Design, Construction and Testing of the Neal Bridge in Pittsfield, Maine

Final Report
October 2009

MaineDOT

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16. Abstract (Limit 200 words)

Highway bridges in the US are quickly becoming deficient due to increasing traffic volumes, rapid deterioration, extended service life, and increasing load requirements. Repair or replacement of deficient structures is expensive, time and labor intensive, and typically results in lengthy road closures during construction. Researchers at the University of Maine's AEWC Advanced Structures & Composites Center have developed a lightweight, corrosion resistant system for short to medium span bridge construction using FRP composite arch tubes that act as reinforcement and formwork for cast-in-place concrete. They are lightweight, easily transportable, rapidly deployable and do not require the heavy equipment or large crews needed to handle the weight of traditional construction materials.

This report describes the design, construction and testing of the Neal Bridge in Pittsfield Maine, the first FRP arch bridge.

17. Document Analysis/Descriptors		18. Availability Statement	
FRP composite arch			
19. Security Class (this report)	20. Security Class (this page)	21. No. of Pages 55	22. Price

AEWC

Advanced Structures & Composites Center

THE UNIVERSITY OF MAINE

Design, Construction & Testing
Of the Neal Bridge, Pittsfield, Maine

AEWC Report

Project 622

October 19, 2009

Submitted by:

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Dan Bannon
Melissa Landon, PhD
Larry Parent, PE

Table of Contents

1 Introduction

This report is organized using the same section headings as the Manual for Bridge Evaluation [AASHTO 2008]. Section 2 contains relevant information for the bridge files. Although the Manual contains Sections 3 through 8, they are not referenced here.

2 Bridge Files

2.1 General

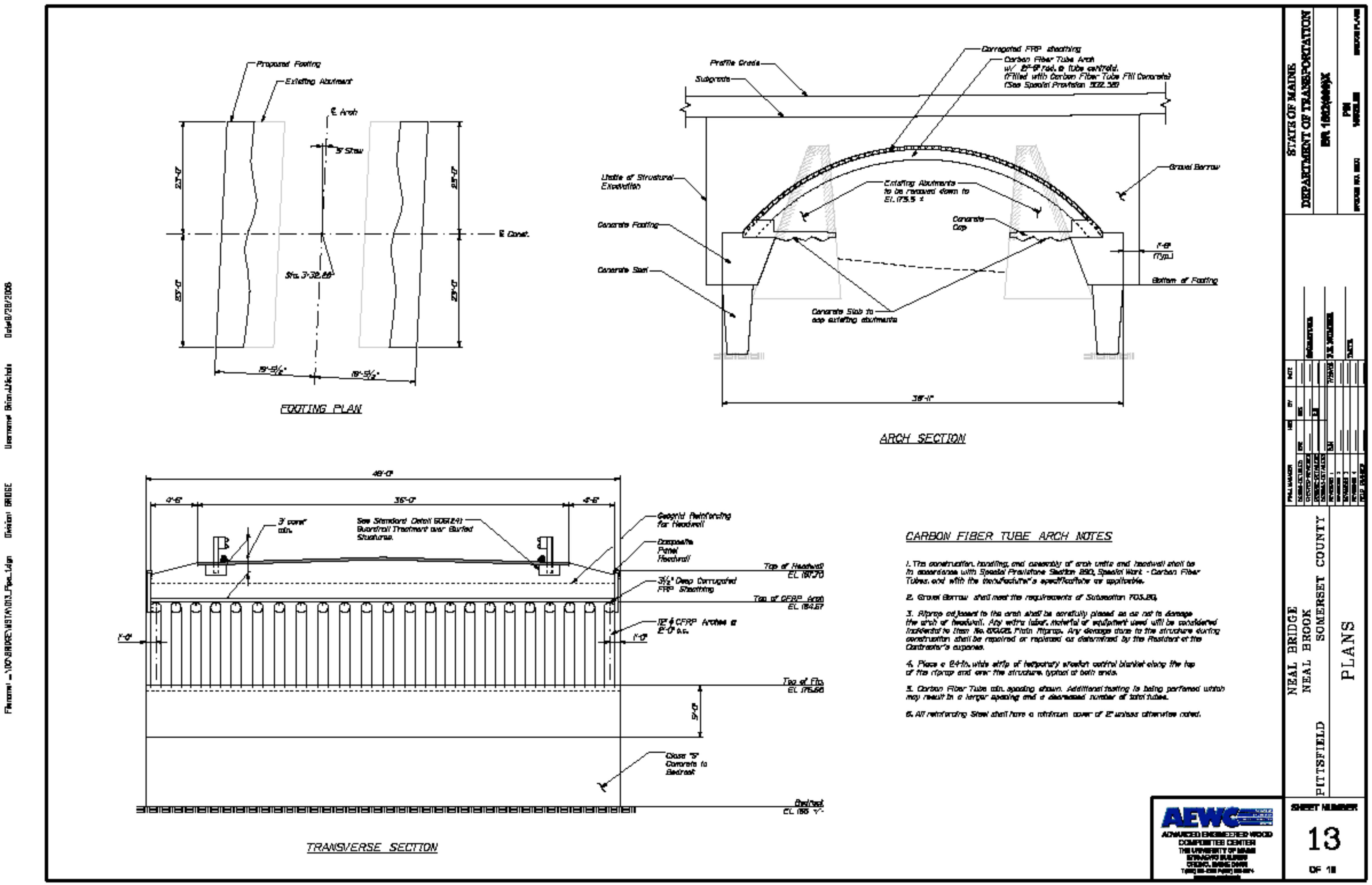
This report compiles the data from AEWC for the structural testing and design work done for the Neal Bridge in Pittsfield, Maine. It follows the sections of the Manual for Bridge Evaluation that are applicable to AEWC's responsibilities. This includes plans, test data, and design of arch structural members, decking and the headwall. Those sections where the work was completed or designed by the Maine DOT are not included in this report.

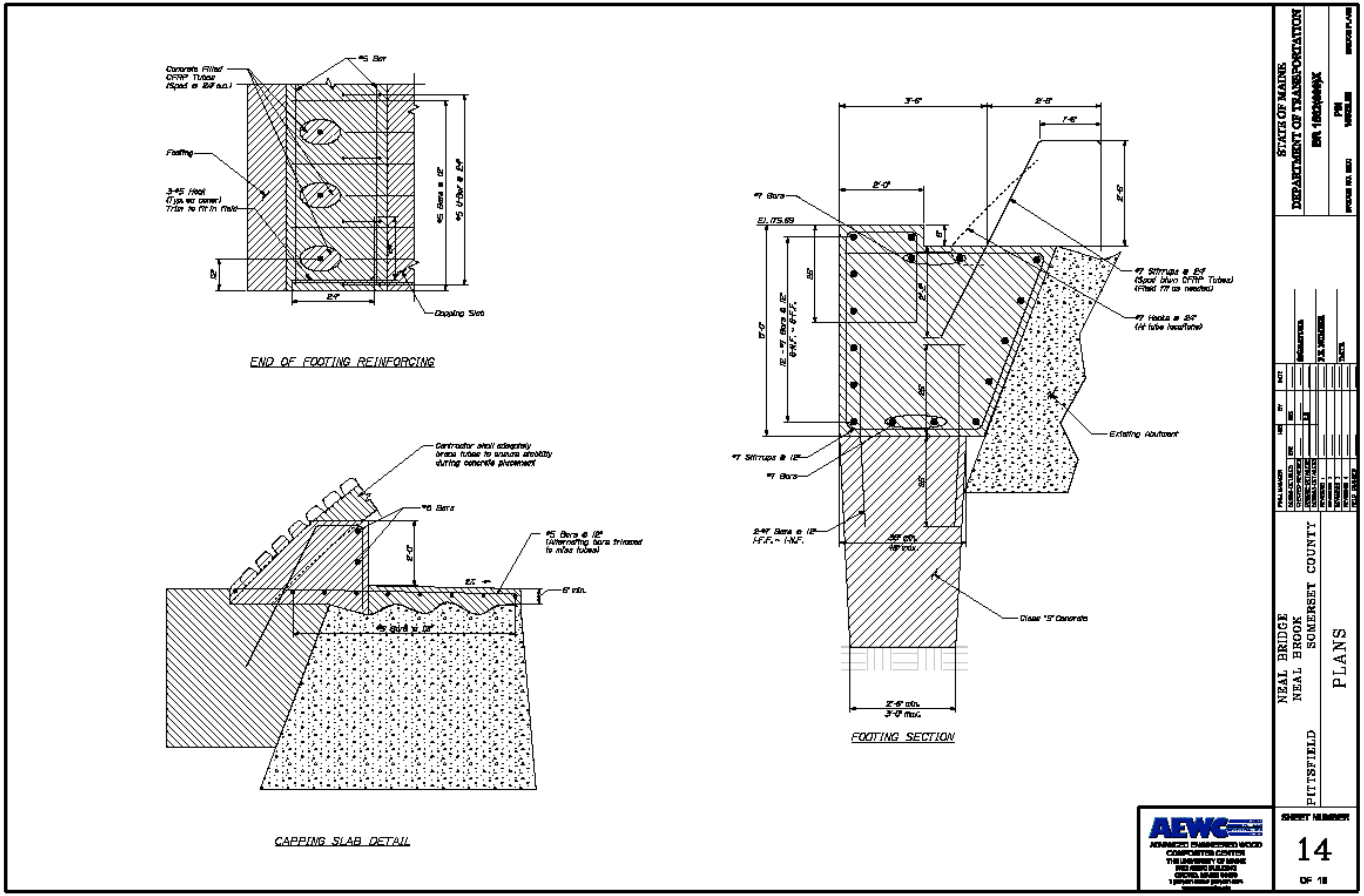
2.2 Components of Bridge Records

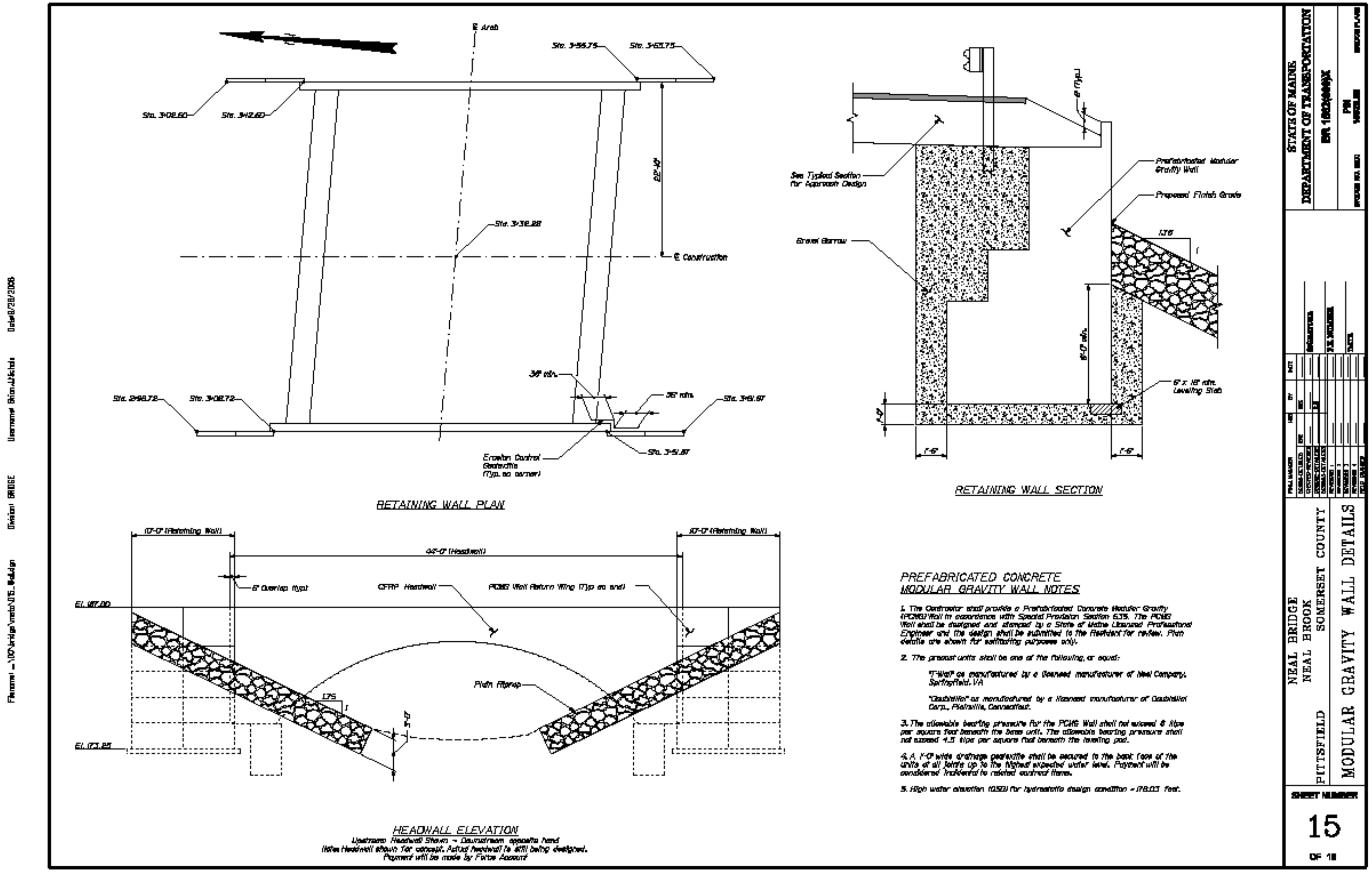
See the following sections for the components included in the bridge records.

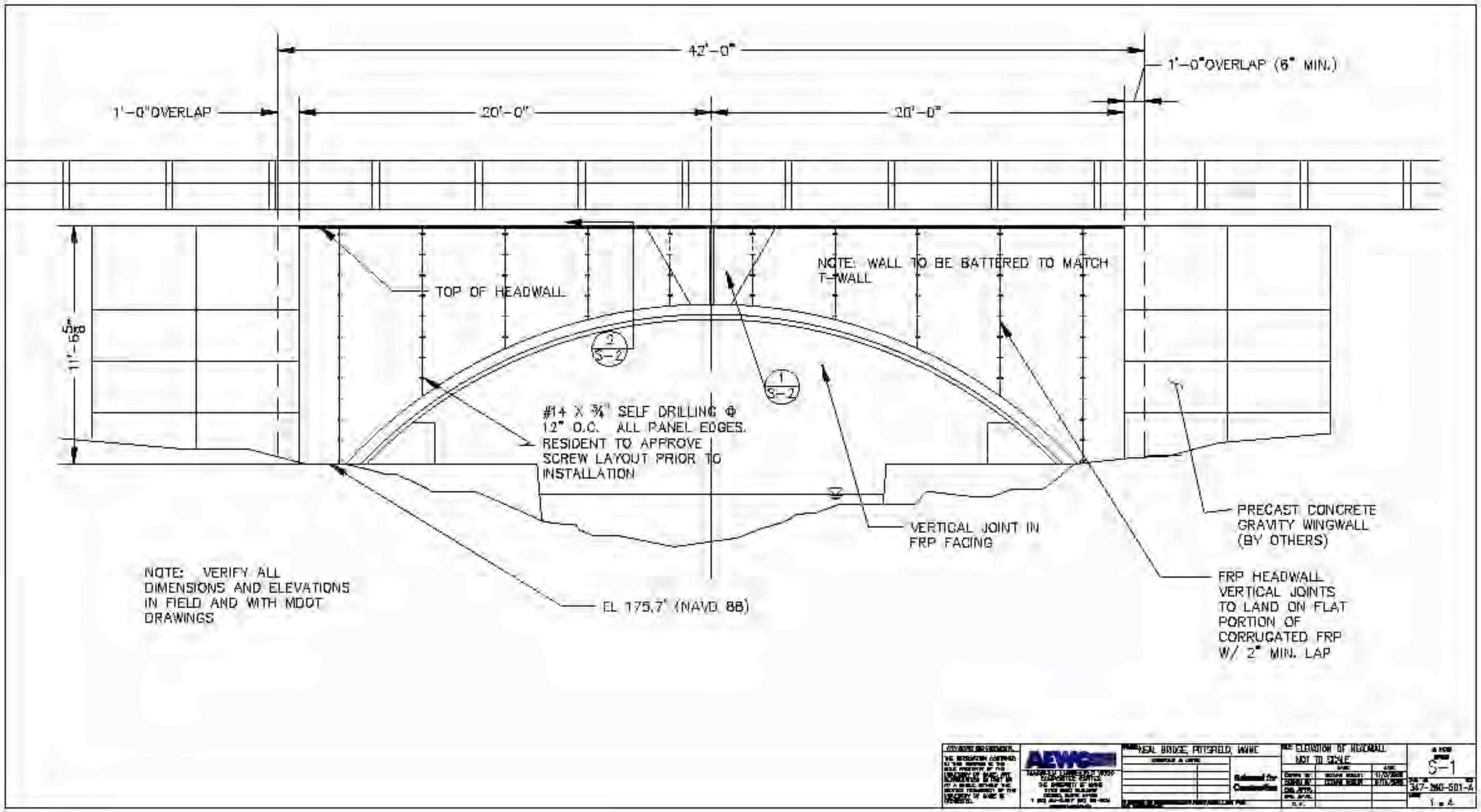
2.2.1 Plans

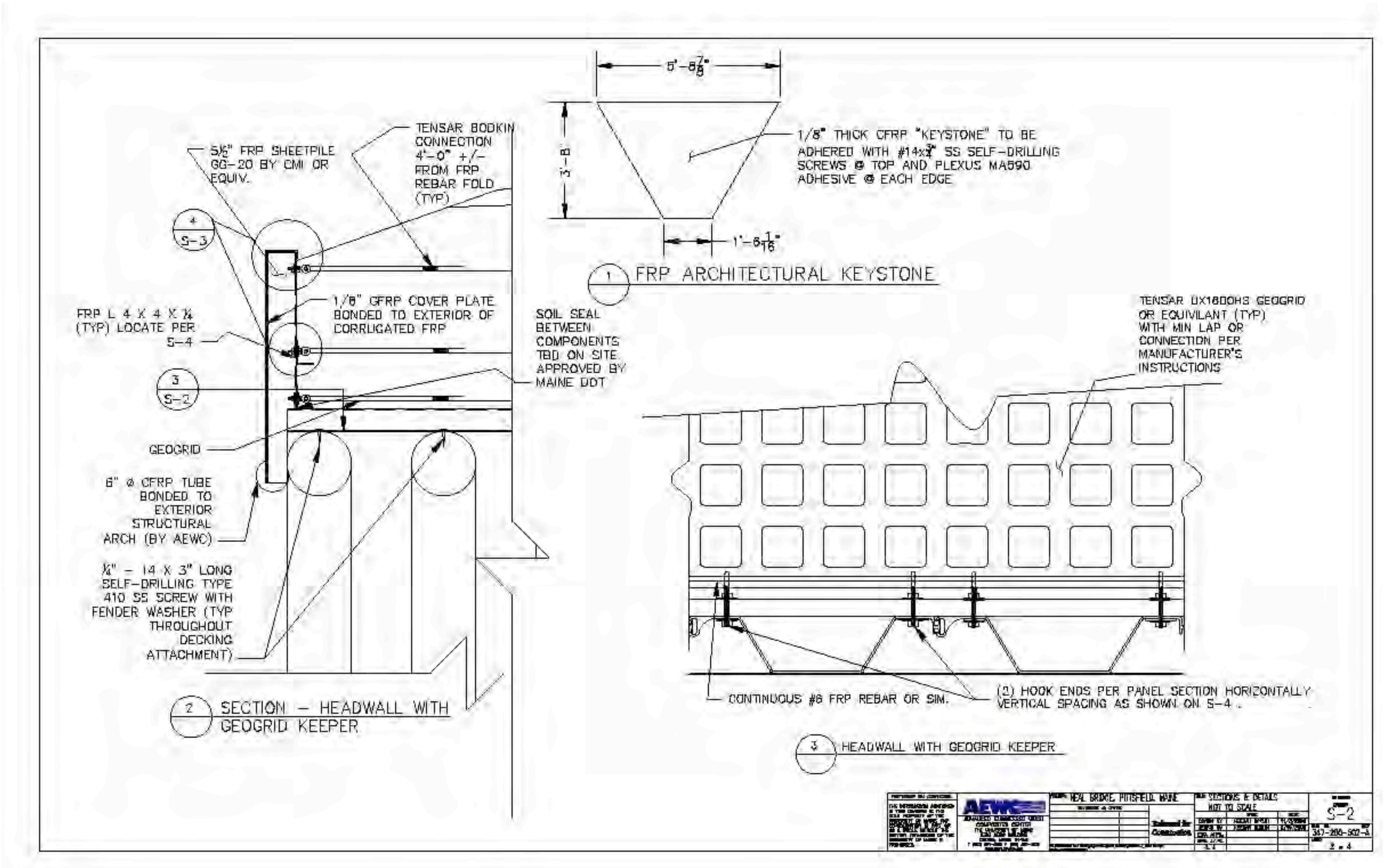
Plans included in this report are design drawings provided by AEWC. They do not include shop drawings by contractor or as-built drawings.

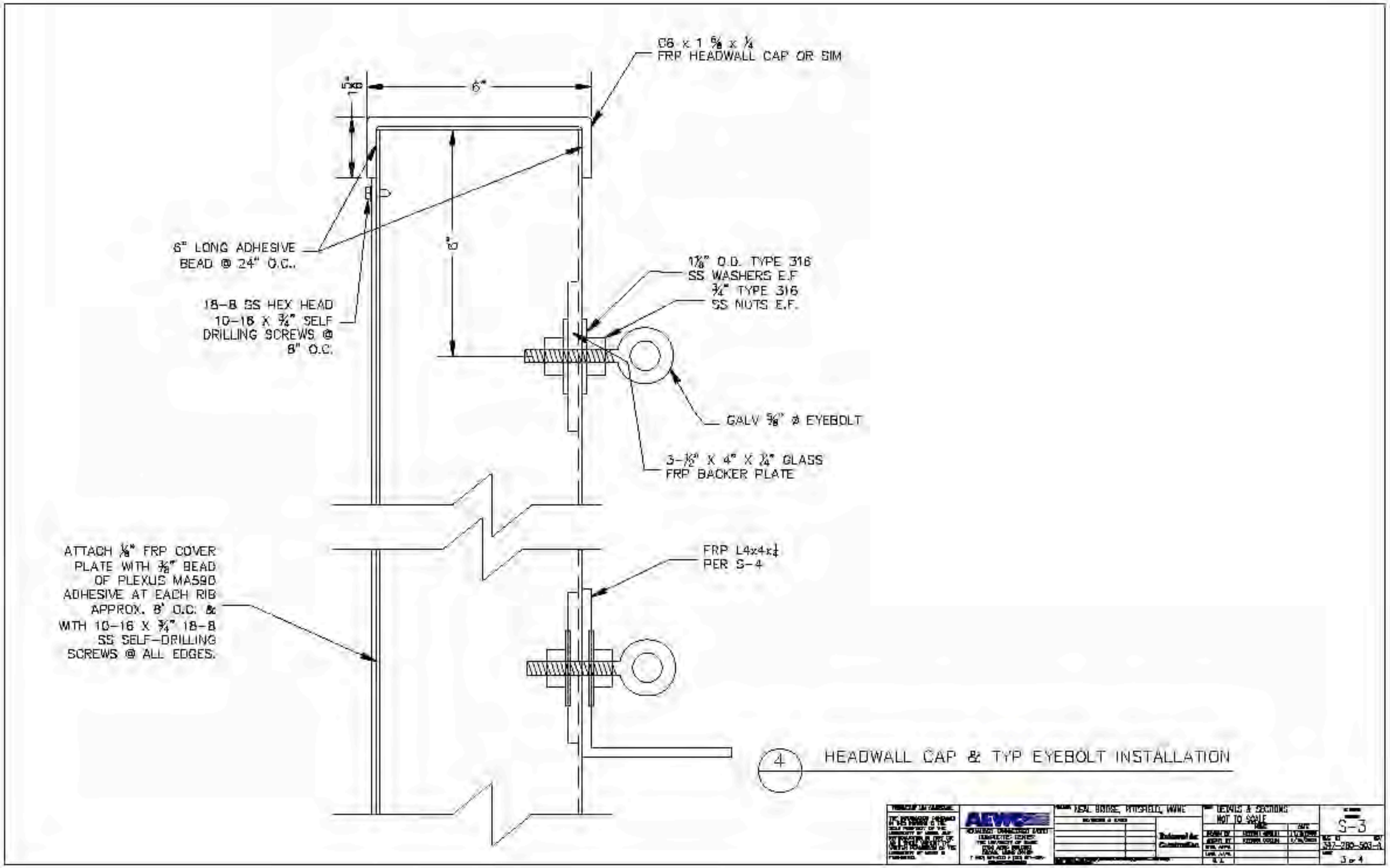

FOOTING PLAN
ARCH SECTION
TRANSVERSE SECTION
Proposed Footing
Existing Abutment
Profile Grade
Subgrade
Corrugated FRP sheathing
Gravel Borrow
Concrete Footing
Concrete Cap
Bottom of Footing
48'-0"
36'-0"
4'-6"
Top of Headwall
Top of CFRP Arch
Top of Ftg.
Bedrock
CARBON FIBER TUBE ARCH NOTES
1. The construction, handling, and assembly of arch units and headwall shall be in accordance with Special Provisions Section 890, Special Work - Carbon Fiber Tubes, and with the manufacturer's specifications as applicable.
2. Gravel Borrow shall meet the requirements of Subsection 703.20.
3. Riprap adjacent to the arch shall be carefully placed so as not to damage the arch or headwall. Any extra labor, material or equipment used will be considered incidental to Item No. 610.08 Plain Riprap. Any damage done to the structure during construction shall be repaired or replaced as determined by the Resident at the Contractor's expense.
4. Place a 24-in. wide strip of temporary erosion control blanket along the top of the riprap and over the structure, typical at both ends.
5. Carbon Fiber Tube min. spacing shown. Additional testing is being performed which may result in a larger spacing and a decreased number of total tubes.
6. All reinforcing Steel shall have a minimum cover of 2" unless otherwise noted.
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
NEAL BRIDGE
NEAL BROOK
PITTSFIELD
SOMERSET COUNTY
PLANS
AEWC
ADVANCED ENGINEERED WOOD COMPOSITES CENTER
SHEET NUMBER
13
OF 18

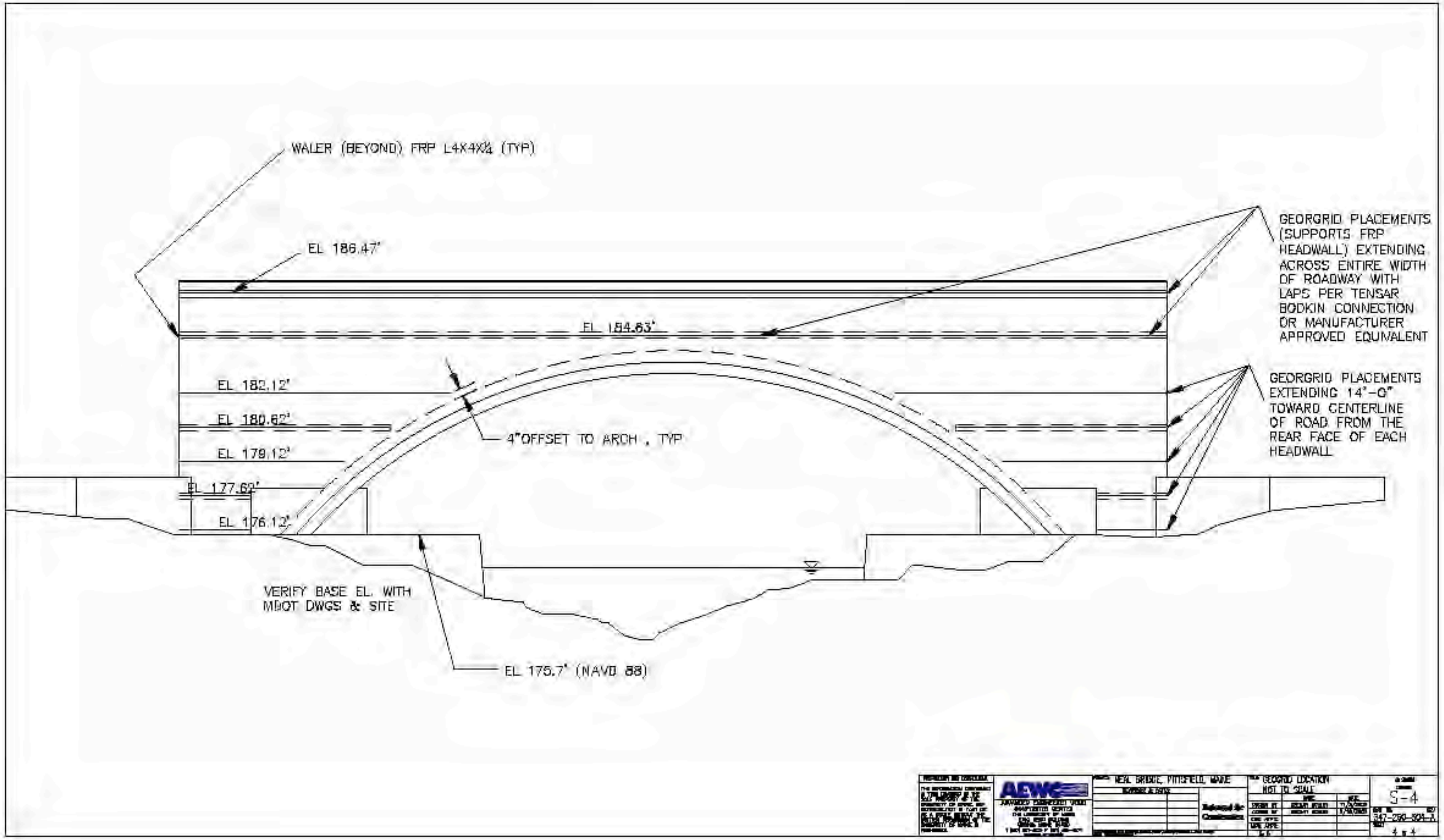

END OF FOOTING REINFORCING
Concrete Filled CFRP Tubes
Footing
Dapping Slot
CAPPING SLAB DETAIL
FOOTING SECTION
Existing Abutment
Class "S" Concrete
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
NEAL BRIDGE
NEAL BROOK
SOMERSET COUNTY
PITTSFIELD
PLANS
AEWC
SHEET NUMBER
14

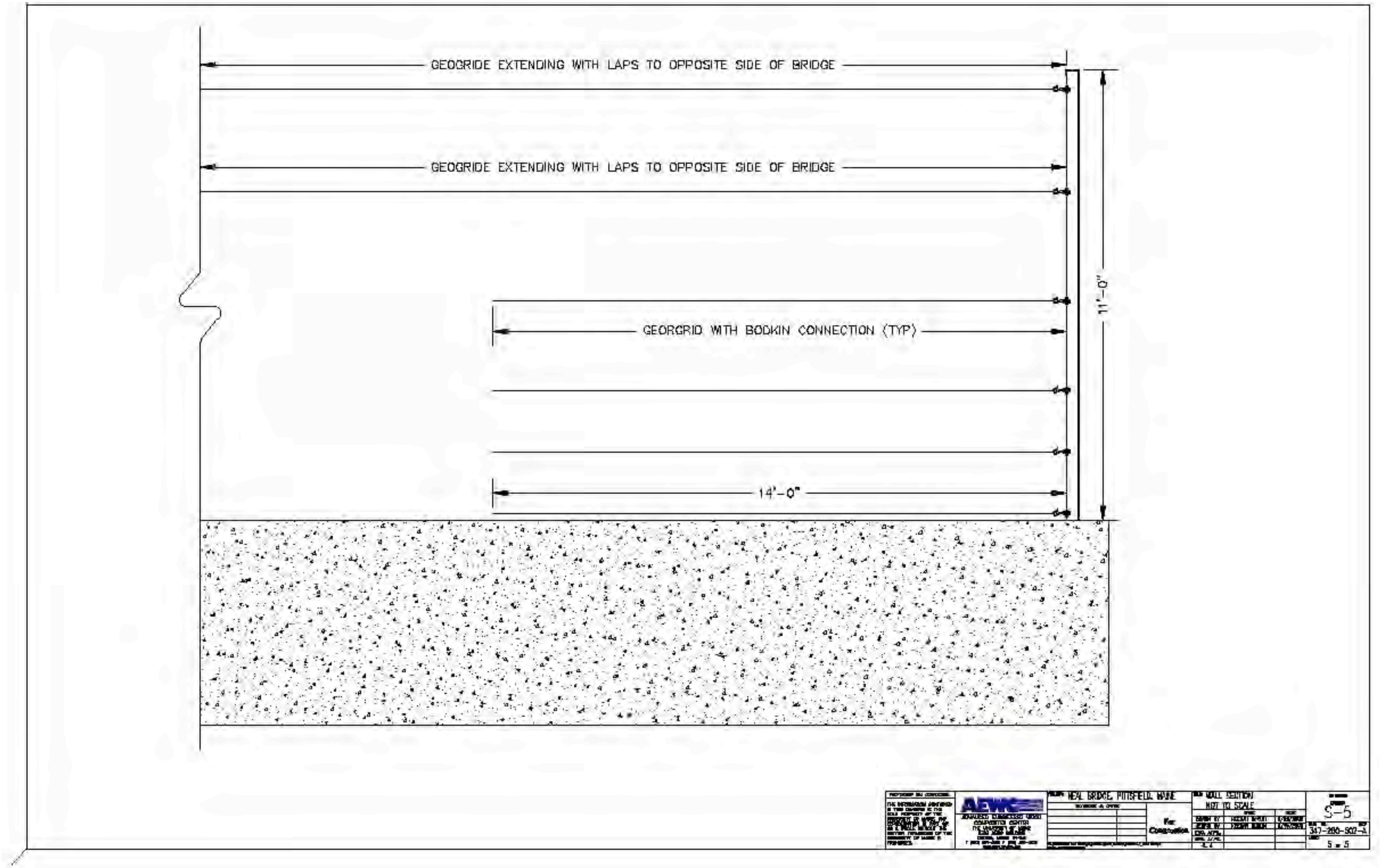

RETAINING WALL PLAN
Sta. 3+02.60
Sta. 3+12.60
Sta. 3+55.75
Sta. 3+63.75
Sta. 3+38.28
22'-10"
℄ Construction
Sta. 2+98.72
Sta. 3+08.72
Sta. 3+51.87
Sta. 3+61.87
Erosion Control Geotextile (Typ. ea corner)
RETAINING WALL SECTION
See Typical Section for Approach Design
Gravel Borrow
Prefabricated Modular Gravity Wall
Proposed Finish Grade
6" x 18" min. Leveling Slab
1'-6"
HEADWALL ELEVATION
Upstream Headwall Shown ~ Downstream opposite hand
Note: Headwall shown for concept. Actual headwall is still being designed.
Payment will be made by Force Account
10'-0" (Retaining Wall)
44'-0" (Headwall)
6" Overlap (typ)
CFRP Headwall
Plain Riprap
El. 187.00
El. 173.85
PREFABRICATED CONCRETE MODULAR GRAVITY WALL NOTES
1. The Contractor shall provide a Prefabricated Concrete Modular Gravity (PCMG) Wall in accordance with Special Provision Section 635. The PCMG Wall shall be designed and stamped by a State of Maine Licensed Professional Engineer and the design shall be submitted to the Resident for review. Plan details are shown for estimating purposes only.
2. The precast units shall be one of the following, or equal:
"T-Wall" as manufactured by a licensed manufacturer of Neel Company, Springfield, VA
"Doublewal" as manufactured by a licensed manufacturer of Doublewal Corp., Plainville, Connecticut.
3. The allowable bearing pressure for the PCMG Wall shall not exceed 6 kips per square foot beneath the base unit. The allowable bearing pressure shall not exceed 4.5 kips per square foot beneath the leveling pad.
4. A 1'-0" wide drainage geotextile shall be secured to the back face of the units at all joints up to the highest expected water level. Payment will be considered incidental to related contract items.
5. High water elevation (Q50) for hydrostatic design condition = 178.03 feet.
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
NEAL BRIDGE
NEAL BROOK
PITTSFIELD
SOMERSET COUNTY
MODULAR GRAVITY WALL DETAILS
SHEET NUMBER
15


42'-0"
1'-0"OVERLAP (6" MIN.)
1'-0"OVERLAP
20'-0"
20'-0"
11'-6 5/8"
TOP OF HEADWALL
NOTE: WALL TO BE BATTERED TO MATCH T-WALL
#14 X 3/4" SELF DRILLING Φ 12" O.C. ALL PANEL EDGES. RESIDENT TO APPROVE SCREW LAYOUT PRIOR TO INSTALLATION
VERTICAL JOINT IN FRP FACING
PRECAST CONCRETE GRAVITY WINGWALL (BY OTHERS)
NOTE: VERIFY ALL DIMENSIONS AND ELEVATIONS IN FIELD AND WITH MDOT DRAWINGS
EL 175.7' (NAVD 88)
FRP HEADWALL VERTICAL JOINTS TO LAND ON FLAT PORTION OF CORRUGATED FRP W/ 2" MIN. LAP


5'-8 7/8"
3'-8"
1/8" THICK CFRP "KEYSTONE" TO BE ADHERED WITH #14x3/4" SS SELF-DRILLING SCREWS @ TOP AND PLEXUS MA590 ADHESIVE @ EACH EDGE
1'-6 1/16"
1 FRP ARCHITECTURAL KEYSTONE
TENSAR BODKIN CONNECTION 4'-0" +/- FROM FRP REBAR FOLD (TYP)
5 1/2" FRP SHEETPILE GG-20 BY CMI OR EQUIV.
4 S-3
1/8" GFRP COVER PLATE BONDED TO EXTERIOR OF CORRUGATED FRP
FRP L 4 X 4 X 1/4 (TYP) LOCATE PER S-4
SOIL SEAL BETWEEN COMPONENTS TBD ON SITE APPROVED BY MAINE DOT
3 S-2
GEOGRID
6" Ø CFRP TUBE BONDED TO EXTERIOR STRUCTURAL ARCH (BY AEWC)
1/4" - 14 X 3" LONG SELF-DRILLING TYPE 410 SS SCREW WITH FENDER WASHER (TYP THROUGHOUT DECKING ATTACHMENT)
2 SECTION - HEADWALL WITH GEOGRID KEEPER
TENSAR UX1600HS GEOGRID OR EQUIVILANT (TYP) WITH MIN LAP OR CONNECTION PER MANUFACTURER'S INSTRUCTIONS
CONTINUOUS #8 FRP REBAR OR SIM.
(2) HOOK ENDS PER PANEL SECTION HORIZONTALLY VERTICAL SPACING AS SHOWN ON S-4 .
3 HEADWALL WITH GEOGRID KEEPER
AEWC
NEAL BRIDGE, PITTSFIELD, MAINE
SECTIONS & DETAILS
NOT TO SCALE
S-2


C6 x 1 5/8 x 1/4
FRP HEADWALL CAP OR SIM
6"
6" LONG ADHESIVE
BEAD @ 24" O.C.
18-8 SS HEX HEAD
10-16 X 3/4" SELF
DRILLING SCREWS @
8" O.C.
1 7/8" O.D. TYPE 316
SS WASHERS E.F
3/4" TYPE 316
SS NUTS E.F.
GALV 5/8" ⌀ EYEBOLT
3-1/2" X 4" X 1/4" GLASS
FRP BACKER PLATE
ATTACH 1/8" FRP COVER
PLATE WITH 3/8" BEAD
OF PLEXUS MA590
ADHESIVE AT EACH RIB
APPROX. 8' O.C. &
WITH 10-16 X 3/4" 18-8
SS SELF-DRILLING
SCREWS @ ALL EDGES.
FRP L4x4x1/4
PER S-4
4
HEADWALL CAP & TYP EYEBOLT INSTALLATION
NOT TO SCALE
S-3
3 of 4


WALER (BEYOND) FRP L4X4X¼ (TYP)
EL 186.47'
EL 184.63'
GEORGRID PLACEMENTS (SUPPORTS FRP HEADWALL) EXTENDING ACROSS ENTIRE WIDTH OF ROADWAY WITH LAPS PER TENSAR BODKIN CONNECTION OR MANUFACTURER APPROVED EQUIVALENT
EL 182.12'
EL 180.62'
4" OFFSET TO ARCH , TYP
EL 179.12'
GEORGRID PLACEMENTS EXTENDING 14'-0" TOWARD CENTERLINE OF ROAD FROM THE REAR FACE OF EACH HEADWALL
EL 177.62'
EL 176.12'
VERIFY BASE EL. WITH MDOT DWGS & SITE
EL 175.7' (NAVD 88)
NEAL BRIDGE, PITTSFIELD, MAINE
GEOGRID LOCATION
NOT TO SCALE
S-4
347-250-S04-A
4 of 4


GEOGRIDE EXTENDING WITH LAPS TO OPPOSITE SIDE OF BRIDGE
GEOGRIDE EXTENDING WITH LAPS TO OPPOSITE SIDE OF BRIDGE
GEORGRID WITH BODKIN CONNECTION (TYP)
14'-0"
11'-0"
AEWC
NEAL BRIDGE, PITTSFIELD, MAINE
WALL SECTION
NOT TO SCALE
For Construction
S-5
347-200-502-A
5 of 5

2.2.2 Specifications

Specifications are included here for those sections that AEWC was responsible for. They include the expansive self-consolidating concrete mix used to fill the arches and the corrugated FRP decking material.

SPECIAL PROVISION
SECTION 502
STRUCTURAL CONCRETE
(Carbon Fiber Tube Fill)

Description: This work shall consist of furnishing and placing a portland cement concrete fill as shown on the plans, or as directed by the Resident. Except as otherwise specified in this Special Provision, all work shall be in conformity with the applicable provisions of Section 502 - Structural Concrete

MATERIALS

Concrete: Concrete shall be as specified below.

Table 1: Arch Fill Concrete Mix

ITEM	WEIGHT PER YD3	NOTES
WATER	378.2 LBS	
TYPE II CEMENT	755.3 LBS	
3/8" COURSE AGGREGATE	1263.3 LBS	Shall satisfy Section 703.02
SAND FINE AGGREGATE	1391.7 LBS	
ADVA 530	98.2 fl oz.	1/4 Added to initial mix with Remainder added on site as needed to obtain 10" slump
DARATARD 17	22.7 fl oz.	Added to initial mix
CONEX	113.3 LBS	Added to initial mix
Entrained Air	0 %to 3%	Target shall be 0%
Min. Compressive Strength	5500 psi (38 MPa)	At 28 days

CONSTRUCTION REQUIREMENTS

Placement of Concrete: The concrete mix shall be placed in a continuous placement operation.

A. General Concrete shall not be placed until arches have been plumbed and checked and approved by the Resident. The method and sequence of placing the concrete shall be approved before any concrete is placed. All concrete shall be placed before it has taken its initial set and, in any case, as specified in Section 502.0701. Concrete shall be placed in such a manner as to avoid separation and segregation. A sufficient number of workers for the proper handling of the concrete is required. Care shall be taken to prevent mortar from spattering on tube members and sheathing. Following the placing of the concrete, all exposed surfaces shall be thoroughly cleaned as required, with care not to injure any surfaces.

B. Pump Truck Pump truck will be the only method of concrete placement allowed. Payment, for Pump truck with operator, will be incidental to this item.

C. Vibrating shall not be allowed when placing concrete into the composite arches. The mix is self-consolidating and separation will occur if vibrated.

Method of Measurement Structural concrete, Carbon Fiber Tube Fill, satisfactorily placed and accepted, will be measured for payment by the cubic yard, in accordance with the dimensions shown on the plans.

Basis of Payment The accepted quantity of Structural Concrete, Carbon Fiber Tube Fill, will be paid for by the cubic yard price.

Payment will be made under:

Pay Item	Pay Unit
502.38 Structural Concrete, Arch Type	Cubic Yard

SPECIAL PROVISION

SECTION 509.60

FRP Panels (Sheathing)

Description.

This work shall consist of the furnishing the FRP panels for the Carbon Fiber Tube Arch in accordance with the plans, specifications and in conformity with instructions supplied by the manufacturer.

Materials.
The materials include the panels, and hardware.

FRP Panels

Fiberglass Reinforced Plastic (FRP) panels and the fasteners required to secure the panels.

Panels shall be Tuff Span® 8.0 Roof Deck Series 700 or an approved equal that conforms to these specifications.

Resin Type

Resin shall be premium grade, chemically resistant Vinyl Ester.

Glass Reinforcement

Reinforcement shall be straight and continuous, with fibers oriented in two directions (along the length and width of unit). Glass content shall be a minimum of 47% by weight.

Flame Spread

Panels shall have a Class 1 flame spread rating (25 or less when tested in accordance with ASTM E-84), shall be listed by UL and bear the UL label.

UV Resistance

Panel material shall be made from a UV stabilized resin modified with acrylic monomers. Additional UV resistance shall come from surfacing mats and a surface coating of an acrylic polymer.

Color

White or other color approved by Owner.

Lengths

Use 16’-0” minimum lengths. Panels shall not be cut; lap panels to fit length. Contractor option to have full width (~45’-0”) panels supplied and delivered.

Structural Fasteners with Washers

Fasteners shall be stainless steel (300/316 series), spaced and installed per manufacturer's recommendations.

Side Lap Fasteners

SB2 grommets, installed per manufacturer's specifications.

Structural Parameters Performance Criteria

Panels shall meet the performance criteria described below for the spans indicated on the drawings (2’-0”). Product compliance with criteria shall be established by full-scale tests for positive and negative loading per ASTM Test Method E-72.

Loads

Decking shall meet the following in single or multiple layers.

Dead (includes soil):	550psf
Live (truck & lane):	2000psf

Allowable Deflections

Decking: L/240

Factors of Safety

Decking

Live loads: FOS = 2.5

Execution

Handling and Storage

Handling

Protect the surface of FRP panels from cuts, scratches, gouges, abrasions, and impacts. Do not use wire slings unless material is fully protected. Use spreader bars when lifting FRP.

Storage

Store panels under cover. Keep panels dry. Stack panels off ground with one end elevated to permit draining of incidental water that can permanently stain panels.

Installation of FRP Panels

3.02.01 Installation Instructions

Installer must follow manufacturer's installation instructions and the shop drawings.

Pilot Holes in Panels

Pilot holes must be drilled in panels for all fasteners. Drill holes with a sharp carbide tipped sheeter's bit. Pilot holes in panels should be sized so that the fastener threads just clear the edges of the hole.

Pilot Holes in Carbon Fiber Tubes

Pilot holes must be drilled in supports for Type A and B stainless steel self-tapping at drill speeds of 500 RPM or less. Pilot holes in carbon fiber tubes shall be sized appropriately for fasteners.

End Laps

End laps for panels shall occur at supports and be 6 inches minimum.

Approved Vendors:

Enduro Composites

16602 Central Green Blvd.

Houston, TX 77032

(713) 358-4000 – Phone

(713) 358-4100 – Fax

Or Equal, approved by University of Maine

Submittals
Shop Drawings submitted for review in accordance with Section 105.7 of the Standard Specifications.

Method of Measurement
The FRP Panels will be measured as one lump sum price in accordance with the plans and specifications.

Basis of Payment
The accepted FRP panels will be paid for at the contract lump sum price, complete and in place.

Payment will be made under:
509.60 FRP Panels (Sheathing) Lump Sum

Other materials were called out on the drawings for items on the headwall. See the drawings for this information on these materials.

2.2.3 Correspondence – This section not included.

2.2.4 Photographs – This section not included.

2.2.5 Materials and Tests

This section gives the results of material and structural level testing for the materials used in the Neal Bridge in Pittsfield, Maine. Coupon tension testing of the laminate used in the arch shell is covered as well as preliminary concrete mix testing conducted in the development of the technology. Material data sheets are attached in Appendix A of this report.

2.2.5.1 Material Test Data

2.2.5.1.1 Coupon Testing
Mechanical testing was performed in accordance with ASTM D3039 on specimens for tensile stiffness and with modified notched specimens for tensile strength. Compressive strength is equated to the tensile strength because local buckling of the FRP is prevented due to the expansive concrete filling the tubes. A comprehensive analysis was carried out using Classical Lamination Theory to validate the results of these tests. Results of coupon tests were in good agreement with theoretically predicted values.

The test method ASTM D3039 determines the in-plane tensile properties of polymer matrix composite materials reinforced by high-modulus fibers. ASTM D3039 was designed to produce tensile property data for material specifications, research and development, quality assurance, and structural design and analysis. The results of the testing are given in Table 2.

Table 2: ASTM D3039 Test Results

Specimen	Tested MOE (ksi)	Poisson's Ratio	Strength (kip/in width)	Strength (ksi)	Thickness (in)	Predicted MOE (ksi)	Percent Difference (MOE)
1	5872.8	0.477	2.831	28.26	0.1002	6722.7	12.64%
3	6558.4	0.435	3.066	30.46	0.1007	6722.7	2.44%
4	5956.1	0.473	2.698	27.30	0.0988	6722.7	11.40%
5	6473.9	0.433	2.996	29.81	0.1005	6722.7	3.70%
6	6373.0	0.352	2.773	27.55	0.1007	6722.7	5.20%
7	6650.5	0.448	2.913	28.29	0.1030	6722.7	1.07%
8	5938.3	0.411	2.572	25.63	0.1003	6722.7	11.67%
9	5739.1	0.404	2.625	27.98	0.0938	6722.7	14.63%
Mean	6195.3	0.429	2.809	28.161	0.0998		7.85%
Std. Dev	355.3	0.04	0.176	1.490	0.0027		
COV	5.73%	9.5%	6.2%	5.3%	2.7%		

In order to obtain the laboratory specimens, flat CFRP panels were manufactured using the Vacuum Assisted Resin Transfer Molding (VARTM) process. Panels were prepared by drawing lengths of braided fabric over thin, rectangular molds, creating very wide, thin tubes. Panels were then infused with resin and allowed to cure under controlled environmental conditions.

The angle and diameter of braided textiles can be adjusted by applying force in the longitudinal direction to the dry fabric. Materials used were 24K tow T-700 carbon fibers braided at approximately [± 45.0°] and 24-inch (300 mm) diameter and vinyl ester resin. The fabric was tensioned during manufacturing to achieve a use angle for the carbon fiber of approximately 20 degrees and an outer diameter of approximately 11.75 inches. From these panels, specimens were cut using a computer controlled water-abrasive cutting machine.

Initially, tension testing was performed in accordance with ASTMD 3039 to characterize both the elastic and the strength properties of the braided composite. ASTM D-3039, however, yielded ultimate tensile strength values an average of 74% below predicted values. This is due to the free edge effects of fibers in the coupon. For this reason, other methods were investigated to determine the tensile strength of the braided composite material. The notched specimen tension test (Figure 1) was developed to more effectively test the strength parameters. A “bowtie” shaped coupon was chosen because fiber continuity is maintained from end to end of the specimen. This eliminated free edge effects and took advantage of the tensile strength of a composite material being a fiber dominated property (only the continuous fibers contribute to strength in measuring gage section area). Results of the coupon tensile strength testing are shown in Table 3.

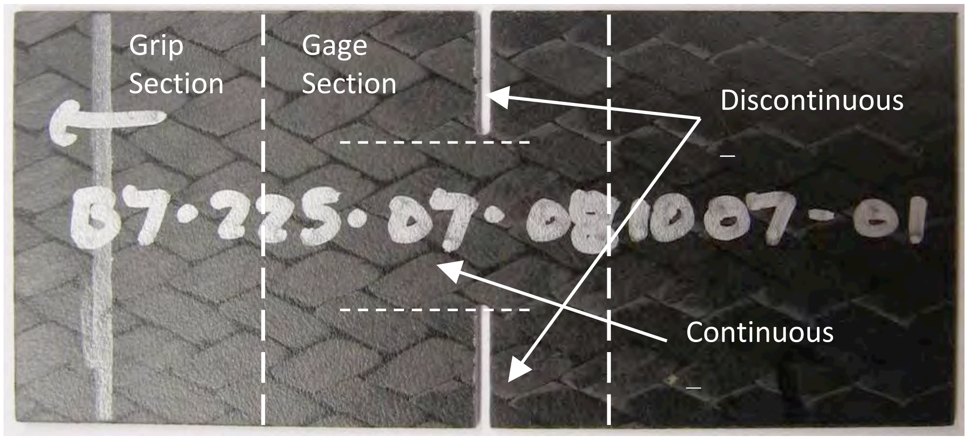


Figure 1. Notched Tension Specimen

Table 3: Results of the Notched Specimen Tension Test

MOE (ksi)	6195.3								
Specimen	**Width (in)**	**Thickness (in)**	**Area (in^2)**	**Max Load (lb)**	**Ult. Strain (in/in)**	**Measured Strength (kip/in)**	**Strength (ksi)**	**CLT Predicted Strength (ksi)**	**Percent Difference Tensile Strength**
1	0.8315	0.0970	0.0807	8027.4	0.016065	9.654119	99.527	121.661	18.19%
2	0.8205	0.0990	0.0812	8658.363	0.017205	10.55254	106.5914	121.661	12.39%
3	0.8110	0.0985	0.0799	9071.571	0.01833	11.18566	113.56	121.661	6.66%
4	0.8520	0.1000	0.0852	8994.823	0.017041	10.5573	105.573	121.661	13.22%
6	0.8145	0.0980	0.0798	9622.614	0.019459	11.81414	120.5524	121.661	0.91%
7	0.8130	0.0985	0.0801	8880.889	0.017901	10.9236	110.8995	121.661	8.85%
8	0.8275	0.0990	0.0819	8190.996	0.016139	9.898485	99.98469	121.661	17.82%
9	0.8230	0.1020	0.0839	8909.703	0.017132	10.82588	106.1361	121.661	12.76%
Mean	0.8241	0.0990	0.0816	8794.5449	0.0174	10.6765	107.8530		
STD. DEV.	0.0133	0.0015	0.0020	506.3311	0.0011	0.6884	7.0112		
COV	1.62%	1.50%	2.45%	5.76%	6.50%	6.45%	6.50%		

2.2.5.1.2 Concrete Testing

The compressive strength of cylindrical concrete specimens was tested in accordance with ASTM C 39-05. This test requires a compressive axial load to be applied to molded cylinders or cores until failure occurs. The compressive strength of the concrete specimen is calculated by dividing the maximum load attained during the test by the cross-sectional area of the specimen. Table 4 shows the results of testing cylinders made with 15% Conex and 1% measured air.

Table 4: Arch Fill Concrete Testing

Cylinder Label	**Length (in)**	**Diameter (in)**	**Cross-Sectional Area (in^2)**	**Max Load (lbs)**	**Compressive Strength (psi)**	**Average Comp. Strength (psi)**
a	8	4	12.57	71190	5665.0	5715.0
b	8	4	12.57	70620	5620.0	
c	8	4	12.57	73640	5860.0	

2.2.5.1.3 Beam Testing

Concrete-filled fiber reinforced polymer (FRP) beam specimens were tested using a four-point bending apparatus. Simple supports were provided at each end of the specimen to provide free rotation and load was applied at the beam third points. Load was applied to the specimens using a 110-kip servo-hydraulic actuator. The actuator is mounted beneath the floor and load is applied by pulling downward on the yoke using a high-strength DYWIDAG Threadbar®. A 110-kip load cell was installed in-line to monitor applied load. A sketch of the test setup is shown in Figure 2. The beam section was of the same construction as the Neal Bridge arch members.

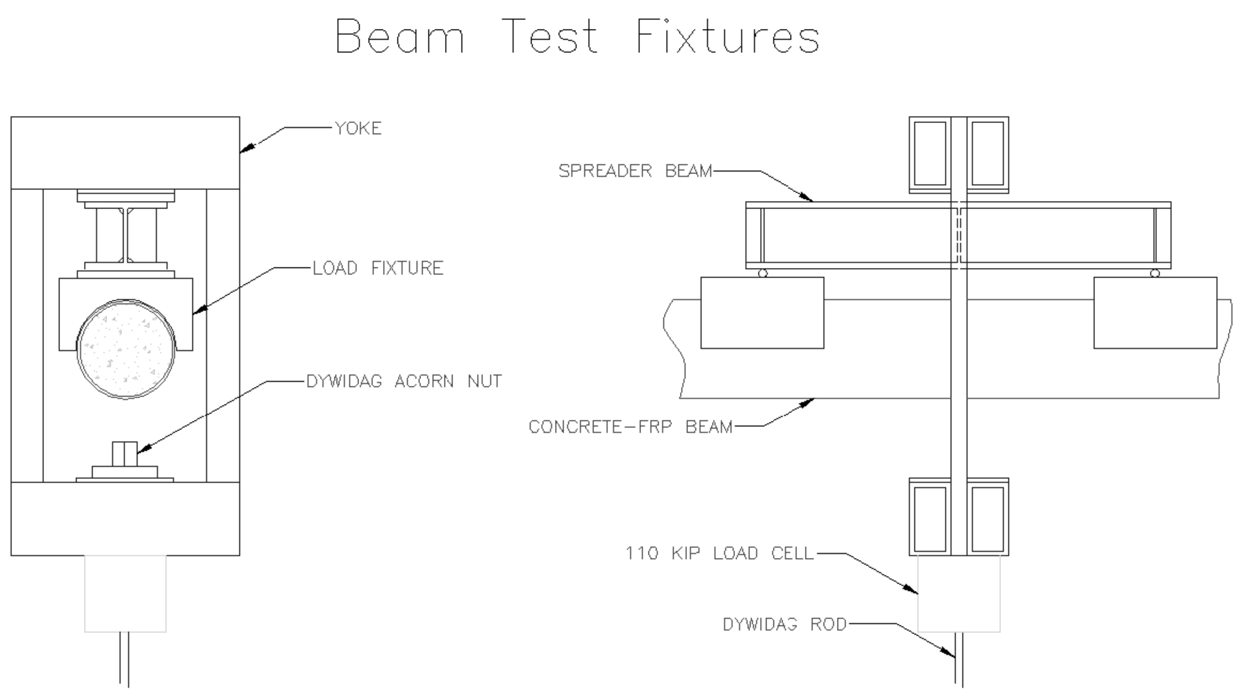


Figure 2: Beam Test Fixtures

Three beams were tested to failure under static load. The average failure load for these three beams was 54.3 kips. Following this, three beams were fatigued for one million cycles. The first of these, fatigue beam 01, was tested monotonically over a 28 kip load range. The second two were tested in both positive and negative bending to a load that was predicted to just achieve the maximum ACI recommended fatigue strain for concrete (0.0015). The testing was performed sequentially: 100,000 cycles in positive bending followed by 100,00 in negative and then back to positive. A summary of the results for the fatigue beam testing is given in Table 4. The average strength of the fatigues specimens was 54.3 kips, indicating that there was no significant loss of strength due to the fatiguing. In fatigue beam 01, a small amount of residual deformation was observed, which increased throughout the test as accumulation of damage occurred. In the remaining two beams, no residual damage was seen.

Table 5: Beam Testing Results

Specimen	Loading Direction	Load Range	Failure Load	Failure Moment
Fatigue Beam 01	Positive Only	+3.1k to +31k	>55k*	---
Fatigue Beam 02	Positive	+2.05k to +20.5k	53.0k	1272 in-k
	Negative	+2.05k to -20.5k	---	---
Fatigue Beam 03	Positive	+2.05k to +20.5k	---	---
	Negative	+2.05k to -20.5k	54.6k	1966 in-k
*Beam could not be broken with equipment as set, load assumed to be 55k (actuator capacity).				

2.2.5.1.4 Arch Testing

Analysis and testing was conducted on arches similar to the Neal Bridge arches but with a gross geometry (span and height) approximately 65% as big, having a tested span of 21'-2 ½". Static and fatigue testing were conducted. A description of the test setup, model, testing procedure, and results are given here.

A nonlinear finite element model was used to predict the response of the arch test specimens. By using symmetry, the model was reduced to a half arch. Two different boundary conditions were modeled at the crown where the arch model was cut. These two cases are representative of the two damage states of the arch specimen. Prior to peak loading (Figure 3A), the crown behaves as a "fixed roller"; that is, rotation and horizontal translation are fixed, and vertical translation is unconstrained. After the FRP has ruptured, the arch forms a hinge at the crown. This condition may conservatively be modeled as a "pinned roller"; rotation and translation in the vertical direction are allowed, while horizontal translation is fixed (Figure 3B). This is a conservative lower bound as the hinge retains some amount of rotational stiffness after sustaining damage. The actual damaged condition is highly variable and cannot adequately be modeled using this simple finite element model.

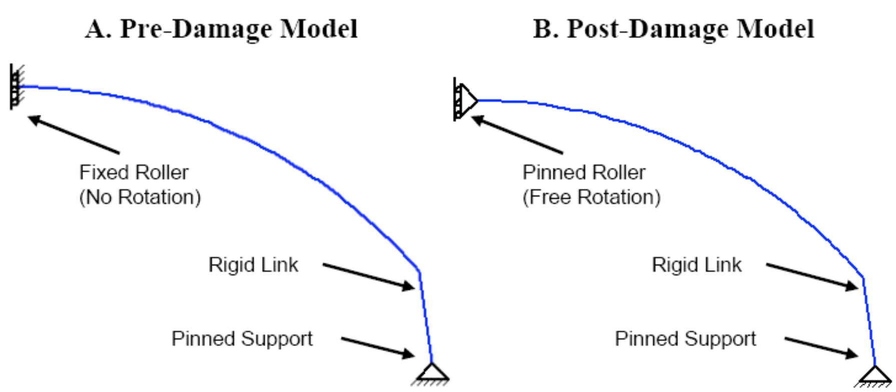


Figure 3: Structural Model of Arch Test

The cracking moment in the arches is greatly under predicted when using the equation given by ACI [ACI 318-08] for modulus of rupture. This is attributed to the under prediction of the concrete's tensile strength in this research. The under prediction of concrete tensile strength has several effects on the predicted arch behavior, including an under predicted linear-elastic region and an under predicted stiffness throughout the flexural response. The following alternate equation was used for the concrete modulus of rupture in predicting the behavior of the arch specimens. The coefficient is based on the experimental results.

$$f_{cr} = 17.5\sqrt{f_c^{'}} \qquad \text{Equation 1}$$

In this equation, f'_c is the 28-day concrete compressive strength; the factor relating to lightweight concrete has been taken as 1.0 for the purposes of this work. The proposed equation represents a 2.3:1 increase over the modulus of rupture predicted using the ACI 318 equation. The moment curvature model developed by Burgueño [1999] accurately predicts both the moment-curvature response and the moment capacity of the concrete-filled FRP beam specimens once this adjustment is made.

All specimens tested were relatively lightly reinforced. The 11.8 in diameter tubes had an average wall thickness of 0.10 in, resulting in a reinforcement ratio, ρ = 3.4%. Due to the minimal reinforcement, all specimens failed due to tensile rupture of the FRP reinforcing shell. This failure mode was consistent with model predictions in all cases.

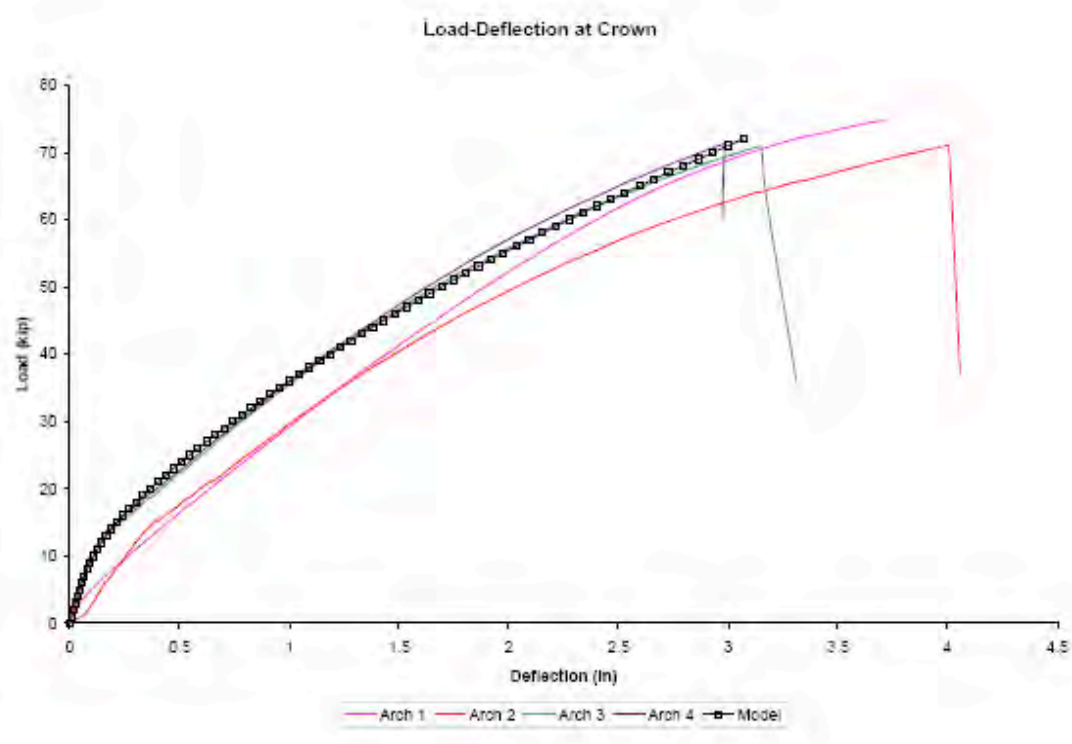


Figure 4: Load- Deflection of Static Arch Testing

Table 6, below, shows results for the static and fatigue arch testing. The peak load at initial fiber rupture on the underside of the crown is given as well as the corresponding moment for that loading region at the crown.

Table 6: Arch Testing Results

	Failure Load (kip)	COV	Corresponding Moment at Crown (kip-in)	Number of Specimens	Percent Difference from Predicted
Static Arch 1	74.7	-----	1440	1	
Static Arch 2	71.0	-----	1370	1	
Static Arch 3	70.9	-----	1370	1	
Static Arch 4	71.2	-----	1380	1	
Static Average	**72.0**	**2.6%**	**1390**	**4**	**4.14%**
Predicted	69.0	-----	-----	-----	-----
Fatigue 1	75.4	-----	1460	1	
Fatigue 2	62.3	-----	1210	1	

After the initial failure, the arch members retained their stability as well as a significant amount of their initial load carrying capacity. The post-peak behavior of the arch members was studied by subjecting the specimens to a secondary static test until complete failure was forced. During the secondary tests, the arch members continued to show ductility and energy absorption.

2.2.5.2 Full Scale Bridge Load Testing

Diagnostic load testing of the Neal Bridge was conducted to increase the understanding of the structural performance of the arches and the load distribution through the soil and to calibrate the analytical load rating of the structure. Digital data acquisition was used to allow for 26 load cases and a large number of instruments. A dynamic test was also conducted following the static testing. Two roughly 66,500 pound double rear axle dump trucks were used to load the bridge for both the static and dynamic tests. A description of the tests conducted, their results, and the corresponding load rating is given in this section. Strain, deflection using linear potentiometer deflection gages, and PONTOS 3D image correlation for deflection were used to collect structural performance data.

2.2.5.2.1 Live Load

Two dump trucks provided by the bridge maintenance division of the Maine DOT were used as the live load for this testing. They were tandem rear axle dump trucks loaded with soil. Axle weights were taken by the Maine State Police. Figure 5 gives the average axle weights for the two trucks. The trucks themselves are shown in Figure 6.

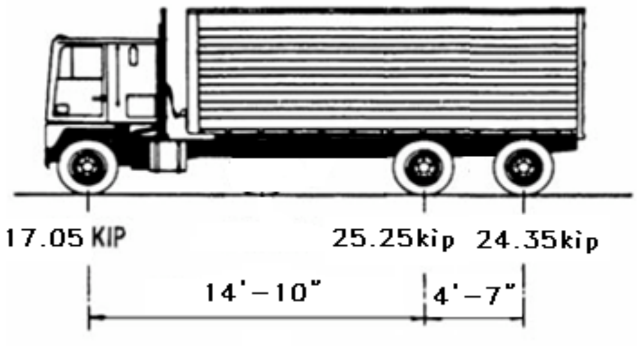


Figure 5: Average Live Load of Two Test Trucks

Figure 6: Trucks Used in Testing of Neal Bridge

2.2.5.2.2 Test Setup

The first set of tests was conducted with two fully loaded dump trucks side by side, 4'-0" apart measured from the outside surface of the outer set of rear tires. The trucks were facing north. Strain, deflection, and soil pressure data were collected for this series of tests. Deflection data were collected using linear potentiometer deflection gages at discrete points along the length of the 10^{th} arch from the downstream face of the bridge. Deflection data were also collected used PONTOS 3D digital image correlation for a region of arches 10 and 11 at the crown. The position of the front axle of the trucks is given in Table 7 and illustrated in Figure 7.

Table 7: Front Axle Position for 1^{st} Series of Static Tests

	Truck Position								
Side by side	1	2	3	4	5	6	7	8	9
Front Axle Location (inches from bridge centerline)	150	210	270	330	390	-31	30	90	150

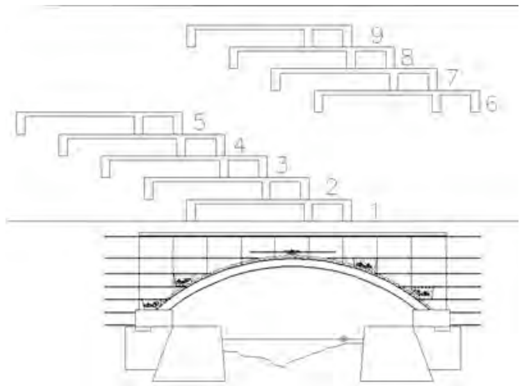


Figure 7: Side-by-side truck locations during load testing (see Table 7)

The second series of tests used both dump trucks in the same downstream lane, both facing south. The trucks were centered over arch 10, 4'-0" downstream of centerline. The positions of the front axle of the lead truck are given in Table 8. The front axle of the second truck was 41'-0" behind the front axle of the first truck.

Table 8: Front Axle Position for Lead Truck during 2nd Series of Static Tests

Tandem	**Truck Position**							
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Front Axle Location of Lead Truck (inches from centerline)	182	122	62	2	-58	-118	-178	-238
	9	**10**	**11**	**12**	**13**	**14**	**15**	**16**
	-298	-358	-418	-478	-538	-598	-658	-718

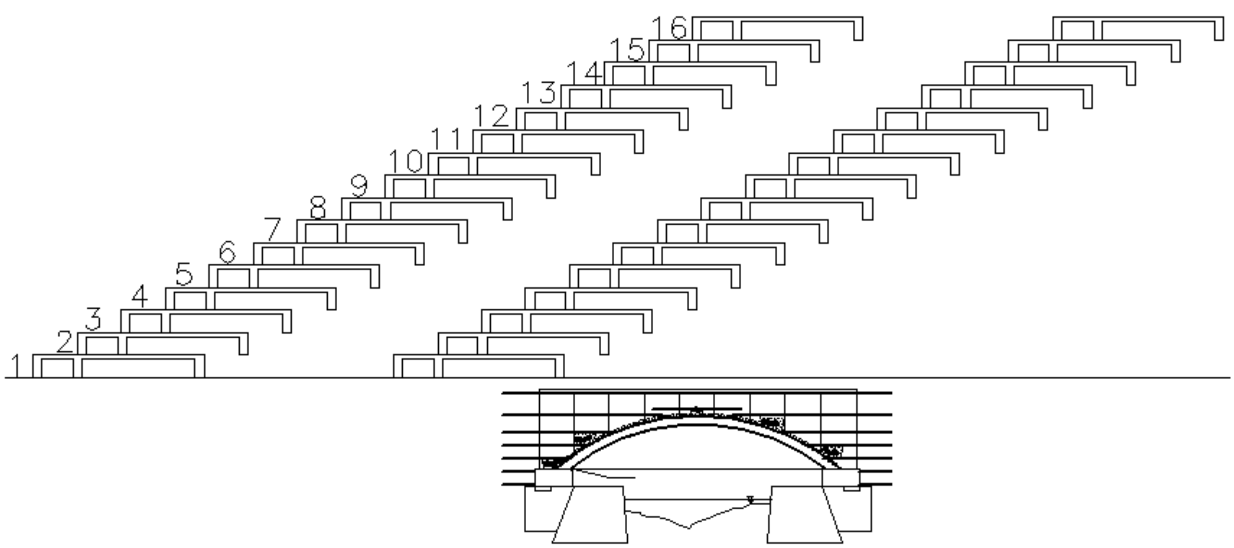


Figure 8: Tandem truck locations during load testing (see Table 8)

The third test was a dynamic test with the trucks traveling at the speed limit of 45 mph across the bridge with headway of approximately 10 feet. The trucks were centered in the downstream lane heading north. PONTOS was unable to collect data for this test.

2.2.5.2.3 Instrumentation

Strain, deflection, and soil pressure data were collected continuously during the static and dynamic tests. The fourth, sixth, eighth, and tenth arches from the downstream side of the bridge were instrumented with strain gages. Deflection gages were placed on arch 10 to measure perpendicular deflections of the arches. Soil pressure gages were embedded in the soil above arch 10. Photos of the deflection gages can be seen in Figure 9. PONTOS was used to collect deflection data for the center 1 meter of span of arches 10 and 11 as well as the decking spanning between those arches.

Figure 9: Location of Deflection Gages: (A) Post carrying deflection gage at shoulder (B) Section of arch 10 monitored with digital image correlation (C) Post for deflection gage at crown (D) Post carrying deflection gage near foundation (E) Location of deflection gage at footing

Strain gages were installed on arches 4, 6, 8 and 10. Strain was recorded on 3 sections of arches 8 and 10 and on 1 section of arches 4 and 6. Three longitudinal strain gages were placed at each location. At the base of each arch one additional strain gage was placed in the hoop direction (opposite E in Figure 6). The three locations on the arches that were instrumented include the crown, the shoulder 85 inches horizontally from the crown and the top of the curb or foundation connection of the arches. Figure 11 shows the global location of the strain and pressure gages. The strain gage location at each arch section is shown in Figure 12.

2.2.5.2.4 Measurement of Soil Pressure

Ten vibrating wire total earth pressure cells (TPCs) were installed along the outside of arch 10 within the backfill to assess the interaction between the structural backfill and the arch during loading (Figure 11). The 9" pressure pad diameter TPCs (Figure 10) were manufactured by Roctest to have a capacity of 4180 psf (200 kPa). At each location, a TPC was installed parallel to the ground surface (horizontal) with the edge between 3" and 6" from the face of the bridge decking. Horizontal TPCs were used to determine the vertical stress distribution beside the arches. In all locations except the centerline of the arch, TPCs were installed parallel to the bridge arch tangent, with the center of the pad nearly 6" vertically above the center of the horizontal TPC pressure pad. These sloped TPCs were installed to assess soil-structure interaction from arch. Additionally, one TPC was installed between arches 9 and 10 to assess loading directly above the less stiff decked area.

As recommended by Roctest (2005), the TPCs were installed between the arch and backfill with a fine sand layer at least 4" thick on the bottom and top of the cell to prevent point loading that could lead to inaccuracy or damage. The sand layers under and above the TPCs were compacted by hand using heavy tamper both before and after the installation to reduce the presence of voids and adhere to construction specifications. Avoidance of vibratory compaction over the TPCs was recommended to avoid damage to the instruments (Roctest 2005). TPC wiring was run through small ports in the decking and connected to a Campbell Scientific datalogging system powered by a battery charged by a solar panel, all of which was secured to the north northbound headwall. Initial TPC data acquisition attempts were problematic, so actual pressures in the cell locations after construction were not determined. Relative pressures during the load test were gathered.

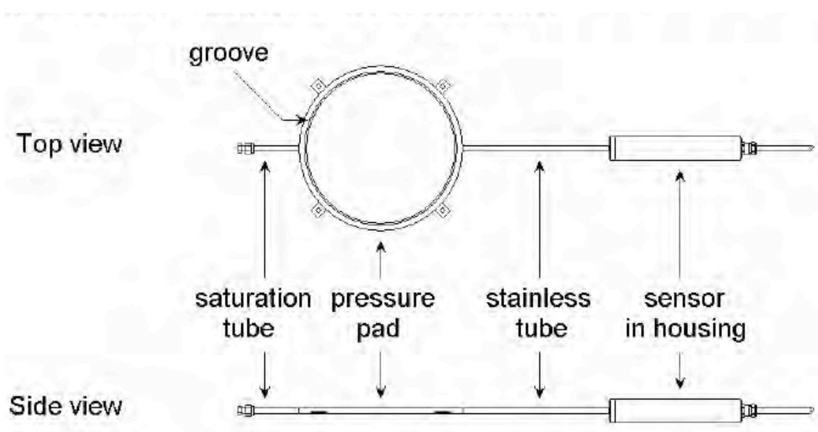


Figure 10: Schematic of a Roctest Total Pressure Cell (Roctest 2005).

Roctest (2005). Instruction Manual: Vibrating Wire Pressure Cells, Model TPC & EPC. < http://www.roctest.com/modules/AxialRealisation/img_repository/files/documents/E1078E-050708b.pdf>, July 2009.

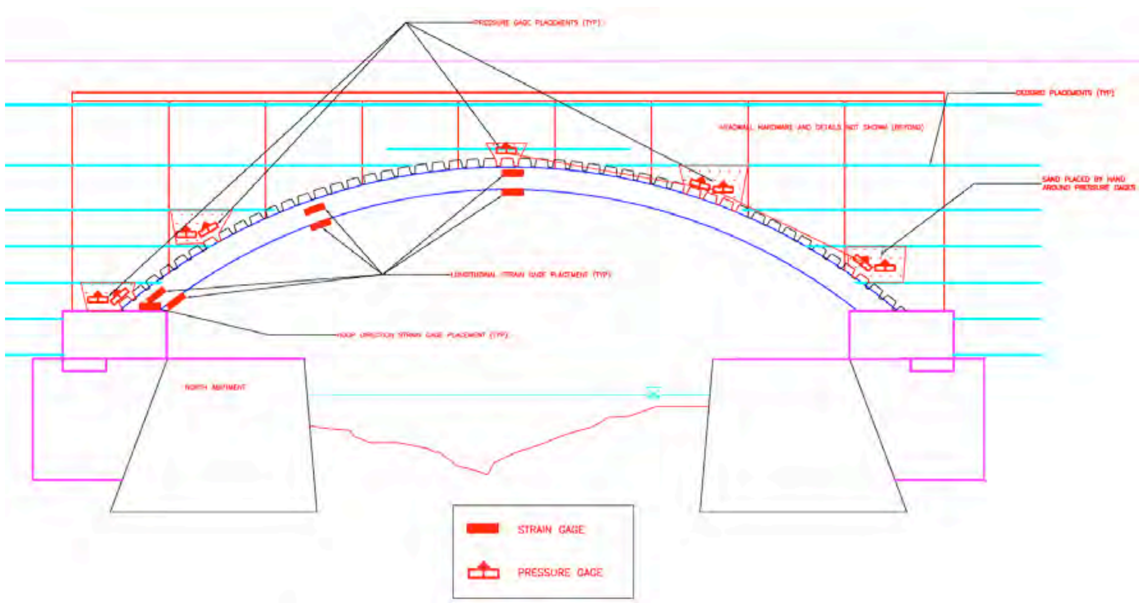


Figure 11: Location of Strain Gages and Total Pressure Cells (TPC)

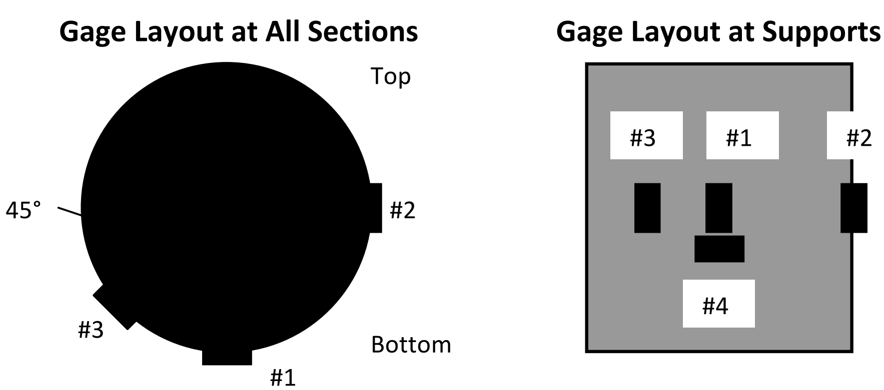


Figure 12: Location of Strain Gages at Each Arch Section

2.2.5.2.5 Results

The main goal of the load testing was to determine the live load effects and load distribution of the two loaded trucks. The maximum strain, deflection and soil pressure measurements are reported here as well as graphs of the strain at each location versus each truck location. Plots of the live load strain for each arch are shown in the following four figures.

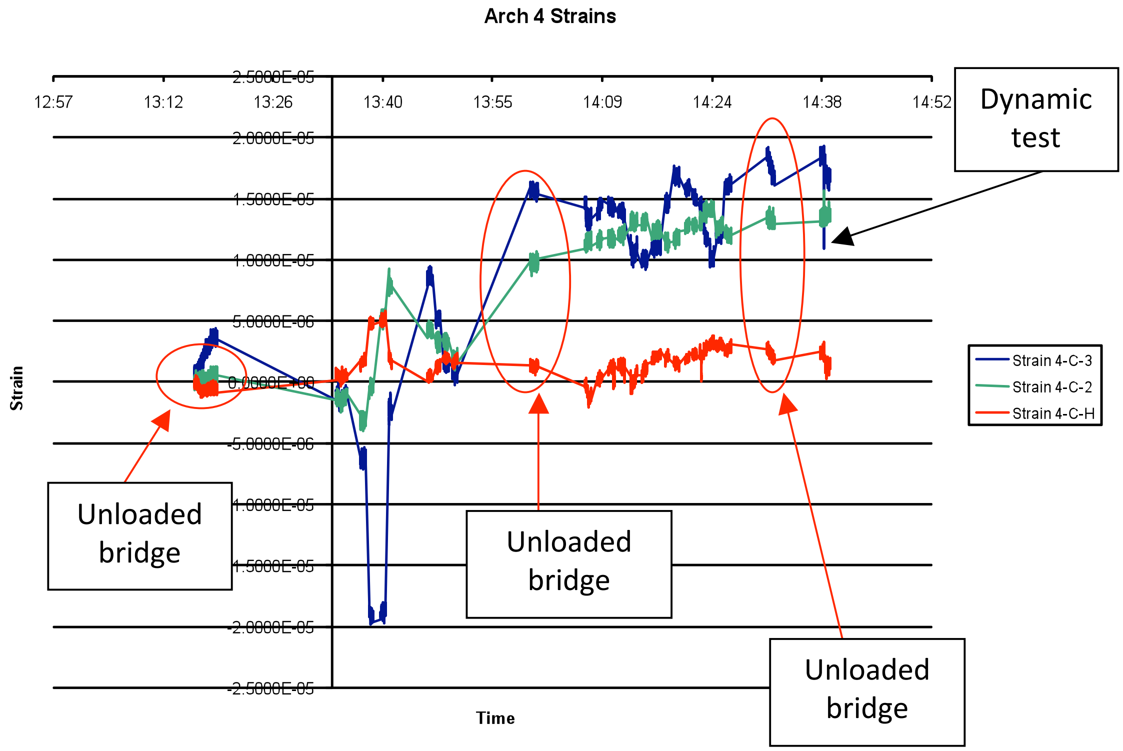


Figure 13: Live Load Strain Data for Section C of Arch 4

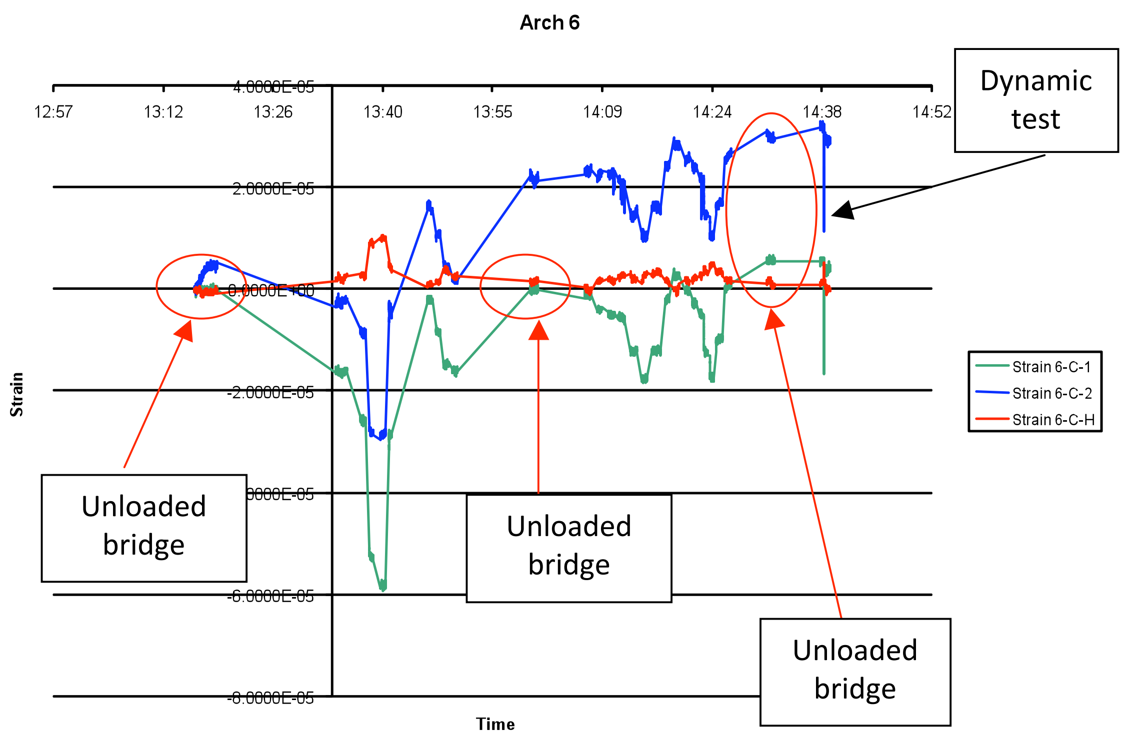


Figure 14: Live Load Strain Data for Section C of Arch 6

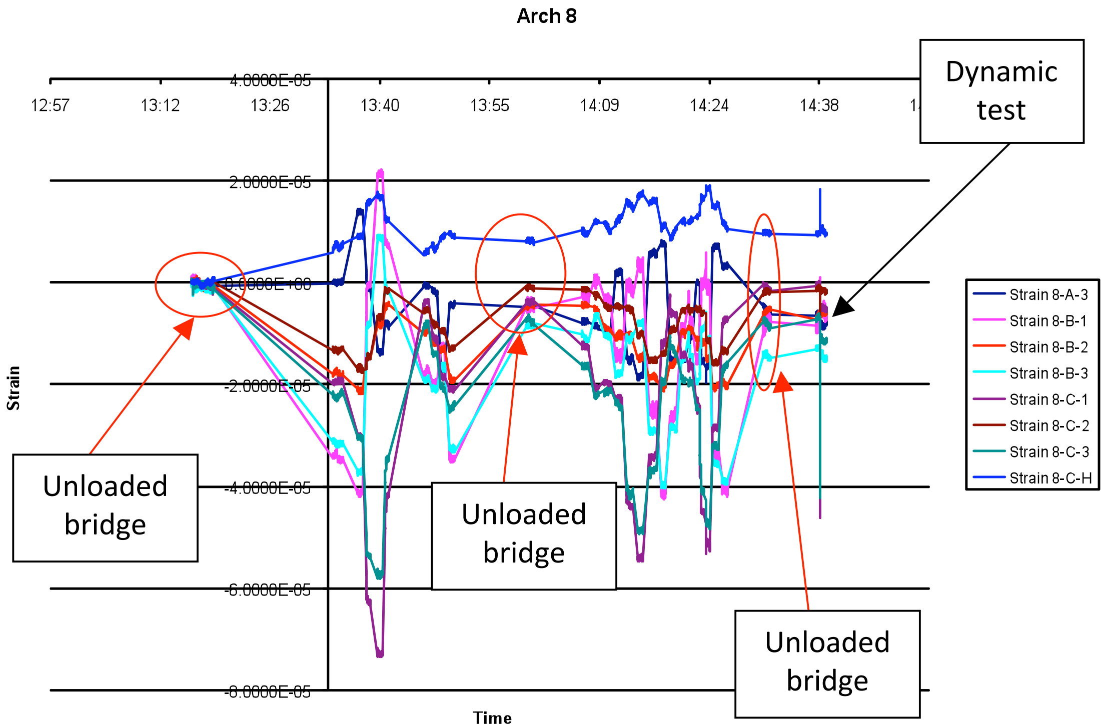


Figure 15: Live Load Strain Data for Sections A, B, C of Arch

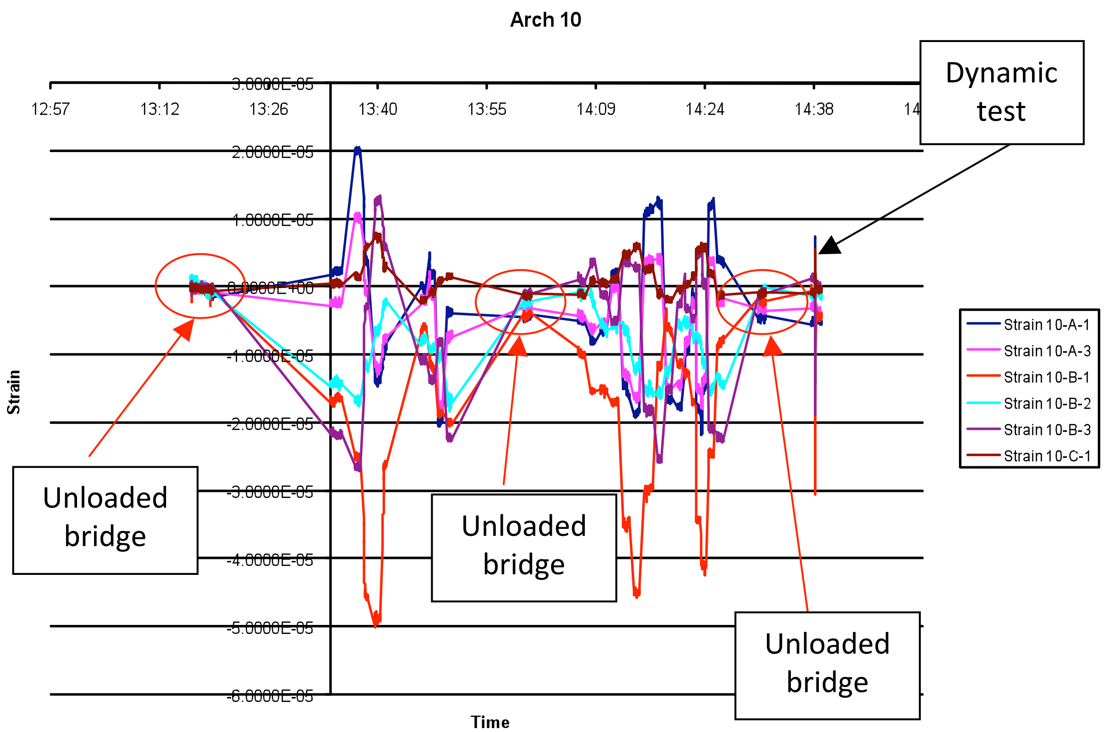


Figure 16: Live Load Data for Sections A, B, C of Arch 10

The maximum live load strain averaged over the time period for each static loading was negative 0.0068%. This is 0.4% of the average failure strain (1.74%) of the laminate from coupon testing.

As can be seen in Figures 10 through 13, the strain gage data appears to indicate both high variability and drift. There are many possible causes of this including temperature variation, under-regulated voltage to the instrumentation and imperfect solder joints at the strain gages.

2.2.5.2.6 Conclusions

The maximum measured values of strain in the FRP and concrete are compared to the strain capacities in Table 9. The values of strains represent 25 truck positions and 21 strain measurement locations. The maximum measured positive strain in the FRP, which was observed in laboratory testing as the critical failure mode, is 621 times less than the tension strain capacity. This high number results from the very low strains measured during field load testing, and illustrates the reserve capacity that the Neal Bridge FRP has in tension.

Table 9: Measured Strains during Field Load Test Series vs. Material Strain Capacity

	Maximum Measured LL Strain (i) (1)	Calculated DL (ii) Strain at location of maximum LL strain (2)	Material Strain Capacity (iii) (3)	Total Strain (DL + LL) (4) = (1) + (2)	Strain capacity / demand (iv) (5) = (3)/(4)
Positive (tension)	2.8E-05	-8.66E-05	0.0174	2.8E-05 (v)	621
Negative (compression) FRP	-6.7E-05	-9.58E-05	-0.0087	-1.63E-04	53
Negative (compression) Concrete	-6.7E-05	-9.58E-05	-0.003 (vi)	-1.63E-04	18

(i) 25 truck positions, 21 strain locations

(ii) Arch weight, backfill, wearing surface, neglecting strain of wet concrete

(iii) Based on 2.2.5.1.1

(iv) Demand based on field test conditions

(v) Conservatively neglect dead load strain

(vi) Based on ACI 318, conservative for confined concrete

Similarly, while the compressive strength of the concrete did not control in the laboratory tests, the maximum compressive strain in the FRP adjacent to the concrete in Table 9 is 18 times less than the compressive strain capacity of unconfined concrete. This further serves to illustrate the reserve capacity of the Neal Bridge.

As discussed, actual pressures in the cell locations after construction could not be determined. Therefore, the change in pressure measured during the load testing was determined relative to pre-load test measurements. Pressures were measured at 5-second intervals, the fastest rate possible. Due to the slow sampling rate, no useful data were collected during the dynamic test.

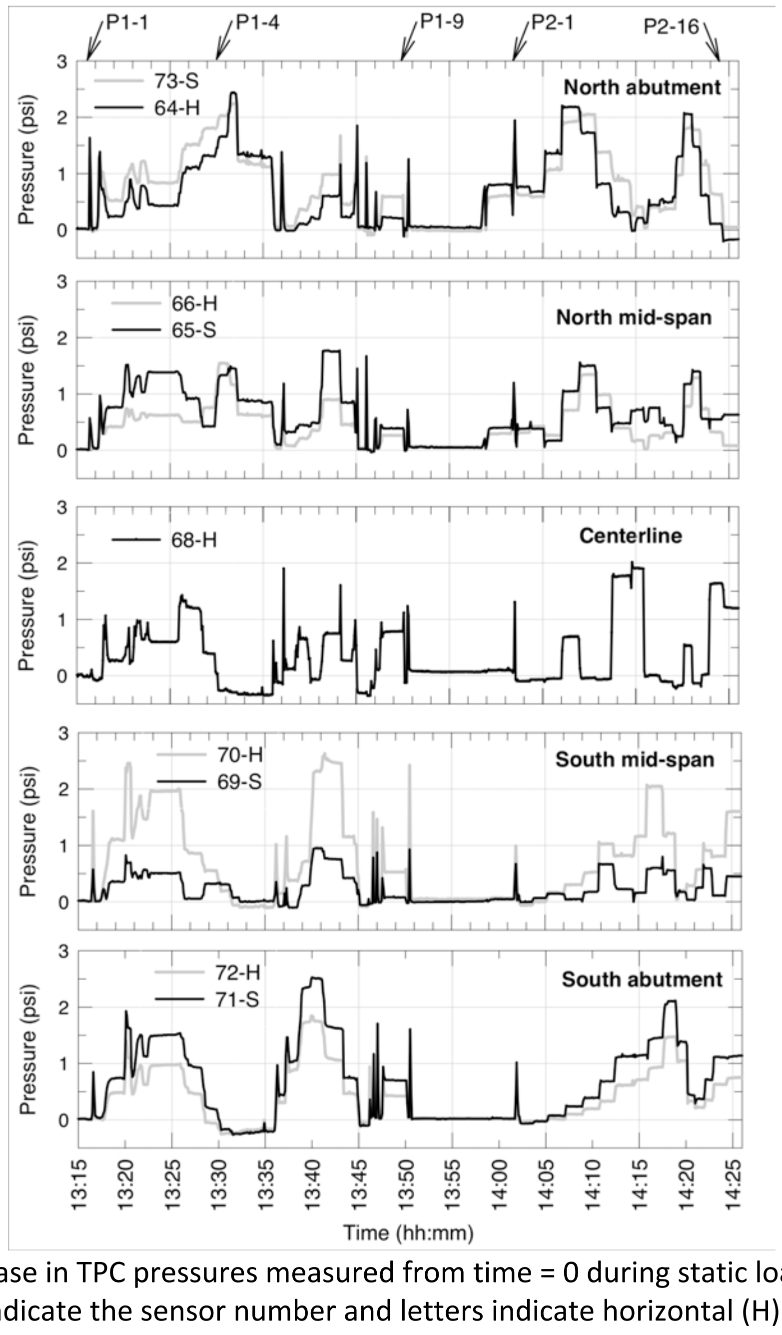


Figure 17: Increase in TPC pressures measured from time = 0 during static load test (Note: legend numbers indicate the sensor number and letters indicate horizontal (H) and sloped (S)).

Figure 17 shows the increase in pressures measured for all TPCs during static load testing. The numbers at the top of the figure indicate the approximate load regime, where P1-1 corresponds with side-by-side pressure loading regime 1 and P2-16 corresponds with tandem pressure loading regime 16 (Table 7 and Table 8).

Results show there is less than a 3-psi increase in pressure in all sensors. These values are small considering that the truck loading is 125 psi, assuming two tire patch areas of 10”x20” per axle. With increasing depth, surface loading is distributed through the soil mass and geogrid structure laterally away from the bridge. It is of interest that the south abutment, north mid-span, and north abutment sloped TPCs have greater measured pressures than the horizontal TPCs. This indicates that bridge deflections during loading engage the soil mass parallel to the arch tangent. It is unknown why the south mid-span horizontal TPC shows greater loading than the sloped TPC.

The pressure increase at each location is dependent on the surface location of the trucks, as expected. Additionally, the sensors directly under the load do not have the greatest pressure differences for a particular load scenario. The greatest pressure is from sensors that are responding to arch deflections. This is best shown for tandem load position 16 (P-2-16). The rear axle of the second truck is directly over the midspan TPC (68-H), while the front axle is south and over TPCs 69 through 72. The sloped TPC (73-S) at the north abutment shows a load of nearly 1.6 psi during this load scenario, even though there is no direct vertical loading at that part of the bridge. Horizontal TPC 64-H only shows a pressure increase of 0.8 psi, further indicating arch deflection is the likely cause of loading TPC 73-S.

Figure 18 shows the difference in pressure measured adjacent to the arch and adjacent to the decking between arches for the northern mid-span location. It illustrates that for most loading scenarios, the TPC along the arch registers higher pressures than the TPC over the decking. During most of the static load testing, it is greater than 75% different (where % change is calculated as the difference between the arch and decking pressure normalized by the arch pressure -

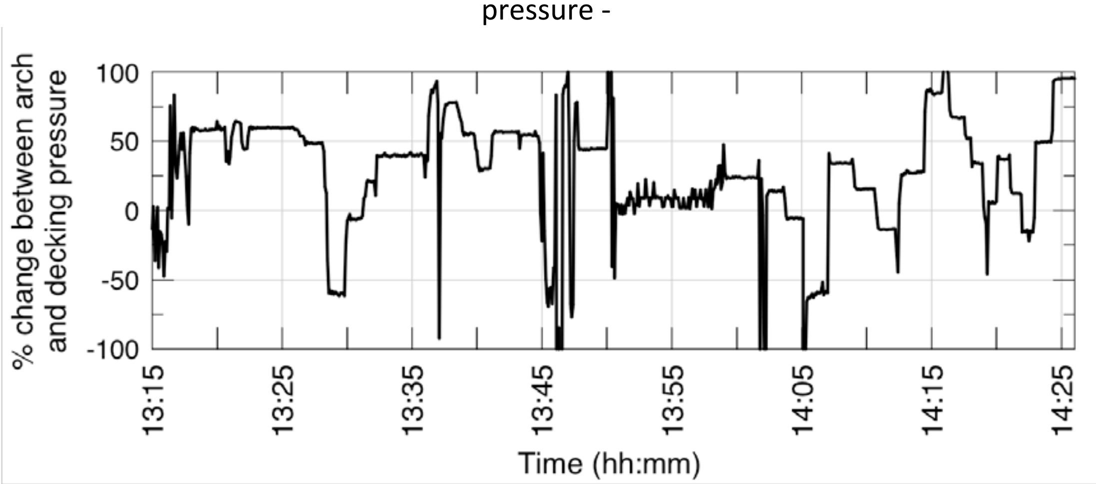


Figure 19). It is interesting to note that during side-by-side load regime 4 (P1-4), which is near the central mid-span of the arches, the pressures are similar. For this scenario, the decking and arch are likely deforming together as there is little soil between the pavement and the arch to

redistribute stresses away from the structure. Similar results occur when loading is near the centerline and just to the southern end of the bridge.

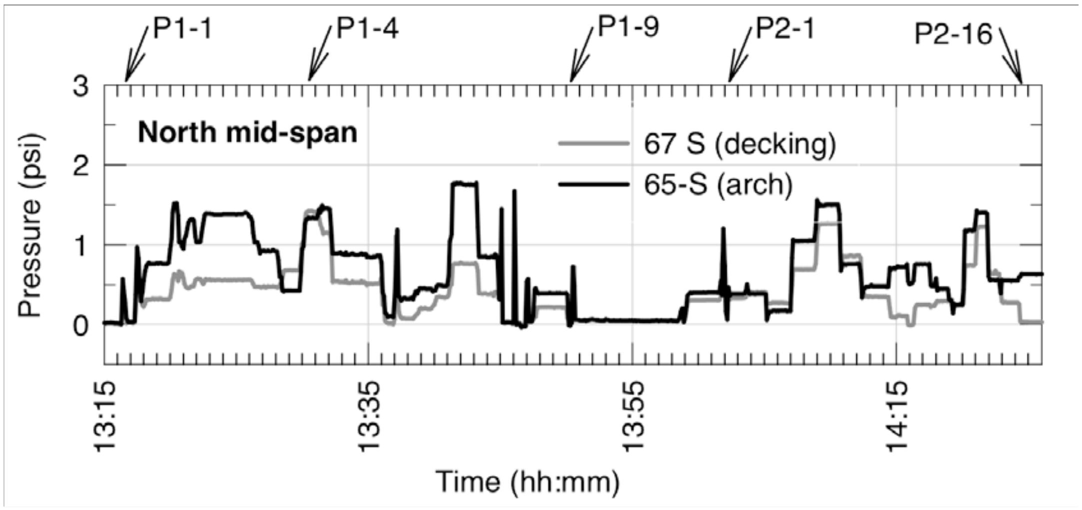


Figure 18: Difference between pressure increases due to loading at the arch and between arches adjacent to the decking.

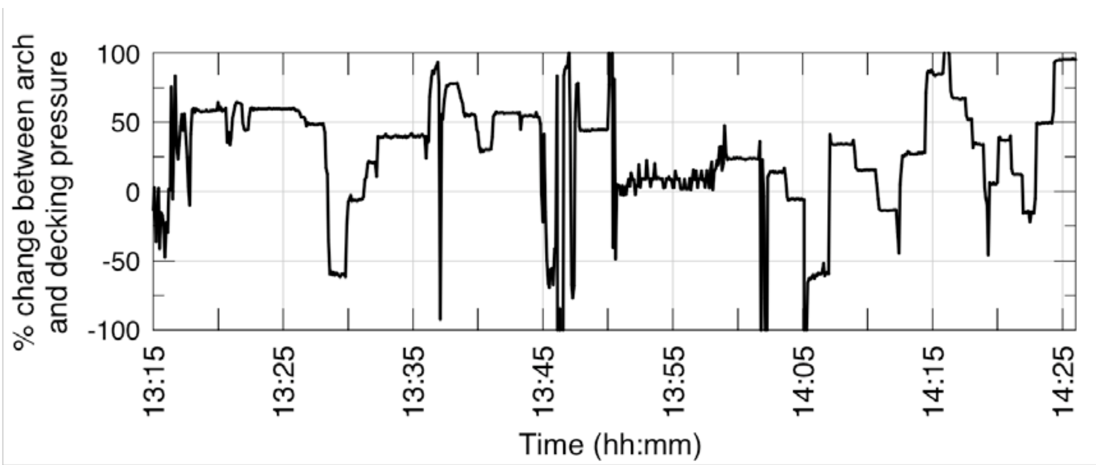


Figure 19: Percent change in pressure measured between arches and over an arch.

It has been shown that all static loading conditions resulted in less than 3 psi increases in pressure, which is significantly less than the applied load at the surface. However, it was of interest to determine how much of a pressure change resulted compared to the in situ dead load at each of these locations. As previously mentioned, actual pressures after bridge completion could not be determined due to data acquisition problems. Therefore, dead loads for the horizontal TPCs were estimated based on the assumed soil density after compaction (using RC= 95% for the dry unit weight and the corresponding average water content from the Standard Proctor compaction curve) and the weight of the paving and base paving layers. Table 10 shows the percent change in pressure determined during loading. As expected, the shallowest locations experience the greatest increase in pressures. This information should be used with caution, however. It is highly likely that greater vertical and horizontal stresses were "locked-in" during compaction than estimated from soil weight. Therefore, the dead loads calculated here represent the lower bound of earth pressure for the unloaded structure. In reality, the percent change in pressure at these locations will be less depending on the actual pressure at these locations.

Table 10: Increase in pressure during loading relative to estimated in situ vertical stresses

TPC	Location	Estimated Dead Load (psf)	Maximum pressure change (psf)	% Pressure Change
72-H	South abutment	1208.8	211.7	17.5
70-H	South mid-span	685.4	348.9	50.9
68-H	Peak	483.2	254.8	52.7
66-H	North mid-span	1013.0	222.1	21.9
64-H	North abutment	1404.5	350.7	25.0

2.2.6 Maintenance and Repair History – This section not included.

2.2.7 Coating History – This section not included.

2.2.8 Accident Records – This section not included.

2.2.9 Posting – This section not included.

2.2.10 Permit Loads – This section not included.

2.2.11 Flood Data – This section not included.

2.2.12 Traffic Data – This section not included.

2.2.13 Inspection History – This section not included.

2.2.14 Inspection Requirements

2.2.14.1.1 Schedule and focus

The Neal Bridge should be inspected every 24 months. Inspection should include, at a minimum, the arches (surface and shape), the headwall (shape), and the decking above the arches (surface and shape). In addition, the condition of selected bolts along the inside of the headwall should be inspected. Three non-adjacent bolts on each side of the bridge should be inspected. If deterioration is found, additional inspection may be required. The fasteners which connect the decking to the arches are primarily for construction, and do not require inspection.

2.2.14.1.2 Arches

In general, it is expected that the tubing material will retain its glossy coat unless subjected to abrasion. It is, however, possible that some areas may produce a chalky surface if exposed to extreme ultra-violet light (UV). This should not be confused with abrasion. Areas of the arches that are no longer glossy should be categorized as follows:

Chalky – these areas are caused by excess UV, not abrasion. It is likely that this chalky surface will protect the material from additional UV but if significant areas are found, additional investigation is warranted.

Loss of sheen – these areas should be noted for follow-up inspection but are not otherwise critical.

Dry undamaged fabric – these areas should be recoated with an appropriate resin system but do not represent structural damage. If the source of abrasion is evident, it should be removed or protected against. The area should be marked for follow-up inspection.

Damaged fabric – torn or cut fibers represent structural damage. These areas should be analyzed to determine percent capacity and a suitable layer of externally bonded reinforcement should be applied. The area should be marked for follow-up inspection.

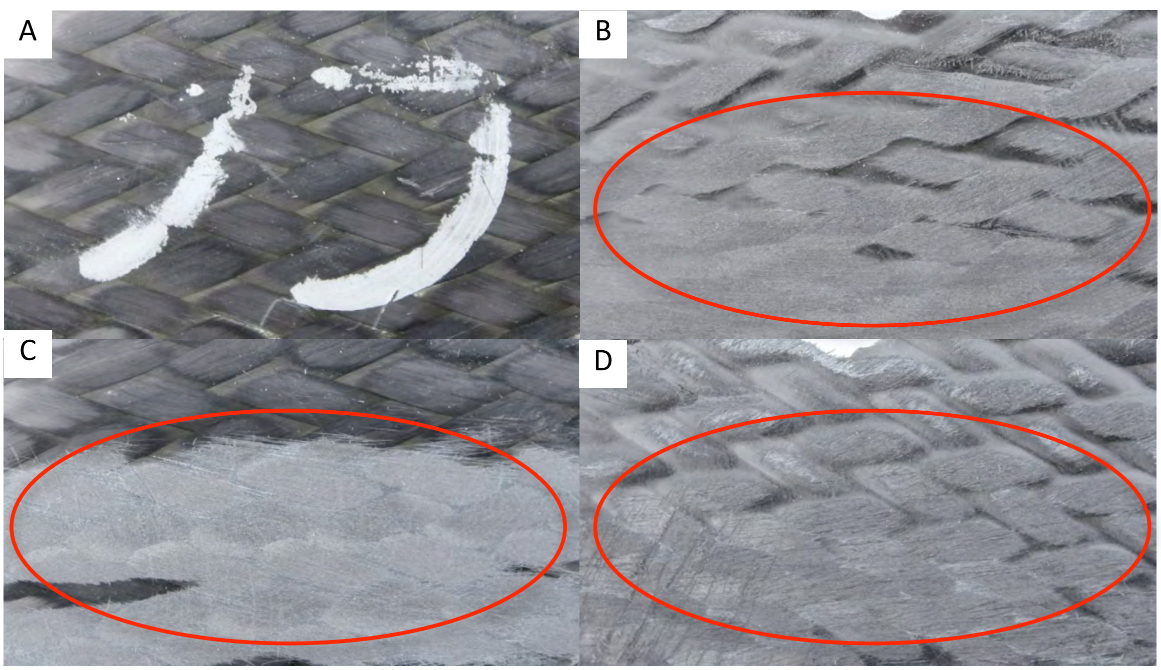


Figure 20 – Approximately 1” x 2” (25 mm x 50 mm) sections of arch in various conditions: (A) No damage (white paint marks) (B) Light loss of sheen, no structural damage, report only (C)

Heavy loss of sheen, light dry undamaged fabric, no structural damage, should be recoated with resin (D) Light damaged fabric, may require structural repair, should be recoated

Arches as installed were not perfectly round either in cross-section or in gross radius. No kinks or other sharp transitions were noted, however, and inspections should include examination of overall geometry for consistency from arch to arch and along each arch. Note that the surfaces of the tubes themselves have some sharp ridges of clear resin. These are part of the manufacturing process and their presence is not of structural concern.

Arches should be tapped lightly with a hard object at roughly 12" (30 cm) intervals along their length, varying location from top to bottom of the arch. Tapping arches should produce a solid sound. Any hollow sounding areas should be reported immediately, and an effort should be made to measure the area that is hollow sounding. Note that some voids were found during construction and filled with resin. These areas may sound slightly different from adjacent areas, but should not sound hollow. The repaired arches are #2, #3, #4, #5, #13 & #15 counting from the downstream end. Voids were found and filled within a foot or two on each side of center as well as directly at the crown. They were all within the top few inches of the tube.

The outermost arch on each side supports the headwall skin. The joint between it and the supporting arch (just to the interior) should be checked for signs of damage including cracks or extreme discoloration.

2.2.14.1.3 Decking

The bottom of the decking spanning between the arches should be glossy white. Areas that are no longer glossy should be categorized and treated the same as the arches.

Decking should run flat from arch to arch. The midspan of the decking should not be more than 1/8" (3.2 mm) below the straight line connecting the two points of contact with adjoining arches.

The following deflection limits are recommended based on a maximum allowable bending stress in the decking panel. This bending stress is given by the manufacturer, Enduro Composites, and uses a safety factor of 2.5. A deflection limit of 3/8 inch is recommended for the panel perpendicular to the arches and a limit of 1/4 inch is given for the bottom flange of the decking in the direction parallel to the arches. A combined total deflection between arches at the center of the bottom flange of 5/8" could be seen and is acceptable. See Figures 15 and 16 for clarification of deflection requirements.

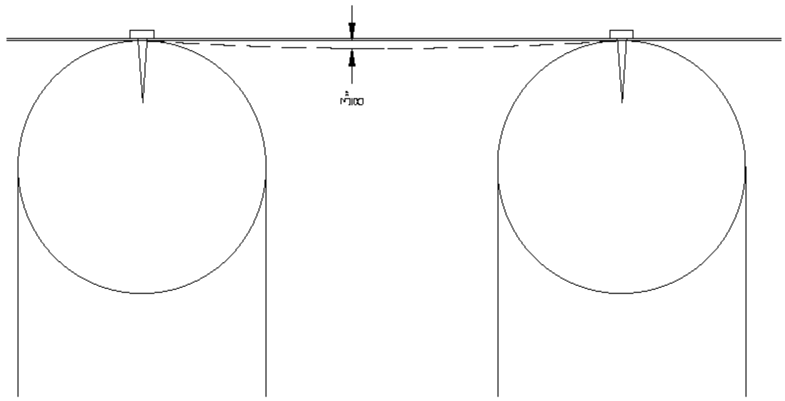


Figure 21: Allowable Decking Deflection Perpendicular to Arches

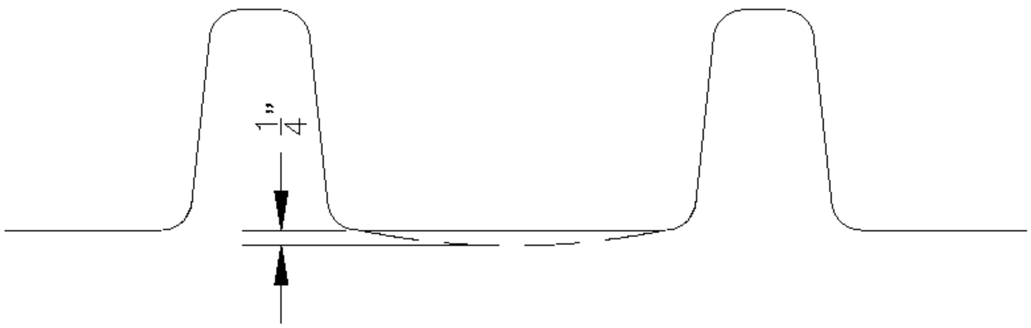


Figure 16: Allowable Decking Bottom Flange Deflection

2.2.14.1.4 Headwall

The headwall should be roughly plumb and planar. The east headwall showed some noticeable bulging, especially at the top, immediately after backfill was complete. The top of the headwall geometry should be recorded in the as-built drawings. Changes from the as-built condition should be recorded. Plumbness should be measured and recorded at four to five locations on each headwall. Appendix D contains images of the inspection notebook from the resident engineer with initial measurements. These measurements were based on a string line. Future measurements should provide accuracy and repeatability within 1/8" such that a 1/4" movement can reliably be measured. Accelerating movement will require corrective action that may include excavation and replacement of either the geogrid or the connections between the headwall and the geogrid. Decreasing movements of up to 1/4" from measurements of 4/9/09 should be acceptable.

2.2.15 Structure Inventory and Appraisal Sheets – This section not included.

2.2.16 Inventories and Inspections – This section not included.

2.2.17 Rating Records – This section not included

2.3 Inventory Data – This section not included

2.4 Inspection Data – This section not included

2.5 Condition and Load Rating Data

2.5.1 General

The load rating for this bridge was based on Section 6 of the Manual for Bridge Evaluation (AASHTO 2008). We followed 6A.1.7.1 – Design Load Rating using the allowance for alternate analysis methods. The analysis method and results are presented below.

2.5.2 Revised Condition and Load Rating Data

The moment-curvature response for the concrete-filled FRP arch tubes was predicted using the model developed by Burgueño [1999]. Material input parameters were determined as described in the respective sections of this report. The ultimate tensile strength of the laminate was reduced using the reduction factor C_E = 0.90, as specified in ACI 440.1R Section 7.2 [ACI, 2006] for carbon composites exposed to earth and weather. The nominal moment capacity, M_n, predicted by the model was reduced by φ = 0.55, as specified in ACI 440.1R Section 8.2.3 for tension controlled sections. The resulting reduced moment capacity is φM_n = 690 in*kip. The predicted moment-curvature response for the arch members is shown in Figure 17.

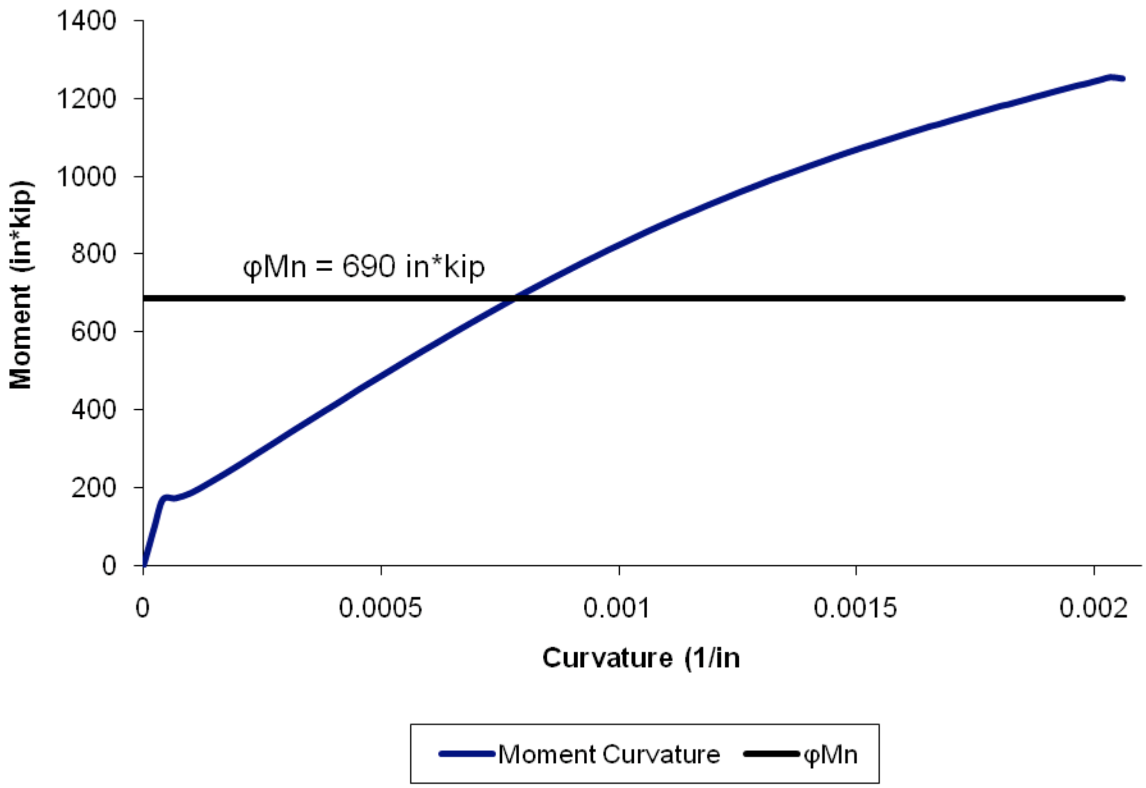


Figure 17. Predicted Moment-Curvature Response and Reduced Capacity for Concrete-Filled FRP Beam Members

The service level load effects were determined for the Neal Bridge structure using the finite element model for calculation of the load rating factors *RF*. The moment envelopes are shown in Figure 15 and the values are given in Table 1. The load-rating factor, *RF*, was calculated using Equation 3.

$$RF = \frac{\phi M_n - \gamma M_D}{\gamma M_L} \qquad \text{Equation 3}$$

The calculated load rating factors are given in Table 8. At each section along the arch, maximum positive and negative moments were calculated. In addition, dead load moments using γ of both 1.25 and 0.9 were calculated. The ultimate dead and live load moments were combined to create the maximum absolute moment. The load-rating factor was taken as the minimum value of *RF* from the analysis. From the values in Table 1 the minimum *RF* = 3.7.

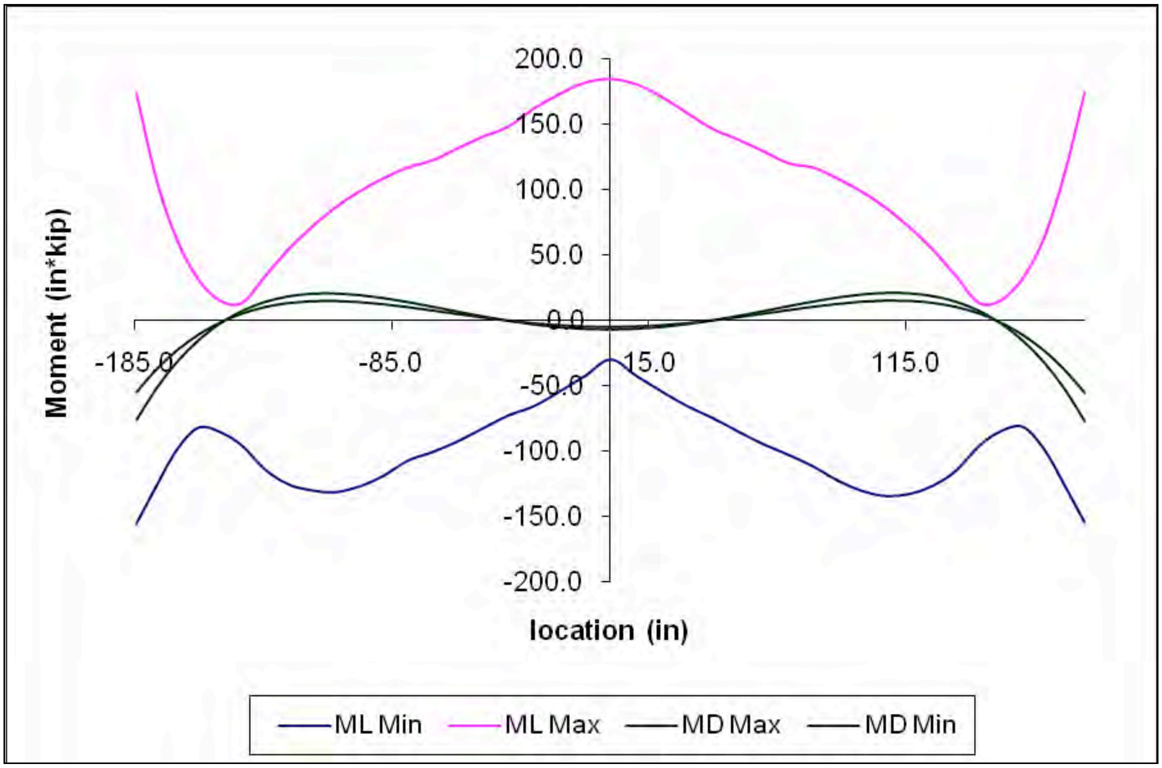


Figure 17. Dead and Live Load Moment Envelopes

Table 8. Moment Envelope and Load Rating Factor

Distance From Crown (in)	M_{Lu} (in*kip)		M_{Du} (in*kip)		Minimum RF
	Maximum Negative	Maximum Positive	γ = 1.25	γ = 0.9	
184.4	-154.3	174.5	-77.2	-55.6	4.3
176.6	-125.8	112.3	-52.5	-37.8	5.8
168.6	-98.1	63.0	-32.0	-23.1	7.2
160.4	-81.1	32.4	-15.4	-11.1	8.6
152.0	-84.3	15.4	-2.3	-1.7	8.2
143.4	-95.6	13.3	7.5	5.4	7.1
134.7	-114.2	33.3	14.4	10.4	5.9
125.7	-125.9	53.6	18.8	13.6	5.3
116.7	-132.4	70.8	21.0	15.1	5.0
107.4	-134.1	85.8	21.4	15.4	5.0
98.1	-130.3	98.3	20.2	14.6	5.1
88.6	-122.5	108.5	17.9	12.9	5.5
79.0	-111.6	117.0	14.9	10.7	5.8
69.4	-102.6	120.3	11.3	8.1	5.6
59.6	-94.5	129.5	7.5	5.4	5.3
49.8	-84.9	138.5	3.8	2.7	4.9
39.9	-74.4	146.5	0.3	0.2	4.7
30.0	-65.4	158.6	-2.6	-1.8	4.3
20.0	-54.1	171.2	-4.8	-3.5	4.0
10.0	-42.1	180.9	-6.2	-4.5	3.8
0.0	-29.5	185.0	-6.8	-4.9	3.7

RF is increased or reduced based on the adjustment factor *K*, which relates the load test results to the calculated response. *K* accounts for two factors: (1) the ratio of computed strain in the member to measured strain, and (2) the ratio of load effects due to the test truck to those due to the design vehicle. *K* was calculated for each section of the bridge, and the minimum was taken as the overall load rating factor for the bridge. The calculations are given below for the strain gage, location and truck position that produced the lowest overall load rating factor. It should be noted that the calculations do not necessarily represent the maximum strain measured during the load test, but rather the combination of measured strain, and predicted strain under both the test vehicle and design vehicle that produce the worst case load rating factor for the structure.

$\varepsilon_t := -7.105 \cdot 10^{-6}$	Measured strain during load test for worst case load rating
$\varepsilon_c := -3.772 \cdot 10^{-6}$	Corresponding calculated strain due to test vehicle
$K_a = \frac{\varepsilon_c}{\varepsilon_t} - 1 = -0.469$	Equation 2: Factor Comparing Predicted and Measured Strain (Ref. 4. 8.8.2.3.1-2)
$T = \varepsilon_c = -3.772 * 10^{-6}$	Calculated strain due to unfactored test vehicle
$W = -5.32 * 10^{-7}$	Calculated strain due to unfactored gross rating load
$K = 0.531$	T/W

$K_b = 1.0$ — K_b – (Ref. 4. Table 8.8.2.3.1-1)

$K = -1 + K_a K_b = 0.531$ — Equation 3: Adjustment Factor (Ref. 4. 8.8.2.3.1-1)

$RF_T = RF_c K = 1.964$ — Equation 4: Load Rating Factor for Live Load Capacity Base on Load Test Results (Ref. 4)

Based on the above calculation, the minimum load rating factor for live load capacity based in the results of the field load test is 1.96. The load rating for the bridge is 1.96 times the design live load. The design tandem produces the worst case load effects in this structure. The resulting load rating for the arch members is 1.96 x 25 kips = 49 kips.

2.5.2.1 Conclusions

Based on the results of field load testing and the analytical load rating a revised load rating has been determined for the bridge. A minimum load rating factor of 1.96 was found. The load rating for the bridge may be taken as 1.96 times the design live load, or 49 kip per tandem axle. The total live load rating for the bridge based on a pair of tandem axles is 98 kip.

2.6 Local Requirements – This section not included

2.7 References

(ACI) American Concrete Institute. (2005). *Building Code Requirements for Structural Concrete (318-05) and Commentary (318R-05),* ACI Committee 318, American Concrete Institute, Farmington Hills, Mich. 2005.

(ACI) American Concrete Institute. (2006). *Guide to the Design and Construction of Structural Concrete Reinforced with FRP Bars (ACI 440.1R-06).* Farmington Hills, MI: American Concrete Institute.

AASHTO (2007). *LRFD Bridge Design Specifications*, 4th ed. AASHTO, Washington D.C. 2007.

AASHTO (2007). *Manual for Bridge Evaluation,* 1st ed. AASHTO, Washington D.C. 2008.

Bannon, D. (2009). *Characterization of Concrete-Filled Fiber Reinforced Polymer Arch Members.* M.S. Thesis. Department of Civil and Environmental Engineering, University of Maine, Orono, 2009.

Burgueño, R. (1999). System Characterization and Design of Modular Fiber Reinforced Polymer (FRP) Short- and Medium-Span Bridges. (Doctoral dissertation, University of California, San Diego, 1999). (UMI No. 9928617).

Appendix A. Material Data Sheets

GRAFIL INC.

PYROFIL™ TR50S 15K

Typical Fiber Properties

Tow Tensile	Strength	710 ksi 4,900 MPa	JISR 7601
	Modulus	35 msi 240 GPa	
Typical Density		0.066 lb/in^3 1.82 g/cm^3	JISR 7601
Typical Yield	15K	496 yds/lb 1,000 mg/m	JISR 7601

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Web: www.grafil.com

ISO 9001:2000
FM 58416

07/2006

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Important: The technical information contained herein is not to be construed as warranties and no patent liability can be assumed. This information can be used for material selection purposes only.

PPG Fiber Glass

TECHNICAL DATA SHEET
Hybon® 2022 Roving

Application: ***Hybon® 2022 Roving*** *is a single end roving for filament winding and weaving/knitting applications and is made of electrical (E) glass fiber. This roving is compatible with polyester, vinyl ester, epoxy, and phenolic resin systems.* ***Hybon® 2022 Roving*** *is designed for applications that require maximum wet-out and wet-out consistency, together with good abrasion resistance and processing characteristics. It is suitable for applications such as piping in oil-field CO_2 gathering systems and pressure cylinders.*

- Provides strand hardness without sacrificing rapid and complete wet-out
- Excellent payout and package transfer
- Low resin demand during processing
- Excellent package transfer efficiency through the use of an outer adhesive film
- Supported by PPG's extensive technical resources
- Product is manufactured in conformance to ISO 9002 requirements

PRODUCT DESCRIPTION

Type of Fiber	E-Glass (ASTM D578-98, paragraph 4.2.2)													
Type of Sizing	Silane													
Roving Yields, nominal ±7% (yd/lb)	103	206	218	225	250	288	330	413	450	675	827	900	1200	1800
Tex, nominal ± 7% (g/km)	4800	2400	2275	2200	1985	1722	1500	1200	1100	735	600	550	413	276
Fiber Diameter, nominal	T	MN	MN	T	M	LM	Q	MN	MN	K	K OR MN	M	K OR MN	K
Micrometers, µm	24	17	17	24	16	15	20	17	17	13	13 OR 17	15	13 OR 17	13
Percent of Sizing (by wt. of glass)	.55	.55	.55	.55	.55	.55	.55	.55	.55	.55	.55	.55	.55	.55

PACKAGING & PALLETIZING DATA

Packaging Option 1:

- Yields: 103, 206, 413, 827 & 900
- 48 packages/pallet
- Pallet weight: 980 kg
- Package weight: 20.4 kg

Packaging Option 2:

- Yields: 218, 225, 250, 288, 330, 450, 675, 1200 & 1800
- 60 packages/pallet
- Pallet Weight: 1,225 kg
- Package Weight: 20.4 kg

A First-In-First-Out (FIFO) stock control system is recommended to minimize the influence of storage conditions.

Storage: These products should be stored at room temperature and at a relative humidity of 65% +/- 10%. To avoid problems with humidity or static electricity, the glass product should be conditioned in the working area prior to use.

Caution: To avoid the possibility of potential injury, maintain column stability by limiting pallet stacking to two high as noted on individual shipping container.

1-800-613-0155 ▪ fgcustservice@ppg.com ▪ www.ppgfiberglass.com

A&P Technology

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Cincinnati, Ohio 45245-1055
Tel: (513) 688-3200 Fax: (513) 688-3201
sales@braider.com www.braider.com

Data Sheet for Product Code: **UM6447**

Machine Size: 272

Raw Materials:

Material Name	Manufacture	Ends per Carrier	Number of Carriers
FG 450 Hybon 2022 Roving	PPG Industries	1	272 (Bias)
Spandex 1120 Denier V800	Asahi Kasei Fibers Corp.	2	136 (Axial)

*Refer to manufacture datasheets for raw material properties.

Nominal Finished Good Properties:

ID	Angle (+/-3°)	Ft/Lb (+/-10%)	GSM (Oz/Yd^2)
8.3	0° +/- 45°	3.25	691 (20)

Calculated Nominal Expanded Condition Properties:

ID	Angle	Ft/Lb	GSM (Oz/Yd^2)
11.6	0° +/- 81°	0.75	2142 (63)

 Issued: 03-10-2009

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sales@braider.com www.braider.com

Data Sheet for Product Code: **UM6448**

Machine Size: 600

Raw Materials:

Material Name	Manufacture	Number of Filaments	Ends per Carrier	Number of Carriers
Carbon TR50S 15K (.8%)	Grafil Inc.	15000	1	600 (Bias)

*Refer to manufacture datasheets for raw material properties.

Nominal Finished Good Properties:

ID	Angle (+/-3°)	Ft/Lb (+/-10%)	GSM (Oz/Yd^2)
24	+/- 45°	1.73	448 (13)

Calculated Nominal Expanded Condition Properties:

ID	Angle	Ft/Lb	GSM (Oz/Yd^2)
11.6	+/- 20°	2.30	698 (21)

A&P Technology Proprietary Information Issued: 03-10-2009

Appendix B. Lessons Learned

A four-hour meeting was held at AEWC to review the Neal Bridge project and document the lessons learned amongst the Maine Department of Transportation, AEWC and Stetson & Watson. One of the key outcomes is that the arches can be installed in half a day for a bridge like the Neal Bridge. The most time consuming procedures were the foundation fabrication and the backfilling.

The arch foundation should be simplified by using a rectangular form encompassing all the arch ends. Several ideas were advanced to facilitate the installation of the arches. Vertical and horizontal alignment jigs could expedite the placement of the arches, improve the accuracy of positioning, and reduce the construction stage on-site solution development. Sheet piles as head wall material are a good solution that perhaps should be extended to wing walls. Backfilling was impeded by the geogrid. Headwall attachment and backfilling presents one of the most significant opportunities for time saving.

For more details and a list of the lessons identified during the meeting please the table on the next page.

Issue	Influencing factors	Solution Employed	Recommendation for future
All steel must be domestic steel, some steel had to be imported			Material selection
Design not complete, changes on the run, many change orders	Tight schedule		Need a complete design from the beginning
Rock (ledge) was too deep			Site selection
Headwall appearance and alignment Need temporary support head wall construction: contract needs proper tools Connection needs a solid connection, should not rely upon geogrid to hold up head wall initially	Once the soil was put down temporary ties could only influence head wall above the soil level	Whalers	1. Different head wall details 2. Need temporary bracing detail 3. Could have rigid temporary bracing that becomes permanent 4. Use Conspan type headwall
Sheet pile Good solution	3D cut for fit		1. Pre-cut sheet pile for a better fit into channel 2. Use concrete filler in channel
Facing sheet on headwall is not necessary and would probably look better if left off.	Color was too white. A different color may be more acceptable	Fascia	1. Let the sheet pile be seen. 2. Use fascia to mask headwall alignment
Foundation - concrete piles	1. Did not have head room drilling rig with overhead wires 2. Ledge variation was 2 to 8 feet 3. Piles would have been too close together 4. DOT geotechs wanted foundation to bedrock 5. More contractors have ability to dig holes and fill rather drill	Full foundation to bedrock	Dig two holes and fill with concrete and cap
Rebar was difficult to work with, difficult to hold everything in place while filling with concrete	Tight schedule		1. Rebar detail should have been in several pieces limited to 90 deg bends 2. Footing should have been longer across the road, one foot on each side to allow forming 3. J hooks connecting footing to arches should have been shorter, 90deg bend inside footing to allow constructability to hold them in place

Arch alignment	Method is not a big time factor	1. 4x4 wood beams strapped to the arches 2. plywood template was used for horizontal alignment	1. Use full length template with composite box bear with integrated straps for full width of bridge for vertical alignment of arches, could be a permanent brace to be left in place for attaching the headwall 2. Should also include radiused cutouts for horizont alignment 3. Include a reference point for alignment, such as a short, several inch, vertical cut, or a reference point the arch to align the arches relative to each other 4. Need individual pieces to space arches during placement, straps are acceptable and do not take lo to employ
Arch handling: difficult to handle without some kind of handle			1. Integrate handles that maybe could be used as spacers
Arch foundation	Irregular cut at end of arch may have been conducive to the arch not floating		1. Set arches on flat footing and use a rectangular fo encompassing all arch ends together 2. Redesign rebar detail to facilitate arch install. Minimize transverse rebar. 3. 1.5" step does not require a bulkhead
Decking - no complaints			Self tapping screws worked well
Slump	1. 10 inch slump specified 2. Sunrise did not do a trial batch		1. Specify slump flow, must have trial batch 2. Specification should accommodate the supplier 3. Need at least a 3 inch hole, preferably 4 inch diameter 4. If flow is slow increase the super
Geo grid attachment Geo grid is very time intensive	1. Geo grid doubled thebackfill 2. A lot of hand compact around layers of geo grid	Eyebolts and bars	1. Use a set of angle bars to clamp geo grid 2. Eliminate geo grid, especially above the arch elevation
Guardrail			Use enough cover so that a guardrail does not have be in concrete, can be driven
Staged construction			The design is amenable to staged construction, one lane at a time
Headwall/Wing wall			Continue headwall design to wingwalls

Appendix C. Construction of the Neal Bridge

One major attribute to the arch technology used in the structure of the Neal Bridge is its lightness and subsequent ease of construction. Demonstrating this attribute was a major goal of this project. This section will highlight the major areas of construction with emphasis on the composite technology components.

Foundation

The foundation for the Neal Bridge was designed as a typical reinforced concrete foundation. Four concrete placements were used in the construction of the foundations. They were the "seal" pour for the concrete beneath the footing and underwater, the footing pour where the base of the arches sat, the "cap" pour to cap the old abutment, and the "curb" pour which encased the base of the arches. The seal pour was Class S concrete to ledge. The other concrete in the footing was Class A structural concrete. Requirements for these two classes of concrete can be found in the MDOT specifications for this project. Other than some trouble placing the concrete in the north abutment seal pour and some redesign that followed, there were no significant problems for the foundation construction.

Arches

The erection of the arches was an important and exciting step in the construction of the Neal Bridge. In one day the arches were shipped from the University of Maine in Orono to the bridge site and erected. Some shimming and preparations were done in the morning of the following day.

The arches were shipped from Orono in a box tractor trailer in two loads. Once on site they were unloaded from the truck, inspected and then laid out on the grass near the bridge side. A boom truck was positioned to pick up the arches from the lay down area and place the arches on the footings. Hand labor was not used to move the arches onto the footing due to safety and walking conditions. Some arches were set in place by hand once they were in the hole. Once on the footing the arches were spaced and braced using wooden jigs. Ratchet straps available at any hardware store, were used to tie the arches together temporarily. Approximately 5 men were used in placing the arches. The upstream arch was the first to be placed and was laterally braced. Subsequent arches were strapped to this first arch and then to each other, therefore bracing the system until decking could be installed.

There were areas for improvement found when placing the arches on the footings. Finding a better way to carry the arches by hand was one suggestion. If hand labor is required to move the arches some sort of handle system is desirable. This will especially be true for larger diameter arches where personnel will be unable to wrap their arms around the diameter of the arches. A second improvement that must be made is the manufacturing tolerances of the arches. As mentioned previously in Appendix B, the difference in the shape of the arches was an issue. The half-day of shimming would have been eliminated if the arches more similar in shape.

Decking

The decking (or sheathing) was attached to the arches in one day. Three-inch long self-drilling stainless steel screws were used to attach the decking panels to the arches. The panels were 45 feet long and trimmed with large circular saw at each end to match the 7-degree skew of the bridge. 3 ½” to 4” of fill concrete was placed on top of the decking prior to backfill. The concrete was placed to protect the decking from large stones in the backfill material as well as from vandalism from underneath the bridge. There were concerns about the durability of the decking due to its thickness. Thicker decking panels or decking panels designed compositely with concrete above will be one solution to this concern.

Headwall and Backfilling

The construction of the headwall and backfilling were significant tasks in the construction of the Neal Bridge. The FRP sheetpile worked well as a wall material. The geogrid and connection scheme did not work as well as expected. The connection of the FRP sheetpile to the arches was also not a great detail. The three-dimensional cuts to fit the pile to the arches were not easy to make. Another concern raised was that the amount of geogrid lengthened the time needed to backfill the bridge with this crew. These points will be discussed more in the following paragraphs.

The backfilling began with the placement of flowable fill behind the footings. Granular backfill was used above the elevation of the top of the footing. Compaction was achieved using hand compactors, plate compactors and vibratory rollers. Maine DOT inspected the densities at each lift of granular backfill.

The geogrid was placed by hand, generally by two laborers. The unidirectional geogrid was cut parallel to the strong axis of the grid to fit past the galvanized eyebolts and wrap around the FRP rebar. See Sheet 2 and Sheet 3 of the headwall drawings on pages 6 and 7 if further clarification is needed. The geogrid was wrapped around the FRP rebar and attached to itself with a Bodkin bar from Tensar. Tension of the geogrid was achieved using steel bars and two labors prying on the geogrid. An excavator buck load of backfill material was then unloaded on the geogrid.

Vibratory rollers and heavy plate compactors were used. In future projects their use should be limited in the specification. Excessive vibrations were seen when the vibratory roller was initially used on the Neal Bridge. Its use was then limited to areas that were roughly 6 to 8 feet from the exposed deck. This seemed adequate in preventing excessive vibrations in the structure. It cannot be said that this will be true for future projects though.

Wingwalls

A precast concrete T-Wall retaining wall system was used as wing walls for the Neal Bridge. Forty sections of T-Wall were used, ten at each corner. The walls were 13'-4" tall with a 6" wide by 18" thick by 10'-0" long concrete leveling pad beneath the walls.

Paving

Paving of the Neal Bridge was conducted in November 2008 for the binder and in May 2009 for the finished wearing surface. The binder was placed November 21st, 2008 with the road opening on November 22nd.

Guardrail

The guardrails were driven as specified in the Maine DOT's standard specifications. The depth of the top of the arches was adequate which prevented the need for the posts at the crown of the arches to be shortened and their bases encased in concrete. This is a detail that is favorable and should be achieved where possible in future FRP arch bridges.

Appendix D. Inspection Notebook – Headwall Measurements

14

12/16/2008

DS HEADWALL

		4/9/09
0'	0"	
5'	7/8"	1 1/8"
10'	13/16"	1"
15'	3/8"	1/2"
20'	9/16"	3/4"
25'	3/4" & 5/8"	7/8" 3/4"
30'	1/2"	1/2"
35'	1/4"	1/4"
40'	0"	0"

DS side to side
T-WALL
14.20 M

US side to side
T-WALL
14.05

US HEADWALL

			4/9/09
0'	0"		0"
5'	~~5/16"~~	1 1/2"	1 1/2"
10'	~~3/4"~~	2 1/4"	2 1/4"
15'	2"		2 1/8"
20'	1 1/8"		1 1/8"
25'	0"		0"
30'	5/8"		5/8"
35'	1"		7/8"
40'	0"		0"