girder flange completely filling the area between the grout dams. The top of the grout bed should be 25 mm (1 in.) below the strand extensions.
4. If leveling screws are used they shall be completely removed after the grouting operation and prior to deck placement.

#### 620.5.3 Haunch Height

The haunch height should take into account the following factors:

1. A minimum midspan blocking distance of 25 mm (1 in.) plus an allowance for cross-slope and camber tolerance is required in order to obtain a minimum grout bed height of 25 mm (1 in.).
2. The flange bolts for field splices of steel girders may interfere with precast concrete deck panels. Provide a haunch depth that accommodates the added thickness of the field splice. Modifying panel thickness to avoid conflict with field splices shall not be allowed. To avoid this conflict, the splice bolts may be oriented to be with the bolt head oriented to be on the top splice plate (i.e. install the bolts down through the splice rather than up through the splice).

#### 620.5.4 Deflection Breakdown Table

A deflection breakdown at each .3, .5, and .7 location, which lists separately the dead load deflection of the deck panels, cast-in-place slab, and composite loads, should be shown within a table on the contract plans.

#### 620.5.5 Deck Reinforcement



The top mat of reinforcing steel as specified in Plate 650.4e(650.4f) should be provided in the cast-in-place portion of the slab.

For continuous bridges with precast deck panels, the requirements of AASHTO 10.38.4.3 should be modified to provide  $\frac{2}{3}$  of the “1% of the cross-sectional area of the concrete slab” in the top mat. The remaining  $\frac{1}{3}$  may be ignored. Maximum bar size allowed is a #6.

**Appendix F**  
**SPR Project # 13733-D**  
**Concrete & Grout Mix Designs**

<b>Precast Concrete Deck Panels</b>		
<b>Mix Design</b>		
<b>f'<sub>c</sub>=6500 psi</b>		
Per Cubic Yard:	Amount	Brand/Source
Cement:	716 lbs	Glenn's Falls Type III
Fly Ash:	126 lbs	East Lake Type F
Water:	229 lbs	On site Well
Fine Agg.:	1010 lbs	J.P. Carrara & Sons
Coarse Agg.:	1879 lbs	J.P. Carrara & Sons
Air Entrainment:	7.5 oz	W.R. Grace – Darex II
HR Water Reducer	105 oz	W.R. Grace – ADVA Flow
Retarder:	25 oz	W.R. Grace – Daratard 17
Corrosion Inhibitor:	512 oz	W.R. Grace - DCI
<b>Cast-in-Place Concrete</b>		
<b>Mix Design</b>		
<b>f'<sub>c</sub>=4000 psi</b>		
Per Cubic Yard:	Amount	Brand/Source
Cement:	320 lbs	Cement Quebec Type II
GGBFS:	320 lbs	Newcem
Water:	279 lbs	Person Concrete, Winnisquam
Fine Agg.:	1190 lbs	Campton Sand & Gravel
Coarse Agg.:	1755 lbs	Hooksett Crushed Stone
Air Entrainment:	4.4 oz	W.R. Grace – Darex II
HR Water Reducer	25 oz	W.R. Grace – Daracem 100
Retarder:	10 oz	W.R. Grace – Daratard 17
<b>Deck Grout</b>		
<b>Mix Design</b>		
<b>f'<sub>c</sub>=4000 psi</b>		
Per Cubic Yard:	Amount	Brand/Source
Cement:	823 lbs	Cement Quebec Type II
Water:	314 lbs	Person Concrete, Winnisquam
Fine Agg.:	1350 lbs	Campton Sand & Gravel
Coarse Agg.:	1350 lbs	Hooksett Crushed Stone
Air Entrainment:	4.4 oz	W.R. Grace – Daravair 1000
Water Reducer	24.69 oz	W.R. Grace – Daracem 65
HR Water Reducer	25 oz	W.R. Grace – Daracem 100

**Table F-1 - Mix Designs**