DESIGN AND CONSTRUCTION OF A LIGHTWEIGHT CONCRETE BRIDGE SUPERSTRUCTURE FOR NCDOT

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ABSTRACT

In response to growing interest in the use of lightweight concrete for bridges, the North Carolina Department of Transportation (NCDOT) selected a two-span bridge for which the entire superstructure would be constructed using lightweight concrete. This project had two firsts for the state: the first use of "sand lightweight concrete" for prestressed concrete girders and the first use of "all lightweight concrete" for the deck and barriers. The prestressed concrete girders were designed using "sand lightweight" concrete with a density of 123 pcf and the deck and parapet were designed using "all lightweight" concrete with a fresh density of 105 pcf and an equilibrium density of 100 pcf. The preliminary design was prepared by an engineer working for the local lightweight aggregate supplier and the final design and plans were prepared by the Department.

The paper reports on the design and construction of this project. Material properties used for design were based on previous test data. Results of design calculations, which were performed using a conventional commercial design program that implemented design factors for lightweight concrete, are reported. Designs using lightweight and normal weight concrete are compared. Material test results, project special provisions, and several lessons learned are presented.

Keywords: Lightweight concrete, prestressed concrete, girders, deck, parapet, design, density, compressive strength, modulus of elasticity, tensile strength, permeability

INTRODUCTION

The North Carolina Department of Transportation (NCDOT) completed a two-span bridge that was used as a demonstration project for the first sand lightweight concrete prestressed concrete girders (lightweight coarse aggregate with conventional sand, with a 123 pcf fresh density) and the first all lightweight concrete deck and parapets (lightweight coarse and fine aggregate, with a 105 pcf fresh density and a 100 pcf equilibrium density) to be used in the state. The Department designed the bridge with assistance from the local lightweight aggregate supplier. This paper discusses girder and deck design, including assumed material properties; a comparison of lightweight and normal weight concrete designs; special provisions; construction and material property test results; and lessons learned.

This paper is an abridged version of a paper was originally presented at the 32nd Annual International Bridge Conference®, in Pittsburgh, PA (USA), June 7-11, 2015. [1] This paper is more focused on issues related to prestressed concrete girders and has been revised and expanded in some areas.

PROJECT DESCRIPTION

The bridge being replaced in this project carries Martin Luther King, Jr. Drive over US 29/ US 70/ I-85 Business on the edge of Thomasville in Davidson County, NC. The bridge carries a two lane road over a four-lane divided highway. The road was closed and traffic was detoured during construction of the new bridge on the same alignment as the existing bridge.

The existing four-span structure was replaced with a two span structure of approximately the same length. Initial preliminary plans for the replacement bridge were developed by NCDOT and used four 63-in.-deep NCDOT Modified Bulb Tee prestressed concrete girders for each span. The total bridge length was set at 188’-0”, with two equal spans of 94’-0”. The overall bridge deck width was 34’-7” with a clear roadway width of 32’-0”.

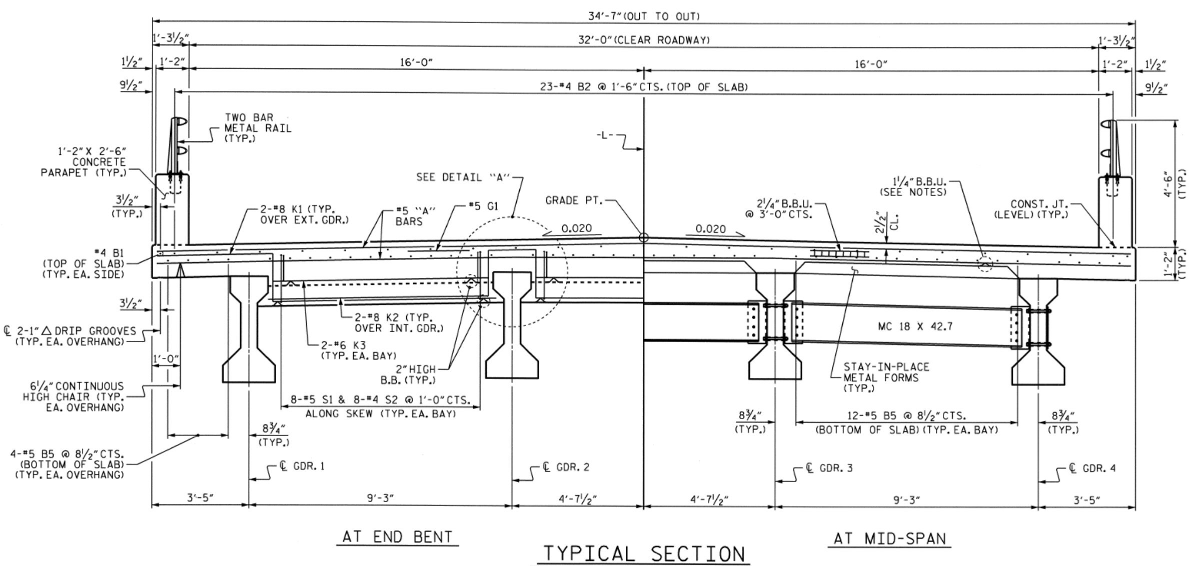
NCDOT asked a bridge engineer working for a local lightweight aggregate supplier to assist in the redesign of the bridge using all lightweight concrete (defined later in paper) for the girders, deck and parapets as a demonstration project. After discussions, it was agreed that the pretensioned girders should utilize sand lightweight concrete because there was no known test data for all lightweight concrete for prestressed girders. The Department requested that the revised design use AASHTO Type III prestressed concrete girders to demonstrate possible savings from using lightweight concrete. The new design still used four girder lines and the same deck width. The total bridge length was reduced slightly to 177’-5¾”, with two equal spans of 88’-8 7/8”. Using NCDOT standard deck design tables, the total deck thickness was set at 8.75 in., which included a ¼ in. integral wearing surface that was disregarded when computing structural section properties. No wearing course was applied to the concrete deck.

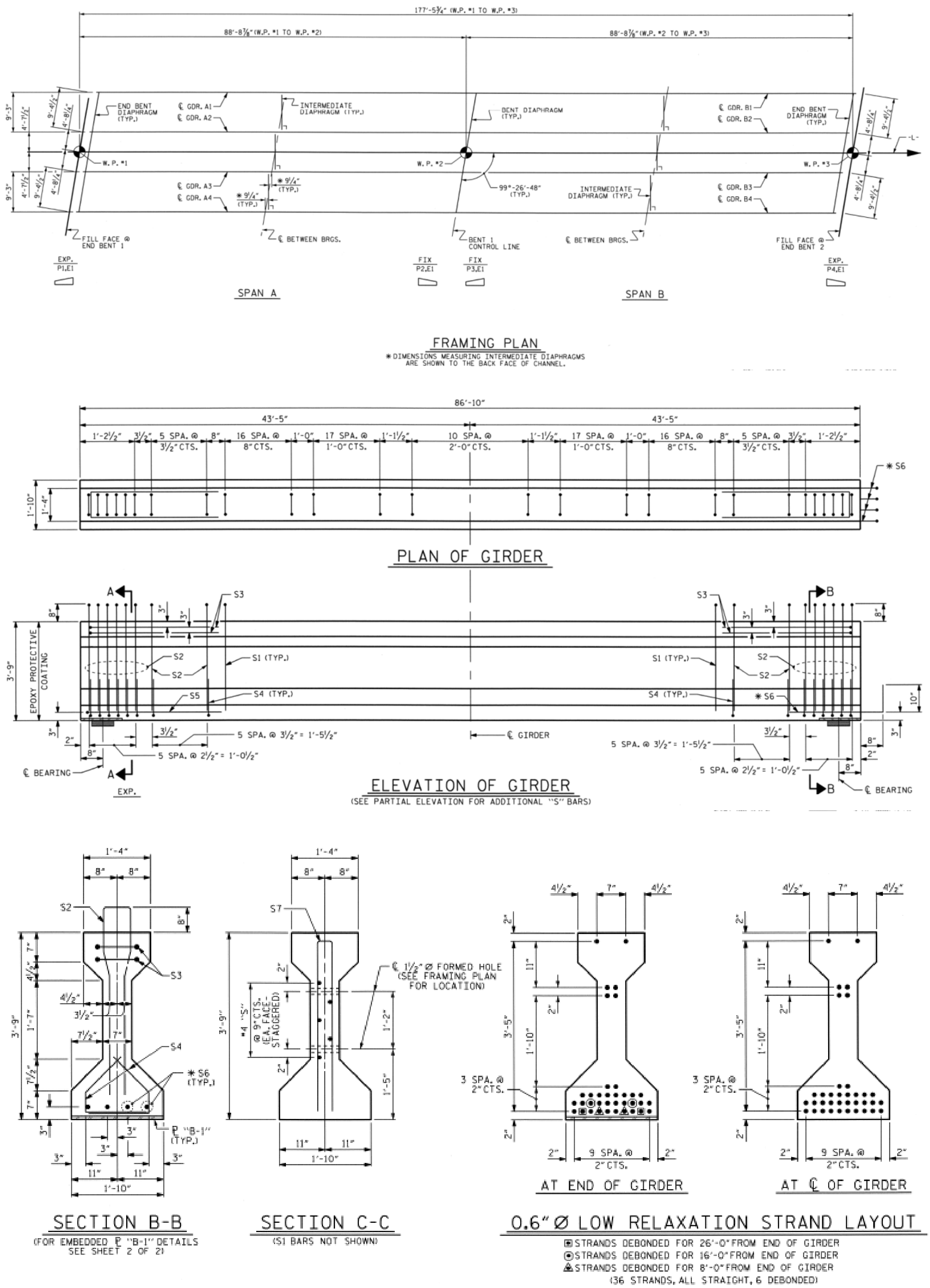
SUPERSTRUCTURE DESIGN

Preliminary superstructure design was performed by the lightweight aggregate supplier working in cooperation with NCDOT. Design was performed in accordance with NCDOT standard practice and the current edition of the *AASHTO LRFD Bridge Design Specifications* at the time of design (2010). A complete package of calculations and specifications were submitted to the Department which was used as the basis to prepare final design calculations and contract plans. Several details from the final plans are shown below.

The deck and parapets were designed using "all lightweight concrete" (ALWC), which is concrete in which all aggregate (both fine and coarse) is lightweight aggregate (LWA) and has the lowest possible density. The prestressed girders were designed using "sand lightweight concrete" (SLWC), which is concrete in which the coarse aggregate is lightweight but the fine aggregate (sand) is normal weight aggregate. Sand lightweight concrete has been more commonly used in recent years and has been used successfully for pretensioned girders and spliced post-tensioned girders. All lightweight concrete has been used infrequently in recent years, but holds promise for applications where maximum weight reduction is desirable. Special aspects of the design of the superstructure of this bridge related to the use of lightweight concrete are described in this paper.

Foundation and substructure designs were completed with the initial preliminary design of the bridge and were not revised because of schedule and other constraints. Savings in foundation and substructure costs may have been realized if they were redesigned accounting for the reduced weight of the superstructure. Cost comparisons between the initial and final designs were not conducted.




FRAMING PLAN
* DIMENSIONS MEASURING INTERMEDIATE DIAPHRAGMS ARE SHOWN TO THE BACK FACE OF CHANNEL.
SPAN A
SPAN B
PLAN OF GIRDER
ELEVATION OF GIRDER
(SEE PARTIAL ELEVATION FOR ADDITIONAL "S" BARS)
SECTION B-B
(FOR EMBEDDED ℙ "B-1" DETAILS SEE SHEET 2 OF 2)
SECTION C-C
(S1 BARS NOT SHOWN)
AT END OF GIRDER
AT ℄ OF GIRDER
0.6" Ø LOW RELAXATION STRAND LAYOUT
STRANDS DEBONDED FOR 26'-0" FROM END OF GIRDER
STRANDS DEBONDED FOR 16'-0" FROM END OF GIRDER
STRANDS DEBONDED FOR 8'-0" FROM END OF GIRDER
(36 STRANDS, ALL STRAIGHT, 6 DEBONDED)

MATERIAL PROPERTIES

The material properties used for design are presented in this section.

Compressive Strength

For design, the minimum 28 day compressive strength (f'_c) for the deck and parapet concrete was 4,000 psi. However, for construction, NCDOT requires the minimum compressive strength for deck concrete to be 4,500 psi at 28 days to provide an extra factor of safety against under-strength concrete in the deck.

A design using a 28 day concrete compressive strength (f'_c) of 8.0 ksi was very slightly overstressed in tension at midspan for the Service III Limit State, so the compressive strength was increased to 8.5 ksi to avoid a rating factor less than one for the new bridge.

To satisfy design requirements, the minimum concrete compressive strength at transfer (f'_{ci}) was set at 6.7 ksi.

Density

Specified densities for the two types of lightweight concrete (LWC) used in the design are shown in Table 1. The concrete densities specified for the project were determined by the local lightweight aggregate supplier after evaluating probable concrete mix designs for the different types and strengths of concrete. Following typical practice, the density of the concrete used for computing dead loads was 5 pcf greater than the densities shown in Table 1 to account for the increased density with steel reinforcement.

Table 1 - Specified Concrete Densities

	Type of LWC	Fresh Density (pcf)	Equilibrium Density (pcf)
Girder	SLWC	123	--
Deck & Parapets	ALWC	105	100

Lightweight aggregate has a higher absorption than normal weight aggregate and is prewetted prior to batching to control the behavior of the fresh concrete mixture. With time, some of the moisture absorbed in the aggregate migrates out of the concrete, reducing the concrete density to what is called the “equilibrium density” which is defined in ASTM C567.

The equilibrium density for the deck and parapet concrete is listed in Table 1. This density is used for design for these elements. An equilibrium density was not specified for girder concrete because calculations indicate that the reduction in density with time is small for the girder concrete. The girder is also precast, so handling loads must be computed using the fresh density rather than equilibrium density since handling occurs before drying occurs. Finally, the girder mix design has a low w/cm, and therefore a low permeability, slowing migration of moisture from the concrete. Therefore, the fresh density was used for the design density for the girder concrete.

The fresh density was specified for both types of concrete because it is needed for quality control during concrete placement.

Modulus of Elasticity

To obtain a modulus of elasticity for use in design for both types of LWC, test data for similar concrete mixtures was used to evaluate LRFD Eq. 5.4.2.4-1 and to determine the aggregate correction factor, K_1. The evaluation indicated that K_1 should be 1.0 for ALWC and 0.85 for SLWC. Computed values for modulus of elasticity used for design were:

- ALWC deck & parapet at 28 days 2,087 ksi
- SLWC girders at transfer 3,132 ksi
- SLWC girders at 28 days 3,528 ksi

DECK SHEAR DESIGN CHECK

Shear is not checked for bridge decks of this type. However, the LRFD Specifications include two factors that reduce the shear capacity of lightweight concrete elements. Therefore, the deck shear was checked for this bridge and found to be have adequate shear capacity. See Reference 1 for further discussion.

GIRDER DESIGN

Design for shear and flexure were performed using a commercial design program that had fully implemented the LRFD design requirements for LWC. Prestress losses were computed using simplified loss estimation procedures with gross section properties. Loads were determined according to NCDOT practice, including composite dead loads being distributed equally to all girders. Using NCDOT standard practice, girders were designed as simple spans but were detailed as continuous for live load.

The final girder design used 36 - 0.6-in.-diameter seven-wire strands. All strands were straight, with 3 pairs of strands debonded for lengths up to 26 ft. Exterior girders controlled the design. The same strand pattern satisfied design requirements for interior and exterior girders. Strands were added to reduce girder camber as discussed later in this paper. Stirrups were provided to satisfy vertical and horizontal shear requirements as well as end splitting provisions.

Girder Camber and Deflection

Estimated girder cambers are shown in Table 2.

Table 2 - Estimated Cambers for SLWC Girder

	Transfer (in.)	Erection (in.)	Final (in.)
Interior Girder	3.23	5.74	3.88
Exterior Girder	3.23	5.74	4.04

Erection and final cambers in Table 2 were computed using the following NCDOT camber multipliers.

- At erection: 1.80 for prestress effects
  1.85 for dead load
- Final conditions: 2.20 for prestress effects
  2.40 for dead load

Adding Strands to Control Camber

The reduced density and modulus of elasticity of the SLWC girder resulted in the camber being significantly greater than for a normal weight concrete girder. The design was modified as described below to reduce the camber and the large build-up that it required.

An initial design with 32 strands was obtained using standard procedures to satisfy design criteria using the minimum number of strands at the greatest eccentricity. However, the number of debonded strands required to meet stress limits at release exceeded the LRFD and NCDOT limits so this design was not allowed. A design with 34 strands was then developed, positioning the additional pair of strands to satisfy stresses and to minimize the final girder camber. The lowest camber achievable with 34 strands was 4.79 inches, which would require a 6 inch buildup. A design with 36 strands was then considered to further reduce the camber. With this design, a final camber of 4.04 inches was achieved, which would allow a buildup of 5 inches. This strand pattern was used as the final design. A design with 38 strands reduced final camber to 3.36 inches, which would allow a 4.5 inch build up. However, the cost of two added strands would be greater than the savings from the ½ inch reduction in buildup. Further evaluation revealed that increasing the number of debonded strands to the maximum limit reduced the camber by an additional ½ inch. However, this approach was not used.

COMPARISON OF LWC AND NWC DESIGNS

After completion of the preliminary design, a girder was designed using normal weight concrete (NWC) for the superstructure. The two designs are compared in Table 3.

Table 3 - Comparison of LWC and NWC Designs

	LWC	NWC	Difference	Percent Diff.
	A	B	A – B	(A-B)/B (%)
General design parameters:				
No. strands	36	40	-4	-10%
f'_{ci} (ksi)	6.7	7.8	-1.1	-14%
f'_c (ksi)	8.5	9.5	-1.0	-11%
Cambers for exterior girders:				
Transfer (in.)	3.23	2.42	0.81	+33%
Erection (in.)	5.74	4.30	1.44	+33%
Final (in.)	4.04	2.77	1.27	+46%
Build up (in.)	5	3.75	1.25	+33%
Reaction at interior bent:				
Total dead load (kips)	306.6	385.1	-78.5	-20%

For general design parameters and the reaction at the interior bent, the LWC design resulted in a reduction in quantity compared to the NWC design. This should result in a more economical design. However, in this case, the reduced superstructure dead load was not considered in the design of the substructure and foundations.

The cambers and build up were increased for LWC compared to NWC, so the concrete quantity for the deck would be increased slightly.

SPECIAL PROVISIONS

NCDOT has used a project special provision (PSP) for SLWC for bridge decks for many years. A review of that document revealed that some additional provisions were needed. Therefore, new special provisions were developed for both the sand lightweight girder concrete and the all lightweight deck and parapet concrete.

In general, requirements for LWC are the same as for NWC. Lightweight aggregates should be required to meet the same physical requirements used for conventional concrete aggregates. The major difference is that a maximum concrete density is specified for LWC. The main items addressed in the special provisions include:

- Use AASHTO M 195 (ASTM C330) for aggregate grading requirements for LWA
- Testing method for concrete density
- Volumetric method for testing for air content rather than the pressure method
- Modified procedure for freeze thaw testing, which is given in AASHTO M 195
- Requirements for moisture conditioning of LWA
- Requirement for contractor have a LWA supplier representative present at the batch plant and/or project site for the first placement of lightweight concrete.

PROJECT ADVERTISEMENT AND BIDDING

The project was included in the NCDOT August 2011 letting. Seven bids were received with a low bid of $1,294,907.25, which was 12.2% below the Engineer's Estimate of $1,474,986.73. The high bid was $1,413,628.87.

Quantities and bid tabs for the SLWC pretensioned girders and the ALWC deck and parapets are shown in Table 4 for the low bidder along with the high and low bids submitted.

Table 4 - Bid Tabs for LWC Items for All Bidders (Source: NCDOT Bid Tabs)

	45" Prestr. Concrete Girder (Sand LWC)	Reinforced Concrete Deck Slab (All LWC)	1'-2" x 2'-6" Concrete Parapet (All LWC)
Quantity	694.7 LF	6065 SF	350.6 LF
Low bidder	$200.00	$20.00	$50.00
Lowest bid for quantity	$175.00	$20.00	$50.00
Highest bid for quantity	$286.75	$33.15	$81.97

CONSTRUCTION

Construction began in early 2012 and was completed in June 2012.

GIRDERS

The prestressed girders were manufactured by Standard Concrete Products at their plant in Savannah, GA. The plant had previous experience with manufacturing prestressed girders using sand lightweight concrete for a VDOT project. The company had also manufactured lightweight concrete girders at their plants in Atlanta, GA, and near Mobile, AL.

Girder Mix Design

The QC department at the plant developed a sand LWC mix design that was approved by NCDOT. The mix design compressive strength was 9,000 psi, although the project specifications required 8,500 psi. Details of the mix design can be found in Reference 1.

The computed plastic density for the mix was 118.6 pcf, which satisfied the specified maximum density of 123 pcf. The equilibrium density, calculated using the mix proportions, was 116.5 pcf, or about 2 pcf less than the fresh density. This confirmed the main reason that only the fresh density was specified for the girder concrete.

Girder Fabrication

The four girders required for each span were cast at the same time in a single bed. LWA supplier representatives were present for pouring the first line of girders. Concrete was batched in the on-site batch plant. Photos of Span A girders are shown in Figure 1.

Figure 1 - Top Surface of Girder after Finishing and Girders after Removal from Bed

The maximum concrete density recorded during pouring both girder lines was 119.7 pcf, less than the specified fresh density of 123 pcf.

The minimum concrete compressive strength at transfer of 6,700 psi was achieved at 21 and 23 hours after concrete placement for Span A and Span B girders, respectively. Strands were detensioned shortly after achieving the release strength. Cambers were then measured and the girders were removed from the bed and placed in the yard for finishing and storage.

Average concrete compressive strengths at 7 days were 8,800 and 8,180 psi for Spans A and B, respectively. Average concrete compressive strengths at 28 days were 10,535 and 10,040 psi for Spans A and B, respectively, which were both well above the specified compressive strength of 8,500 psi.

Girder Erection

Girders were erected at night with one span being placed each night. The computed weight of each girder was nearly 22 tons based on a concrete density of 128 pcf that included a 5 pcf reinforcement allowance. A NWC girder would have weighed approximately 26 tons.

The crane operator appreciated the reduced weight of the girders, stating that he often works near crane capacity limits, so even a small reduction in weight was helpful for him.

DECK AND PARAPETS

The deck and parapets were constructed using an ALWC mixture that was supplied by a ready mix plant. Contract plans indicated that the total volume of ALWC for the deck was 224.6 CY. The total volume of ALWC required for the two parapets was 42 CY.

Deck and Parapet Mix Design

The ready mix concrete supplier was familiar with using sand LWC. They developed an ALWC mix design that was approved by NCDOT. Details of the mix design can be found in Reference 1.

The computed fresh density for the mix was 100.9 pcf, which satisfied the specified maximum density of 105 pcf. The equilibrium density, calculated using the mix proportions was 96.6 pcf, or 4.3 pcf less than the fresh density. The difference is greater than what was computed for the SLWC because there is more LWA with absorbed water in the ALWC.

Placing Deck and Parapet Concrete

Contract plans showed the deck being placed in two pours, with an optional pouring sequence with three pours. The contractor elected to use the optional sequence with three pours, where the final pour was the closure diaphragm and a portion of the deck each side of the diaphragm.

A trial placement with the ALWC deck mix was performed at the ready mix concrete plant several days before the first scheduled deck pour.

Two deck pours were attempted using a concrete pump to deliver the concrete. However, the air content and density were out of range so the placement was halted. Subsequent placements using conveyors were successful. These difficulties are discussed later in the paper. A power screed was used to finish the deck concrete.

The contractor elected to form and pour the rectangular parapets rather than to use slip forming. This avoided adjusting the slump for the deck mix that was also being used for the parapets. The parapets were cast with no issues. A fresh density of 101 pcf was reported.

Compressive strength test results for deck and parapet concrete were available for eight days on which concrete placements were attempted or made. The average 28 day compressive strength for all of these dates was 5,650 psi, with a range in daily averages of 5,210 to 6,580 psi. Therefore, all reported daily average compression test results exceeded the minimum specified compressive strength of 4,500 psi.

Grinding and Grooving of Deck

The surface of the deck was rough in several areas near construction joints. These areas were ground as shown in Figure 2. The only cracks observed in the deck were at construction joints, and these cracks were very tight. . Several tight cracks were also observed in the parapets. Transverse grooves were cut into the entire deck as shown in Figure 4.

Figure 2 – Deck after Grinding at Construction Joint and Deck after Grooving

COMPLETION AND OPENING OF BRIDGE

NCDOT records indicate that the bridge was accepted on June 29, 2012 and the bridge was opened to traffic. Two photographs of the completed bridge about 3 months after completion are shown in Figure 5. Since traffic was on the bridge, a close inspection of the deck and parapets for cracking was not possible.

Figure 3 - Photos of Completed Bridge

MEASURED CONCRETE PROPERTIES

Measured properties are presented in this section for the girder concrete. Data were not available for deck and parapet concrete properties other than the compressive strengths already discussed.

GIRDER CONCRETE PROPERTIES

Density

The girder fabrication section reports that the maximum fresh density for both pours was 119.7 pcf, below the specified maximum of 123 pcf. The computed fresh density from batch quantities was 118.6 pcf, so the measured fresh density was close to the predicted value.

To observe the drying behavior of the SLWC, several 4 x 8 in. cylinders were dried in an oven. After 14 days, the density changed as shown in Table 5. This change is significantly greater than the 5.2 pcf estimated for the mix design using procedures of ASTM C567. Oven drying began two days after the concrete had been batched, so the cement and fly ash would not have had time to fully hydrate or react, so some water that would have eventually been bound by reaction products may have left the concrete.

Five 4 x 8 in. cylinders were also allowed to dry in the laboratory, and three more were allowed to dry outside, with drying beginning at two days after the concrete was batched. Average initial and final densities for these cylinders also appear in Table 5. Final readings for the sets of cylinders were taken after drying for 194 or 196 days. The change in density for air dried cylinders and the computed (theoretical) equilibrium density are similar, as shown in Table 5, showing that the method of ASTM C567 for estimating equilibrium density worked well for the girder concrete mixture.

Table 5 – Change in Density with Drying

	Initial Density (pcf)	Final Density (pcf)	Change in Density (pcf)
Oven Dried			
Theoretical	118.6	113.4	-5.2
Actual	120	111	-9
Air Dried			
Theoretical	118.6	116.5*	-2.1
Indoor	121.0	117.7	-3.3
Outdoor	120.6	118.0	-2.6

* - Equilibrium density

Compressive Strength

Compressive strength test results for girders were discussed in the section "Girder Fabrication."

Tensile Strength

Tensile splitting test results are shown in Table 6. The expected tensile splitting strength was 525 psi, based on the design compressive strength of 8500 psi and using the 0.85 reduction factor for sand LWC. The measured tensile strengths exceeded this value by a wide margin.

Table 6 – Results of Splitting Tensile Tests

Age at Test (days)	No. of Cylinders	Average f_{ct} (psi)	Max. f_{ct} (psi)	Min. f_{ct} (psi)
28	9	720	810	630
56	7	751	923	619

Table 7 shows the average measured splitting tensile stress in terms of the square root of the average concrete compressive strength at the indicated age. Values of the ratio $f_{ct} / \sqrt{f'_c}$ for both ages exceed 6.7, which is the ratio assumed in the design specifications for NWC. This means that the splitting tensile stress for this LWC mixture exceeds the expected value for NWC. If contract documents had specified the splitting tensile strength to be at least $6.7\sqrt{f'_c}$ (which would be 618 psi using the design compressive strength of 8500 psi), the shear reduction factor would have been 1.0 rather than 0.85. [Note: These calculations use psi units while the *AASHTO LRFD Bridge Design Specifications* use ksi units, so the coefficient of 6.7 changes to 1 / 4.7 = 0.21 for ksi units (see Art. 5.8.2.2).]

Table 7 – Compare Splitting Tensile Tests to $\sqrt{f'_c}$

Age at Test (days)	Average f_{ct} (psi)	Average f'_c (psi)	Ratio $f_{ct} / \sqrt{f'_c}$
28	720	9,219	7.50
56	751	10,234	7.42

Modulus of Elasticity

Modulus of elasticity test results are shown in Table 8. Each row presents results for the test of a single cylinder. After testing for modulus, the cylinder was unloaded and tested to failure to obtain the compressive strength. The following modulus of elasticity values were used for design, and were computed using LRFD Equation 5.4.2.4-1 with a K_1 value of 0.85:

- SLWC girders at transfer 3,132 ksi
- SLWC girders at 28 days 3,528 ksi

Measured values are compared to values computed using Equation 5.4.2.4-1 in the *LRFD Specifications* in Table 9. Using a K_1 value of 0.85 to compute the modulus of elasticity, as was assumed for design, the measured modulus was close to, but always greater than, the computed modulus for all cylinders tested. When the measured modulus is compared to the modulus used for design for early and later ages, the measured value is again very close to the design value, with one cylinder falling slightly below the design value. Therefore, the actual behavior, such as cambers and deflections, should be close to that predicted in design.

Table 8 – Results of Modulus of Elasticity Tests

Age at Test (days)	Density (pcf)	Compressive Strength (psi)	Measured E_c (ksi)
2	117.0	7,018	3,174
2	117.0	7,422	3,300
7	116.9	7,997	3,384
7	119.4	7,900	3,285
28	117.0	9,432	3,707
28	117.0	8,726	3,377
56	117.0	9,634	3,656
105	120.2	10,047	3,747

Table 9 – Comparison of Measured, Computed and Design Values of Modulus of Elasticity

Age at Test (days)	Measured E_c (ksi)	Computed E_c (ksi)	Measured vs. Computed	Measured vs. Design
2	3,174	2,975	6.7%	1.3%
2	3,300	3,059	7.9%	5.4%
7	3,384	3,168	6.8%	
7	3,285	3,254	0.9%	
28	3,707	3,448	7.5%	5.1%
28	3,377	3,317	1.8%	-4.3%
56	3,656	3,485	4.9%	3.6%
105	3,747	3,706	1.1%	6.2%

If the default value of $K_1 = 1.0$ had been used in design, the measured modulus of elasticity values would have been from 10 to 15% lower than the assumed modulus. This is well within the range of variation that is anticipated for modulus of elasticity computed using the LRFD equation. Therefore, using the default value of $K_1 = 1.0$ could also be expected to provide behavior predictions that are reasonably close to measured values.

Permeability

Two 4x8 cylinders from one of the girder batches were sent to an independent testing laboratory for permeability testing by ASTM C1202-12. The average test result at 56 days after casting was 1093 coulombs, which indicates low chloride permeability.

MEASURED GIRDER CAMBERS & DEFLECTIONS

Midspan cambers were measured by the girder producer when girders were removed from the bed. For the eight girders, the average initial camber was 2.61 in. with a range of 2.50 to 2.75 in., while the predicted camber was 3.23 in. This data also shows that the initial cambers were very consistent between girders, with a range of only 0.25 in.

Shortly after the girders were erected (about 45 days after casting), the contractor surveyed the girders so they could calculate required deck form elevations. From this data, girder cambers at midspan were computed. Survey shots were taken on the roughened top surface of the girder, so there was some variation in elevation because of surface texture. The average midspan camber after erection was 4.38 in., with a range of 4.14 to 4.56 in., indicating that the cambers were still very consistent between girders. The estimated camber at erection shown on the plans was 5.74 in., so measured cambers were lower than anticipated by about 1.5 in.

At the author’s request, the contractor surveyed the bottom of the girders near midspan using before and after the deck and parapet concrete was placed. While the survey methods did not provide very accurate readings, they were adequate to provide an estimate of the dead load deflection experienced by the girders. Combining this survey data with the top flange survey results allowed an estimate for the remaining girder camber after the deck and parapet had been placed. With all dead load in place, the average midspan camber was 2.79 in, with a range of 2.34 to 3.84 in. The wider variability is most likely due to the methods used in measuring the camber of the deflected girders. The plans showed an estimated final camber of 3.95 in., so all final cambers were less than expected.

Measured and predicted cambers are summarized for the stages of construction in Table 10.

Table 10 – Summary of Measured and Predicted Cambers

Stage	Average (in.)	Range (in.)	Predicted (in.)	Predicted - Average
Initial	2.61	2.50 – 2.75	3.23	0.60
Erection	4.38	4.14 – 4.56	5.74	1.36
Completed	2.79	2.34 – 3.84	3.95	1.16

Based on these values, the elastic deflection from the deck and parapet load was about 1.6 in. The deflection for deck and parapet shown on the plans was 2.8 in. Therefore, the deflection due to dead load was less than expected by slightly more than 1 in. Since test data indicate that the densities and the modulus of elasticity from cylinders were close to design values, these quantities do not appear to be the source of the difference in deflection.

Final cambers shown in the plans were computed using multipliers that anticipated additional deflection with time that had not yet occurred when the final survey was taken. Therefore, comparing the long-term computed values to the measured values right after the bridge was completed is not exactly valid since it is comparing two different conditions. The final camber estimated from the survey data was consistently less than the final camber on the plans by an average of 1.16 in. The additional long-term deflection was very close to 1 in., so the final camber should be close to what was anticipated.

LESSONS LEARNED

Some lessons learned from this project from the perspective of the LWA supplier are presented in this section. Where possible, recommendations for improving future projects are given.

CONCRETE MIX DESIGN

The LWA supplier was not directly involved in the mix design for either type of concrete used in this project. While both mixtures performed well, it is suggested that concrete suppliers should involve the LWA supplier at early stages of mix design. It is recognized that some concrete suppliers may have substantial experience with LWC mixtures for building applications, and they may be able to provide a properly proportioned mixture without assistance from the LWA supplier.

GIRDER WEIGHT

While the design comparison between a LWC and NWC design for this bridge did not show a significant difference in the girder design, there may be instances where the use of LWC girders could be beneficial to the owner and/or contractor. The crane operator was certainly pleased that the girders were lighter – it made his life easier, giving him more flexibility in handling the girder during erection. This may be especially significant for girders crossing railroad tracks or in other situations where crane placement may be difficult or the size of crane may be limited. In some cases, there may also be significant savings in shipping costs if LWC girders are used, especially if girders must be shipped through several states, so several permits are required. Savings in the foundations or substructure may also be realized when LWC girders are used. When combined with a LWC deck, a shallower girder or a reduced number of girders may be possible when using LWC.

DECK CONCRETE PLACEMENT

The LWA supplier was not involved in prepour meetings or trial placements for this project. While LWA is simply a lighter rock and should behave that way in concrete, there are some potential challenges that could be avoided or minimized if all parties understood the nature of the material and how to handle and test it.

In particular, it is important that all parties, including the contractor, concrete supplier, concrete pumping subcontractor and DOT QC personnel, understand the issues that can be important for a successful LWC placement. Since the density is specified, it is important that those testing density and those supplying the LWC understand the effects of their actions on the acceptability of the concrete. Several examples follow.

Holding Back Mix Water

The concrete supplier must understand that holding back water increases concrete density. Therefore, while this is a common practice for NWC mixes, little if any water should be held back from a LWC mix.

Testing Concrete Density

The calibration factor for a unit weight bucket is especially important for LWC because density is being used for acceptance, where for NWC, density is only used as a check on the consistency of the concrete being delivered. The ASTM standard test method for unit weight of concrete provides the density of water to 5 significant figures for calibration of a unit weight bucket. Therefore, the calibration factor for a bucket should have at least 3 significant figures. For early pours on this project, the calibration factor for the unit weight bucket that was being used for acceptance of the concrete had only 2 significant figures. As it turns out, adding another significant digit (from 0.33 to 0.334) made a difference of 1.3 pcf in the unit weight being determined, which affected the acceptance of a number of loads of concrete. To demonstrate, assuming that the concrete in the unit weight bucket weighed 35 lbs, using a factor of 0.33 would indicate a unit weight of 106.1 pcf, while using a factor of 0.334 would indicate a unit weight of 104.8 pcf, or a difference of 1.3 pcf. This seemingly insignificant detail had significant implications for this project.

Placement Method for Concrete

Concrete for most bridge decks is placed by pumping, and LWC for bridge decks has been successfully pumped for many projects across the US. Since LWC is also regularly pumped to the top of high-rise buildings, pumping LWC for bridge decks should certainly be successful. However, it is well known that pumping can have a significant effect on the air content of any type of concrete. With LWC, loss of entrained air is especially important because entrained air is not only important for durability, but is also used to reduce the density of LWC. So if a LWC mixture loses entrained air, it may exceed the required maximum fresh density. Therefore, the contractor and pumping subcontractor must be aware of this issue and use proper equipment and procedures to avoid loss of air during pumping, just as they should for NWC. The concrete supplier must also be aware that proper moisture conditioning of the LWA prior to batching will also have an effect on the pumpability of the LWC.

Testing Air Content

An issue that is often raised by QC personnel when use of LWC is mentioned is the test method for air content. The required volumetric test for LWC using a roll-o-meter is definitely a physically taxing test method. However, experience has shown that using unit weight as an indicator of air content can be successfully employed in the field as long as the batch weights are adequately controlled. While this would not eliminate use of the roll-o-meter, it can be used as an indicator that may reduce the frequency of these tests.

Summary

In summary, it is recommended the LWA supplier be involved early in the project so that potential batching, testing and placement issues for LWC can be addressed and understood prior to the first placement of concrete on the project. Prepour meetings for LWC placements should include the LWA supplier.

DECK FINISHING

The contractor reported some difficulty in achieving a smooth finish on the deck. This was evident from visual inspection of the deck prior to grooving. However, the difficulties encountered were not severe enough to require grinding of significant areas of the deck prior to transverse grooving. From the author’s brief inspection of the deck prior to grinding, it appeared that most if not all grinding was done near the construction joints.

The supervisor for the contractor indicated that he had no prior experience with LWC. With a limited volume of concrete on this bridge, there was little opportunity to develop experience with LWC on the project. Some difficulties could have been avoided if the LWA supplier had been involved earlier in the project.

CONCLUSIONS

This demonstration project for NCDOT included the first use in the state of sand lightweight concrete (123 pcf fresh density) for prestressed concrete bridge girders and the first use in the state of all lightweight concrete (100 pcf equilibrium density) for a deck and parapets.

While some difficulties were encountered during construction, most of which were related to the deck, it appears that the project was an overall success demonstrating that these reduced density concretes could be used for bridge construction. It appears that all parties involved benefited from gaining experience with the materials used. This is demonstrated by the report of successful placements for the ALWC parapets after the concrete supplier, testing personnel and contractor had become more familiar with the ALWC. It was also good to note that all concrete used in the bridge easily met the specifications for compressive strength.

A report on the project is being prepared by the first author which will contain more details about the project and its design and construction.

ACKNOWLEDGEMENTS

The cooperation and assistance of design and construction engineers at NCDOT, and the staff of Lee Construction Company of the Carolinas and Standard Concrete Products – Savannah, GA plant, was greatly appreciated as the design was completed and data was collected during the construction of this project. In addition to these organizations, the following companies also contributed in some way to the data collected in this paper.

- PSBeam prestressed girder design software – Eriksson Technologies, Tampa, FL
- Lightweight aggregate – Carolina Stalite Company, Salisbury, NC
- Ready mixed concrete – Cemex, Archdale, NC
- Permeability testing - TEC Services, Lawrenceville, GA

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