CRACK CONTROL IN TOPPINGS FOR PRECAST FLAT SLAB BRIDGE DECK CONSTRUCTION

By

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A THESIS PRESENTED TO THE GRADUATE SCHOOL
OF THE UNIVERSITY OF FLORIDA IN PARTIAL FULFILLMENT
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To my late grandparents, Antonio Amado Alfonso and Ermigio Gonzalez

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This thesis presents the results of several techniques that were evaluated to provide crack control in the cast portion of a precast flat slab bridge. Poor curing techniques and improper placement of the reinforcement has caused excessive shrinkage cracking in a number of flat slab bridges in Florida. In conjunction with the Florida Department of Transportation (FDOT), four full-scale flat slab bridge spans were constructed to test the field performance of the toppings. FDOT guidelines were followed in the design and construction of the decks. The toppings incorporated either steel fibers, synthetic fibers, a steel/synthetic fiber blend, carbon-fiber grid, or a shrinkage-reducing admixture. The toppings were monitored visually for cracking for 30 weeks and are currently under observation in Tallahassee, Florida.

As of March 2005, no cracks had developed in the toppings, because insufficient tensile stresses were generated. Fiber reinforced mixtures performed better in reducing

average crack width using the restrained ring test, and their performance improved with increasing fiber volume. Crack control treatments did not affect concrete modulus of elasticity or tensile strength. The results presented herein were based on observations during construction, results of materials tests, and performance of the toppings.

CHAPTER 1
INTRODUCTION

Background

Precast flat slab bridges are a practical alternative to traditional deck/girder designs used for short span bridges. Using precast slabs reduces the price of bridge construction by virtually eliminating the need for formwork thus making it economically attractive. It allows for faster construction time and quicker project turnover. According to the FDOT's Structures Manual (2004b), the price of the superstructure on a flat slab bridge is the least per square foot, when compared to other designs used in Florida.

Flat slab bridges consist of prestressed, precast concrete deck panels that span from bent to bent. The panels act as permanent forms for a cast-in-place deck. The top surface of the flat slab is roughened to transfer horizontal shear. In some cases, transverse reinforcement is placed, to ensure horizontal shear transfer. A topping is then placed over the precast flat slab, which allows the composite to act as a single unit. Some panels incorporate a shear key to transfer transverse shear. The keys usually contain welded wire mesh, reinforcing bars, or both as well as non-shrink grout. The topping contains transverse and longitudinal reinforcement intended to provide crack control and lateral transfer of shear between the panels. Figure 1 shows recently erected prestressed slabs before topping placement. These panels have horizontal shear reinforcement and shear keys.

Poor curing techniques and improper placement of reinforcement has caused excessive shrinkage cracking in a number of flat slab bridges in Florida. Excessive cracking is unsightly, can affect the durability of the wearing surface, and can lead to corrosion of the reinforcement.

Figure 1. Typical prestressed slab panels

Purpose of Study

The focus of this research was to evaluate techniques for providing crack control in the cast portion of a precast flat slab bridge. A review of methods that have been used to control cracking on bridge decks was conducted. Several systems were considered and chosen for use in the experimental program based on their effectiveness, ease of implementation and application, and effect on the labor and construction cost of the bridge. These systems were then evaluated on full-scale precast flat slab bridge spans. Specimen size and shape were chosen to closely match existing field conditions and steps were taken to ensure that toppings were exposed to similar curing conditions. They were left outside to weather, and were monitored visually for cracking. Crack width, crack distribution, ease of application, and the overall cost of each system were compared and ranked based on performance. Recommendations are made for changes to flat slab bridge construction techniques based on their performance.

CHAPTER 2
SITE EVALUATIONS

Site visits were conducted by the author to assess crack patterns on selected existing flat slab bridges. Three Central Florida bridges were visited: Turkey Creek Bridge (No. 700203), Mill Creek Bridge (No. 364056), and Cow Creek Bridge (No. 314001). All of these have reflective longitudinal cracks over the joints in the flat slabs, and transverse cracks over the bents.

Turkey Creek Bridge

The Turkey Creek Bridge is located on US1 south of Melbourne. It is a simply supported, six-span bridge with 12 in deep precast flat slabs with shear keys and an 8 in topping. The topping is reinforced with No. 5 bars at 12 in on center in each direction. The topping has extensive longitudinal cracks that vary in size. Reflective cracks are located over each flat slab joint. Many of the cracks have been repaired with epoxy (Figure 2) and show no signs of continued cracking.

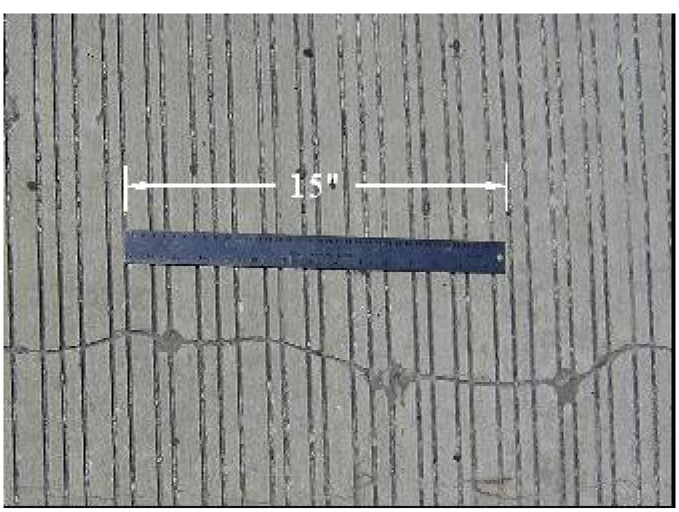

Figure 2. Repairs to cracks on Turkey Creek Bridge

A large number of vehicles were using the bridge on the day of the visit. In addition to showing the most cracking, it also carries the largest traffic volume of the three bridges.

Mill Creek Bridge

The Mill Creek Bridge is located on CR318 north of Ft. McCoy. It is a simply supported, two-span bridge composed of 15 in deep precast flat slabs. The topping has a reflective crack over each flat slab joint (Figure 3) that measures an average of 0.016 in. Cracks were also noted over the middle bent where the flat slabs meet end to end. The control joint is located at the center and runs with the span of the bridge. All of these cracks are relatively small and have not affected the performance of the bridge. No construction drawings were available for this bridge.

Figure 3. Reflective crack on topping of Mill Creek Bridge

Cow Creek Bridge

Cow Creek Bridge is located on CR 340 just west of High Springs. It is a five-span bridge with 12 in deep flat slabs with shear keys and a 6 in topping (Figure 4). The flat slabs have horizontal shear reinforcement and the topping has No. 5 reinforcing bars at 6 in on center in the transverse direction and at 12 in on center in the longitudinal direction. Previous assessment by the FDOT showed that the reinforcement bars in the topping were incorrectly installed at 4 to 5 in below the topping. The topping has a reflective longitudinal crack over each joint in the flat slab. These cracks measured an average of 0.028 in. It also has cracks along most of the saw-cut joints located over the bents. Figure 5 shows the typical saw cut and bearing located over every bent. Concrete has spalled in some areas adjacent to the cuts (

Figure 6). This type of cracking occurs when the control joints are cut after the concrete has set. The longitudinal cracks do not appear to have affected the performance of the bridge.

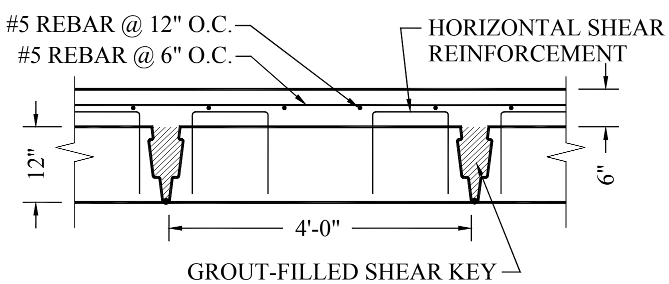

Figure 4. Cow Creek Bridge cross-section

Summary

Three precast flat slab bridges with reinforced concrete toppings were visually inspected. The Cow Creek and Turkey Creek bridges had shear keys built into the prestressed slabs. Slab depth varied from bridge to bridge. All of the bridges had a reflective longitudinal crack over each flat slab joint and multiple transverse cracks over

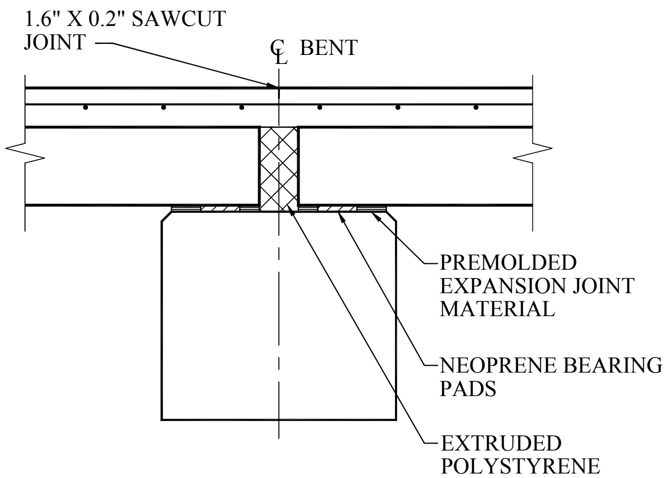

Figure 5. Control joint and bearing detail

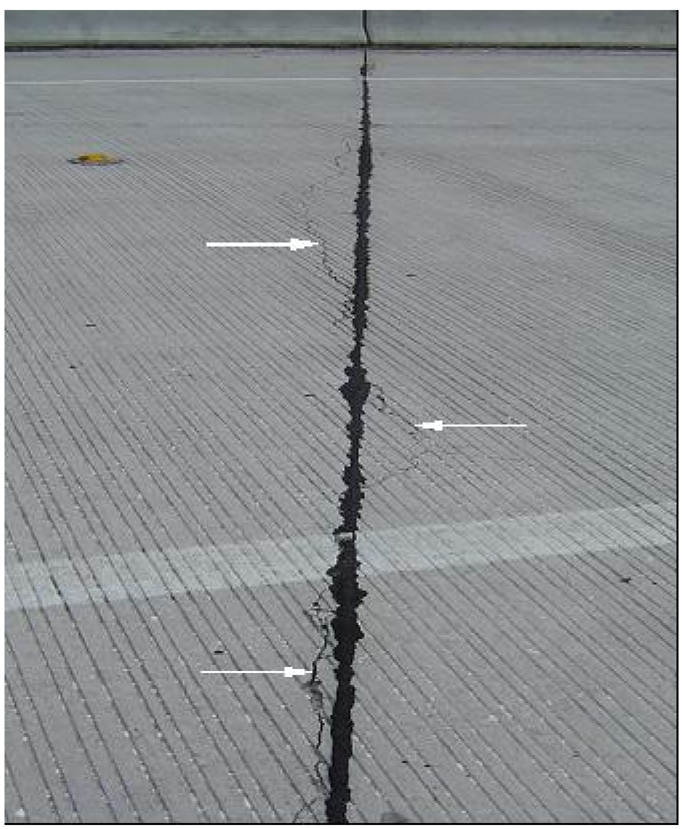

Figure 6. Transverse cracks at a control joint on the Cow Creek Bridge

the bents where the topping goes into negative moment. The topping on the Cow Creek Bridge was spalling at these locations. The Turkey Creek Bridge showed the most cracking and is the only one to have been repaired.

CHAPTER 3
LITERATURE REVIEW

Cracking of bridge decks is not a problem that is specific to flat slab bridges. Although limited research has been conducted dealing specifically with cracking on this type of bridge, a good deal of research has been performed on deck cracking of traditional slab/girder and deck slab bridges. Several of the factors listed by Issa (1999) are common causes of deck cracking.

- Poor curing procedures which promote high evaporation rates and a large amount of shrinkage.
- Use of high slump concrete
- Excessive amount of water in the concrete as a result of inadequate mixture proportions and re-tempering of concrete.
- Insufficient top reinforcement concrete cover and improper placement of reinforcement.

Cracks may not be the result of bad design but rather an outcome of poor construction practice.

Researchers have tested several methods to control cracking that can be easily implemented and though they do not increase the tensile strength of the concrete, they do improve its shrinkage and post crack behavior. Many of these have been implemented by transportation departments and have proven to work in the field.

The New York Thruway Authority (NYTA) and the Ohio Turnpike Commission (OTC) have successfully used shrinkage compensating concrete (SCC) to control shrinkage cracking on bridge decks (Ramey, Pittman, and Webster 1999). Although the

NYTA had problems with deck scaling in the bridge decks that used SCC it was determined not to be a factor. The OTC had the greatest success with SCC. They have replaced 269 bridge decks with SCC and only 11 have shown minor or moderate cracking with none showing severe cracking. This same study also showed that good quality SCC requires continuous curing to activate the ettringite formation. The OTC requires contractors to use fog spraying under certain weather conditions, always use monolecular film to retard evaporation, and control the curing water temperature to avoid thermal shock. They also require wet curing for seven days, which is necessary because SCC will crack if any ettringite is activated after the concrete hardens. Use of SCC requires strict curing techniques to effectively eliminate shrinkage cracks.

Research has shown that shrinkage reducing admixtures (SRA) effectively reduce drying shrinkage of concrete and, subsequently, cracking. Tests show a reduction in drying shrinkage of about 50 to 60% at 28 days, and 40 to 50% after 12 weeks (Nmai et al. 1998). Restrained ring tests showed that concrete mixtures with SRA decrease the rate of residual stress development by decreasing the surface tension of water by up to 54% (Pease et al. 2005). A considerable reduction in crack width occurs as compared with normal concrete depending on the type and amount of SRA used (Shah, Karaguler, and Sarigaphuti 1992). SRA can be integrated in the mixture or applied topically to the concrete surface after bleeding stops. Better results are obtained with larger surface application rates. Mixing SRA integrally, however, is more effective.

Rectangular slabs and ring type specimens have been used to demonstrate the ability of synthetic fibers to control cracking resulting from volume changes due to plastic and drying shrinkage. Synthetic fibers were shown to reduce the amount of plastic shrinkage cracking when compared to the use of welded wire mesh (Shah, Sarigaphuti, Karaguler 1994). They tested polypropylene, steel, and cellulose fibers using a restrained ring test at 0.5%, 0.25%, and 0.5% by volume, respectively. The maximum crack width was reduced by 70% at those dosage rates. The ability of the fibers to control cracking is partially due to the decrease in the amount of bleed water (Nanni, Ludwig, and McGillis 1991; Soroushian, Mirza, and Alhozaimy 1993). The authors suggested that the presence of fibers reduced settlement of the aggregate particles, thus eliminating damaging capillary bleed channels and preventing an increase in inter-granular pressures in the plastic concrete. Adding synthetic fibers also decreases the initial and final set times of the concrete. Decreasing the time that the concrete is left exposed to the environment in a plastic state promotes reduced shrinkage cracking.

A series of tests run by Balaguru (1994) on steel, synthetic, and cellulose fibers reveals that the fiber's aspect ratio (length/diameter) seems to be a major factor contributing to crack reduction. An increase in fiber content also contributed to a smaller crack area and width. The same results were obtained by Banthia and Yan (2000), and Grzybowski and Shah (1990) (Figure 7-Figure 10). Fibers with a high aspect ratio have more contact area with the concrete mixture consequently, more stress is transferred by the fiber before pull-out. Increases in fiber content usually lead to smaller crack widths. Too much fiber, however, may affect the workability of the concrete mixture and cause entanglement into large clumps. Fiber length, volume, and specific fiber surface (total surface area of all fibers within a unit volume of composite) are all major contributing factors to the amount of cracking.

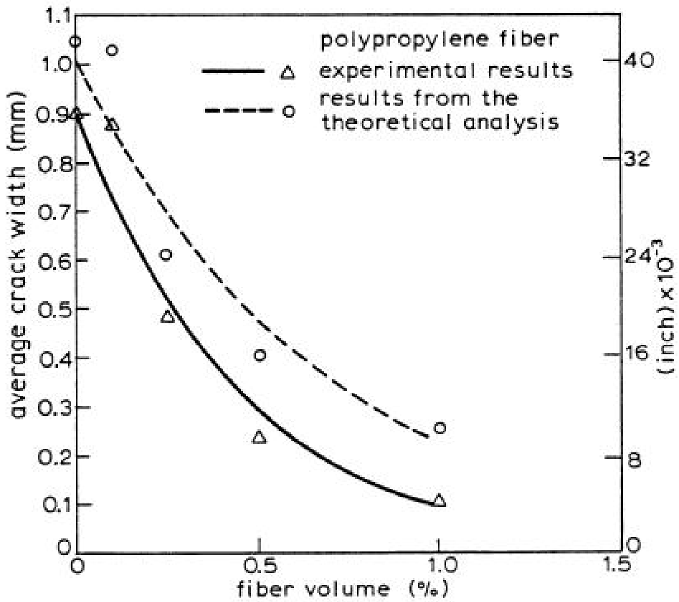

Figure 7. Average crack width vs. fiber volume for polypropylene fibers (Grzybowski and Shah 1990)

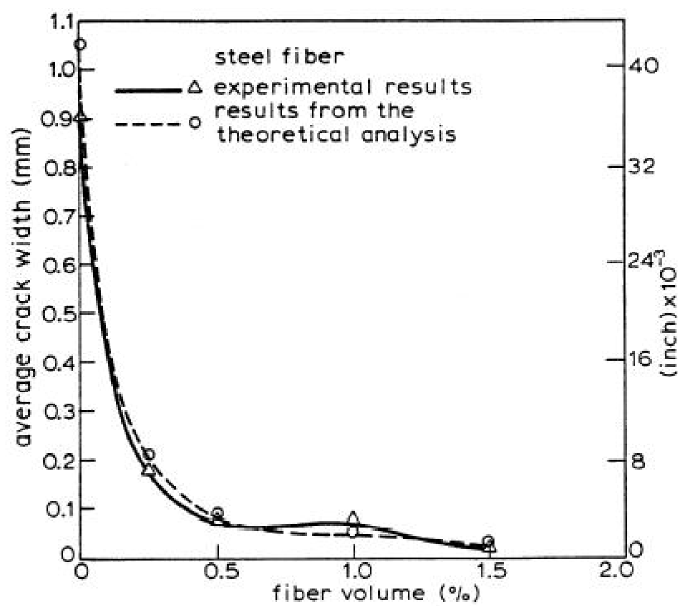

Figure 8. Average crack width vs. fiber volume for steel fibers (Grzybowski and Shah)

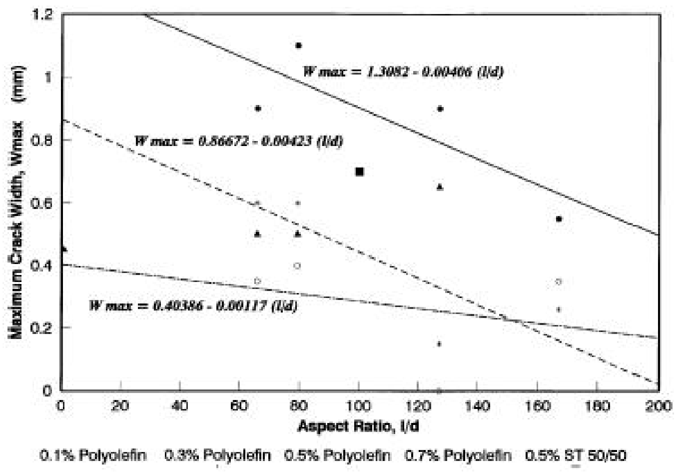

Figure 9. Maximum crack width vs. aspect ratio (Grzybowski and Shah 1990)

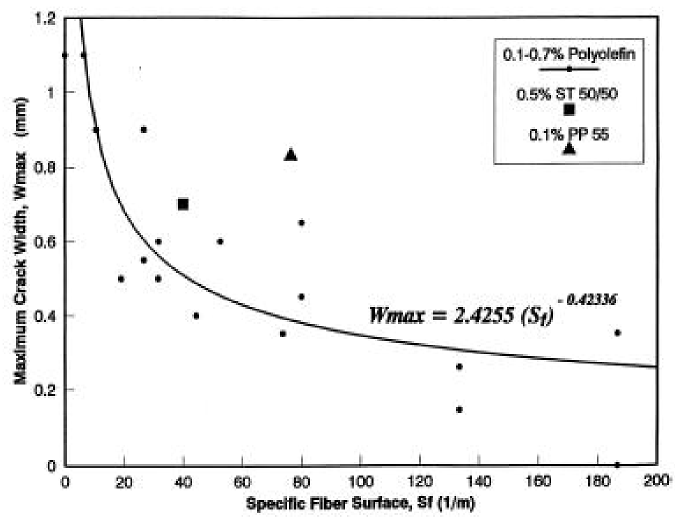

Figure 10. Maximum crack width vs. specific fiber surface (Grzybowski and Shah 1990)

Little research was found on use of a rigid carbon fiber reinforced polymer (CFRP) composite grid to control bridge deck cracking. A CFRP grid would make it possible to reinforce the concrete near the surface. Flexure testing by Makizumi, Sakamoto, and Okada (1992) placed a carbon-fiber grid, prestressed strands, and in some cases, reinforcing bars, in small beams. The grid was placed 3mm from the extreme face in tension. Cracks were reduced by half in cases with reinforcing bars. Specimens that contained only grid and prestressing met the minimum crack size requirements proposed by the Japan Society of Civil Engineers (JSCE).

CHAPTER 4
EXPERIMENTAL PROGRAM

Introduction

Several methods of controlling cracking were considered for testing (Table 1). The concrete toppings that were evaluated contained either synthetic fiber, steel fiber, a blend of steel and synthetic fibers, a shrinkage reducing admixture, or a carbon-fiber grid. They were selected based on their ease of application and their effect on the construction and labor cost of the bridge deck. Many of these are presently used in the construction industry. A standard FDOT Class II (bridge deck) mixture was also used as a basis for comparison.

Table 1. Methods considered for controlling shrinkage cracking

Method of control	Advantages	Disadvantages	Comments	Test
Control	n/a	n/a	n/a	Yes
Transverse post-tensioning – precast panels are post-tensioned together before topping is placed.	Reduce transverse reinforcement requirements.	Difficult and costly on small, low-volume projects Curing must still be carefully implemented	n/a	No
Shrinkage compensating cement: Concrete will increase in volume after setting and during early age hardening by activation of ettringite (ACI 223-98)	No special equipment or techniques are needed	Delay in pouring causes loss in slump (ACI 223-98) Curing must be carefully monitored	Concrete must remain as wet as possible during curing in order to activate ettringite. Concrete expands during wet cure No effect on creep (ACI 223-98) No modification of formwork is needed (ACI 223-98) Used to control dry shrinkage	No
Shrinkage reducing admixtures: Reduces capillary tension that develops within the concrete pores as it cures (Pease, Shah, Weiss 2005)	Easily mixed in at jobsite or at cement plant Considerable reduction in crack width as compared with plain concrete (Shah, Karaguler, and Sarigaphuti 1992)		Volume of water added into mix must be reduced by volume of admixture added into mix (Pease, Shah, Weiss 2005)	Yes

Table 1. Continued

Method of control	Advantages	Disadvantages	Comments	Test
Fiber reinforced concrete: Randomly distributed fibers carry tensile stresses after cracking	Discontinuous and distributed randomly Loss in slump, not in workability (ACI 544.1R) Easily incorporated into mix	Balling may become a problem if fiber lengths are too long (ACI 544.1R)	Many types and lengths available All bonding is mechanical (ACI 544.1R)	
Synthetic fibers: Commercially available fibers shown to distribute cracks and decrease crack size (ACI 544.1R)			Most fibers will not increase the flexural or compressive strength of the concrete (ACI 544.1R) Fiber dimensions influence shrinkage cracking Mostly used in flat slab work to control bleeding and plastic shrinkage (ACI 544.1R)	
Acrylic		Not much research has been conducted	Has been used to control plastic shrinkage (ACI 544.1R)	No
Aramid		Expensive when compared to other fibers	Mostly used as asbestos cement replacement in high stress areas (ACI 544.1R)	No
Carbon	Reduces creep Reduces shrinkage significantly (ACI 544.1R)	Difficult to achieve a uniform mix (ACI 544.1R)	Research shows that carbon fibers have reduced shrinkage of unrestrained concrete by 9/10 (ACI 544.1R)	No
Nylon	Widely used in industry	Moisture regain must be taken into account at high fiber volume content (ACI 544.1R)	Shown to have decreased shrinkage by 25% (ACI 544.1R)	No

Table 1. Continued

Method of control	Advantages	Disadvantages	Comments	Test
Polyester		No consensus on long term durability of fibers in portland cement concrete (ACI 544.1R)	Not widely used in industry	No
Polypropylene	Significantly reduces bleed water (ACI 544.1R) Widely used in industry		Shown to reduce total plastic shrinkage crack area and maximum crack width at 0.1 % fiber volume fraction (Soroushian, Mirza, and Alhozaimy 1995)	Yes
Steel fibers	Many shapes and sizes available Use of high aspect ratio fibers provide high resistance to pullout (ACI 544.1R) Widely used in industry	Surface fibers will corrode (surface staining?) If large cracks form, fibers across opening will corrode (ACI 544.1R)	May not reduce total amount of shrinkage but increase number of cracks reducing crack size (ACI 544.1R)	Yes
Natural fibers	Very inexpensive	Requires special mix proportioning to counteract retardation effects of glucose in fibers (ACI 544.1R)	Not widely used in industry	No

Table 1. Continued

Method of control	Advantages	Disadvantages	Comments	Test
Carbon FRP Grid: Grid system carries tensile stresses after cracking at depth of installation	Available in different sizes Can be placed at a specific depth	May not be available in large sheets Manufacturer recommended that concrete be screeded at level where mesh is placed	Not much information available on its use to control cracking FDOT allows placement of grid at ½ in below surface	Yes
Glass FRP Grid: Grid system carries tensile stresses after cracking at depth of installation	Available in different sizes Can be placed at a specific depth	Concrete may need to be screeded at level where mesh is placed	Not much information available on its use to control cracking FDOT allows placement of grid at ½ in below surface	No

Each concrete mixture that was used for the precast slabs and the toppings conformed to the parameters set forth in the FDOT Standard Specifications for Road and Bridge Construction (2004a) (Table 2, Table 3, and Table 4). All of the concrete toppings had the same proportion of ingredients within acceptable tolerances. They varied only in the type of system that was incorporated into the mixture to control cracking.

Table 2. Concrete type for bridge superstructures

Component	Slightly Aggressive Environment	Moderately Aggressive Environment	Extremely Aggressive Environment
Precast Superstructure and Prestressed Elements	Type I or Type II	Type I or Type III with Fly Ash or Slag, Type II, Type IP, Type IS, or Type IP(MS)	Type II with Fly Ash or Slag
C.I.P. Superstructure Slabs and Barriers	Type I	Type I with Fly Ash or Slag, Type II, Type IP, Type IS, or Type IP(MS)	Type II with Fly Ash or Slag

Table 3. FDOT structural concrete specifications

Class of Concrete	Specified Minimum Strength (28-day) (psi)	Target Slump (in)	Air content Range (%)
II (Bridge Deck)	4,500	3*	1 to 6
IV	5,500	3	1 to 6
*The engineer may allow higher target slump, not to exceed 7 in when a Type F or Type G admixture is used.			

Table 4. Master proportional limits

Class of Concrete	Minimum Total Cementitious Materials lbs/yd^3	*Maximum Water Cementitious Materials Ratio lb/lb
II (Bridge Deck)	611	0.44
IV	658	0.41
*The calculation of the water to cementitious materials ratio (w/cm) is based on the total cementitious material including silica fume, slag, fly ash, or Metakaolin.		

Four full-scale bridge decks were constructed to test the performance of the toppings. The Cow Creek Bridge was selected as a model for the design because it displays the type of crack patterns that this project is investigating and it has similarities in design with the other evaluated bridges and other existing flat slab bridges in Florida. A redesign of the bridge deck was conducted to ensure that the full-scale model conforms to the latest design codes. Each deck was approximately 12 ft wide and spanned 30 ft. The toppings were 6 in deep and exposed to similar environmental conditions as existing flat slab bridges in Florida.

A linear elastic finite element analysis was performed on a preliminary design to model drying shrinkage at 50% and 80% humidity. The concrete topping was divided into three sub-layers with an overall thickness of 6 in and the precast flat slab assumed to yield no shrinkage. Partial symmetry finite element models were used due to the plane symmetry of the geometry. The model was 8 ft. long and 4 ft. wide.

Three boundary conditions along edges of the slabs were considered. The first, Model A (Figure 11), imposed vertical and translational constraints on the bottom plane of the precast concrete deck while the second, Model B (Figure 12), only had vertical constraints. Model C (Figure 13) restrained translational movement and allowed vertical motion.

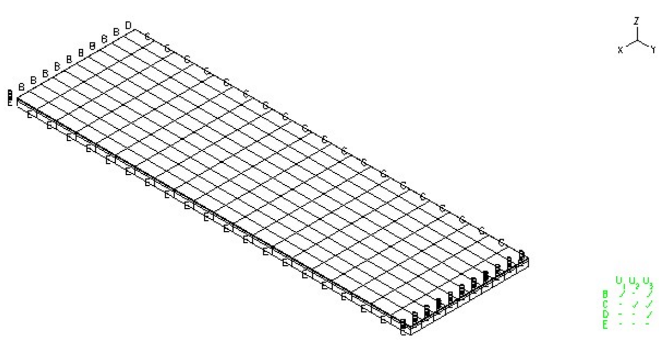

Figure 11. Finite element analysis Model A

As shown in Table 5, the first two models generated similar maximum principal stresses in the topping. However, the direction of the stresses was dependent on the boundary constraint imposed on the bottom of the precast flat slab. More severe tensile stresses were generated at the corners in the contact zone of the topping and the flat slab in Model C (Table 6).

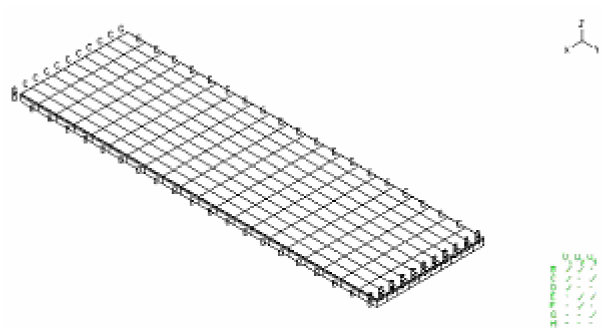

Figure 12. Finite element analysis Model B

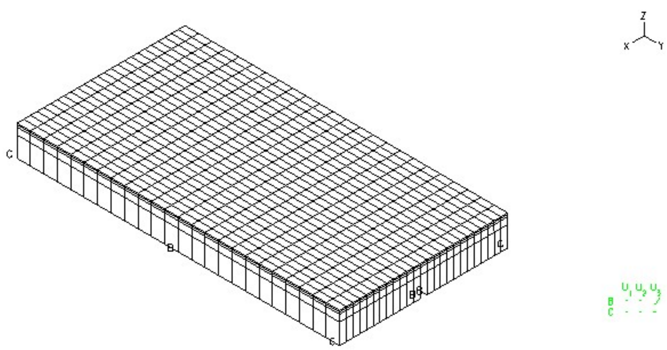

Figure 13. Finite element analysis Model C

Table 5. Maximum tensile stresses developed in topping

Model	Relative Humidity	Time (days)	Maximum Tensile Stress (psi)	Maximum Stress Component
A	80	10	351.0	σ_{xx}
		20	648.3	
		30	913.7	
A	50	5	556.9	σ_{xx}
		10	1054.4	
		30	2741.2	
B	80	10	337.9	σ_{yy}
		20	622.2	
		30	871.7	
B	50	5	536.6	σ_{yy}
		10	1013.8	
		30	2616.5	

Table 6. Maximum principal stress in model C

Model	Relative Humidity	Time (days)	Maximum Principal Tensile Stress (psi)
C	50	5	536.6
		10	967.4
		30	2278.5
C	80	10	310.4
		20	565.6
		30	760.0

Design and Fabrication

The flat slab analysis and design was done using LRFD Prestressed Beam Program v1.85 (Mathcad based computer program) developed by the FDOT Structures Design Office. The program analyzes prestressed concrete beams in accordance with the AASHTO LRFD Specification (2001) and the FDOT's Structures Manual (2004b). Input and output from the program are found in Appendix A.

Twelve full-scale precast slabs were constructed by Dura-Stress Inc., a Precast/Prestressed Concrete Institute (PCI) certified plant, in Leesburg, Florida. The slabs were similar in size and design to the Cow Creek slabs with a length of 30-ft. Unlike the Cow Creek Bridge, the flat slabs used to construct the test specimens did not have shear keys. The Texas DOT has had success with flat slab bridges without shear keys (Cook and Leinwohl 1997) and eliminating them would help reduce labor and construction costs. Each slab had twelve ½ in diameter lo-lax prestressing strands tensioned to 31 kips each. The two center strands were debonded 3 ft. from each end of the slab. The slabs were also reinforced with mild steel. Vertical shear reinforcement was provided every 12 in. U-shaped reinforcing bars, spaced at 12 in, provided horizontal shear reinforcement. Mild steel was also provided at each end of the slabs for confinement. All of the steel had a minimum concrete cover of 2 in. Reinforcement details are shown in Figure 14 and Figure 15. Complete reinforcement details are found in Appendix F. Figure 16 and Figure 17 show the constructed reinforcement system.

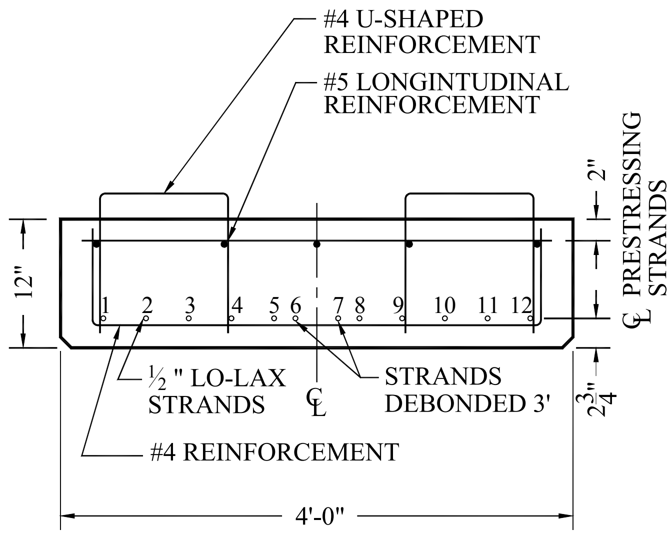

Figure 14. Typical cross-section through precast slab specimen

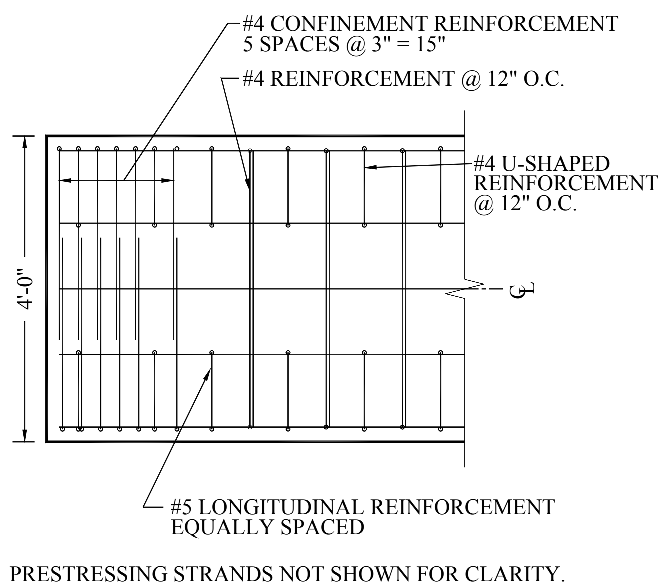

Figure 15. Reinforcement detail at end of slab

The concrete used for the slabs was a Class IV FDOT concrete mixture. The mixture design provided by the manufacturer is shown in Table 7. Based on the specifications found in Table 2, the concrete is intended for use in a mildly aggressive environment as defined by the FDOT's Standard Specification for Road and Bridge

Construction (2004a). It was batched onsite and delivered to the casting bed in trucks equipped with pumps to place the concrete.

Figure 16. Reinforcement at end of flat slab

Figure 17. Flat slab reinforcement layout

Table 7. Concrete mixture components for precast slabs

Material	Type	Amount per CY
Cement	AASHTO M-85 Type II	800 lbs
Mineral Admixture	NA	NA
Water	--	308 lbs
Aggregate	Sand 2	1150 lbs
Aggregate	#67 Granite 2	1750 lbs
Admixture	Air Entraining	0 oz
Admixture	Water Reducer	24 oz
Admixture	Superplasticizer	72 oz

The slabs were constructed in three groups of four as indicated in Table 8. The layout on the casting bed is shown in Figure 18. Steel plates and plywood were used as formwork for the slabs. A truck pumped the concrete onto the bed starting at slab No. 4 and moved towards slab No. 1 as the concrete was placed (Figure 19). Each truck transported approximately 5 cubic yards (CY) of concrete. One truck immediately continued placing concrete as the previous one finished. A total of three deliveries were needed to complete the casting of one group of slabs. The concrete was not screeded as it was placed. Personnel from the prestressing yard raked the concrete into place as it was pumped onto the casting bed. The surfaces were raked to ensure a rough finish to aid in horizontal shear transfer from the topping to the slab and a hoisting anchor was embedded into each corner of the precast slabs (Figure 20). Curing agents were not applied to the surface of the concrete.

Cylinders were taken to ensure adequate strength at release, document 28-day strength, and for possible future use. The cylinders collected for future use have yet to be tested. Additionally, plant quality control personnel collected five cylinders from each group to check the release and 28-day strength. The designed minimum release strength and 28-day strength were 4500 psi and 5500 psi respectively.

Table 8. Flat slab identification number and location

Designation	Casting Date & Time	Location on Casting Bed	1 Day Compressive Strength	Release Date & Time	28-Day Compressive Strength
FS1-1	5/5/2004 1:30PM	1	3873 psi	5/7/2004 ≈ 7:00AM	8963 psi
FS1-2		2			
FS1-3		3			
FS1-4		4			
FS2-1	5/11/2004 10:30AM	1	3403 psi	5/13/2004 ≈ 7:00AM	8403 psi
FS2-2		2			
FS2-3		3			
FS2-4		4			
FS3-1	5/14/2004 11:00AM	1	3685 psi	5/17/2004 ≈ 7:00AM	7975 psi
FS3-2		2			
FS3-3		3			
FS3-4		4			

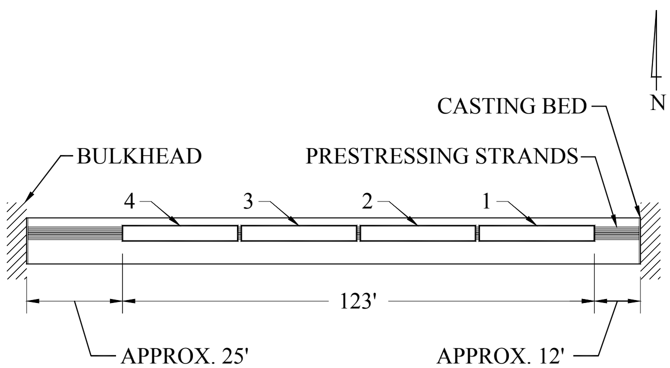

Figure 18. Typical slab layout on casting bed

Figure 19. Casting of flat slabs

Figure 20. Finished flat slab with hoisting anchors installed

Two cylinders were tested 24 hours after casting to determine the strength of the slabs. None of the slabs attained the minimum release strength within 24 hours. They remained on the casting bed for an additional day to allow the concrete to gain strength. It was assumed that the minimum release strength would be exceeded 48 hours after casting; therefore, additional cylinders were not tested to verify it. Twenty-eight day strength, transfer dates and times are shown in Table 8.

The precast slabs were stored at the prestressing yard for approximately six weeks while the test site was prepared. The slabs were stored in three stacks. Each stack contained four flat slabs. The slabs and the cylinders were exposed to the environment during this period.

Site Layout

Four single span flat slab bridge superstructures were constructed at the FDOT Maintenance Yard located at 2612 Springhill Rd. in Tallahassee, FL. Reinforced concrete supports for the flat slabs were constructed by the FDOT Structures Lab personnel to elevate the slabs to a convenient working height above the ground. The precast slabs were supported by neoprene bearing pads placed using a three-point system shown in Figure 21. This pattern was used on the Cow Creek Bridge and is currently used successfully by the Texas DOT (Cook & Leinwohl 1997). A view of the site before the placement of the precast slabs is shown in Figure 22. Each specimen consisted of three flat slab panels to ensure the possibility that at least one of the two joints would produce reflective cracks

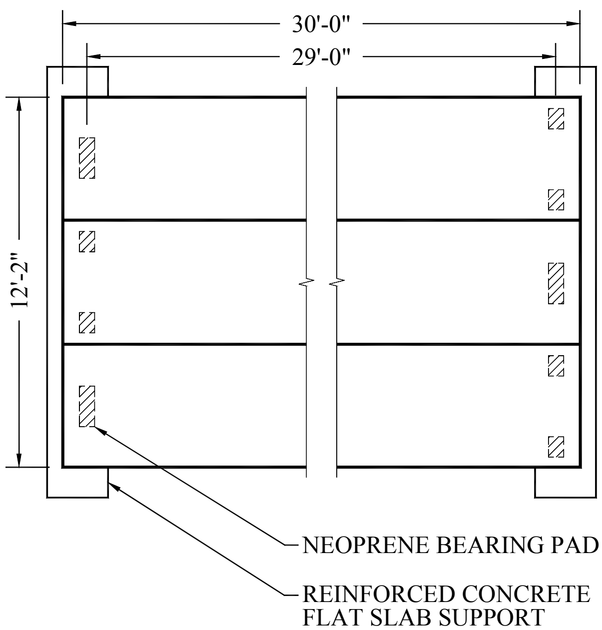

Figure 21. Typical bearing pad placement

Slab Placement

The flat slabs were delivered and placed on June 29, 2004. The panels were transported to the site on flat bed trailers. Each trailer carried two flat slabs. The first delivery was at 9:00 AM and approximately every half hour thereafter.

Figure 22. Concrete supports with neoprene bearing pads before placement of precast slabs

A crane was onsite to unload and place the flat slabs on the supports. The panels were unloaded and installed in the order that they arrived. Concrete cylinders that were cast along with the slabs were also brought to the site and placed near the precast slabs. Figure 23 shows an overview of the specimens and flat slab orientation that made them up. A single specimen was composed of three adjacent flat slabs with a 1 in gap between them. A 1-½ in diameter backer rod was installed between the panels near the surface of the precast slab to retain the fresh concrete (Figure 24).

Formwork was erected on the edges of each deck for the placement of the topping. It was composed of ¾ in plywood that had one side sealed to prevent moisture absorption from the concrete mixture (Figure 25). Once the formwork was erected the topping reinforcement was installed. The formwork was removed seven days after casting the toppings.

Topping Reinforcement

The size and spacing of the reinforcement was designed using the AASHTO LRFD Specification (2001) and the FDOT Structures Manual (2004b). No. 5 reinforcing bars were installed in the longitudinal and transverse directions spaced at 12 in on-center with 2 in of concrete cover. This spacing is the minimum reinforcement required for shrinkage and temperature control. The maximum allowable spacing was used to maximize the shrinkage tensile stresses in the concrete.

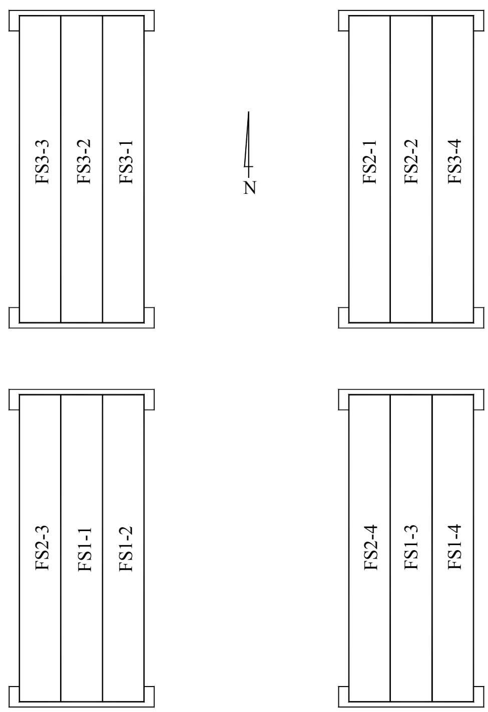

Figure 23. Slab site layout

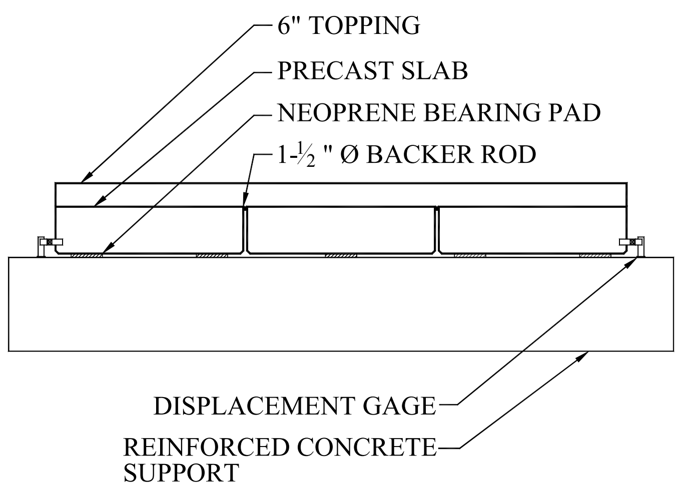

Figure 24. Typical superstructure end elevation view

Figure 25. Reinforcement and formwork on precast slabs before topping placement

The longitudinal reinforcement was placed first and tied to the flat slab's horizontal shear reinforcement with wire ties. The transverse reinforcement was then placed over it and tied (Figure 26).

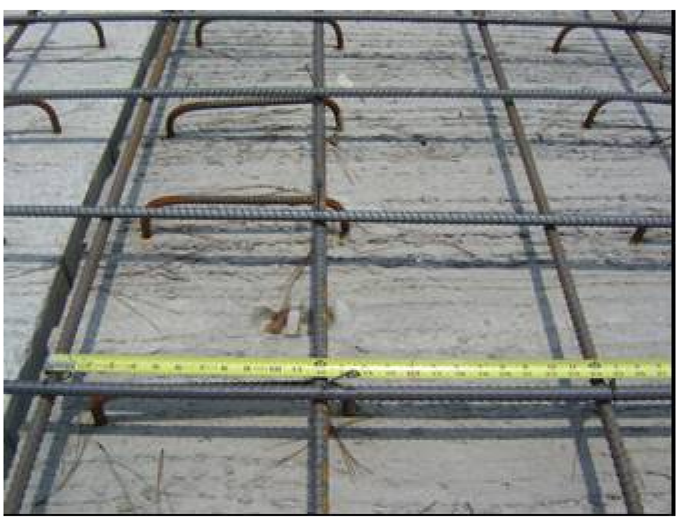

Figure 26. Topping reinforcement layout

Topping Placement

The toppings were cast daily during the week of July 26, 2004. Figure 27 shows the layout of the toppings with their respective designations shown in Table 9. Toppings that had a similar mixture were paired up to minimize shrinkage-cross-over effects over a span.

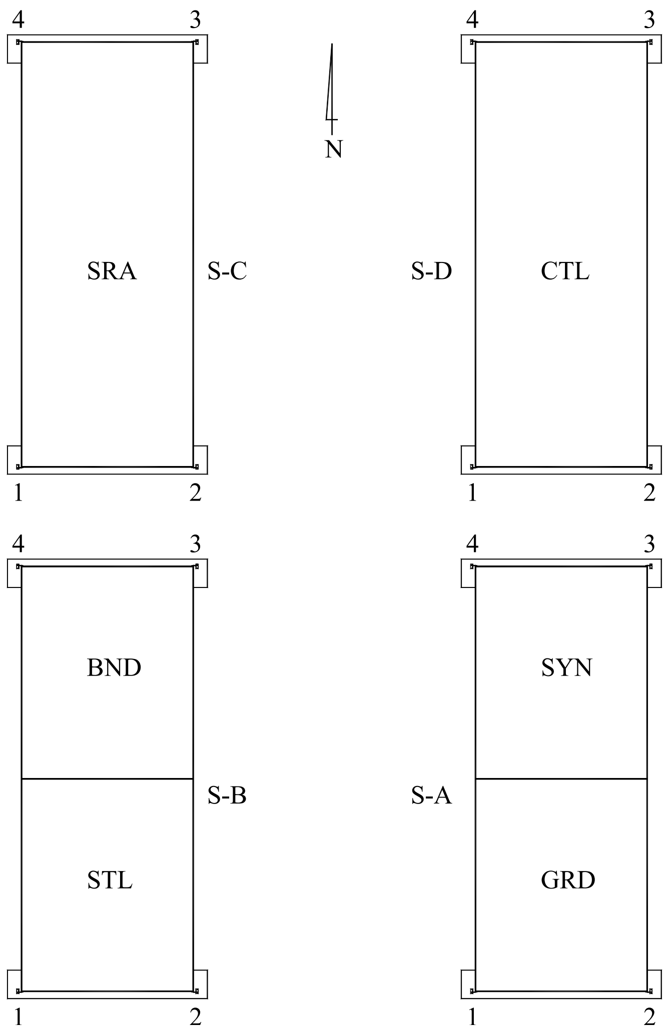

Figure 27. Displacement gage locations and superstructure and topping designation

Table 9. Specimen designation and topping treatment

Symbol	Topping Treatment
SYN	Synthetic fibers
BND	Blended fibers
GRD	Carbon fiber grid
STL	Steel fibers
SRA	Shrinkage reducing admixture
CTL	None

The STL and BND toppings were combined because each had steel fibers incorporated into their concrete mixtures. To ensure that the CTL topping was not affected by cross-over effects and that it remained valid as a basis for comparison it was cast on a single span. The SRA topping was also cast on a single span because of the lower overall shrinkage expected of this type of concrete. The remaining two toppings, GRD and SYN, were cast on a single span. Any toppings that shared a span were cast within 2 days of each other.

The toppings were exposed to direct sunlight from sunrise to sunset except for the CTL topping. A large tree located on the northeast corner of S-D (Figure 27) cast a large shadow on the topping until early afternoon. The CTL topping was purposefully located on S-D to see if it would develop cracks under the best curing conditions available at the site. Ideally, if the CTL topping cracked, the other toppings would have either cracked or restrained the formation of cracks.

Before the concrete placement, the surface was cleaned of debris with a blower and then wetted to prevent excessive water absorption from the fresh concrete topping. Front or rear discharge ready-mix trucks delivered the concrete to the site. Addition of water to the concrete mixes was performed by the concrete plant's personnel. Following the addition of the topping treatment the truck deposited the concrete directly onto the slabs. The concrete was leveled with a vibratory screed and finished with a 3 ft bull float. A curing compound was sprayed on the surface after the bleed water, if any, had evaporated. The compound was manufactured by W.R. Meadows and met the standards of the FDOT Standard Specification for Road and Bridge Construction (2004a).

The fresh concrete was tested for air content and slump in accordance with ASTM C173 and ASTM C143, respectively. The initial slump was measured upon delivery and after the addition of water and/or crack control system. The air content was measured after all modifications were made to the delivered mix.

Twenty-seven cylinders were cast for each topping in accordance with ASTM C31. Lids were place on the cylinders after collection and removed the following day. The cylinders remained in their molds and allowed to cure on their respective topping until they were tested. Tests were conducted for compressive and tensile strength as well as for modulus of elasticity at the ages shown in Table 10.

Table 10. Cylinder test schedule

Cylinder Age (days)	Pressure Tension Test	Compressive Test ASTM C39	Elastic Modulus ASTM C469
3	yes	NA	NA
7	yes	yes	NA
28	yes	yes	yes
56	yes	yes	yes

Tensile strength was measured using the pressure tension test (Figure 28). The equipment consisted of a cylindrical chamber for pressurizing the specimen, nitrogen filled tank, collars for the ends of the specimen, and a computer that records data supplied by a pressure transducer. This procedure required the operator to open a valve by hand to apply pressure to a 4 in by 8 in concrete cylinder for each test. The load rate was determined by watching a monitor that plotted a load versus time line, which should be in the range of 35 psi/sec. Li (2004) details the test equipment and procedure.

Workability of the fresh mixture was ranked by the author from 1 to 4 according to the scale outlined in Table 11. The rankings were subjective, based on visual and physical observations as well as feedback from personnel casting the topping.

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Figure 28. Pressure tension testing equipment (Li 2004)

Table 11. Workability ranking scale

Rank	Workability
1	Very good
2	Good
3	Poor
4	Very poor

Very good workability is defined as a mixture that easily flowed down the chute and consolidated around reinforcement with little to no vibration. A mixture with good workability flowed down the chute and consolidated around the reinforcement with some vibration. If the mixture flowed down the chute with aid and consolidated around reinforcement with vibration it was classified as having poor workability. A mixture with very poor workability required physical effort to aid it down the chute and required excessive vibration to consolidate it.

Synthetic fiber (SYN)

Polypropylene\polyethylene monofilament fibers (Figure 29) were used in the SYN topping at a dosage rate of 6 lbs/CY. The material properties provided by the fiber's manufacturer are given in Table 12 and the concrete mixture's constituents are shown in Table 13.

Figure 29. Synthetic fibers used in SYN topping

Table 12. Material properties for fibers used in SYN topping.

Specific Gravity	0.92
Absorption	None
Modulus of Elasticity	1,378 ksi
Tensile Strength	90 ksi
Melting Point	320°F
Ignition Point	1,094°F
Alkali, Acid and Salt Resistance	High

Twenty-four pounds of fibers were fed into the mixing drum over a period of 4 min. They were dispersed manually to prevent balling and allowed to mix for 70 revolutions of the drum as per manufacturer's recommendations. Even after mixing, however, some of the fibers were entangled and not fully coated with cement paste. Seven gallons of water was added to the mixture after a slump test measured 1¾ in. This volume of water was based on the delivery ticket, which subsequently was discovered to have been incorrect.

Table 13. Mixture proportions for SYN topping

Material	Design Qty.	*Required	Batched	Difference	Difference (%)	Moisture (%)
#57 Stone (lbs)	1640	6685	6620	-65	-0.97	1.90
Sand (lbs)	1324	5460	5430	-30	-0.55	3.10
Cement (lbs)	495	1980	1965	-15	-0.76	NA
Fly Ash (lbs)	120	480	345	-135	-28.13	NA
Air (oz)	1.8	7.2	7	-0.20	-2.78	NA
WR (oz)	33.8	135.2	135	-0.20	-0.15	NA
Water (gal)	25	65.58	65	-0.58	-0.89	NA
*Amount required for 4 CY. Quantities provided by ready-mix plant.						

Consequently, the actual w/c ratio was 0.38, which was significantly lower than the target value. At the time of casting, the mixture had a slump of 3¼ in and an air content of 2.5%.

The workability of the SYN mixture was less than ideal. The fresh concrete did not flow down the chute and required excessive raking and vibrating during placement. Low w/c ratio, low air content, and incorrect amount of fly ash and cement contributed to poor workability. Following screeding, only a light sheen formed on the surface with no bleed water or bleed channels visible.

Blended fiber (BND)

The BND topping was a blended fiber concrete mixture composed of synthetic (Figure 30) and steel fibers (Figure 31). The synthetic fibers were ¾ in long multifilament nylon fibers while the steel fibers were 2 in long with a crimped profile.

Table 14 andTable 15 outline the material properties of the synthetic and steel fibers provided by the manufacturer. Synthetic and steel fibers were used at a dosage rate of 1 lb/CY and 25 lbs/CY respectively. Table 16 shows the batched quantities of the ingredients in the BND mixture.

Synthetic fibers were incorporated into the mixture first so that the steel fibers would help disperse them in the mixture. A slump test, run after the drum revolved 70 times, measured 3¾ in. Eight gallons of water were added to the mixture to increase the workability and the w/c ratio. The concrete mixture had a final w/c ratio of 0.44, air content of 3.5%, and slump of 4¾ in.

The mixture flowed down the chute without any agitation and had good workability. It was easily screeded and finished. Bleed water or bleed channels were not visible on the surface of the topping.

Table 14. Properties for synthetic micro fibers

Specific Gravity	1.16
Absorption	4.5%
Modulus of Elasticity	750 ksi
Tensile Strength	130 ksi
Melting Point	435°F
Ignition Point	1,094°F
Alkali and Acid Resistance	High
Filament Diameter	23 microns
Fiber Length	0.75 in

Table 15. Properties for steel fibers used in BND and STL toppings

Specific Gravity	7.86
Absorption	None
Modulus of Elasticity	29,000 ksi
Tensile Strength	Minimum 100 ksi
Melting Point	2,760°F
Fiber Length	2 in
Equivalent Diameter	0.035 in
Aspect Ratio	57

Figure 30. Synthetic fibers used in BND topping

Figure 31. Steel fibers used in BND and STL toppings

Table 16. Mixture proportions for BND topping

Material	Design Qty.	*Required	Batched	Difference	Difference (%)	Moisture (%)
#57 Stone (lbs)	1640	6672	6700	28	0.42	1.70
Sand (lbs)	1324	5455	5420	-35	-0.64	3.00
Cement (lbs)	495	1980	1985	5	0.25	NA
Fly Ash (lbs)	120	480	445	-35	-7.29	NA
Air (oz)	1.8	7.2	7	-0.20	-2.78	NA
WR (oz)	33.8	135.20	135	-0.20	-0.15	NA
Water (gal)	31	88.60	89	0.40	0.45	NA
*Amount required for 4 CY. Quantities provided by ready-mix plant.						

Carbon-fiber grid (GRD)

A 1.6 in by 1.8 in carbon-fiber grid (Figure 32) was embedded in the GRD topping (Figure 33) to provide crack control near the surface of the topping. Results from tensile tests performed on grid specimens are shown in Table 17. The material properties supplied by the manufacturer are listed in Table 18. The grid was placed one inch below the surface of the topping to prevent spalling or delamination. This positioned it below the minimum ½ in wearing surface required by the FDOT Structures Manual (2004b). The concrete was screeded at the embedment depth to provide a level surface for the placement of the grid. A float was used to fully coat the grid with concrete paste. The topping placement was then completed with a 1 in layer of concrete placed over the grid. Bleed water was clearly visible on the surface of the topping as it cured.

An initial slump of 4¾ in was measured before any water was added to the mixture. Five gallons of water were added to increase the w/c ratio to 0.40, which brought the slump to 6¾ in.

Figure 32. Carbon-fiber grid used in GRD topping

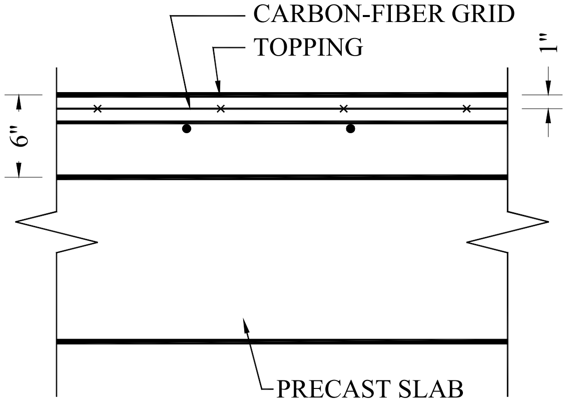


Figure 33. GRD topping grid location cross-section

Table 17. Carbon-fiber strand strength.

Specimen	Fiber Direction	Strength (ksi)	Tensile Modulus (ksi)
*1	Vertical	68.5	7665
2	Vertical	126.2	8549
3	Hoop	98	9671
4	Hoop	110.8	11516
*Specimen had a thick epoxy layer that increased the cross-sectional area used to determine strength therefore underestimating strength.			

It could not be increased any further because the mixture would have become too fluid and possibly segregated. Table 19 shows the batched constituents that make up the GRD concrete mixture. At the time of casting, the concrete had a slump of 6¾ in and 3% air content. The fresh concrete had good workability and flowed easily into place.

Table 18. Physical properties for carbon-fiber grid

Fiber Type	Carbon
Grid Spacing (in)	1.6 x 1.8
% of Grid Openness	69
Nominal Tensile (lbs/strand: warp x fill)	1000 x 1000
Nominal Tensile (lbs/foot)	6,650 x 7,500
Crossover Shear Strength (lbs)	40
Resin Type	Epoxy
Fabric Weight (oz/SY)	11

Table 19. Mixture proportions for GRD topping

Material	Design Qty.	*Required	Batched	Difference	Difference (%)	Moisture (%)
#57 Stone (lbs)	1640	6678	6760	82	1.23	1.80
Sand (lbs)	1324	5455	5410	-45	-0.82	3.00
Cement (lbs)	495	1980	2005	25	1.26	NA
Fly Ash (lbs)	120	480	465	-15	-3.13	NA
Air (oz)	1.8	7.2	7.0	-0.20	-2.78	NA
WR (oz)	34	136	136	0.00	0.00	NA
Water (Gal)	31	80.81	81	0.19	0.24	NA
*Amount required for 4 CY. Quantities provided by ready-mix plant.						

Steel fiber (STL)

The STL and BND toppings contained the same type of steel fibers. Their properties are listed in Table 15 and batched quantities are shown in Table 20. A dosage rate of 60 lbs/CY was used in order to provide a high fiber count per CY and better performance comparison with the SYN and BND toppings. Unlike the previous toppings, water was added to the mixture before the fibers. Sixteen gallons of water were added to the mixture to overcome the decrease in workability and slump caused by the fibers. The fibers were separated as they were deposited into the mixing drum to prevent balling within the mixture. Unlike any of the other toppings, heat generated by the hydration of the cement was felt as it was mixed. It was believed that an incorrect amount of water was added after seeing the consistency of the mixture. The concrete was extremely stiff and did not flow down the chute or consolidate around the reinforcement and formwork. Eight gallons of water was added but the concrete was still not workable. No more water was added because the concrete was already at a w/c ratio of 0.44.

The workability of the STL mixture was poorer than the BND mixture. Like the BND topping, the concrete did not flow down the chute and needed to be raked and vibrated into place. It was extremely difficult to screed and level off the concrete. The poor workability was attributed to an incorrect water dosage and low air content. A high range water reducer could be added to help reduce friction within the mixture thereby improving workability. No bleed water was visible on the surface of the topping.

Table 20. Mixture proportions for STL topping

Material	Design Qty.	Required	Batched	Difference	Difference (%)	Moisture (%)
#57 Stone (lbs)	1640	6678	6670	-8	-0.12	1.80
Sand (lbs)	1324	5455	5430	-25	-0.46	3.00
Cement (lbs)	495	1980	2110	130	6.57	NA
Fly Ash (lbs)	120	480	465	-15	-3.13	NA
Air (oz)	1.8	7.2	7.0	-0.20	-2.78	NA
WR (oz)	34	136	136	0.00	0.00	NA
Water (gal)	31	80.81	80	-0.81	-1.00	NA
*Amount required for 4 CY. Quantities provided by ready-mix plant.						

Shrinkage reducing admixture (SRA)

A shrinkage reducing admixture (SRA) was added to a concrete mixture at a recommended dosage rate of 1-$^{7}/_{8}$ gal/CY. Table 21 shows the batched materials for the SRA topping. Slump tests conducted before and after dosing indicated that the SRA did not affect the slump. Twenty gallons of water were added to increase the w/c ratio to a level comparable to the other toppings. The mixture easily flowed down the chute and around the reinforcement. It had very good workability and was screeded and finished without any difficulty.

Table 21. Mixture proportions for SRA topping

Material	Design Qty.	*Required	Batched	Difference	Difference (%)	Moisture (%)
#57 Stone (lbs)	1640	13356	13330	-26	-0.19	1.80
Sand (lbs)	1324	10910	10810	-100	-0.92	3.00
Cement (lbs)	495	3960	4030	70	1.77	NA
Fly Ash (lbs)	120	960	930	-30	-3.13	NA
Air (oz)	1.8	14.4	14	-0.40	-2.78	NA
WR (oz)	33.8	270.4	270	-0.40	-0.15	NA
Water (gal)	31	145.62	145	-0.62	-0.43	NA
*Amount required for 8 CY. Quantities provided by ready-mix plant.						

Control topping (CTL)

The same concrete mixture that was used for the GRD topping was ordered for the CTL topping (Table 22). Like the SRA topping, 20 gallons of water were added to increase the w/c ratio. The final mixture had very good workability and easily flowed around the reinforcement. Bleed channels were clearly visible on the topping as the bleed water surfaced and ran off the sides of the topping. This topping produced the most bleed water.

Table 22. Mixture proportions for CTL topping

Material	Design Qty.	*Required	Batched	Difference	Difference (%)	Moisture (%)
#57 Stone (lbs)	1640	13774	13670	-104	-0.76	1.80
Sand (lbs)	1324	11251	11150	-101	-0.90	3.00
Cement (lbs)	495	4083.8	4045	-38.8	-0.95	NA
Fly Ash (lbs)	120	990	940	-50	-5.05	NA
Air (oz)	1.8	14.85	15	0.15	1.01	NA
WR (oz)	33.8	278.85	279	0.15	0.05	NA
Water (gal)	31	167.30	167	-0.30	-0.18	NA
*Amount required for 8¼ CY. Quantities provided by ready-mix plant.						

Summary

While these topping treatments can easily be incorporated into a concrete mixture, the variability in workability between the topping treatments needs to be addressed. As Table 23 shows, there was a correlation between the workability rating and the slump. The mixtures that received a poor or very poor rating had slumps less than 3¼ in and low air contents when compared to the 6% allowed by the FDOT Standard Specifications for Road and Bridge Construction (2004a) (Table 3). The effect of the air content is more pronounced in the poorly rated mixtures because of the friction caused by the presence of fibers. Higher air contents would provide more air bubbles that act like ball bearings for the fibers to slide against which would reduce friction within the fresh concrete mixture. The workability of the SYN topping was also affected by the 28% shortage of fly ash in the mixture (Table 13). This shortage prevented the fibers from being fully coated with cement paste after initial mixing thus degrading its workability. Its workability was partially improved by adding water to the mixture to ensure that the fibers were coated but it could have been further improved by adding enough water to increase the w/c ratio to 0.44. Some of the workability issues in the STL topping may be attributed to an incorrect water dosage. This was based on observing the mixture during slump test No. 3. The workability of the concrete would have improved after adding 24 gal of water. The workability of the poorly rated mixtures could have been improved by increasing the amount of air-entraining admixture, water-reducing admixture or adding a high-range-water-reducing admixture.

Table 23. Workability rating/slump relationship

Topping	Workability Rating	Slump (in)
SYN	3	3¼
BND	2	4¾
GRD	1	6¾
STL	4	2
SRA	1	5
CTL	1	5

A summary of the test results and tasks completed with each topping is outlined in Table 24. The air content of all the toppings was low given that the FDOT allows up to 6%. Table 25 documents a timeline for tasks completed on each topping. The batched and cast w/c ratios of the concrete mixtures are shown in Table 26.

Table 24. Concrete mixture summary

Topping	Slump Test #1 (in)	Admixture (Gal)	Fiber Amount (lbs/CY)	Slump Test #2 (in)	Additional Water (gal)	Slump Test #3 (in)	Air Content (%)
SYN	4½	NA	6	1 ¾	7	3¼	2.5
BND	2¾	NA	1 micro 25 steel	3 ¾	8	4¾	3.5
GRD	4¾	NA	NA	NA	5	6¾	3
STL	2	NA	60	NA	24	2	2
SRA	1¾	15	NA	2	20	5	1.5
CTL	2¾	NA	NA	NA	20	5	1

Table 25. Timeline from batching to casting

Topping	Delivery	Batch Start	Plant Departure	Arrival Time	Casting Start
SYN	July 26th	8:47AM	8:57AM	9:10AM	9:45AM
BND	July 27th	8:42AM	8:50AM	9:07AM	9:35AM
GRD	July 28th	8:45AM	8:57AM	9:07AM	9:22AM
STL	July 28th	9:56AM	10:15AM	10:26AM	10:58AM
SRA	July 29th	8:32AM	8:49AM	9:05AM	9:35AM
CTL	July 30th	8:30AM	8:50AM	9:02AM	9:20AM

Table 26. Concrete mixture w/c ratios

Topping	Batched w/c Ratio	Jobsite w/c Ratio
SYN	0.36	0.38
BND	0.42	0.44
GRD	0.39	0.40
STL	0.37	0.44
SRA	0.35	0.39
CTL	0.39	0.43

As Figure 34 shows, workability issues with the STL and SYN mixtures affected the finishing time of the toppings. Toppings with fiber treatments took the longest to complete. Screeding of the toppings commenced once casting was approximately half completed except on the BND topping which started immediately after it was cast. More time was spent screeding the GRD topping because it was performed twice, once to level the surface for placement of the grid, and a second time to level off the concrete. The time it took to install the grid includes the screeding time yet it was completed faster than the others because of good workability of the mixture. Timeline data for the SRA and CTL toppings were not listed for comparison because they were twice the size of the documented toppings.

Though the most expensive of the topping treatments tested, the SRA required the least amount of effort to incorporate into the mixture. The SRA was packaged in 5 gal pails that were easily poured into the mixing drum.

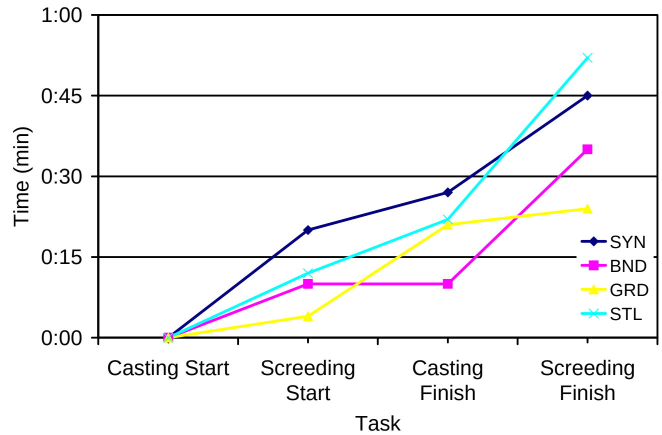


Figure 34. Normalized timeline for construction of the half-span toppings

This treatment should have minimal impact on the labor cost as it only took an additional 10 min. to incorporate and mix into the concrete. Some ready-mix plants will deliver a concrete mixture with SRA. No shrinkage-reducing admixtures are currently on the FDOT's qualified products list and will need to be approved before they can be used in the field.

The fiber treatments were the least expensive measure tested to control cracking. They are available from numerous manufacturers in a variety of materials and lengths, and due to their popularity, fiber reinforced mixtures can be ordered from ready-mix plants. If fibers are added at the job site, they should be scattered by hand as they are placed in the mixing drum to prevent balling. Mixtures with higher fiber volumes such as those used for the SYN and STL toppings should incorporate a high-range-water-reducer to improve the workability. This will reduce the risk of an excessive amount of water added to the mixture at the job site.

Carbon-fiber grids are not as commonly available as the other methods that were tested and, if not planned for ahead of time, projects may experience delays because they must be obtained from a specialty supplier. Constructing a GRD topping in the field requires more time to implement than the other treatment methods due to the double screeding of the topping. Quality control plays a larger role with this system because the grid must be installed at the specified depth to be effective. If it is placed too deep in the topping it will not provide its maximum reinforcement potential. An advantage of this system is that no modifications need to be made to current FDOT approved mixtures and it allows the designer to specify where the crack control system should be installed.

Instrumentation

The bridge decks were instrumented to monitor temperature gradients through the depth of the toppings and displacements at the corners. The temperature was monitored at three locations in the toppings during the placement of the concrete. Displacement gages were installed at the corners of the bridge deck to measure movement due to curling or thermal changes.

Type K thermocouples were installed at three locations in each topping (Figure 35). Each monitoring location consisted of three thermocouples distributed in the vertical plane through the depth of the topping (Figure 36). Each set of thermocouples was tied to a 5 in long No. 3 reinforcing bar to keep them in place while the concrete was placed. The No. 3 bar was tied to the topping reinforcement or the flat slab's horizontal shear reinforcement. The wires ran along the top of the flat slab to the nearest joint. They were fed past the backer rod and ran towards the side of the specimen. All the wires for a given topping were tied together and labeled with the location that was being monitored. Male type K plugs were installed at the ends of the wires.

Nine locations were monitored for each topping (Figure 37). Two four channel data loggers (eight total channels) were used to record the temperature data. One of the channels was used to monitor the temperature at two locations.

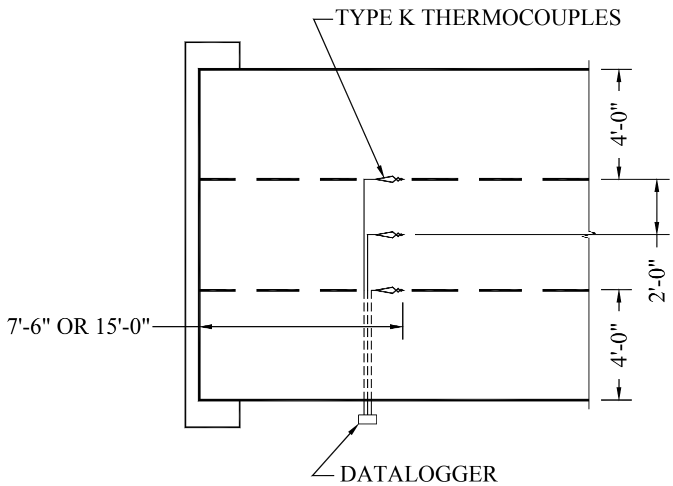


Figure 35. Partial plan view of specimens with typical thermocouple layout

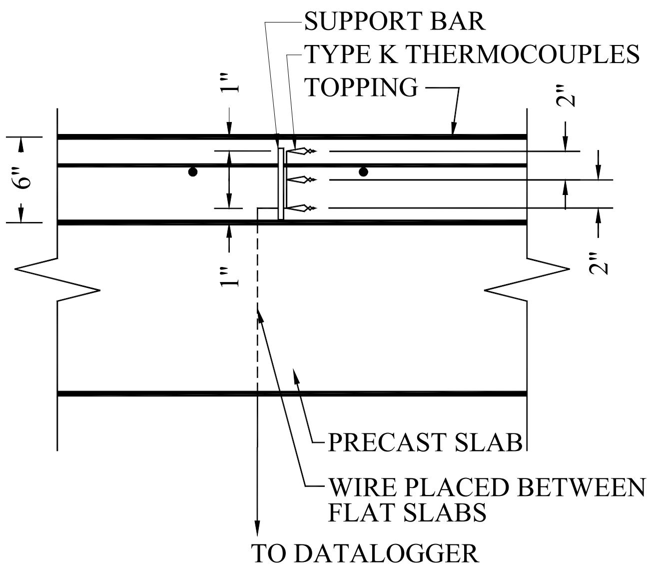


Figure 36. Partial section view of specimen with typical thermocouple profile layout

The plugs were alternated on this channel approximately every half hour. The time and wire label was documented every time they were alternated. The data loggers were not left on-site overnight due to security concerns therefore temperature data was collected for approximately 8 to 10 hours on the days of the topping placement. Since the CTL and GRD toppings are the same FDOT approved mixture, temperature data was only collected for the CTL topping.

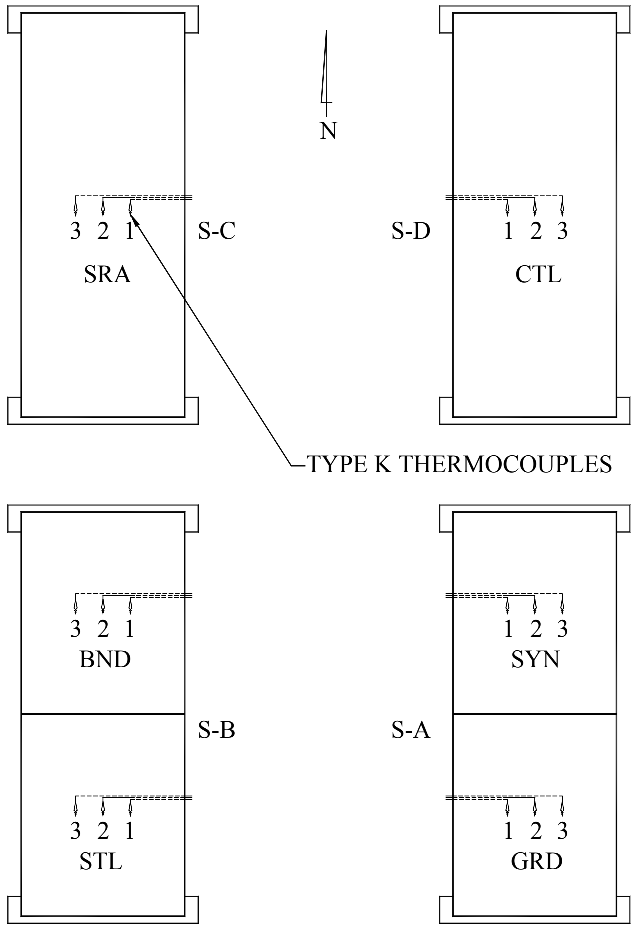


Figure 37. Monitored locations for each topping

Displacement gages were installed at the corners of the bridge decks to monitor vertical or in-plane movement (Figure 27). They were manufactured by Preservation Resource Group, Inc. and had a measurement range of 0.79 in in the vertical direction and 1.57 in in-plane. As shown in Figure 38, steel brackets were used to mount the gages to the superstructure support. The opposite end of the gage was attached to the flat slab with screws (Figure 39).

Figure 38. Displacement gage attachment bracket

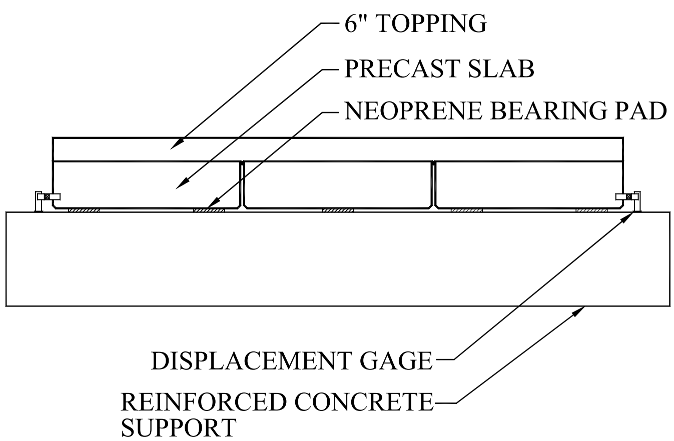


Figure 39. Profile view of displacement gage placement at span end

Restrained Shrinkage Rings

A restrained shrinkage ring test was performed on all of the toppings. The test was used to compare the time to cracking and the number and size of cracks between the concrete mixtures used for the toppings. The test was modeled after a ring test used to measure the cracking potential of concrete and mortar (See, Attiogbe, and Miltenberger

2003). The dimensions of the apparatus were similar but, unlike the test it was modeled after, strain gages were not used and the tests were conducted outdoors, exposed to changing temperature and humidity levels (Figure 40 & Figure 41). A concrete ring was cast for each of the toppings and the top of the ring was sealed with a curing compound to induce drying from the outer surfaces only. The formwork was removed from the ring after 24 h. They were measured weekly for two months and biweekly thereafter with a shop microscope.

The ring with the GRD mixture was the only one that did not incorporate its respective crack control treatment. Hence, the results do not take into account the performance of the carbon-fiber grid.

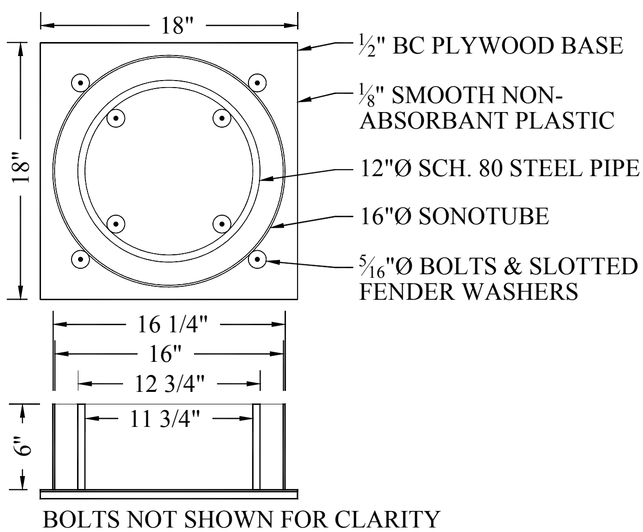


Figure 40. Restrained shrinkage ring

Figure 41. Typical restrained ring specimen

CHAPTER 5
RESULTS AND DISCUSSION

Compressive Strength and Modulus of Elasticity

Cylinder tests were conducted at 3, 28, and 56 days for compressive strength and at 28 and 56 days for modulus of elasticity in accordance with ASTM C39 and ASTM C469, respectively. Results are based on an average of three tests.

Table 27 shows the results of the compressive strength for each of the toppings. The CTL topping had a 28-day compressive strength of 6156 psi, well above the 4500 psi design strength. The STL topping had the highest compressive strength of all the toppings due to the presence of steel fibers and an over-dosage of cement (Table 20). However, steel fibers in the BND mixture did not correlate with an increase in strength. The lower overall strength of the SYN topping may be attributed to an under-dosage of fly ash and cement in the mixture (Table 13). Low w/c ratios did not indicate a higher strength concrete.

Table 27. Compressive strength of concrete cylinders

Topping	3-Day (psi)	28-Day (psi)	56-Day (psi)	w/c ratio
SYN	3614	5756	6376	0.38
BND	2769	6004	6572	0.44
GRD	3128	6501	7068	0.40
STL	4021	7123	8141	0.44
SRA	3129	6290	6488	0.39
CTL	2923	6156	7061	0.43

The modulus of elasticity results are shown in Table 28. Different testing equipment was used to conduct 28 and 56-day modulus and may account for the slight decrease in modulus within some of the toppings. Results indicate that the treatments had a minimal effect on the modulus of elasticity.

Table 28. Modulus of elasticity of concrete cylinders

Topping	28-Day Modulus (ksi)	56-Day Modulus (ksi)
SYN	4219.6	4263.0
BND	4331.3	4208.6
GRD	4328.4	4371.3
STL	4696.5	4403.0
SRA	4636.6	4264.4
CTL	4442.9	4204.9

Pressure Tension Test

The concrete tensile strength was measured using the pressure tension test. Results were based on an average of three tests and are shown in Figure 42 and Table 29. Unexpectedly, the tensile strengths of the specimens were found to decrease over time. The decrease was attributed to the variability inherent in the system because it was difficult to maintain the same load rate for each specimen, and throughout a test. The load rates were analyzed and their coefficients of variation (COV) are presented in Figure 43. As more tests were conducted, the COV of the load rates decreased. The COV within each test, made up of three specimens, was calculated and found not to be largely affected by the variability in the load rate (Figure 44). Based on the results of the 56 day test, the treatments had a minimal effect on the tensile strength of the concrete.

Table 29. Tensile strength of concrete cylinders using pressure tension test

Topping	3-Day (psi)	7-Day (psi)	28-Day (psi)	56-Day (psi)
SYN	656	659	839	667
BND	744	738	526	604
GRD	705	702	570	649
STL	752	613	607	691
SRA	806	794	563	655
CTL	657	728	638	658

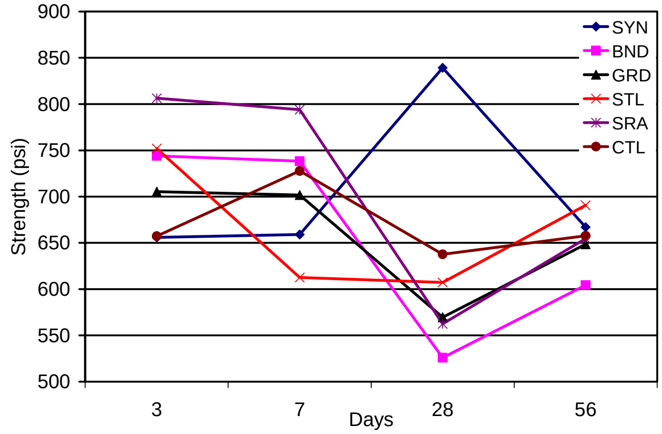


Figure 42. Tensile strength using pressure tension test

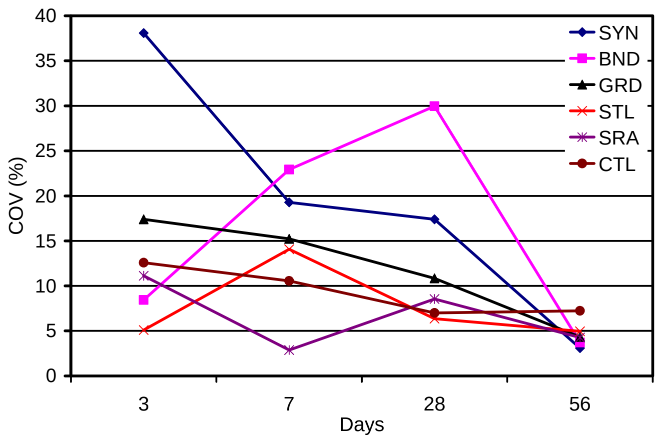


Figure 43. Coefficient of variation for load rate using pressure tension test

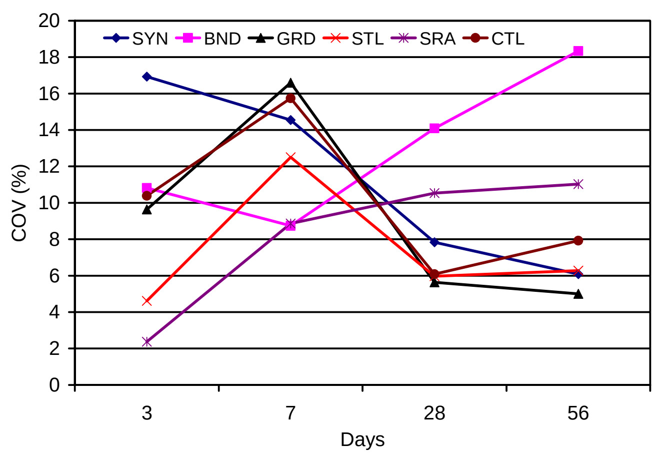


Figure 44. Coefficient of variation for tensile strength using pressure tension test

Restrained Ring Test

Cracks were first observed on the SYN, BND, GRD, and STL rings approximately 60 days after casting. Though microcracks may have been present, cracks became visible after the humidity levels remained below 70% for an eight day period (Figure 45). The BND and GRD rings had two cracks, one across from the other, while the SYN and STL rings had one. No cracks were observed on the concrete toppings. Approximately 40 days later, cracks were observed on the SRA and CTL rings, after the humidity level went below 70%. Again, no cracks were observed on the toppings. The variability in the humidity and temperature at the site contributed to the long time to cracking of the rings when compared to research that shows cracking at much earlier ages when the rings are kept in a controlled environment (Grzybowski and Shah 1990; Shah, Karaguler, Sarigaphuti 1992).

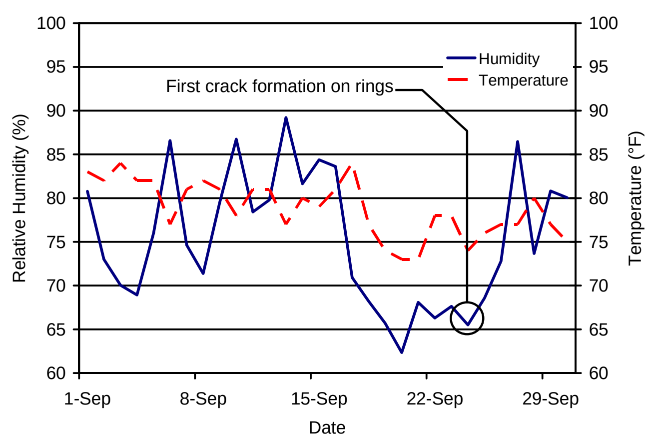


Figure 45. Humidity and temperature for Sept. 2004

Average crack widths are presented in Table 30 and Table 31. Crack widths on the STL ring were smaller than the other rings and consistent with previous research (Grzybowski and Shah 1990). Their research showed decreasing average crack widths with increasing fiber volume. This was confirmed in comparing the performance of the

STL and BND rings. Ignoring the presence of synthetic micro fibers in the BND ring, the STL ring, with the higher fiber volume, performed better in reducing crack width.

Table 30. Average crack width for GRD, SRA, and CTL rings

Approx. Days After Casting	GRD (in)		SRA (in)		CTL (in)
	No. 1	No. 2	No. 1	No. 2	No. 1
57	0.004	0.003	NA	NA	NA
64	0.004	0.003	NA	NA	NA
83	0.004	0.003	NA	NA	NA
99	0.004	0.004	0.002	0.002	0.008
113	0.008	0.005	0.002	0.002	0.008
127	0.008	0.005	0.002	0.002	0.008
141	0.01	0.006	0.002	0.002	0.028
160	0.01	0.006	0.002	0.002	0.028
169	0.01	0.006	0.002	0.002	0.028

Table 31. Average crack width for SYN, BND, and STL rings

Approx. Days After Casting	SYN(in)	BND (in)		STL (in)	
	No. 1	No. 1	No. 2	No. 1	No. 2
57	0.004	0.001	0.001	0.001	NA
64	0.004	0.001	0.001	0.001	NA
83	0.005	0.002	0.001	0.001	NA
99	0.006	0.004	0.002	0.001	0.001
113	0.006	0.004	0.002	0.001	0.001
127	0.006	0.004	0.002	0.001	0.001
141	0.007	0.004	0.002	0.001	0.001
160	0.007	0.005	0.002	0.001	0.001
169	0.007	0.005	0.002	0.001	0.001

Crack widths on the SRA ring were significantly smaller than those on the untreated mixtures. The rings with the two unmodified mixtures, CTL and GRD, had the widest cracks of all the rings. The GRD ring unexpectedly developed a second crack opposite of the first one possibly due to restraint at the concrete/steel interface. As previously stated, the results of the GRD ring do not take into account the effectiveness of the carbon-fiber grid.

Thermocouple Data

Temperature data measured through each topping's depth at the time of casting is presented in Appendix F. While most of the toppings had a temperature difference of approximately 5°F, a 13.2°F temperature gradient was measured approximately five hours after casting in the SRA topping (Figure 46) at location 3. This may promote the formation of internal micro cracks in hot weather concreting.

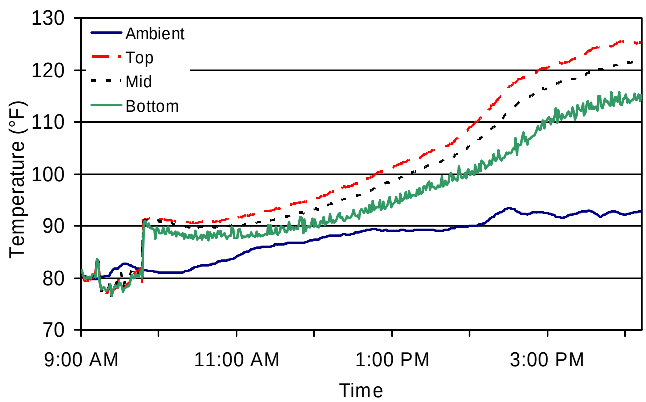


Figure 46. Temperature data through depth of topping for SRA-3

Topping Observations

After 30 weeks of observation, no cracks in the topping, over the flat slab joints, were visible. Several factors inherent in the design and construction may have prevented the formation of cracks.

The FDOT's Standard Specification for Road and Bridge Construction (2004a) was strictly adhered to. All of the concrete mixtures were at or below the maximum 0.44 w/c ratio and were within tolerances allowed for air content and slump. Reinforcement in the toppings was also installed with 2 in of cover as outlined in the FDOT's Structures Manual (2004b). These factors provided a bridge deck that was in compliance with current FDOT standards.

Use of a curing compound may have aided in the prevention of cracks. An FDOT approved compound was sprayed on the topping after the bleed water, if any, had evaporated. It sealed the surface and prevented water from evaporating out of the topping in the first few weeks after casting which is when the majority of drying shrinkage occurs.

Finally, the restraint of the specimens may not have matched the restraint provided on existing flat slab bridges. For cracks to develop, the system must be restrained to induce internal tensile stresses in the concrete as it tries to shrink. The bearing pads may not have provided adequate restraint for the bridge deck. The neoprene pads were 1½ in thick whereas those used on the Cow Creek Bridge measured 1 in thick. The pads may have undergone a shear deformation to accommodate the shrinking topping. The displacements would be too small measure with the gages. They also showed no signs of lifting or curling at the corners (Figure 47-Figure 50). The readings provide clues that show the system either acted in an unrestrained manner or insufficient strain was generated in the topping. Furthermore, measurements show that the superstructures with continuous toppings along the span, S-C and S-D, had a negative displacement while the discontinuous toppings did not.

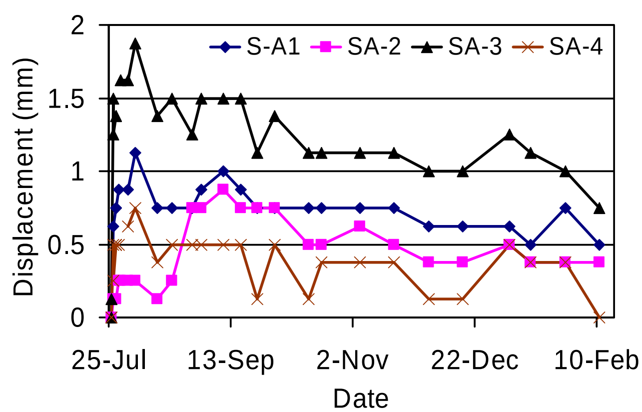


Figure 47. Displacement of superstructure S-A

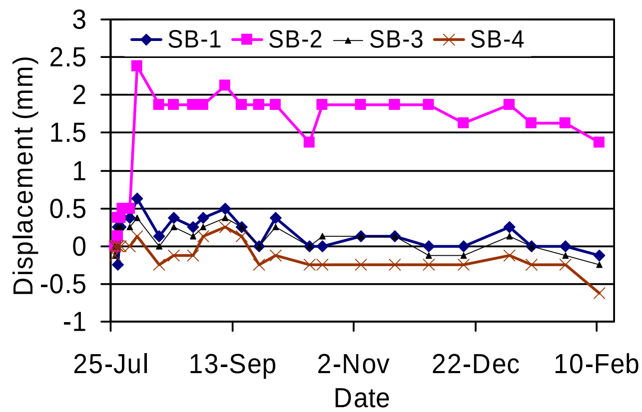


Figure 48. Displacement of superstructure S-B. Gage SB-2 was bumped on August 5, 2004

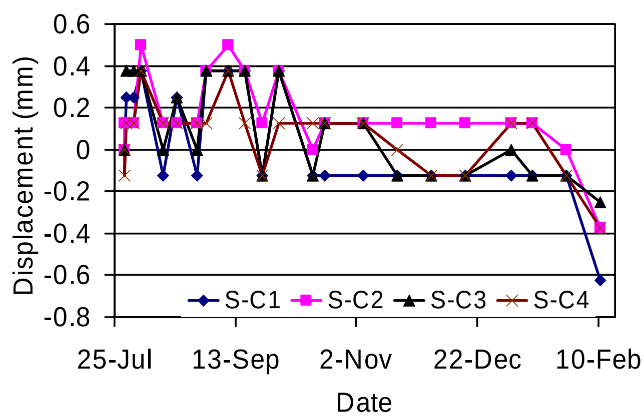


Figure 49. Displacement of superstructure S-C

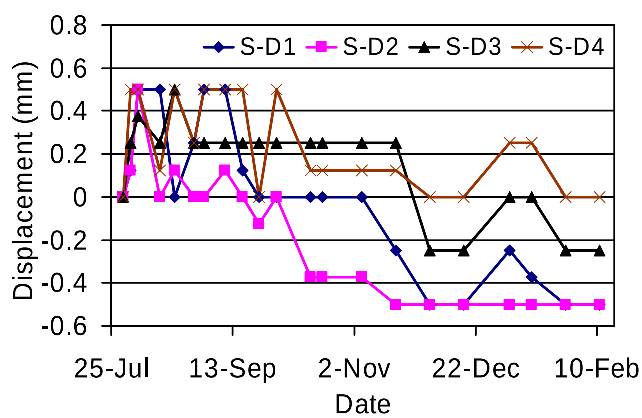


Figure 50. Displacement of superstructure S-D

CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

The focus of this research was to evaluate techniques for providing crack control in the topping of a precast flat slab bridge. Crack control treatments were selected based on their effectiveness, ease of implementation and application, and effect on the labor and construction cost of the bridge. The toppings incorporated either: steel fibers, synthetic fibers, steel/synthetic fiber blend, carbon-fiber grid, or a shrinkage-reducing-admixture.

Four full-scale bridge superstructures were constructed to evaluate the crack control treatments. Each superstructure was composed of three adjacent flat slabs with a 6 in concrete topping. The treatments were each incorporated into a standard FDOT approved concrete mixture and cast on-site.

Cylinder tests were conducted for compressive and tensile strength, and modulus of elasticity. The cracking performance of the treatments was evaluated using a restrained ring test.

After 30 weeks of observation, cracks were not visible in the topping over the flat slab joints. Plastic shrinkage cracks were visible in the CTL, SRA, and GRD toppings. Therefore, it is recommended that the bearing pads be relocated to the center flat-slabs and the toppings be stressed by mechanical means. The results of the restrained ring test will then be correlated to the performance of the toppings. The performance of the carbon fiber will also be compared to the other toppings and recommendations will be made to changes in flat-slab bridge construction.

Based on observations during construction, the results of the materials tests, and the performance of the toppings, the following is concluded:

- Insufficient tensile stresses were generated in the toppings to induce cracking.
- Fiber reinforced concrete with fiber volumes such as those used for the STL and SYN toppings should incorporate a high-range-water reducer to improve workability
- The crack control treatments did not affect the concrete’s modulus of elasticity or tensile strength.
- The STL, SYN, and BND mixtures performed better in reducing the average crack width than the CTL mixture, using the restrained ring test.
- Smaller average crack widths were attained with higher fiber volumes using the restrained ring test.

APPENDIX A
FLORIDA DEPARTMENT OF TRANSPORTAION PSBEAM PROGRAM

LRFD English Prestressed Beam Program
Data Input

Project = "Research Design"
DesignedBy = "Laz Alfonso"
Date = "Dec 12, 2003"

ExistingDataFile ≡ vec2str(READPRN("PbeamFileName.dat")) DataFileToBeCreated := vec2str(READPRN("PbeamFileCreated.dat"))

ExistingDataFile = "C:\FDOT_STR\Programs\LRFDPbeamE1.85\4 ft original span.dat"

DataFileToBeCreated = "C:\FDOT_STR\Programs\LRFDPbeamE1.85\4 ft original span.dat"

Comment = "4 ft wide 12 inch thk 30 ft span"

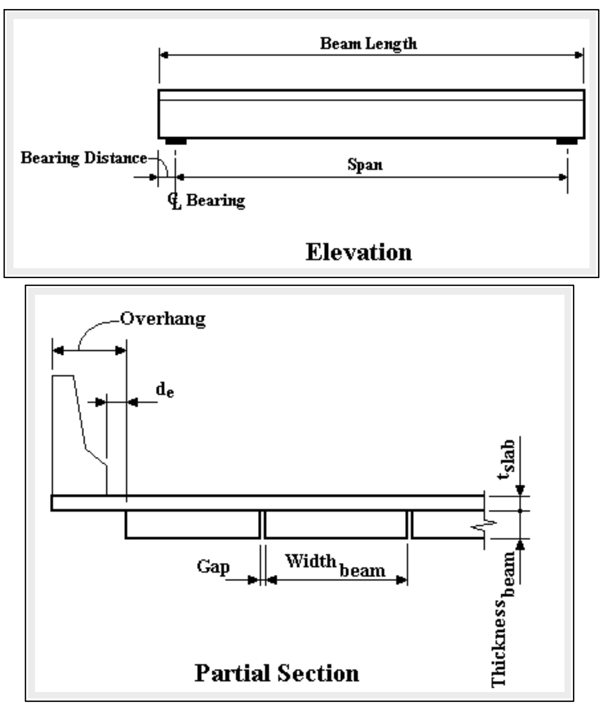
newComment := "4 ft wide 12 inch thk 30 ft span"

Only change the *new* *values, if current data values are OK, leave the double X* (XX) *in the* newData *field.*

Enter or Change Project Data

newProject := "XX" newDesignedBy := "XX" newDate := "XX"

Plan, Elevation, and Cross Section Data



The top of the precast beam is the location of the origin for the coordinate system.

Figure 51. LRFD PSBeam input 1

Echo of Input		*Input New Values*
$L_{beam} = 30\,ft$	*see Beam Elevation*	$newL_{beam} := 30 \cdot ft$
$BearingDistance = 6\,in$	*see Beam Elevation*	$newBearingDistance := XX \cdot in$
$PadWidth = 6\,in$	*width of the bearing pad - used in the shear calculations - see Beam Elevation*	$newPadWidth := XX \cdot in$
$Width_{beam} = 4\,ft$	*see Partial Section*	$newWidth_{beam} := XX \cdot ft$
$Width_{adj.beam} = 4\,ft$	*used to calculate the live load distribution to exterior beams. Not used for interior beams*	$newWidth_{adj.beam} := XX \cdot ft$
$Overhang = 0\,ft$	*see Partial Section*	$newOverhang := XX \cdot ft$
$t_{slab} = 6\,in$	*see Partial Section, not including integral WS*	$newt_{slab} := XX \cdot in$
$t_{slab.delta} = 1\,in$	*maximum additional slab thickness over support to accomodate camber, used for additional DL only*	$newt_{slab.delta} := XX \cdot in$
$d_e = -1.5\,ft$	*see Partial Section (3 ft max).* *(LRFD 4.6.2.2.1)* *corrected to ASSHTO definition internally*	$newd_e := XX \cdot ft$
BeamPosition = "interior"	*This should be either* "interior" *or* "exterior"	newBeamPosition := XX
$Thickness_{beam} = 12\,in$	*see Partial Section*	$newThickness_{beam} := XX \cdot in$
$Gap = 1\,in$	*see Partial Section* *(LRFD 3.6.1.1.1)*	$newGap := XX \cdot in$
$t_{integral.ws} = 0.5\,in$	*wearing surface thickness cast with the deck* *(SDG 7.2.1)*	$newt_{integral.ws} := XX \cdot in$
$Weight_{future.ws} = 0.015 \frac{kip}{ft^2}$	*future wearing surface* *(SDG Table 3.1)*	$newWeight_{future.ws} := XX \cdot \frac{kip}{ft^2}$
NumberOfBeams = 11	*number of beams in the span cross section (LRFD 4.6.2.2.1)*	newNumberOfBeams := XX
SectionType = "transformed"	*transformed =* "transformed" *gross =* "gross"	newSectionType := XX
$Skew = 0\,deg$	*see Plan View*	$newSkew := 0 \cdot deg$

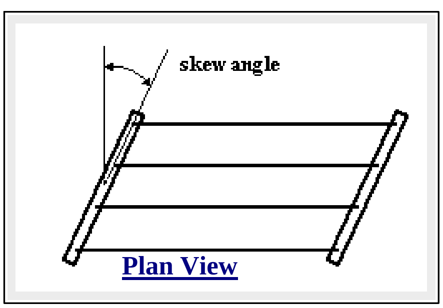


Figure 52. LRFD PSBeam input 2

Permit Truck Axle Loads and Spacings

$PermitAxles = 2$

This is the number of wheel loads that comprise the permit truck, max for dll is 11. A value must be entered for newPermitAxles *for changes to* newPermitAxleLoad *or* newPermitAxleSpacing *to register*

$newPermitAxles := XX$

$Toggle_{permit.only} = 0$

If this value is 1 only the permit live load is considered otherwise the HL-93 live load is used for stresses and the worst case for Strength checks

$newToggle_{permit.only} :=$ XX

$Permit\_uniform\_LL = 0\frac{lbf}{ft}$

Uniform live load to be considered in conjuction with the Permit Vehicle (per lane)

$newPermit\_uniform\_LL := XX \cdot \frac{lbf}{ft}$

Indexes used to identify values in the P and d vectors

$newPermitAxles := if(newPermitAxles = XX, 1, newPermitAxles)$

$q := 0..(newPermitAxles - 1)$ $qt := 0..newPermitAxles$

$$PermitAxleLoad = \begin{pmatrix} 8 \\ 32 \end{pmatrix} kip$$

$newPermitAxleLoad_q :=$

XX·kip
XX·kip
XX·kip
XX·kip
XX·kip
XX·kip
XX·kip
XX·kip
XX·kip

$newPermitAxleSpacing_{qt} :=$

0·ft
XX·ft
XX·ft
XX·ft
XX·ft
XX·ft
XX·ft
XX·ft
XX·ft
0·ft

The PermitAxleSpacing *vector contains the spacings between the concentrated loads. The first and last values are place holders and should always be zero*

$$PermitAxleSpacing = \begin{pmatrix} 0 \\ 14 \\ 0 \end{pmatrix} ft$$

Material Properties - Concrete

$AggregateType = "Standard"$

This should be either "Florida" *or* "Standard" *depending on the type of course aggregate used.*

$newAggregateType :=$ XX

$f_{c.slab} = 4.5ksi$ — *strength of slab concrete* — $newf_{c.slab} := XX \cdot ksi$

$f_{c.beam} = 5.5ksi$ — *strength of beam concrete* — $newf_{c.beam} := 5.5 \cdot ksi$

$f_{ci.beam} = 4.5ksi$ — *release beam strength* — $newf_{ci.beam} := 4.5 \cdot ksi$

$\gamma_{slab} = 0.15\frac{kip}{ft^3}$ — *density of slab concrete, used for load calculations* — $new\gamma_{slab} := XX \cdot \frac{kip}{ft^3}$

$\gamma_{beam} = 0.15\frac{kip}{ft^3}$ — *density of beam concrete, used for load calculations* — $new\gamma_{beam} := XX \cdot \frac{kip}{ft^3}$

$Environment = "moderately"$

This should be either "slightly" *,* "moderately" *or* "extremely"

$newEnvironment :=$ XX

Material Properties - Prestressing Tendons

$f_{pu} = 270ksi$ — *tendon ultimate tensile strength, used for stress calcs* — $newf_{pu} := XX \cdot ksi$

$E_p = 28500ksi$ — *tendon modulus of elasticity* — $newE_p := XX \cdot ksi$

Figure 53. LRFD PSBeam input 3

Material Properties - Mild Steel

$f_y = 60\,ksi$	*mild steel yield strength*		$newf_y := XX \cdot ksi$
$E_s = 29000\,ksi$	*mild steel modulus of elasticity*		$newE_s := XX \cdot ksi$
$H = 75$	*% relative humidity*	*(LRFD 5.9.5.4.2)*	$newH := XX$
$t_j = 1.5$	*time in days between jacking and transfer*	*(LRFD 5.9.5.4.4b)*	$newt_j := XX$
$A_{slab.rebar} = 0.31 \frac{in^2}{ft}$	*area of longitudinal slab reinf per unit width of slab, both layers combined*		$newA_{slab.rebar} := XX \cdot \frac{in^2}{ft}$
$d_{slab.rebar} = 2.5\,in$	*distance from top of slab to centroid of longitudinal steel*		$newd_{slab.rebar} := XX \cdot in$
$A_{s.long} = 1.55\,in^2$	*area of longitudinal mild reinforcing in the flexural tension zone of the beam*		$newA_{s.long} := 1.55 \cdot in^2$
$d_{long} = 2\,in$	*absolute distance from top of the beam to the centroid of the longitudinal steel in the flexural tension zone*		$newd_{long} := 2 \cdot in$
$BarSize = 5$	*Size of bars used to create* $A_{s.long}$ *needed to calculate development length*		$newBarSize :=$ XX

Loads

Composite and non-composite dead loads are calculated based on the provided data and FDOT standards. In the main and detailed programs are locations where changes to the non-composite or composite dead loads can be made. These locations are noted as Add_w $_{noncomp}$ and Add_w $_{comp}$ for non-composite and composite loads respectively. Loads can be added by setting these values equal to positive values and subtracted by setting them equal to a negative value. The program will calculate and apply the HL-93 live load automatically. Additional permit loads must be listed in the permit truck section above.

end of data input

Figure 54. LRFD PSBeam input 4

LRFD English Prestressed Beam Design Program

Project = **"Research Design"**
DesignedBy = **"Laz Alfonso"**
Date = **"Dec 12, 2003"**

Legend Tan = DataEntry Yellow= CheckValues Grey = Comments + Graphs

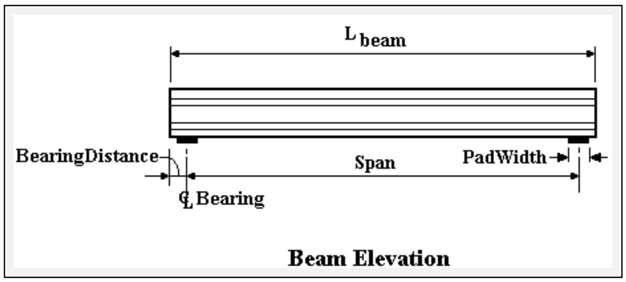
The CR values displayed are Capacity Ratios which give the ratio of the provided capacity divided by the required

Bridge Layout and Dimensions Reference:C:\FDOT_STR\Programs\LRFDPbeamE1.85\ProgramFiles\section1.mcd(R)

Comment = **"4 ft wide 12 inch thk 30 ft span"**

filename = **"C:\FDOT_STR\Programs\LRFDPbeamE1.85\4 ft original span.dat"**

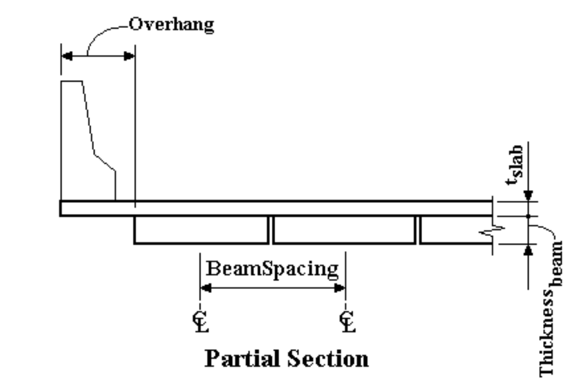
The top of the precast beam is the location of the origin



WRITEPRN("be

DataMessage = **"This is a 4 feet wide, 12 inch thick, flat slab section design "**

L_{beam} = **30** ft BearingDistance = **6** in Span = **29** ft PadWidth = **6** in



WRITEPRN("coo

$Picture_{section}$

WRITEPRN("loca

Overhang = **0** ft BeamSpacing = **4.083** ft t_{slab} = **6** in $h_{buildup}$ = **0** in

Skew = **0** deg $t_{integral.ws}$ = **0.5** in NumberOfBeams = **11** $t_{slab.delta}$ = **1** in

BeamTypeTog = **"FLT12"** *These are typically the FDOT designations found in our standards. The user can also create a coordinate file for a custom shape. In all cases the top of the beam is at the y=0 ordinate.*

BeamPosition = **"interior"** *For calculating distribution factors must be either interior or exterior*

SectionType = **"transformed"**

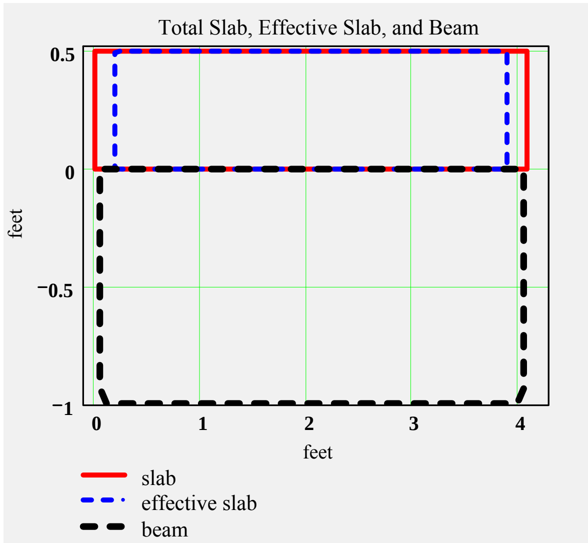
b_e = **4.083 ft** *effective slab width*

LRFD 4.6.2.6

$user\_g_{mom}$ ≡ **0** $user\_g_{shear}$ ≡ **0** *If* $user\_g_{mom}$ *(the moment distribution factor) or* $user\_g_{shear}$ *(the shear distribution factor) is set to zero the program's calculated value will be used. If they are other than zero then this user inputed value will be used.*

Figure 55. LRFD PSBeam output 1

Section Properties - Beam and Slab



Material Properties - Concrete

Corrosion Classification	Environment = **"moderately"**	*density of slab concrete*	$\gamma_{slab} = \mathbf{0.15}\ \frac{kip}{ft^3}$
strength of slab concrete	$f_{c.slab} = \mathbf{4.5}$ ksi		
strength of beam concrete	$f_{c.beam} = \mathbf{5.5}$ ksi	*density of beam concrete*	$\gamma_{beam} = \mathbf{0.15}\ \frac{kip}{ft^3}$
release beam strength	$f_{ci.beam} = \mathbf{4.5}$ ksi	*weight of future wearing surface*	$Weight_{future.ws} = \mathbf{0.015}\ \frac{kip}{ft^2}$
initial conc. modulus of elasticity	$E_{ci} = \mathbf{3861}$ ksi	*used in distribution calculation*	$n_d = \mathbf{1.106}$
concrete modulus of elasticity	$E_c = \mathbf{4268}$ ksi		
type of course aggregate, either **"Florida"** *or* **"Standard"**	AggregateType = **"Standard"**	*relative humidity*	H = **75**

Material Properties - Prestressing Tendons and Mild Steel

tendon ultimate tensile strength	$f_{pu} = \mathbf{270}$ ksi	*tendon modulus of elasticity*	$E_p = \mathbf{28500}$ ksi
time in days between jacking and transfer	$t_j = \mathbf{1.5}$	*ratio of tendon modulus to beam concrete modulus*	$n_p = \mathbf{6.677}$
mild steel yield strength	$f_y = \mathbf{60}$ ksi	*mild steel modulus of elasticity*	$E_s = \mathbf{29000}$ ksi
ratio of rebar modulus to beam concrete modulus	$n_m = \mathbf{6.794}$		
d distance from top of slab to centroid of slab reinf.	$d_{slab.rebar} = \mathbf{2.5}$ in	*area per unit width of longitudinal slab reinf.*	$A_{slab.rebar} = \mathbf{0.31}\ \frac{in^2}{ft}$

Figure 56. LRFD PSBeam output 2

d distance from top of beam to centroid of mild flexural tension reinf. $d_{long} = -2$ in

area of mild reinf lumped at centroid of bar locations $A_{s.long} = \mathbf{1.55}\ \text{in}^2$

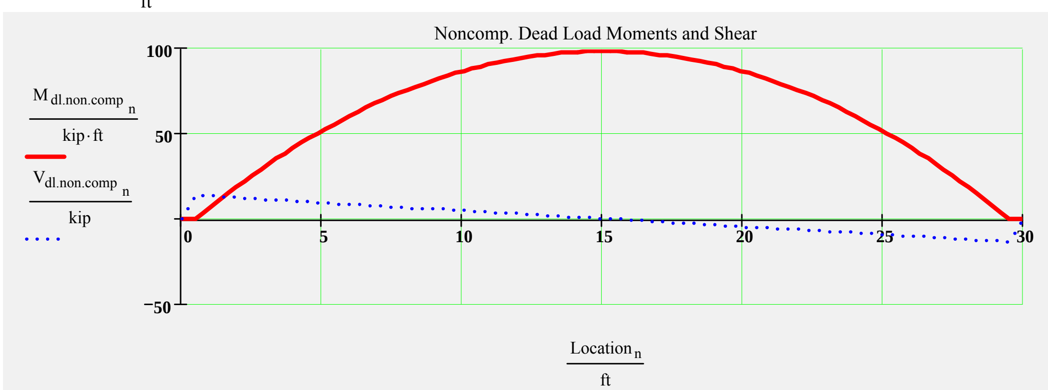
Permit Loads

Number of wheel loads that comprise the permit truck $\text{PermitAxles} = \mathbf{2}$ $\text{PermitUniformLoad} = \mathbf{0}\ \frac{\text{lbf}}{\text{ft}}$

$\text{PermitAxleLoad}^T = (\mathbf{8}\ \ \mathbf{32})\ \text{kip}$ $\text{PermitAxleSpacing}^T = (\mathbf{0}\ \ \mathbf{14}\ \ \mathbf{0})\ \text{ft}$

Loads - Release, Non composite, Composite, and Live Load (truck and lane)

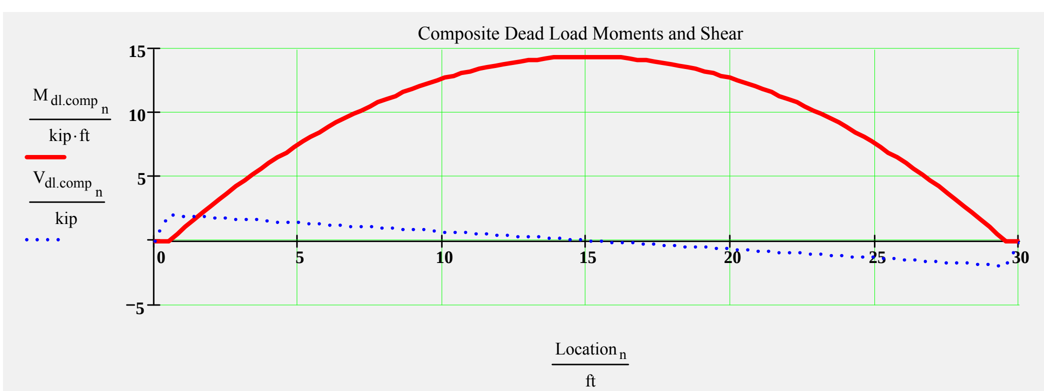
$w_{beam} = \mathbf{0.599}\ \frac{\text{kip}}{\text{ft}}$ $\max(M_{release}) = \mathbf{67.4}\ \text{kip}\cdot\text{ft}$ *note: at release, span length is the full length of the beam*



$w_{slab} = \mathbf{0.332}\ \frac{\text{kip}}{\text{ft}}$ $w_{beam} = \mathbf{0.599}\ \frac{\text{kip}}{\text{ft}}$ $w_{forms} = \mathbf{0}\ \frac{\text{kip}}{\text{ft}}$ $w_{noncomposite} = \mathbf{0.931}\ \frac{\text{kip}}{\text{ft}}$ $\text{Add\_w}_{noncomp} \equiv \mathbf{0}\cdot\frac{\text{kip}}{\text{ft}}$

(w_{slab} includes buildup)

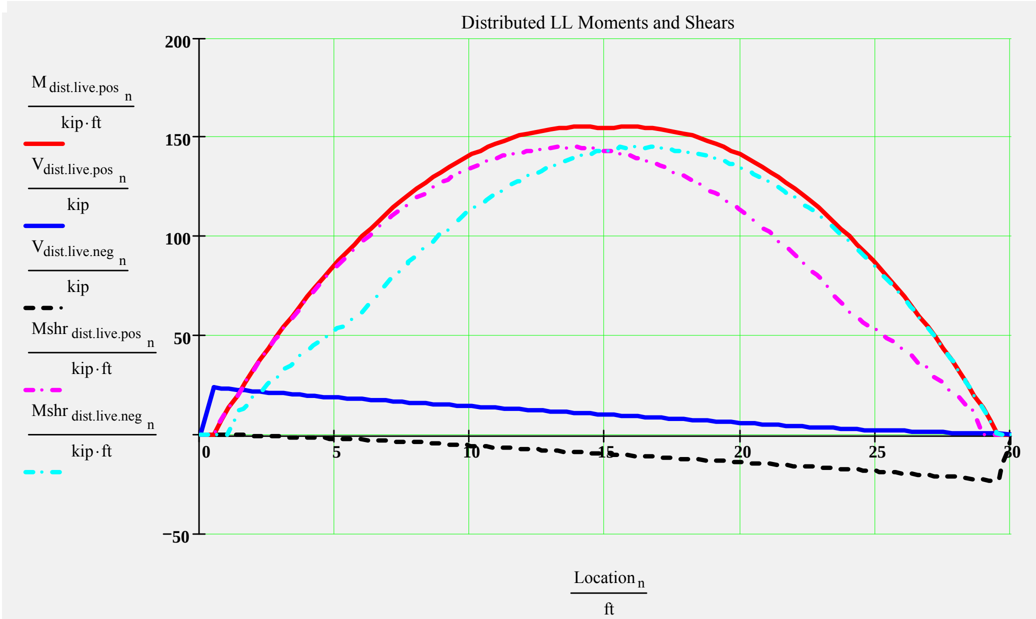
$\max(M_{dl.non.comp}) = \mathbf{98.5}\ \text{kip}\cdot\text{ft}$ $\max(V_{dl.non.comp}) = \mathbf{13.5}\ \text{kip}$



$w_{barrier} = \mathbf{0.076}\ \frac{\text{kip}}{\text{ft}}$ $w_{future.ws} = \mathbf{0.061}\ \frac{\text{kip}}{\text{ft}}$ $w_{composite} = \mathbf{0.14}\ \frac{\text{kip}}{\text{ft}}$ $\text{Add\_w}_{comp} \equiv \mathbf{0}\cdot\frac{\text{kip}}{\text{ft}}$

$\max(M_{dl.comp}) = \mathbf{14}\ \text{kip}\cdot\text{ft}$ $\max(V_{dl.comp}) = \mathbf{2}\ \text{kip}$

Figure 57. LRFD PSBeam output 3



Live load distribution factors BeamPosition = **"interior"** g_{shear} = **0.32** g_{mom} = **0.32**

Reaction $_{LL}$ = **24.381** kip *(service value includes truck impact)* Reaction $_{DL}$ = **16.026** kip *(service value)*

A suggested method of iteration is to fill the beam with tendons beginning in the middle of the bottom row, filling the row outward, then continuing on to the middle of the next lowest row. Typically ,the minimum number of tendon is reached when midspan tensile stress is below the LRFD Service III Limit stress. Next, tendons should be debonded in pairs according to the Structures Design Guidelines until the end compression stress are below the LRFD Service I Limit stress. These two limits typically control the design (see graph below).

Design Prestress Tendon Geometry

Double click on the **Strand Geometry** icon to specify type, location, size, and debonding of strands. Then click on Standdata and press F9 to read in the data.

Strand Geometry

Reference:C:\FDOT_STR\Programs\LRFDPbeamE1.85\ProgramFiles\section

Stranddata :=
a ← READPRN(**"tendsect.dat"**)
w ← READPRN(**"strand.dat"**)
x ← READPRN(**"area.dat"**)
y ← READPRN(**"shield.dat"**)
z ← READPRN(**"distance.dat"**)
(w x y z a)

Summary of Initial Compression and Final Tension Prestress for Iteration Purposes. These two stress checks usually control . See graphs in proceeding sections for full details.

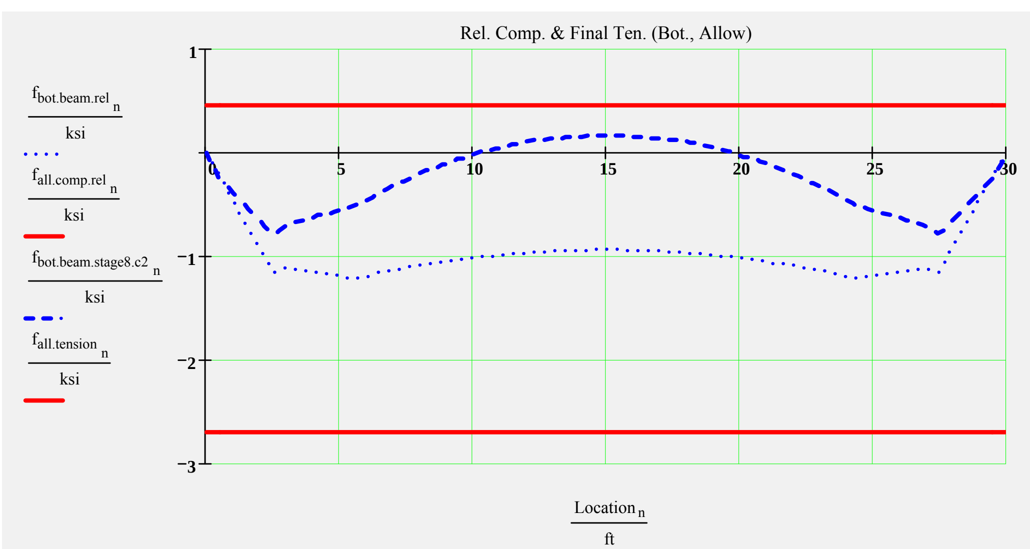


Figure 58. LRFD PSBeam output 4

$\min(CR\_f_{comp.rel}) = \mathbf{2.212}$ $\quad$ $Check\_f_{comp.rel} = \mathbf{"OK"}$

$\min(CR\_f_{tension.stage8}) = \mathbf{2.894}$ $\quad$ $Check\_f_{tension.stage8} = \mathbf{"OK"}$

check strand pattern for debonding limits (per row and total) and for debonded strands on outside edge of strand pattern
Check0 - No Debonded tendon on outside row, Check1 - less than 40% Debonded in any row, Check2 - less than 25% Debonded total

$CheckPattern_0 = \mathbf{"OK"}$

$CheckPattern_1 = \mathbf{"OK"}$

$CheckPattern_2 = \mathbf{"OK"}$

Section and tendon properties

$A_{beam} = \mathbf{3.996}\ ft^2$ *Concrete area of beam* $\quad$ $I_{beam} = \mathbf{6.893} \times \mathbf{10}^3\ in^4$ *Gross Moment of Inertia of Beam*

$y_{comp} = \mathbf{-3.152}\ in$ *Dist. from top of beam to CG of composite section* $\quad$ $I_{comp} = \mathbf{2.24} \times \mathbf{10}^4\ in^4$ *Gross Moment of Inertia Composite Section*

$A_{deck} = \mathbf{1.847}\ ft^2$ *Concrete area of deck slab* $\quad$ $A_{ps} = \mathbf{1.8}\ in^2$ *total area of strands*

$d_{b.ps} = \mathbf{0.5}\ in$ *diameter of Prestressing strand* $\quad$ $\min(PrestressType) = \mathbf{0}$ *0 - low lax 1 - stress relieved*

$f_{py} = \mathbf{243}\ ksi$ *tendon yield strength* $\quad$ $f_{pj} = \mathbf{203}\ ksi$ *prestress jacking stress*

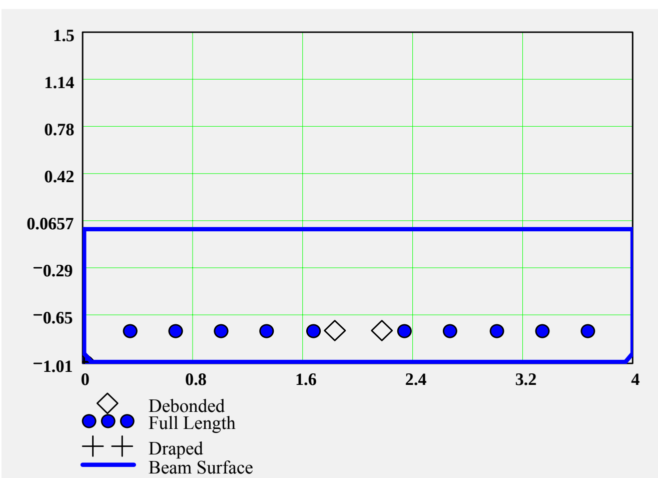
$L_{shielding}^T = (\mathbf{3}\ \ \mathbf{0})\ ft$

$A_{ps.row}^T = (\mathbf{0.3}\ \ \mathbf{1.5})\ in^2$

$d_{ps.row} =$ (ft)

	0	1	2	3	4	5	6	7
0	-0.771	-0.771	-0.771	-0.771	-0.771	-0.771	-0.771	-0.771
1	-0.771	-0.771	-0.771	-0.771	-0.771	-0.771	-0.771	-0.771

Tendon Layout



$TotalNumberOfTendons = \mathbf{12}$

$NumberOfDebondedTendons = \mathbf{2}$

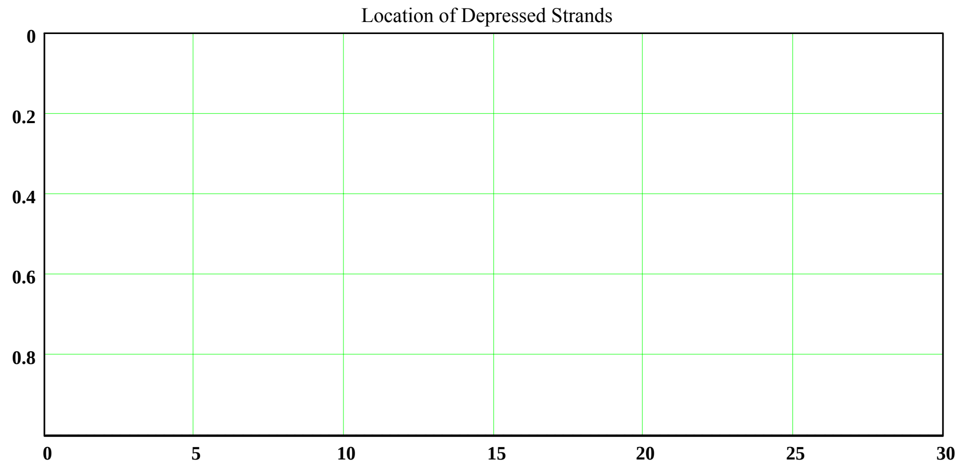
$NumberOfDrapedTendons = \mathbf{0}$

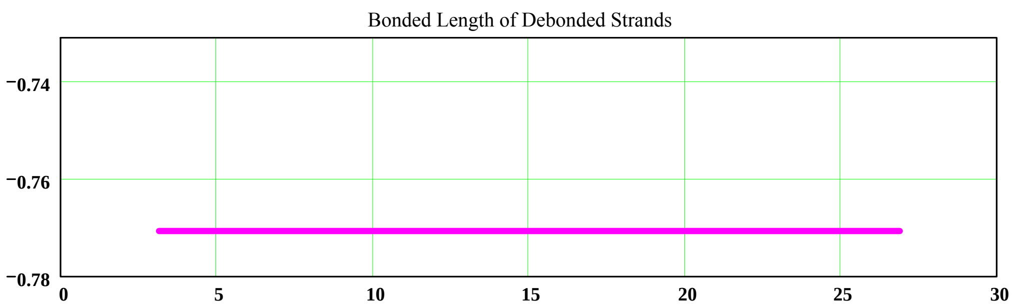
$StrandSize = \mathbf{"1/2\ in\ \ low\ lax"}$

$StrandArea = \mathbf{0.153}\ in^2$

$JackingForce_{per.strand} = \mathbf{30.982}\ kip$

Figure 59. LRFD PSBeam output 5





SERVICE LIMIT STATE

$\max(M_{pos.Ser1}) = \mathbf{268}\ \text{kip}\cdot\text{ft}$ $\max(M_{pos.Ser3}) = \mathbf{237}\ \text{kip}\cdot\text{ft}$

Prestress Losses (LRFD 5.9.5)

$f_{pj} = \mathbf{202.5}\ \text{ksi}$ $\Delta f_{pR1} = \mathbf{-2.2}\ \text{ksi}$ $\Delta f_{pES} = \mathbf{-5.8}\ \text{ksi}$ $\Delta f_{pi} = \mathbf{-8}\ \text{ksi}$ $f_{pi} = \mathbf{194}\ \text{ksi}$

$\Delta f_{pCR} = \mathbf{-7.9}\ \text{ksi}$ $\Delta f_{pSR} = \mathbf{-5.8}\ \text{ksi}$ $\Delta f_{pR2} = \mathbf{-4.5}\ \text{ksi}$ $\Delta f_{pTot} = \mathbf{-26}\ \text{ksi}$ $f_{pe} = \mathbf{176}\ \text{ksi}$

percentages $\frac{\Delta f_{pi}}{f_{pj}} = \mathbf{-3.976}\ \%$ $\frac{f_{pi}}{f_{pj}} = \mathbf{96.024}\ \%$ $\frac{\Delta f_{pTot}}{f_{pj}} = \mathbf{-12.929}\ \%$ $\frac{f_{pe}}{f_{pj}} = \mathbf{87.071}\ \%$

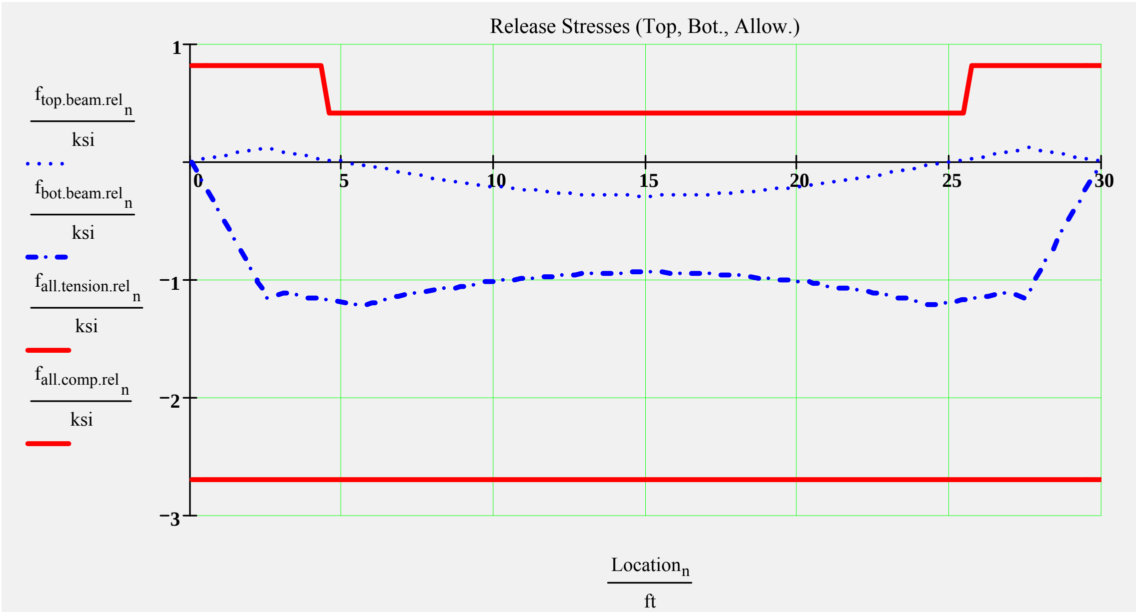
Figure 60. LRFD PSBeam output 6

Stress Limitations for P/S tendons (LRFD 5.9.3)

Check_f_{pt} = **"OK"** $0.8 \cdot f_{py}$ = **194** ksi   Check_f_{pe} = **"OK"**

Stress Limitations for Concrete - Release and Final (LRFD 5.9.4)

Release



$\min(CR\_f_{tension.rel})$ = **7.041**   Check_$f_{tension.rel}$ = **"OK"**

$\min(CR\_f_{comp.rel})$ = **2.212**   Check_$f_{comp.rel}$ = **"OK"**

Final

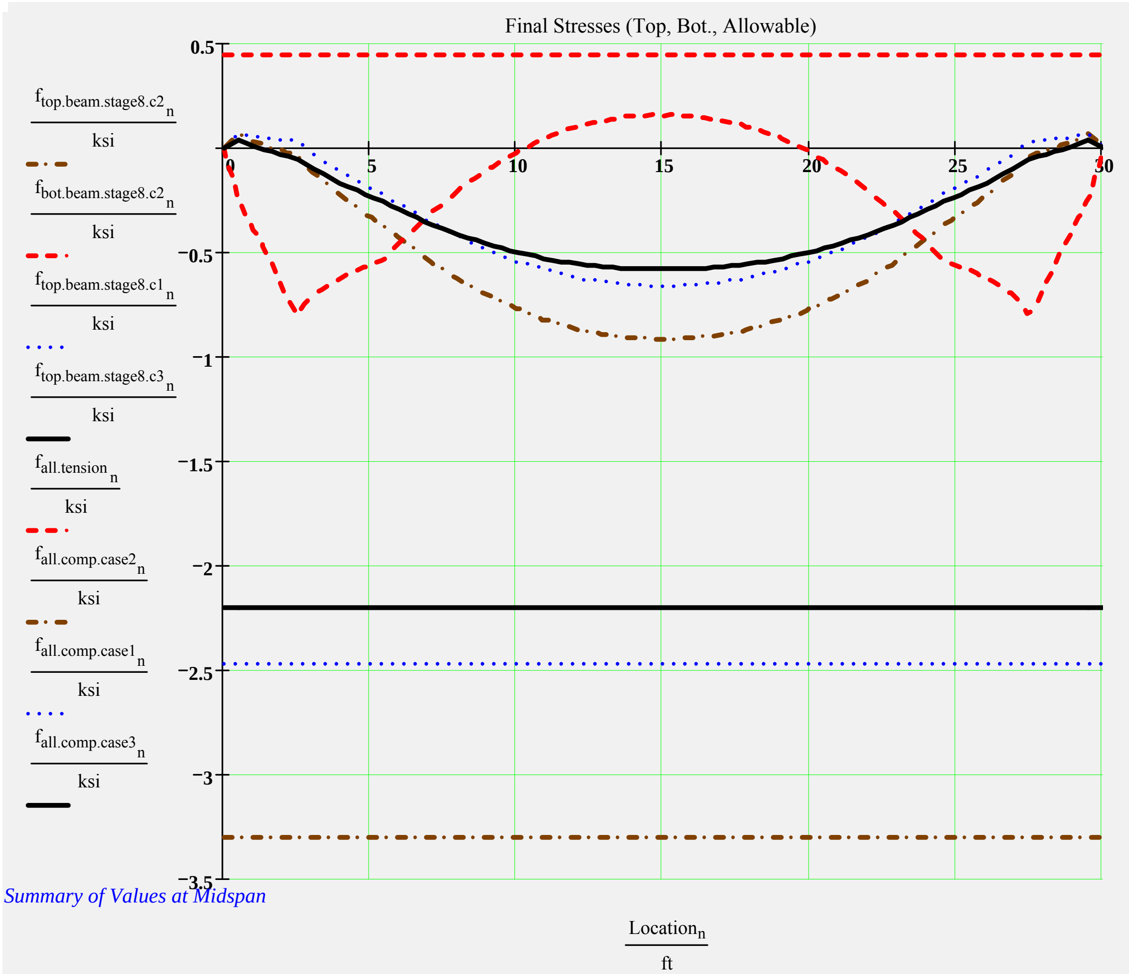
$\min(CR\_f_{tension.stage8})$ = **2.894**   Check_$f_{tension.stage8}$ = **"OK"**   *(Service III , PS + DL +LL*0.8)*

$\min(CR\_f_{comp.stage8.c1})$ = **3.729**   Check_$f_{comp.stage8.c1}$ = **"OK"**   *(Service I , PS + DL)*

$\min(CR\_f_{comp.stage8.c2})$ = **3.603**   Check_$f_{comp.stage8.c2}$ = **"OK"**   *(Service I , PS + DL +LL)*

$\min(CR\_f_{comp.stage8.c3})$ = **3.766**   Check_$f_{comp.stage8.c3}$ = **"OK"**   *(Service I , (PS + DL)*0.5 +LL)*

Figure 61. LRFD PSBeam output 7



Summary of Values at Midspan

Stresses =

"Stage "	"Top of Beam (ksi)"	" "Bott of Beam (ksi)"
1	−0.293	−0.943
2	−0.328	−0.794
4	−0.283	−0.839
6	−0.64	−0.481
8	−0.916	0.153

Compression stresses are negative and tensile stresses are positive

Stage 1 ---> At release with the span length equal to the length of the beam. Prestress losses are elastic shortening and overnight relax

Stage 2 ---> Same as release with the addition of the remaining prestress losses applied to the transformed beam

Stage 4 ---> Same as stage 2 with supports changed from the end of the beam to the bearing locations

Stage 6 ---> Stage 4 with the addition of non-composite dead load excluding beam weight which has been included since Stage 1

Stage 8 ---> Stage 6 with the addition of composite dead load and live loads applied to the composite section

PrestressForce =

"Condition "	"Axial (kip)"	"Moment (kip*ft)"
"Release"	−357.3739	−99.0955
"Final (about composite centroid)"	−323.7221	−164.6951

Properties =

"Section "	"Area (in^2) "	"Inertia (in^4) "	"distance to centroid from top of bm (in)"
"Net Beam "	582.58	7014.2	−5.92
"Transformed Beam "	594.84	7147.13	−5.99
"Composite "	877.07	23140.02	−3.14

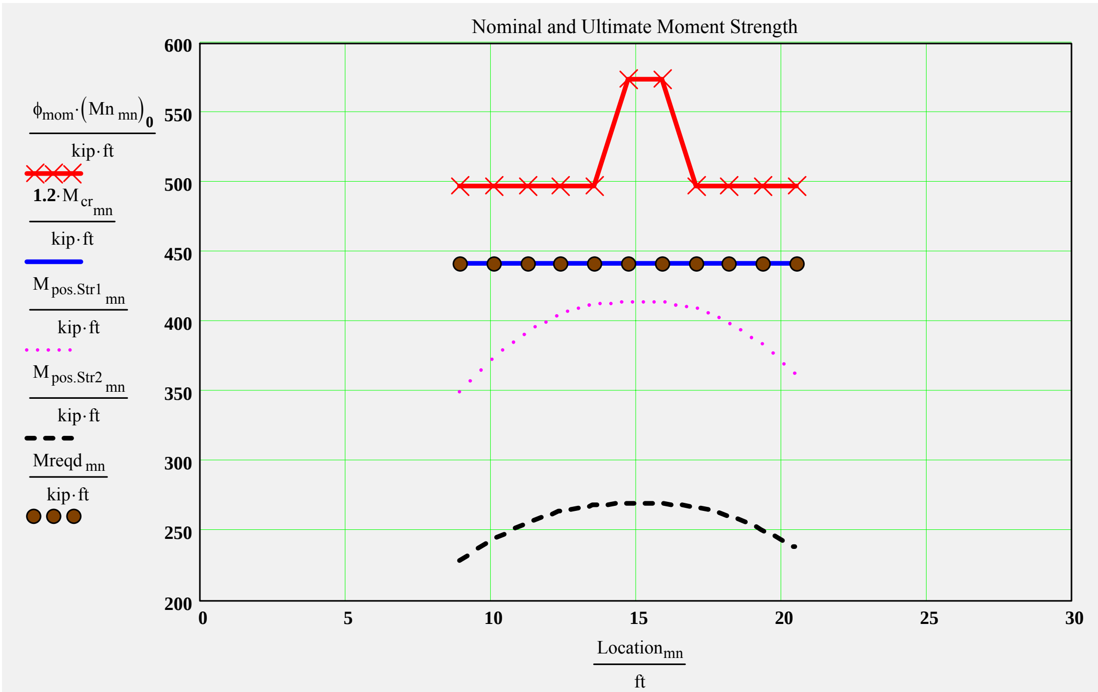
Figure 62. LRFD PSBeam output 8

$$\text{ServiceMoments} = \begin{pmatrix} \text{"Type "} & \text{"Value (kip*ft)"} \\ \text{"Release"} & 67.4 \\ \text{"Non-composite (includes bm wt.)"} & 98.5 \\ \text{"Composite"} & 14.4 \\ \text{"Distributed Live Load"} & 154.5 \end{pmatrix}$$

STRENGTH LIMIT STATE

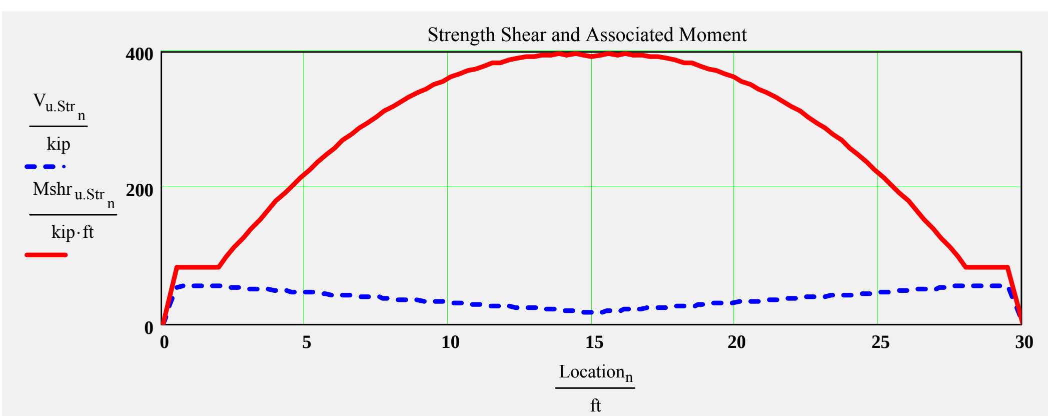
Reference:C:\FDOT_STR\Programs\LRFDPbeamE1.85\ProgramFiles\section3.mcd(R)

Moment Nominal Resistance versus Ultimate Strength Cases I and II



$\max(M_{pos.Str1}) = \mathbf{414}$ kip·ft $\quad \min(CR_{str1.mom}) = \mathbf{1.127} \quad$ CheckMomentCapacity = **"OK"**

Strength Shear and Associated Moment



$\max(V_{u.Str}) = \mathbf{56}$ kip $\quad \max(Mshr_{u.Str}) = \mathbf{396}$ kip·ft

Figure 63. LRFD PSBeam output 9

Check and Design Shear, Interface and Anchorage Reinforcement
Locally assigned stirrup sizes and spacings (Values less than 0 are ignored)
To change the values from the input file enter the new values into the vectors below. Input only those that you wish to change, values that are less than one will not alter the original input values.

The interface_factor accounts for situations where not all of the shear reinforcing is embedded in the poured in place slab

	user_s nspacings :=	user_NumberSpaces nspacings :=	user_A stirrup nspacings :=	interface_factor nspacings :=
A stirrup	XX·in	XX	$XX \cdot in^2$	0.5
S1 stirrup	XX·in	XX	$XX \cdot in^2$	1
S2 stirrup	XX·in	XX	$XX \cdot in^2$	1
S3 stirrup	XX·in	XX	$XX \cdot in^2$	1
S4 stirrup	XX·in	XX	$XX \cdot in^2$	1

Reference:C:\FDOT_STR\Programs\LRFDPbeamE1.85\ProgramFiles\section4.mcd(R)

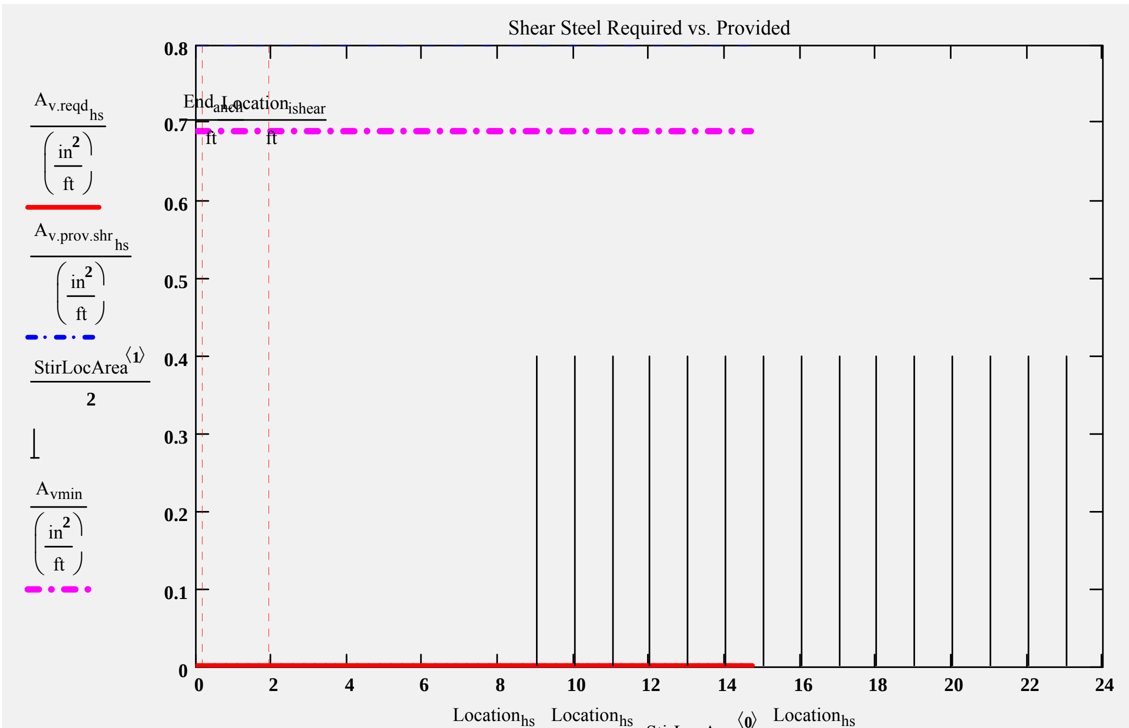
Stirrup sizes and spacings used in analysis

	s	NumberSpaces	$A_{stirrup}$
A stirrup	12	0	0
S1 stirrup	12	0	0
S2 stirrup	12	0	0
S3 stirrup	12	0	0
S4 stirrup	12	15	0.8

$$s = \begin{pmatrix} 12 \\ 12 \\ 12 \\ 12 \\ 12 \end{pmatrix} in \qquad NumberSpaces = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 15 \end{pmatrix} \qquad A_{stirrup} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0.8 \end{pmatrix} in^2$$

EndCover = **0** in

The number of spaces for the S4 stirrup is calculated by the program to complete the half beam length



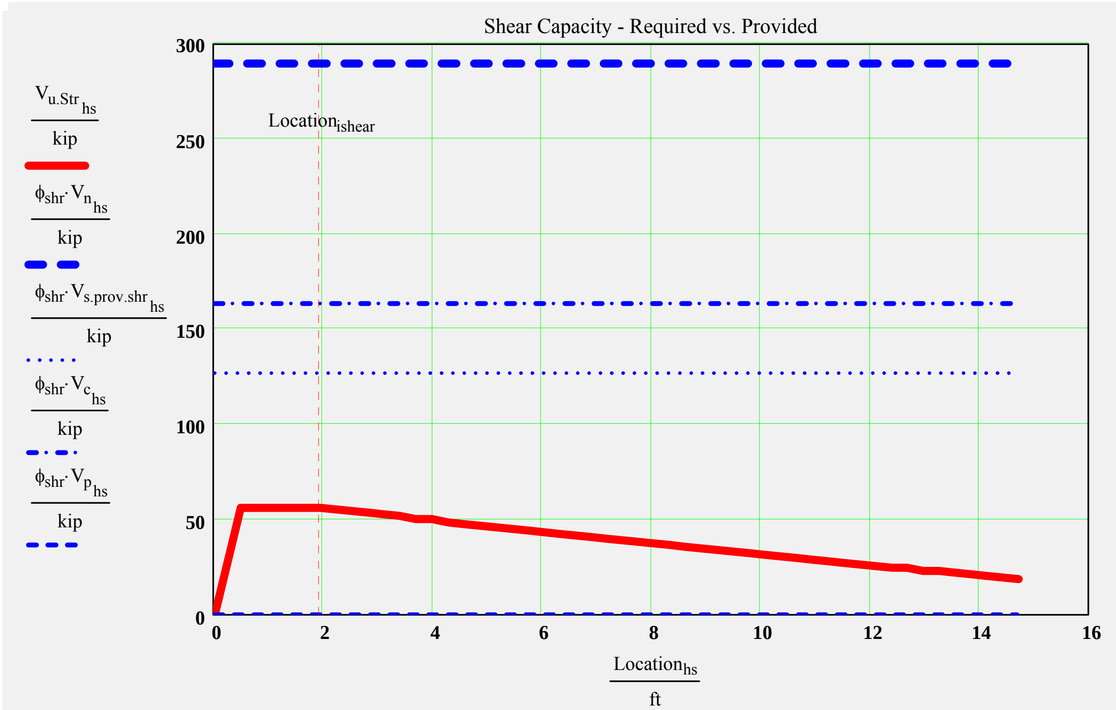
$min(CR_{ShearCapacity}) = \mathbf{2}$ CheckShearCapacity = **"N.A."**

$min(CR_{StirArea}) = \mathbf{10}$ CheckStirArea = **"N.A."**

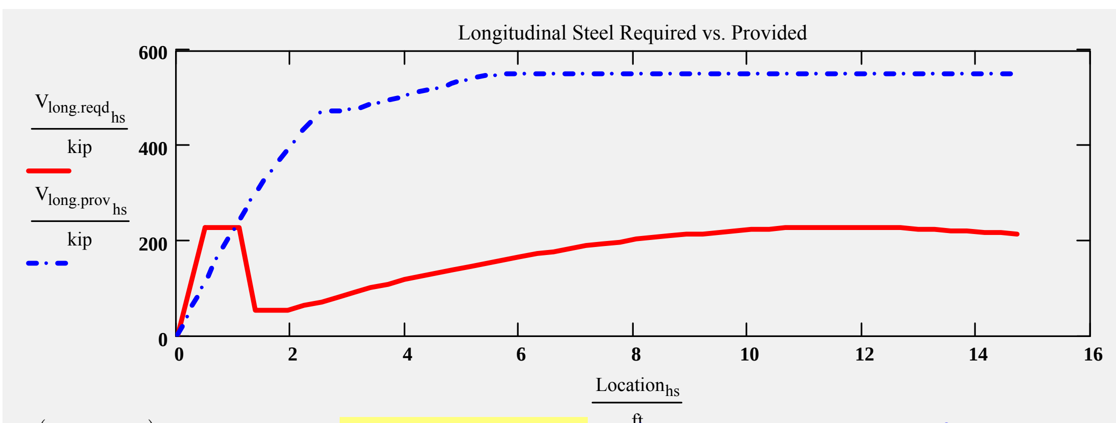
$min(CR_{StirrupArea}) = \mathbf{1.161}$ CheckMinStirArea = **"N.A."**

CheckMaxStirSpacing = **"N.A."** CheckAnchorageSteel = **"N.A."**

Figure 64. LRFD PSBeam output 10



Check Longitudinal Steel



$\min(CR_{LongSteel}) = \mathbf{0.5}$ CheckLongSteel = **"N.A."** *If NG can also adjust with shear reinforcing*

Check Interface Steel

Typically shear steel is extended up into the deck slab. These calculations are based on that assumption that the shear steel functions as interface reinforcing. The interface_factor *can be used to adjust this assumption*

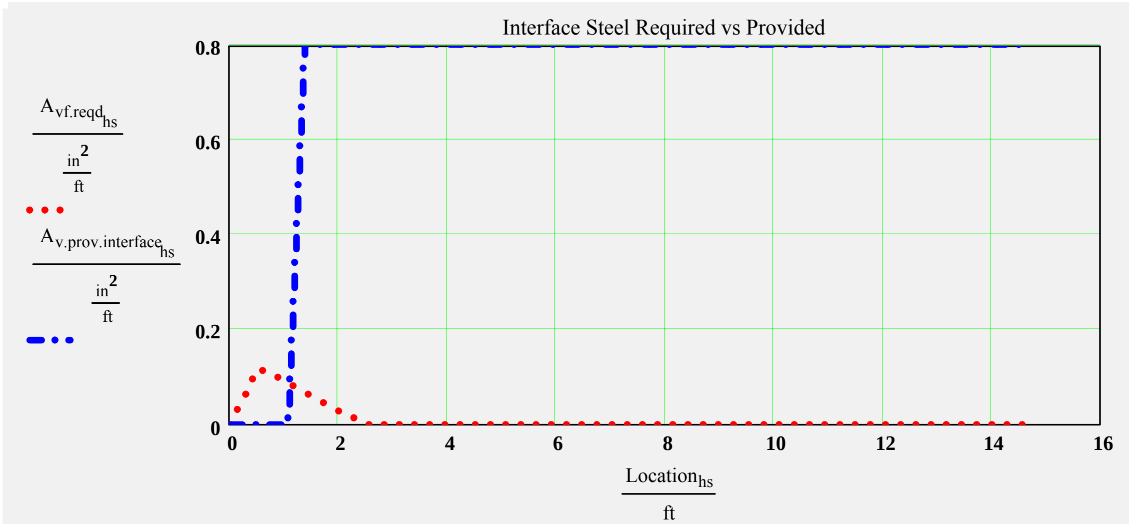
MinInterfaceReinfReqd = **"N.A."**

$A_{vf.min} = \mathbf{0}\ \frac{in^2}{ft}$ $\max(A_{vf.des}) = \mathbf{0.1}\ \frac{in^2}{ft}$

If Avf.design or Avf.min is greater than 0 in ²/ft, interface steel is required.

MinLegsPerRow = **0** CheckInterfaceSpacing = **"N.A."**

Figure 65. LRFD PSBeam output 11



$CheckInterfaceSteel := if\left(\frac{TotalInterfaceSteelProvided}{TotalInterfaceSteelRequired + \mathbf{0.001} \cdot in^2} \geq \mathbf{1}, \textbf{"OK"}, \textbf{"No Good"}\right)$ $CheckInterfaceSteel = \textbf{"OK"}$

Check Anchorage Steel for Bursting and Calculate Confinement Steel

$CheckAnchorageSteel = \textbf{"N.A."}$

use #3 bars @ 6 in for confinement $TotalNoConfineBars = \mathbf{8}$ *value includes bars at both ends*

Summary of Design Checks

AcceptInteriorM = **"OK"**	AcceptExteriorM = **"OK"**	AcceptInteriorV = **"OK"**
$Check\_f_{pt}$ = **"OK"**	$Check\_f_{pe}$ = **"OK"**	$Check\_f_{tension.rel}$ = **"OK"**
$Check\_f_{comp.rel}$ = **"OK"**	$Check\_f_{tension.stage8}$ = **"OK"**	$Check\_f_{comp.stage8.c1}$ = **"OK"**
$Check\_f_{comp.stage8.c2}$ = **"OK"**	$Check\_f_{comp.stage8.c3}$ = **"OK"**	CheckMomentCapacity = **"OK"**
CheckMaxCapacity = **"N.A."**	CheckStirArea = **"N.A."**	CheckShearCapacity = **"N.A."**
CheckMinStirArea = **"N.A."**	CheckMaxStirSpacing = **"N.A."**	CheckLongSteel = **"N.A."**
CheckInterfaceSpacing = **"N.A."**	CheckAnchorageSteel = **"N.A."**	CheckMaxReinforcement = **"OK"**
CheckInterfaceSteel = **"OK"**	CheckStrandFit = **"OK"**	

TotalCheck = **"OK"**

Figure 66. LRFD PSBeam output 12

APPENDIX B
TOPPING PLACEMENT DAILY SUMMARY

Synthetic Fiber Topping

- Flat slabs were cleaned with a blower
- Concrete batched at 8:47AM
- Truck leaves plant at 8:57AM
- Truck arrived at site at 9:10AM. Truck #118, Tag N2322B
- Driver did not have material delivery ticket
- Driver's ticket lists a 4" slump was delivered
- Flat slabs were sprayed with water
- Slump test #1 performed at 9:20AM
- 4-1/2" slump
- Started adding Strux 90/40 fibers 9:20AM-9:24AM
- Fibers were introduced by hand into the drum mixer. They were dispersed manually as they were deposited.
- Counted 70 revolutions from 9:24AM to 9:28AM
- Slump test was attempted to see the effect the fibers had on the mix. The fibers were not uniformly mixed in. There was a lot of bundling.
- Slump test #2 performed at 9:30AM
- 1-3/4" slump
- Instructed driver to add 6 gal to achieve a .44 w/c. This was based on a mixture proportions I obtained from Tallahassee Redi Mix (TRM) on a visit last Monday, July 19th.
- Slump test #3 performed at 9:40AM
- 3-1/4" slump

- Placed concrete from 9:45AM-10:12AM
- Workability was terrible. The concrete was raked and vibrated down the shute. It was then raked into place. Most of the concrete was moved between 4’ & 5’ to its final position. It was then vibrated.
- Screeding started as when the concrete placement was halfway down the topping.
- Screeding finished at 10:30AM
- Floating started as screeding took place. Finished floating at 10:32AM
- An air content of 2.5% was measured
- 27 cylinders were collected and capped. They were collected late in the cycle of events. The collection of cylinders will take place at an earlier time on the remaining toppings.
- The steel ring was cast
- There has not been any bleed water visible on the surface of the topping
- Curing compound was applied at 12:20PM
- Clouds rolled in at 12:36PM and blocked out the sun
- Went to TRM to obtain a copy of the batched materials for today’s concrete mixture. Turns out we were low on the amount of water we could add to the mix.

Blended Fiber Topping

- Met with Casey Peterson, Quality Control Manager for TRM at about 7:45AM
- Based on yesterday’s problems with placing the concrete and the low w/c ratio we wanted to discuss our options to improve the workability of the mixture. He said he could modify the mixture any way we wanted to. We discussed the possibility of reducing the amount of water reducer so as to maximize our w/c ratio while still having a reasonable slump…4”-6”. Based on conversations with Dr. Hamilton, I instructed Casey to send the same mix. We would control the w/c ratio at the site.
- Flat slabs were cleaned with a blower
- Concrete batched at 8:42AM
- Truck left plant at 8:50AM
- Truck arrived at the site at 9:07AM

- Flat slabs were sprayed with water
- Collected material ticket from driver and calculated allowable additional water
- Form was filled out incorrectly and we worked under the assumption that we only had 7 oz of water reducer in the mix. This did not affect our calculations and was discovered later on that afternoon.
- Driver's delivery ticket lists a 4" slump was delivered
- Slump test #1 performed at 9:15AM
- 2-3/4" slump
- Fibers were added to the concrete mixture
- Synthetic micro fibers were added at 9:16AM. 1lb/CY
- Steel fibers were added at 9:16AM-9:22AM. 25 lbs/CY
- The steel fibers were added second so that they would help separate the already present micro fibers
- Counted 70 revolutions from 9:22AM to 9:26AM
- Slump test #2 performed at 9:26AM
- 3-3/4" slump
- Instructed driver to add 8 gal to mixture. Based on 1" slump loss for every gallon of water per CY. We were shooting for a .44 w/c and a 5-3/4 slump.
- Slump test #3 performed at 9:35AM
- 4-3/4 slump
- Placed concrete from 9:35AM – 9:45AM
- Concrete had very good workability. It flowed down the shute easily. Most of the concrete was moved between 2' & 3' to its final position. It was then vibrated.
- Backer rod fell through and was reinstalled and secured from 9:45AM until 9:55AM
- Screeding started when the concrete placement was ¾ of the way down the topping.
- Floating started as screeding took place. Floating started at 10:06AM and finished at 10:17AM

- Screeding was finished at 10:10AM
- An air content of 3.5% was measured
- 27 cylinders were collected and capped while the concrete was placed
- The steel ring was cast while the concrete was placed
- There has not been any bleed water visible on the surface of the topping
- Curing compound was applied at 1:10PM
- Clouds rolled in at 1:20PM and rain started at 1:30PM. Some of the curing compound was washed off.

GRD Topping

- Both flat slabs were cleaned with a blower
- Concrete batched at 8:45AM
- Truck left plant at 8:57AM
- Truck arrived at the site at 9:07AM
- Flat slabs were sprayed with water
- • Collected material ticket from driver and calculated allowable additional water
- Form was incorrectly filled out again. This was noticed immediately and did not affect any calculations.
- Driver's delivery ticket lists a 4" slump was delivered
- Slump test #1 performed at 9:11AM
- 4-3/4" slump
- Instructed driver to add 5 gal of water to mix. This would put us at a .44 w/c based on the delivery ticket.
- Slump test #2 performed at 9:16AM
- 6-1/4" slump
- Placed concrete from 9:22AM – 9:29AM

- Screeding took place as concrete was placed. This finished the concrete 1” below its final surface to allow for grid installation
- Wooden 2”x6” screed was run over the topping two times
- This process was much easier than I expected
- Grid was laid out from9:30AM – 9:35AM
- Grid is 42” wide. There is a grid joint at the center with a two hole overlap. The outer strips overlap about 8” with the inner strips
- Grid was floating lightly to have it “stick” to concrete. All the grid came in contact with the concrete. There was no loss of contact due to the grid wanting to roll up.
- Concrete was topped off from 9:35AM – 9:43AM
- Driver was extremely good at placing concrete where it was needed. He backed the truck up and swung the shute as the concrete was placed
- Concrete was screeded as it was topped off.
- The final screeding finished at 9:46AM
- Floating was done from 9:49AM – 9:55AM
- An air content of 3% was measured
- 27 cylinders were collected while the concrete was placed. They were not capped
- The steel ring was cast while the concrete was placed
- Bleed water was visible on the surface as it cured
- Curing compound was applied at 2:00PM
- It started to rain at 3:05PM

Steel Fiber Topping

- Concrete batched at 9:56AM
- Truck left plant at 10:15AM
- Truck arrived at the site at 10:26AM
- Flat slabs were sprayed with water

- Collected material ticket from driver and calculated allowable additional water
- Form was incorrectly filled out
- Driver's delivery ticket lists a 4" slump was delivered
- Slump test #1 performed at 10:31AM
- 2" slump
- Instructed driver to add 16 gallons of water. This was based off of the delivery ticket. It would put us at a .44 w/c
- A slump test was not taken after the water was added
- Fibers added to the mix from 10:37AM – 10:49AM
- I could feel the heat generated by the mix as I was adding the fibers
- Counted 70 revolutions from 10:49AM to 10:53AM
- Slump test #2 performed at 10:54 AM
- 2" slump
- Placed concrete at 10:58AM
- The mix was extremely stiff. It seems like there is not enough water in the mix. One wouldn't be able to tell that 16 gallons of water were added to the mix. The mix was raked and vibrated down the shute. This mix is much more difficult to work than the synthetic mix.
- Instructed the driver to add 8 gallons of water at 11:03AM. Based on 1" slump loss for every gallon of water per CY. We were shooting for a 4" slump and expected the w/c ratio to go over the max of .44. A slump test was not performed after the water was added.
- Placement continued at 11:10AM. The mix was somewhat workable after the water was added. It still required the vibrator and the rake to get it down the shute. Most of the concrete was moved between 4' & 5' to its final position.
- Topped off at 11:20AM
- Screeded from 11:25AM – 11:50AM
- Concrete was floated but most of it was difficult to finish. There were many voids on the surface in the area of the initial pour.

- An air content of 2% was measured
- 27 cylinders were collected and capped while the concrete was placed. They were collected after the final 8 gallons of water were added.
- The steel ring was cast while the concrete was placed, after the final 8 gallons of water were added.
- No bleed water was seen on the surface
- Curing compound was applied at 2:40PM
- It started to rain at 3:05PM. At 3:18PM some of the curing compound was washed off

SRA Topping

- I called the plant earlier to request a 2" slump concrete because we did not know the effect the SRA would have on the mix
- Flat slabs were cleaned with a blower
- Concrete was batched at 8:32AM
- Truck left the plant at 8:49AM
- Truck arrived at the site at 9:05AM
- Collected material ticket from driver and calculated allowable additional water
- Slump test #1 performed at 9:13AM
- 1-3/4" slump
- Added 15 gallons of SRA from 9:16AM – 9:21AM while truck was mixing at high speed
- Much easier to add when compared to fibers. Not as worried about integration into mixture.
- Slump test #2 performed at 9:24AM
- 2" slump
- Instructed driver to add 20 gallons of water at 9:26AM. Based on 1" slump loss for every gallon of water per CY. We were shooting for a 4" slump.
- Slump test #3 performed at 9:30AM

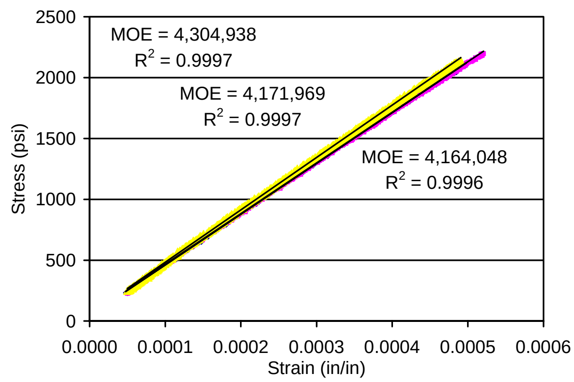
- 5” slump
- Placed concrete from 9:35Am – 9:55AM
- Concrete flowed easily down the shute. Most of the concrete was raked between 2’ & 3’ to its final position. It had very good workability.
- An air content of 1.5% was measured
- 27 cylinders were collected and capped while the concrete was placed.
- The steel ring was cast while the concrete was placed
- Screeded from 9:48AM – 10:10AM
- Floating was done by a different person today. This may have an effect on plastic cracking.
- Noticed bleed water on the surface
- I left site in order to run p. t. tests in Gainesville
- Curing compound applied by structures lab personnel.

Control Topping

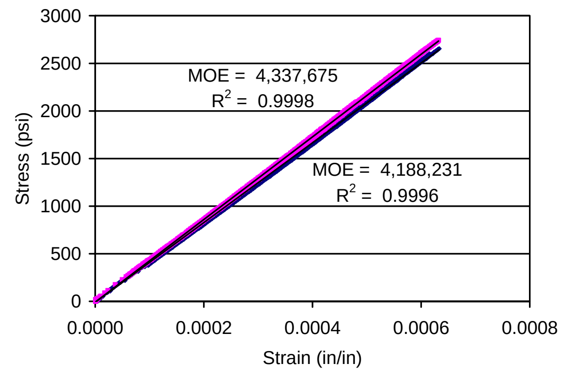
- Flat slabs were cleaned with a blower
- Concrete was batched at 8:30AM
- Truck left the plant at 8:50AM
- Truck arrived at the site at 9:02AM
- Collected material ticket from driver and calculated allowable additional water
- Slump test #1 performed at 9:04AM
- 2-3/4” slump
- Instructed driver to add 20 gallons of water to mixture
- Slump test #2 performed at 9:15AM
- 5” slump
- Placed concrete from 9:20AM -9:34AM

- Concrete had good workability
- Concrete screeded from 9:27AM – 9:45AM
- Floating was done by a different person today. This may have an effect on plastic cracking.
- 27 cylinders were collected and capped while the concrete was placed.
- Measured an air content of 1%
- The steel ring was cast while the concrete was placed
- There was a lot of bleed water on the surface. The bleed channels were clearly visible. Water was running off the sides of the formwork.
- Left site in order to run pressure tension tests in Gainesville
- Curing compound applied by structures lab personnel

APPENDIX C
CYLINDER TEST RESULTS

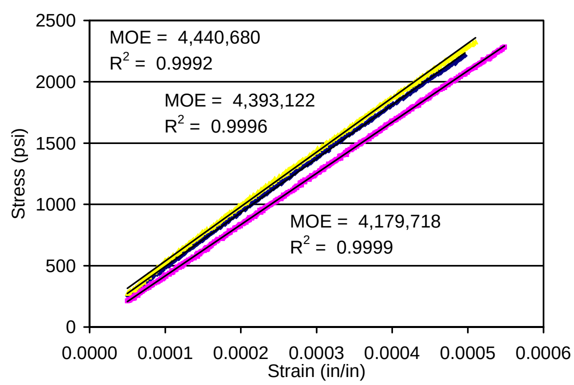


(a)

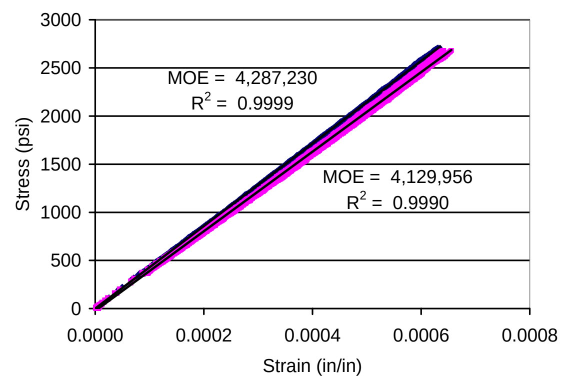


(b)

Figure 67. Modulus of elasticity charts for SYN topping. a) 28-day, b) 56-day

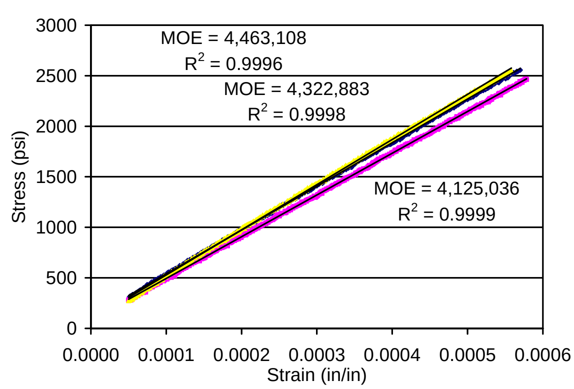


(a)

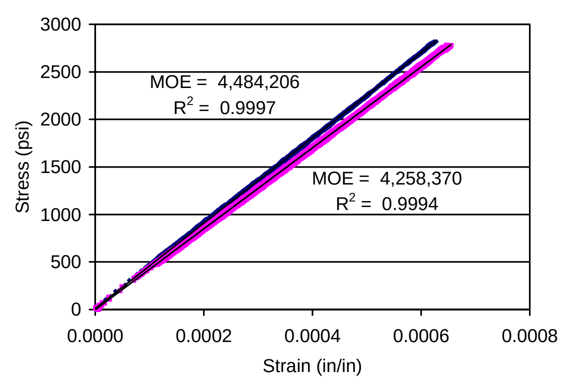


(b)

Figure 68. Modulus of elasticity charts for BND topping. a) 28-day, b) 56-day

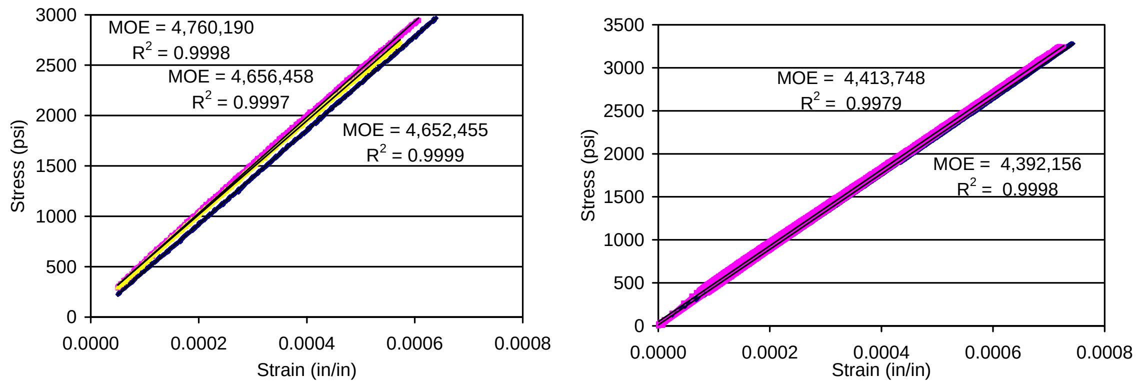


(a)



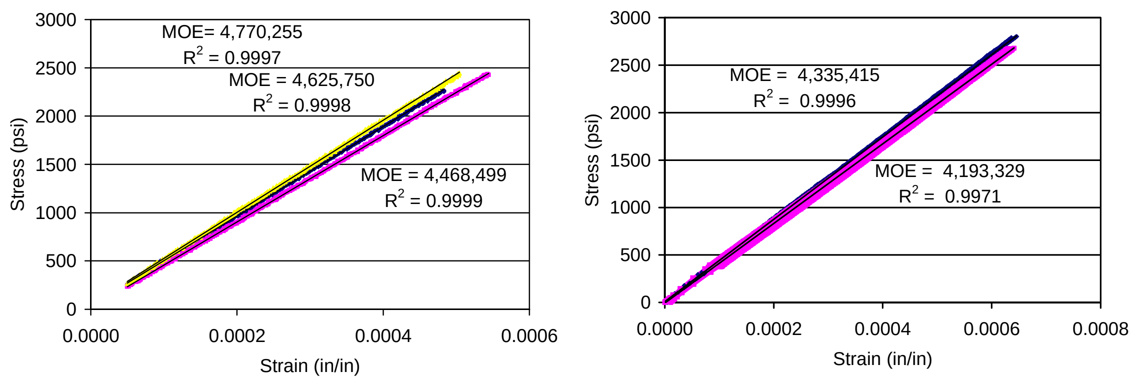
(b)

Figure 69. Modulus of elasticity charts for GRD topping. a) 28-day, b) 56-day



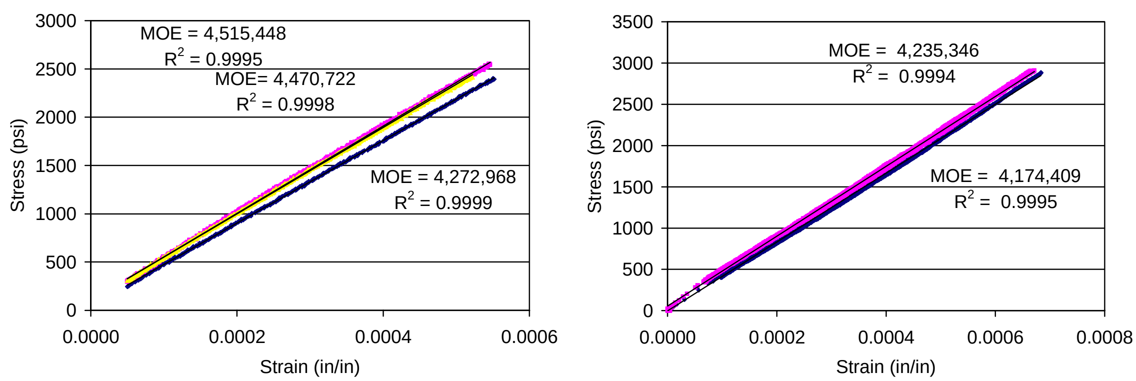
(a) (b)

Figure 70. Modulus of elasticity charts for STL topping. a) 28-day, b) 56-day



(a) (b)

Figure 71. Modulus of elasticity charts for SRA topping. a) 28-day, b) 56-day



(a) (b)

Figure 72. Modulus of elasticity charts for CTL topping. a) 28-day, b) 56-day

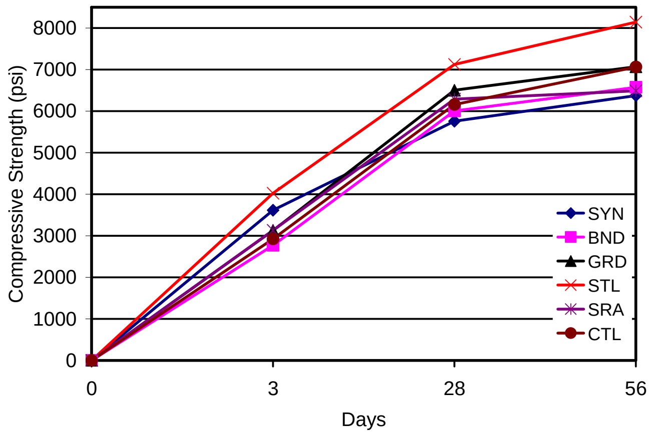


Figure 73. Compressive strength of cylinders at 3, 28, & 56-days

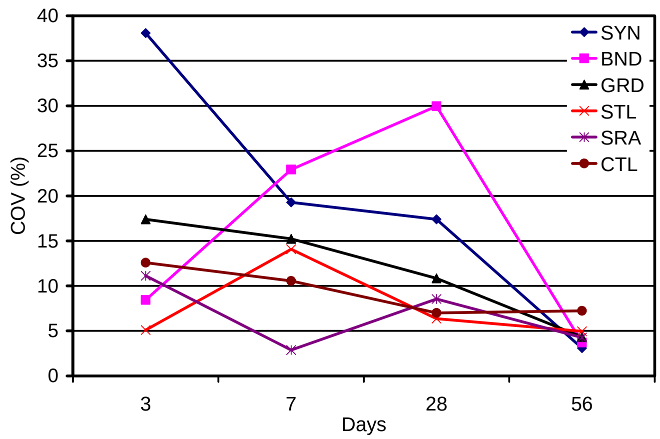


Figure 74. Coefficient of variation for load rate using pressure tension test

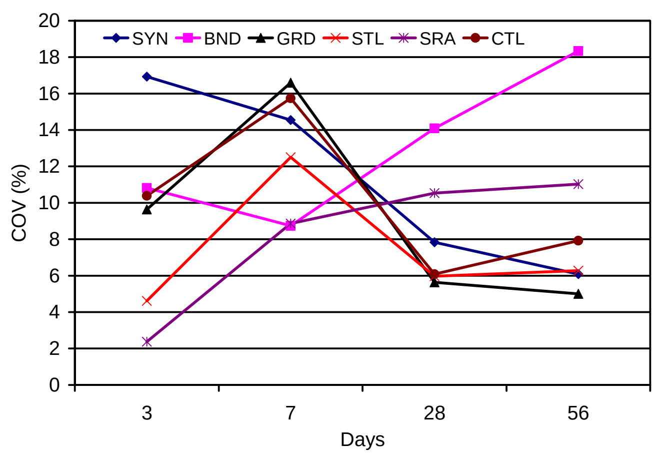


Figure 75. Coefficient of variation for strength using pressure tension test

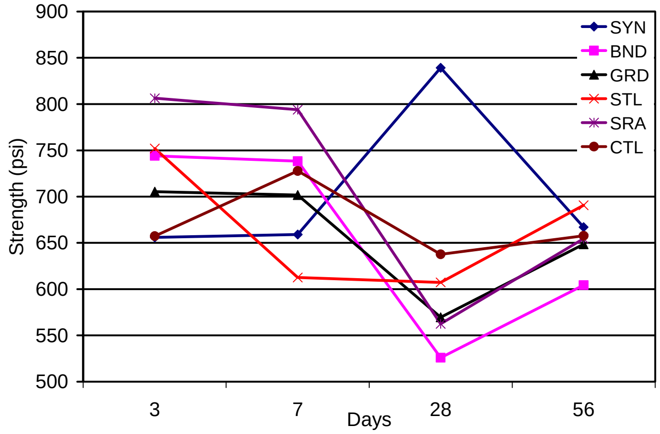


Figure 76. Tensile strength using pressure tension test

APPENDIX D
WEATHER DATA

Temperature and relative humidity data was collected from a weather station located approximately 2 miles away at the Tallahassee Regional Airport. It is operated by the National Climatic Data Center.

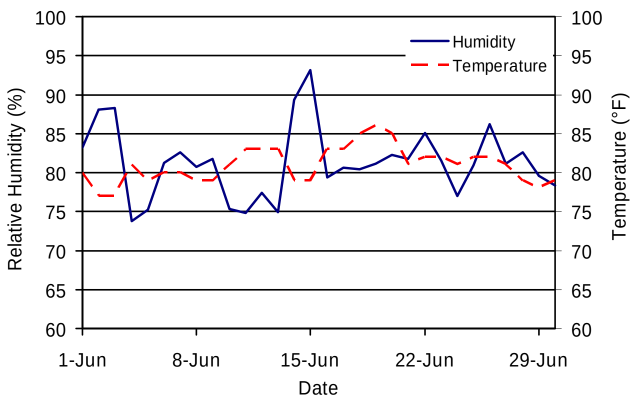


Figure 77. June 2004 humidity and temperature data

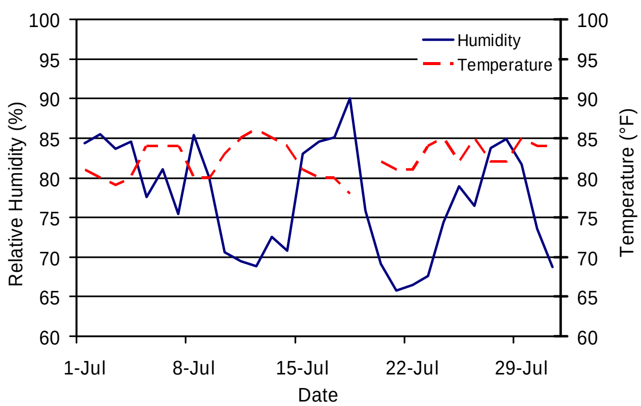


Figure 78. July 2004 humidity and temperature data

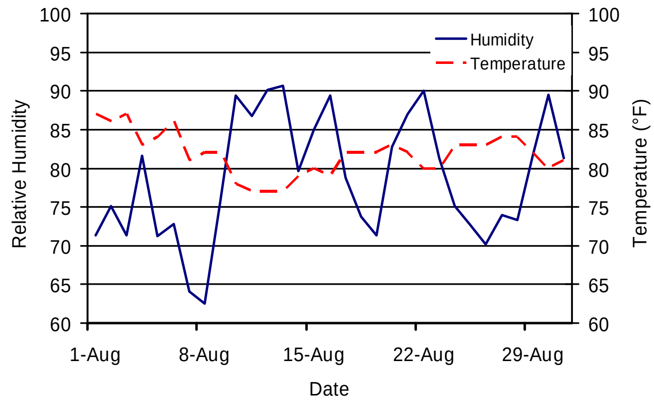


Figure 79. August 2004 humidity and temperature data

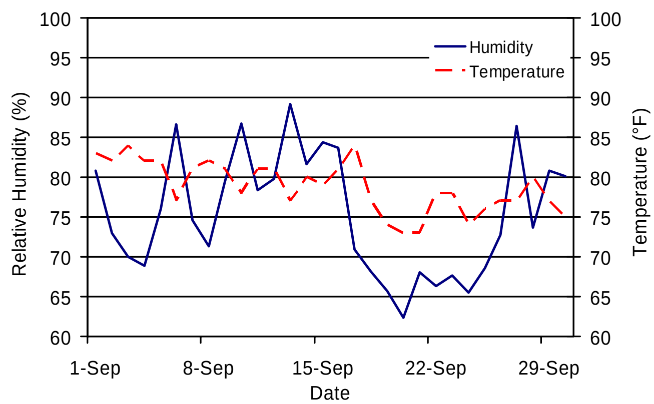


Figure 80. September 2004 humidity and temperature data

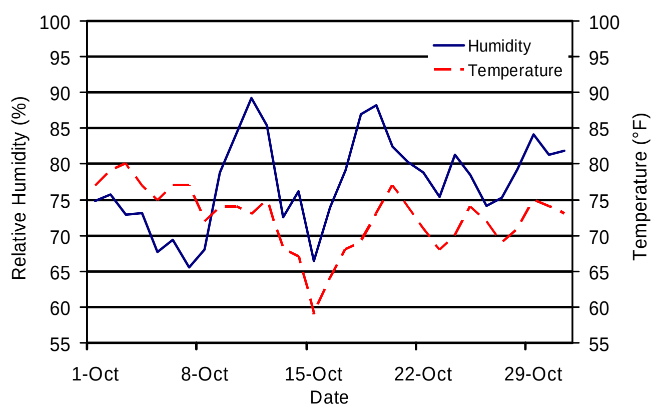


Figure 81. October 2004 humidity and temperature data

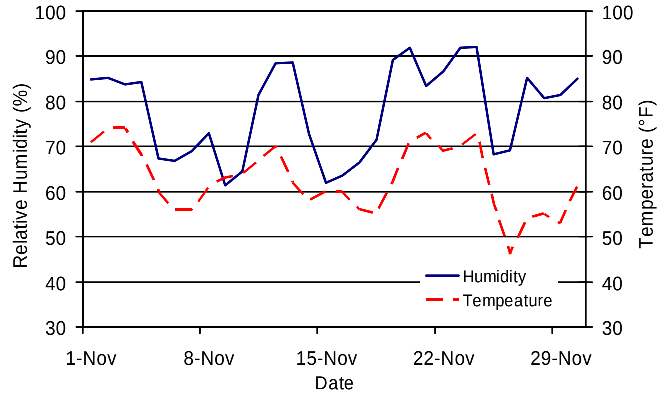


Figure 82. November 2004 humidity and temperature data

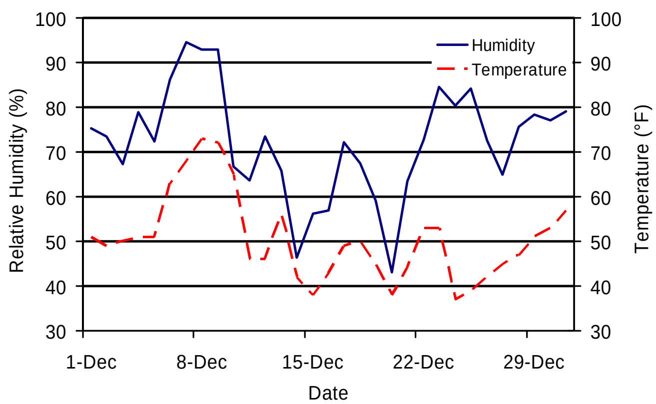


Figure 83. December 2004 humidity and temperature data

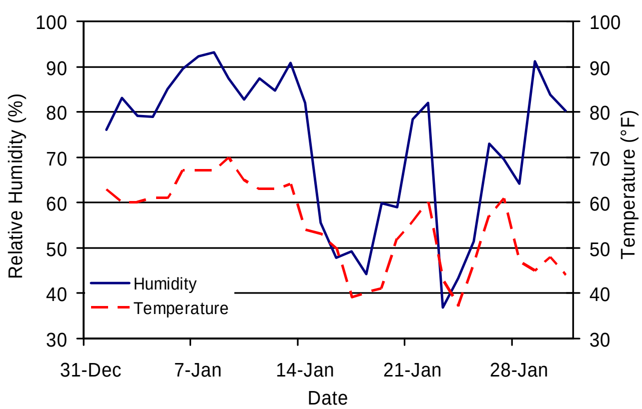


Figure 84. January 2005 humidity and temperature data

APPENDIX E
THERMOCOUPLE DATA

Synthetic Fiber Topping

See Figure 36 and Figure 37 for location of thermocouples within topping.

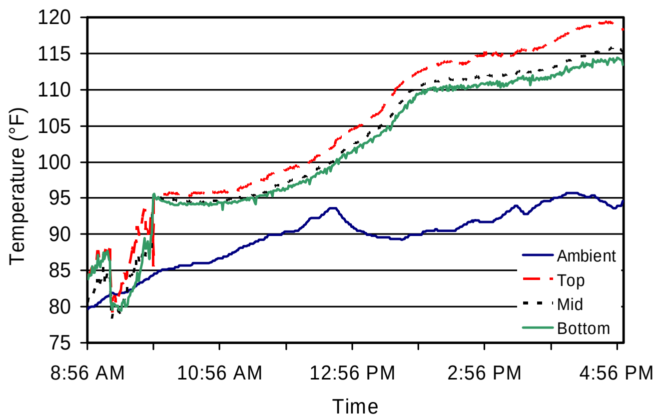


Figure 85. SYN-1 curing temperatures

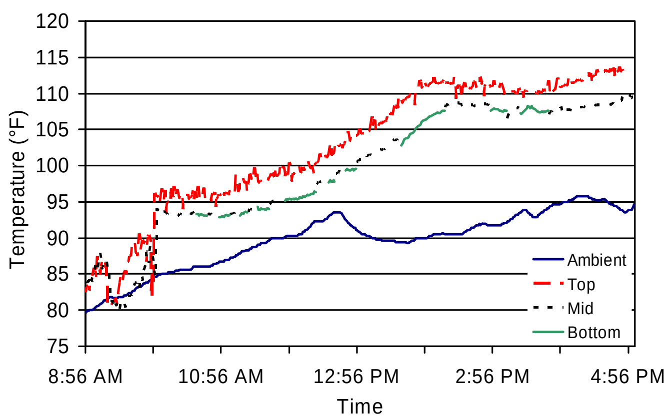


Figure 86. SYN-2 curing temperatures

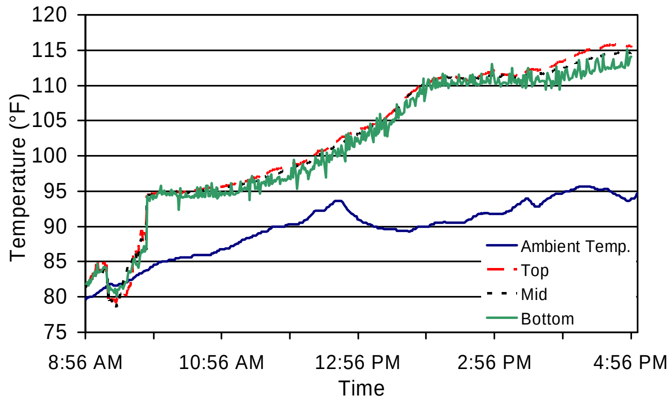


Figure 87. SYN-3 curing temperatures

Blended Fiber Topping

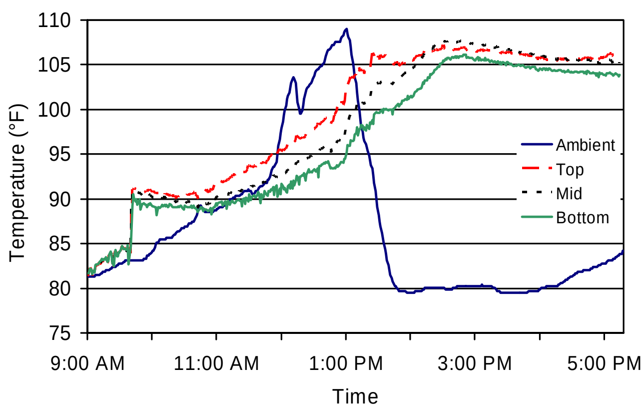


Figure 88. BND-1 curing temperatures

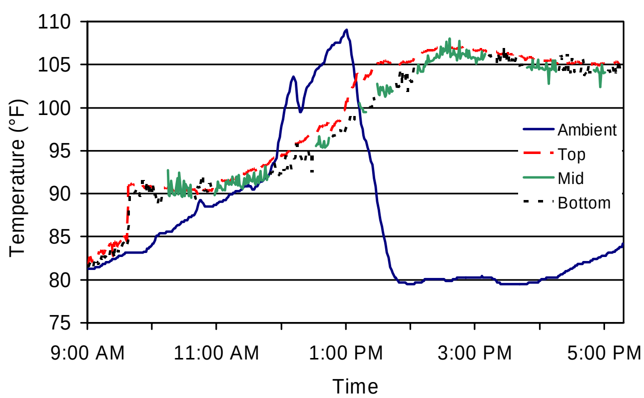


Figure 89. BND-2 curing temperatures

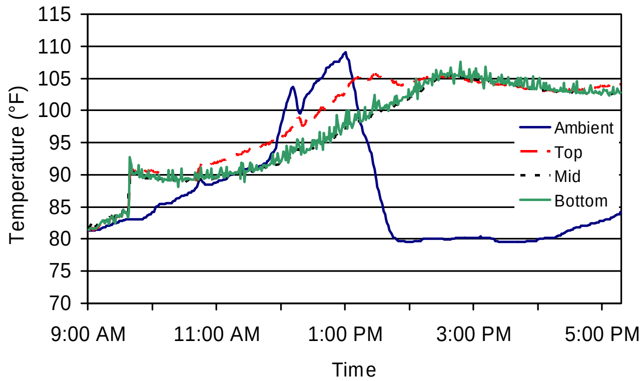


Figure 90. BND-3 curing temperatures

Steel Fiber Topping

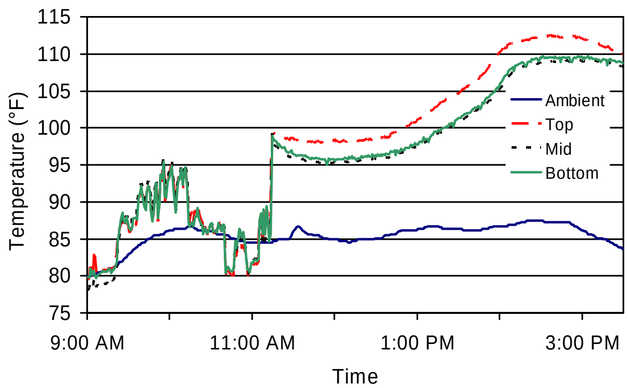


Figure 91. STL-1 curing temperatures

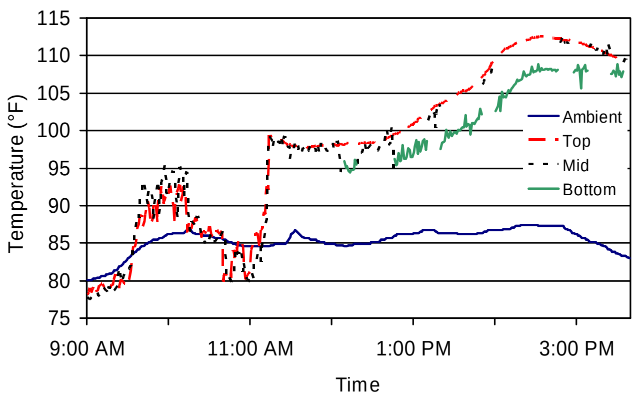


Figure 92. STL-2 curing temperatures

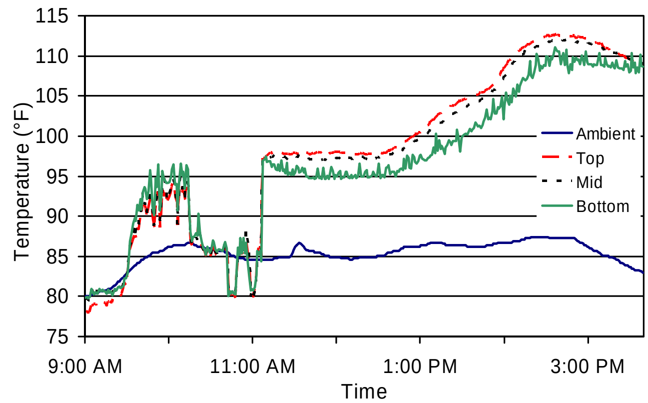


Figure 93. STL-3 curing temperatures

SRA Topping

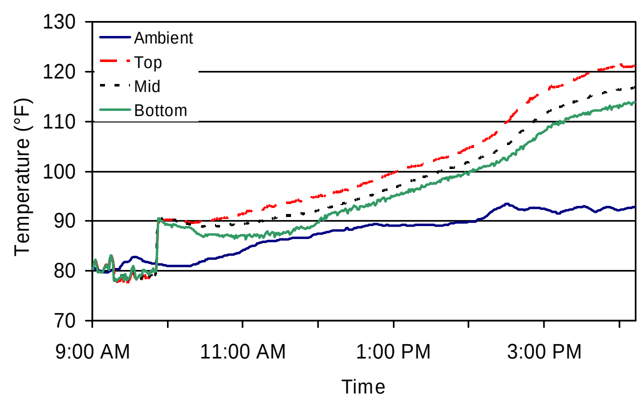


Figure 94. SRA-1 curing temperatures

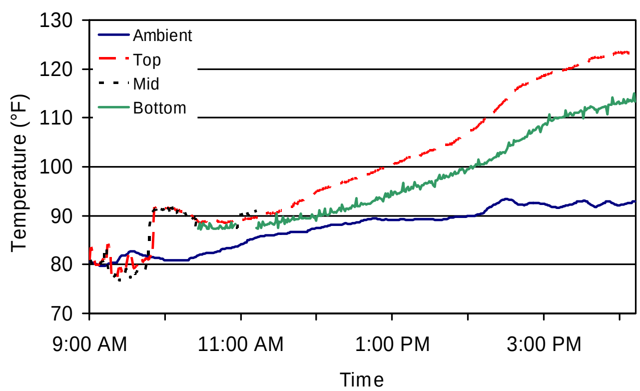


Figure 95. SRA-2 curing temperatures

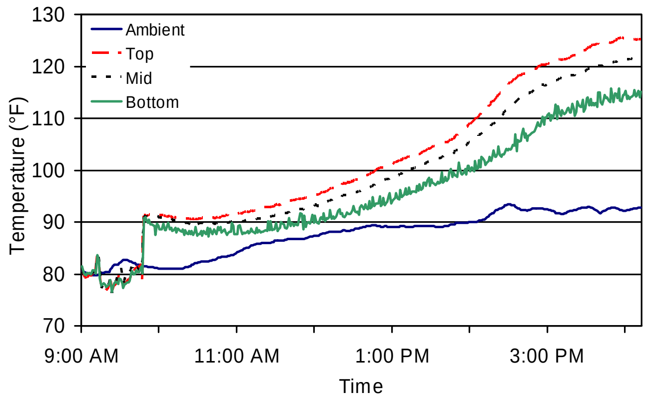


Figure 96. SRA-3 curing temperatures

Control Topping

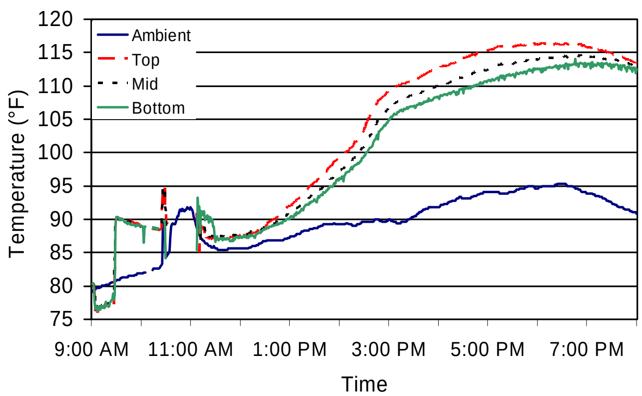


Figure 97. CTL-3 curing temperatures

APPENDIX F
CONSTRUCTION DRAWINGS

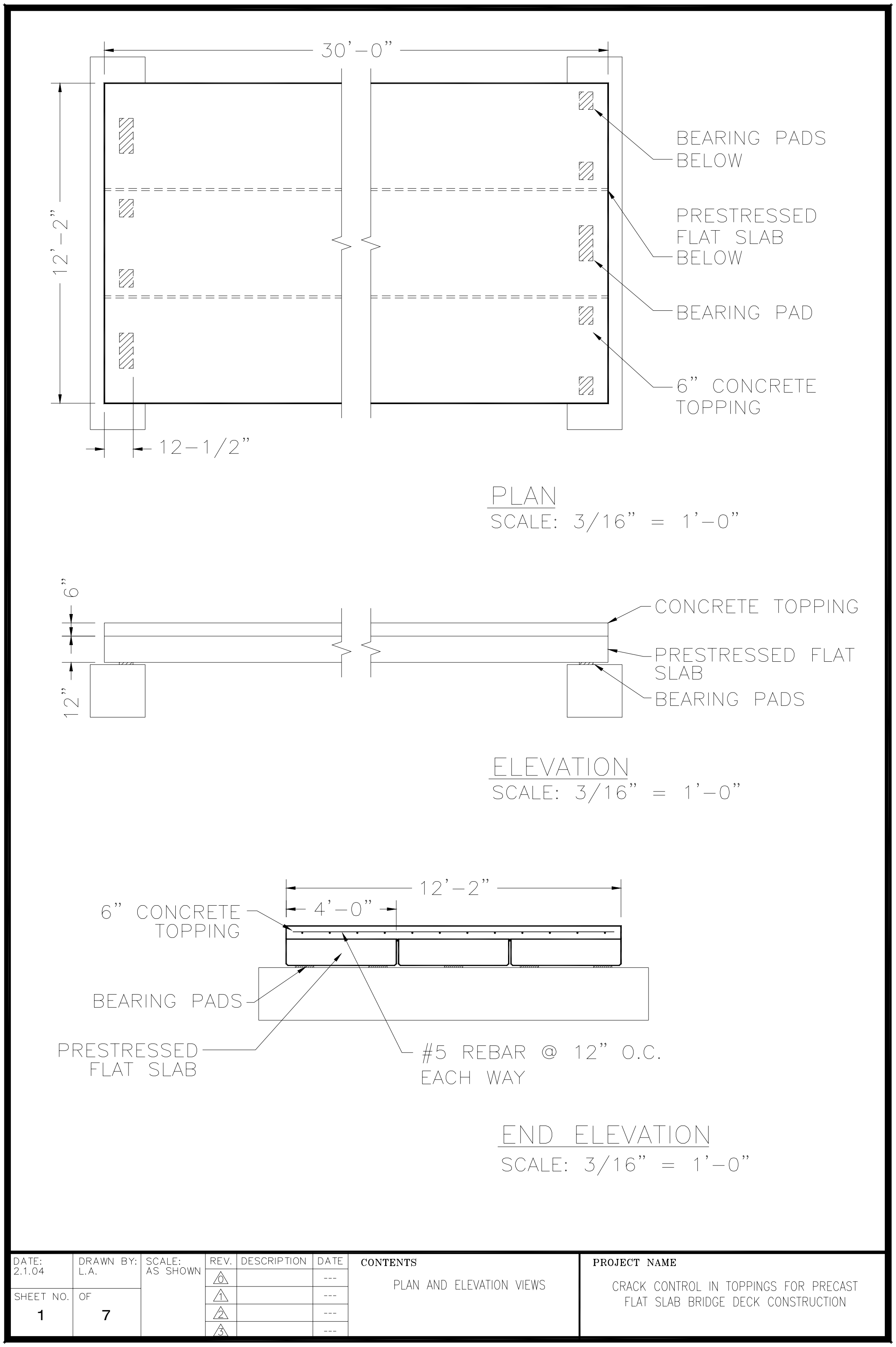


Figure 98. Plan and elevation views of specimens

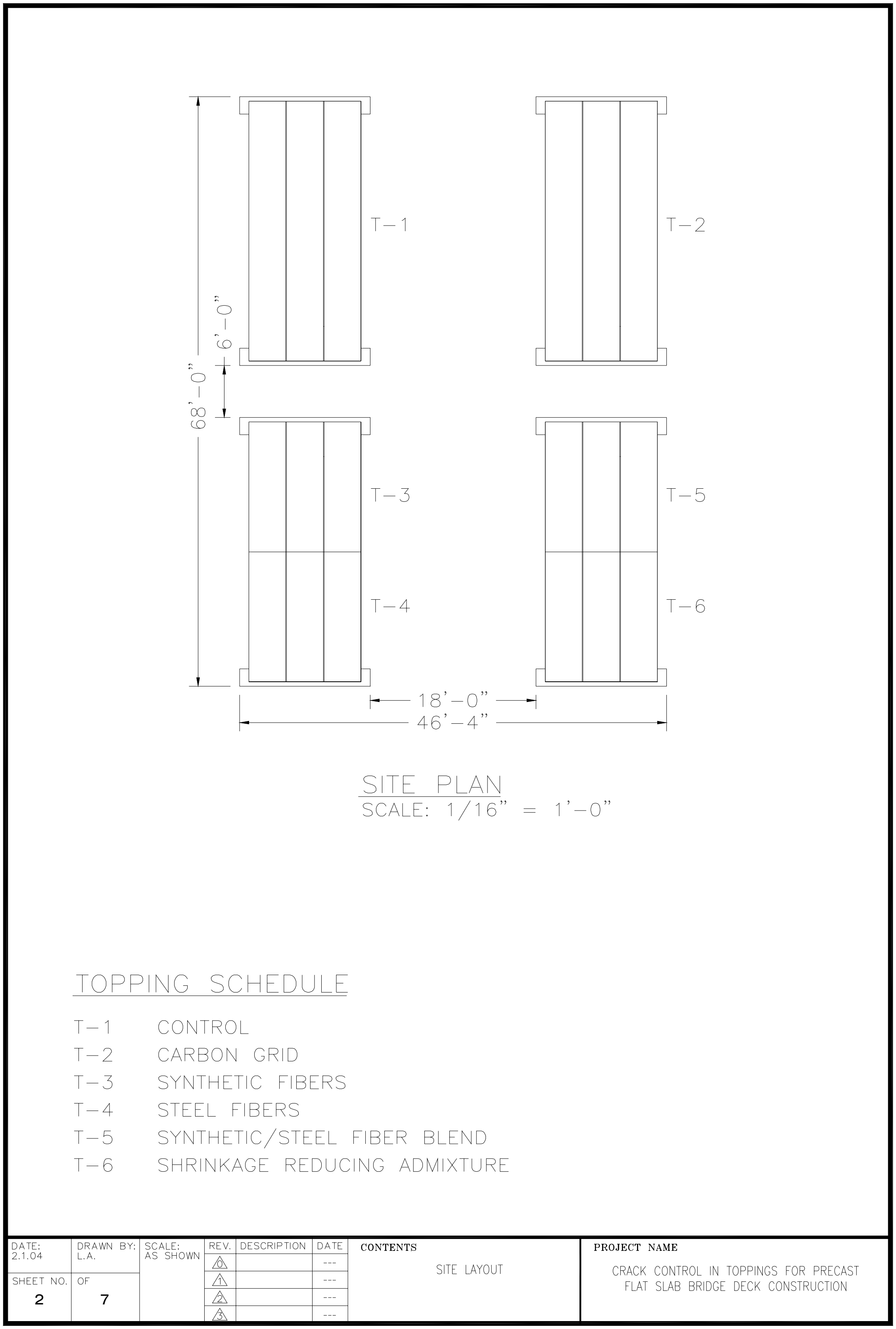
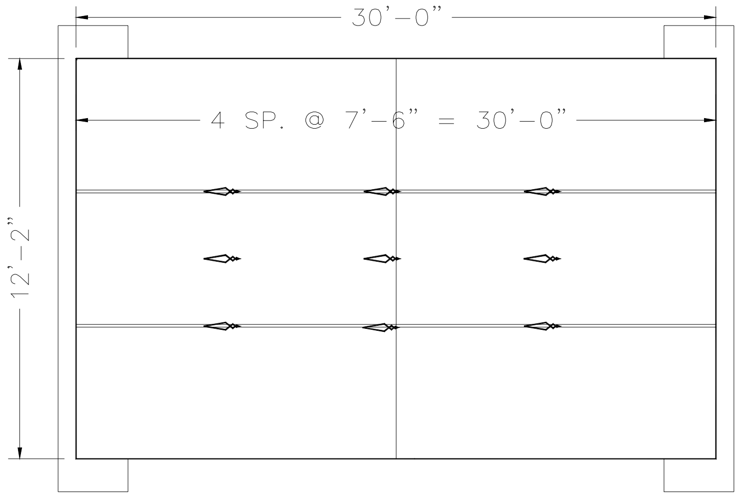


Figure 99. Site layout of specimens



LEGEND

THERMOCOUPLES FOR T-3 THRU T-6

THERMOCOUPLES FOR T-1 & T-2

INSTRUMENTATION PLAN

SCALE: N.T.S.

INSTRUMENTATION & TESTING NOTES:

1 WEATHER CONDITIONS WILL BE MONITORED FOR FIRST 7 DAYS & AT A PERIODIC RATE THEREAFTER.

2 CONCRETE TOPPING TEMPERATURE WILL BE MONITORED AT PRECAST/TOPPING CONTACT AREA , AT MID-DEPTH, AND AT SURFACE FOR FIRST 7 DAYS.

3 CRACKS WILL BE LABELED, DATED, & MEASURED (LENGTH & WIDTH) FOR FIRST 7 DAYS AND AT A PERIODIC RATE THEREAFTER.

4 CONCRETE CYLINDER TESTING WILL BE PERFORMED ON PRECAST FLAT SLAB CONCRETE AND ON ALL TEST SPECIMEN CONCRETE.

5 SHRINKAGE TESTS WILL BE PERFORMED ON CONCRETE TOPPINGS USING A RESTRAINED RING TEST.

DATE: 2.1.04	DRAWN BY: L.A.	SCALE: AS SHOWN	REV.	DESCRIPTION	DATE	CONTENTS	PROJECT NAME
			0		---	INSTRUMENTATION	CRACK CONTROL IN TOPPINGS FOR PRECAST FLAT SLAB BRIDGE DECK CONSTRUCTION
SHEET NO. 3	OF 7		1		---		
			2		---		
			3		---		

Figure 100. Instrumentation and testing notes

CONCRETE PLACEMENT & FINISHING NOTES:

NOTES TAKEN FROM FDOT'S 2004 EDITION STANDARD SPECIFICATIONS FOR ROAD & BRIDGE CONSTRUCTION, SECTION 350.

1 DISTRIBUTE CONCRETE ON THE FLAT SLABS TO SUCH DEPTH THAT, WHEN IT IS CONSOLIDATED AND FINISHED, THE SLAB THICKNESS REQUIRED BY THE PLANS WILL BE OBTAINED AT ALL POINTS AND THE SURFACE WILL AT NO POINT BE BELOW THE DEPTH SPECIFIED FOR THE FINISHED SURFACE.

2 DEPOSIT THE CONCRETE ON THE FLAT SLABS IN A MANNER THAT WILL REQUIRE AS LITTLE REHANDLING AS POSSIBLE.

3 IMMEDIATELY AFTER PLACING THE CONCRETE, STRIKE-OFF, CONSOLIDATE, AND FINISH IT TO PRODUCE A SMOOTH FINISHED PAVEMENT.

4 PERFORM THE SEQUENCE OF OPERATIONS AS FOLLOWS: STRIKE-OFF; CONSOLIDATION; SCREEDING; FLOATING; REMOVAL OF LAITANCE; STRAIGHTEDGING; AND FINAL SURFACE FINISH.

5 FOR PURPOSES OF THIS PROJECT, FINISHING MAY BE DONE BY HAND METHODS IF FINISHING MACHINES ARE NOT AVAILABLE.

STRIKE-OFF & SCREEDING: USE A SCREED THAT IS SUFFICIENTLY RIGID TO RETAIN ITS SHAPE AND IS AT LEAST 2 FEET LONGER THAN THE MAXIMUM WIDTH OF THE STRIP TO BE SCREEDED. MOVE THE SCREED FORWARD ON THE FORMS WITH A COMBINED LONGITUDINAL AND TRANSVERSE SHEARING MOTION. IF NECESSARY, REPEAT THIS UNTIL THE SURFACE IS OF UNIFORM TEXTURE, TRUE TO GRADE AND CROSS-SECTION, AND FREE FROM POROUS AREAS.

CONSOLIDATION: USE HAND-OPERATED SPUD-TYPE VIBRATORS TO CONSOLIDATE.

FLOATING: USE LONG-HANDLED FLOATS TO FLOAT THE CONCRETE. TAKE THE NECESSARY CARE TO AVOID DEPRESSIONS OR RIDGES DURING THIS OPERATION.

CONCRETE FORM WORK & CURING NOTES:

NOTES TAKEN FROM FDOT'S 2004 EDITION STANDARD SPECIFICATIONS FOR ROAD & BRIDGE CONSTRUCTION, SECTION 350.

1 AFTER COMPLETING THE FINISHING OPERATIONS AND AS SOON AS THE CONCRETE HAS HARDENED SUFFICIENTLY APPLY A FDOT APPROVED CURING COMPOUND.

2 APPLY A CURING COMPOUND TO THE SURFACE IN A SINGLE COAT, CONTINUOUS FILM, AT THE MINIMUM RATE OF 0.005 GAL/FT^2, BY A MECHANICAL SPRAYER.

3 FORM WORK MAY BE REMOVED UNTIL THE CONCRETE HAS SET FOR AT LEAST 12 HOURS. AFTER REMOVING THE FORMS, IMMEDIATELY CURE THE SIDES OF THE SLAB IN THE SAME MANNER AS THE SURFACE OF THE TOPPING.

DATE: 2.1.04	DRAWN BY: L.A.	SCALE: AS SHOWN	REV.	DESCRIPTION	DATE	CONTENTS	PROJECT NAME
SHEET NO. 4	OF 7		0		---	NOTES	CRACK CONTROL IN TOPPINGS FOR PRECAST FLAT SLAB BRIDGE DECK CONSTRUCTION
			1		---		
			2		---		
			3		---		

Figure 101. Concrete placement, finishing, and curing notes

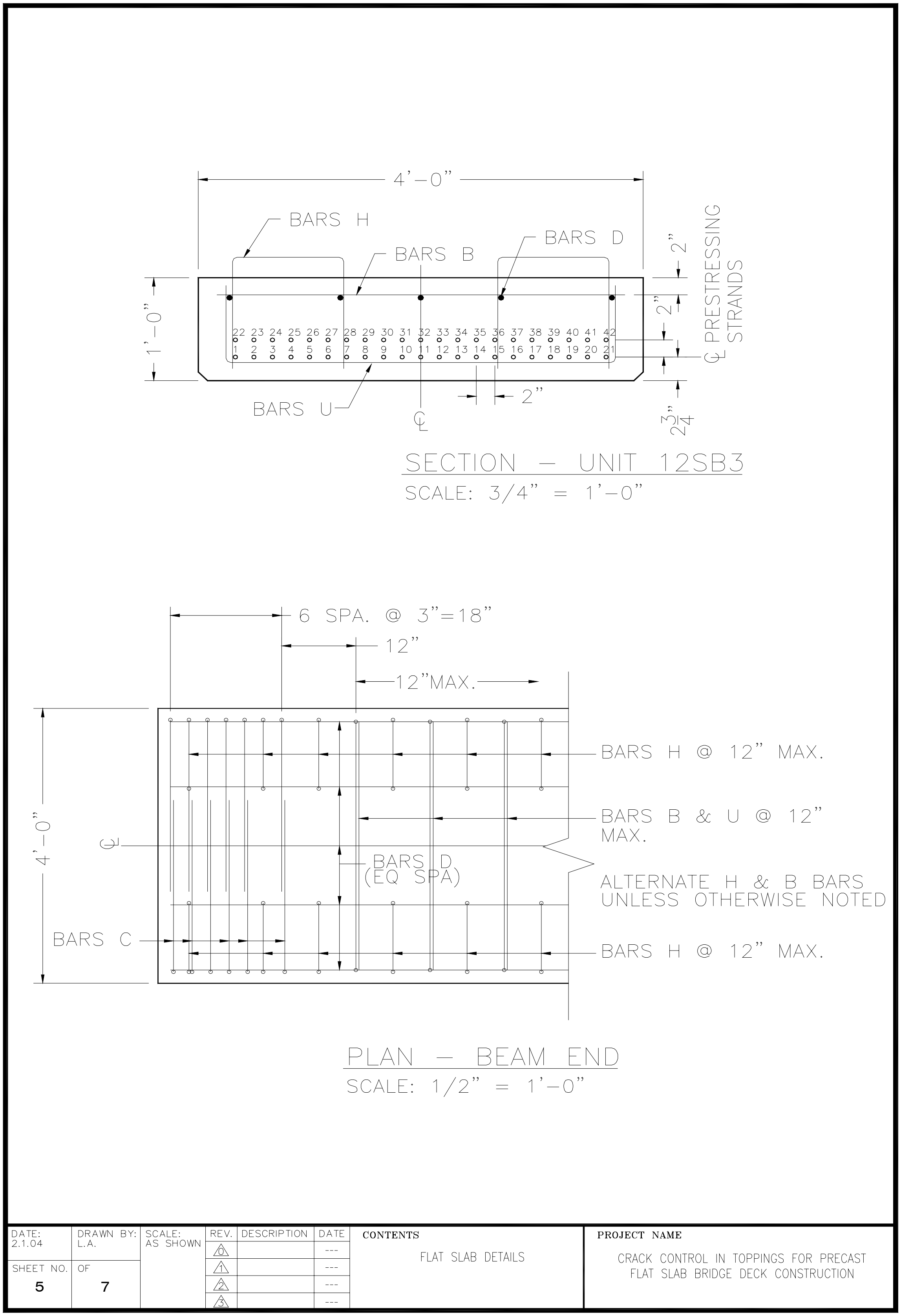
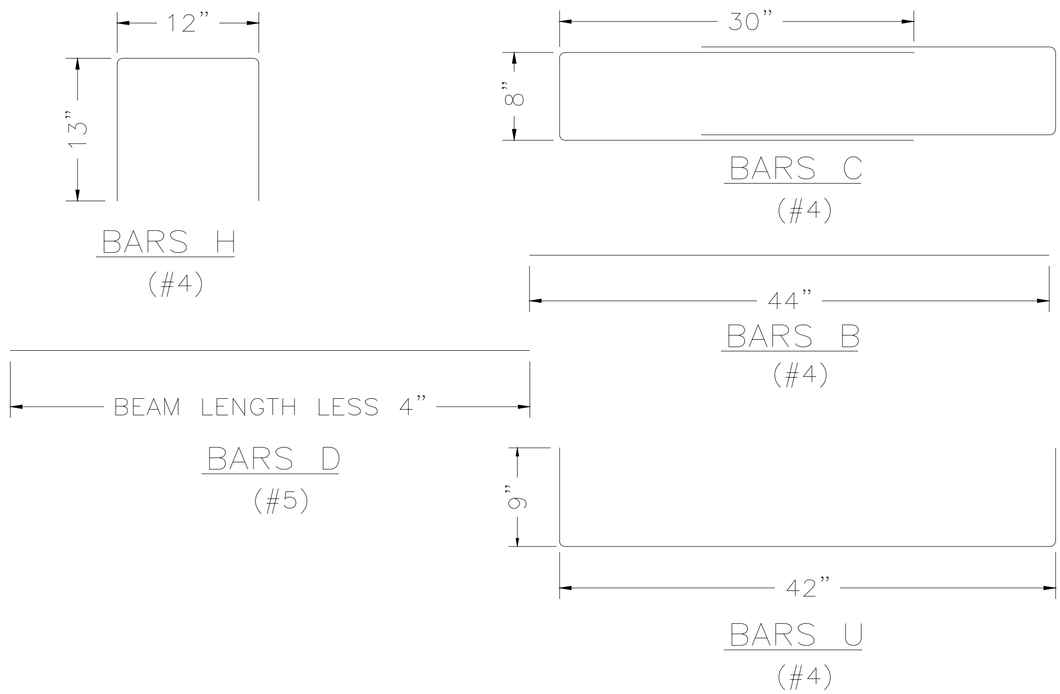


Figure 102. Flat slab detail drawings



BEAM TYPE	BEAM LENGTH	PRESTRESSING STRANDS		DEBONDED PRESTRESSING STRANDS			STRAND
		LOCATION MARK No.	LINE QTY.	LOCATION MARK No.	DISTANCE FROM END	LINE QTY.	TOTAL QTY.
12SB3	30'	1,3,5,7,9	5	10,12	3'	2	12
		13,15,17,19,21	5				

NOTES:

1 ALL REINFORCING STEEL TO BE ASTM A615/A 615M GRADE 420 DEFORMED BARS.

2 ALL PRESTRESSING STRANDS TO BE ASTM A416, GRADE 270 KSI, 1/2" LOW RELAXATION STRANDS.
INITIAL PULL IS 31 KIPS PER STRAND.

3 CONCRETE SHALL BE TYPE IV AND IN ACCORDANCE WITH SPECIFICATIONS SECTION 346.
THE 28 DAY CONCRETE STRENGTH TO BE f'c = 5.5 KSI FOR SLAB BEAMS. THE RELEASE STRENGTH TO BE f'ci = 4.5 KSI.

4 THESE SLABS ARE DESIGNED FOR A 6" THICK COMPOSITE CONCRETE OVERLAY. THE 28 DAY STRENGTH OF THE OVERLAY TO BE f'c = 4.5 KSI.

5 EDGES OF SLAB BEAM TO BE STRAIGHT.

6 TOP SURFACE OF SLAB BEAM TO BE ROUGHENED.

DATE: 2.1.04	DRAWN BY: L.A.	SCALE: AS SHOWN	REV.	DESCRIPTION	DATE	CONTENTS	PROJECT NAME
SHEET NO. 6	OF 7		0		---	FLAT SLAB DETAILS & NOTES	CRACK CONTROL IN TOPPINGS FOR PRECAST FLAT SLAB BRIDGE DECK CONSTRUCTION
			1		---		
			2		---		
			3		---		

Figure 103. Flat slab reinforcement details

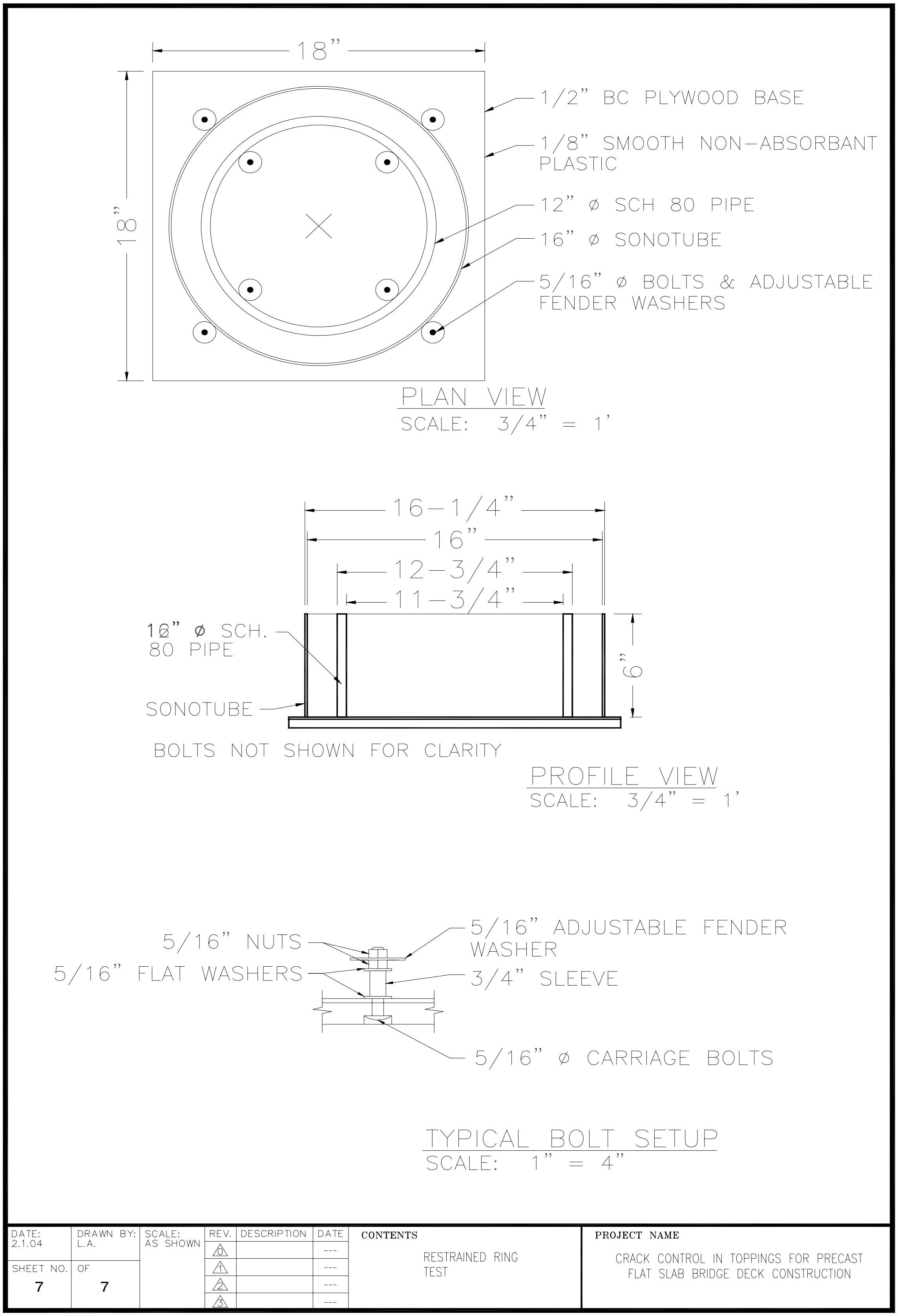


Figure 104. Restrained ring test fabrication drawing

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BIOGRAPHICAL SKETCH

Lazaro Alfonso was born on July 4^{th}, 1974, in Weehawken, New Jersey to parents Lazaro M. and Grisel Alfonso. In 1981 he moved to Miami, Florida. After graduating high school, he practiced as a licensed contractor in South Florida while attending Miami-Dade Community College where he earned an Associate in Arts Degree in 1996. He was admitted to the College of Engineering at the University of Florida in 2001. After graduating with a bachelor's degree in civil engineering in 2003, he entered graduate school at the University of Florida in the department of Civil and Coastal Engineering. After earning a Master of Engineering degree with emphasis in structures in May 2005, he will work at a structural engineering consulting firm in West Palm Beach, Florida.