Issues with shear key cracking led the SCDOT to explore options that address the reflective cracking problem while maintaining the benefits of ABC. After joining this venture with Clemson University, SCDOT settled on the Northeast Extreme Tee (NEXT)-D girder section because of its increased rotational and translational stiffness at the shear key, geometric adaptability, and ability to employ ABC techniques. SCDOT explored the utilization of ultra high performance concrete (UHPC) as the “fill” for the shear key rather than traditional grout. The Hanging Rock Creek Bridge (HRB) located just south of Kershaw, South Carolina was selected as a demonstration project for the NEXT-D girder with UHPC shear keys. Of particular interest to SCDOT was the longterm behavior of the UHPC joints and the transverse load distribution of this new bridge system.

Results from periodic visual bridge inspections and live load testing show that the UHPC shear keys are performing well and show little if any deterioration after about 2 years of service life, and that this bridge system had transverse load distribution best predicted by typology “k” from ASSHTO.
Performance Evaluation of the SCDOT NEXT-D Beam Bridge

Final Report

By

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INTRODUCTION

According to the National Bridge Inventory (NBI), 39% of the United States’ bridges exceed 50 years of age. Additionally, another 15% of US bridges are between the ages of 40 and 49, and in 2016 an average of 188 million trips were made across structurally deficient bridges daily (Bridges, 2017). Most of the nation’s aging bridges were designed considering a 50-year life-span. The NBI provides a guide to the terms it uses to describe bridges and defines “structurally deficient” as “a status used to describe a bridge that has one or more structural defects that require attention” (A Guide, 2018). Within the state of South Carolina, 964 of the state’s 9358 bridges are considered structurally deficient (Deficient, 2018).

In 2017, South Carolina passed a bill which raises the gas tax by a total of 12 cents in 2 cent per year intervals. With the passing of the aforementioned bill, the South Carolina Department of Transportation (SCDOT) is financially positioned to improve much of the state’s infrastructure, including the state’s bridge network. As part of the state’s strategic plan for the next ten years, SCDOT plans to “increase the efficiency and reliability of our road and bridge network” and “focus our bridge replacement program to target our structurally deficient bridges” (Rebuilding, 2018). For many bridges, the state employs accelerated bridge construction (ABC) techniques to quickly rebuild unsatisfactory bridges. ABC techniques reduce on-site construction time with the intent of reducing costs and impact to transportation system users. Prefabricated components and systems are an important tool for ABC.

As a part of ABC, SCDOT has used hollow core and solid precast-pre-stressed slabs which are connected transversely by grouted shear keys. These slabs can be immediately placed in the field rather than cast-in-place concrete slabs, which significantly reduces construction time. As beneficial as these slabs can be to ABC, the shear keys connecting these slabs can be problematic, sometimes even soon after construction. Cracks often form at the shear key-precast interface as shown in figure 1a, which leads to longitudinal reflective cracks developing on the bridge deck as shown in figure 1b. Although initially these cracks are simple maintenance issues, they provide a route for water and contaminants on the roadway to access the mild and prestressed reinforcement in the deck and slabs.
Issues with shear key cracking led the SCDOT steering committee to explore options that address the reflective cracking problem while maintaining the benefits of ABC. After joining this venture with Clemson University, SCDOT settled on the Northeast Extreme Tee (NEXT)-D girder section because of its increased rotational and translational stiffness at the shear key, geometric adaptability, and ability to employ ABC techniques, among other benefits (Dreery, 2010). Additionally, several other states have begun employing the NEXT section and have observed that bridges containing NEXT girders have improved ability to resist reflective cracking as compared to other common girder sections employed. SCDOT is also exploring the utilization of ultra high performance concrete (UHPC) as the “fill” for the shear key rather than traditional grout.
After selecting the NEXT-D girder section as a candidate for short-to-medium span bridges, SCDOT was in need of further studies to investigate the long-term performance NEXT-D girder bridges, particularly at the transverse shear key. Studies were also necessary to evaluate the durability of the UHPC used in the shear key. Finally, the distribution factors of the NEXT-D girder section are not clearly defined in the American Association of State Highway and Transportation Officials LRFD Bridge Design Specification (AASHTO LRFD, 2014). Currently, recommendations on distribution factors have been provided by a Precast Concrete Institute (PCI) technical committee and other researchers (Northeast, 2017). For these reasons, further study of transverse load distribution and shear key performance in bridge systems containing NEXT-D girders is warranted. To achieve this goal a demonstration bridge which would provide a testbed for investigating these issues was selected by SCDOT.

**Hanging Rock Creek Bridge**

The bridge selected is the Hanging Rock Creek Bridge (HRB), displayed in figure 2, located over Hanging Rock Creek just south of Kershaw, South Carolina. The bridge has four simple spans; two 40 feet outer spans and two 70 feet inner spans. A cross section of each span is provided in figure 3. The southern outer span contains pre-stressed solid slab sections and the northern outer span has NEXT-D girders. The inner spans have pre-stressed voided slabs. Each span has an asphalt overlay wearing surface varying transversely from approximately 2” to 7” thick.
Figure 2: Hanging Rock Creek Bridge
The bridge is ideal for load testing because of its low average daily traffic (approximately 650 vehicles per day), variance in girder section types, and inclusion of a NEXT-D span. This allows for live load tests to investigate the behavior of the bridge span containing NEXT-D girders. For direct comparisons of reflecting cracking incident in the NEXT-D and solid slab spans.

The geometry of the NEXT-D section differs significantly from the voided and solid slab sections and uses a double-tee shaped section. Figure 4 provides a cross section of the interior NEXT-D girder section. The NEXT-D exterior section has only minor geometric differences from the interior section and thus is not shown. The NEXT-D shear key is over the full depth of the flange and the shear key width is much wider than the shear key widths of the voided and solid sections. Figure 5 displays the shear key of the NEXT-D section. The longitudinal bars (designated G1604 in Figure 5) were inadvertently omitted from the shear keys between girders 1 & 2 and between girders 2 & 3.
The bridge shear keys are comprised of UHPC provided by Lafarge. Lafarge markets UHPC under the brand name Ductal. The Ductal mix design used in the HRB contained 2% by volume steel fibers. Relative to traditionally-used grouts, the increased tensile, compressive, and bond strengths of UHPC significantly improve the strength and ductility of the transverse connection between the girders. Increased bond strength is especially important because it decreases the likelihood for cracking between the UHPC
and precast girder, thereby decreasing the likelihood for the bridge deck to experience reflective cracking. Additionally, the load distribution between the girders is maintained when cracks do not form at the UHPC-precast girder interface.

**Purpose and Scope**

The purpose of this project is to evaluate the transverse durability of the UHPC shear keys over the first 16 months of life of the HRB and to assess the transverse load distribution behavior of the spans containing NEXT-D. The transverse durability of the UHPC shear key is evaluated through live-load tests and verification of the material properties of the UHPC shear key. Specifically, this research also aims to determine live load distribution factors for moment (DFM) of the NEXT-D span. The experimental distribution factors are determined via live load tests. Strain transducers are placed on the girders to aid in determining the experimental distribution factors and data from the strain transducers is recorded while loaded trucks drive slowly (<5mph) across the HRB. These data are then compared to design values for DFM from AASHTO LRFD.

Secondary objective of the project is assessment of the durability of the UHPC shear keys. Information on this objective is provided in an appendix.

**DISTRIBUTION FACTORS**

**Transverse Load Distribution Factors**

As live load is applied to a bridge deck, a portion of the load is imparted into each bridge girder. In theory, if a particular girder in the structure experiences ductile yielding, the other girders will support more of the load allowing the structure to be redundant. With the ultimate failure load much greater than the yield load, it seems simple enough to safely design bridge girders and begs the question: what is the need for such an in-depth analysis to develop distribution factors? The reason lies in the fact that sometimes failure may be brittle rather than ductile and that the limit states considered are often “related to serviceability and service-level loads” (Barker, 2007). To that end this project aims to experimentally determine distribution factors that can be applied to the design of NEXT-D bridges.

Vehicular traffic loads (live loads) are not entirely distributed to the girder closest to the wheel line, but rather follow the relative stiffness theory. As a result, a bridge deck distributes percentages of the traffic load to each of the girders according to relative stiffness (Barker, 2007). The AASHTO live load distribution factors for shear and moment (DFV and DFM) are used to help determine what the maximum moment or shear an interior
or exterior girder will experience. Generally, as a deck increases in stiffness the distribution factors decrease since load is more evenly distributed in a stiffer deck. Stiffness of a given girder also influences how must load is distributed to it, however generally each girder in a bridge span has the same inherent stiffness. In addition to the deck and girders, the diaphragms, barriers, bridge geometry, and other bridge components also contribute to the overall stiffness of the system and therefore contribute to how the load is distributed.

Once distribution factors are determined, they are multiplied by the maximum load effects and other various safety factors to determine the design moments and shears for an individual girder. The AASHTO LRFD Bridge Design Specification (AASHTO LRFD, 2014) provides a process for calculating the shear and moment distribution factors used to determine design loads. Next, the design load is multiplied by the dynamic load allowance factor to determine the design live load of a typical interior or exterior girder. On a bridge span, the exterior girders are located at the transverse edges of the bridge and the remaining girders are interior girders.

AASHTO LRFD Moment Distribution Factors

The AASHTO LRFD DFM calculation process will be outlined in this section for the AASHTO LRFD “i”, “k”, and “g” girder section types. These girder section types are shown in figure 6 (AASHTO LRFD, 2014, Table 4.6.2.2.1-1). The PCI bridge technical committee recommended section type “i” and “k” for NEXT beam transverse load distribution design (Guidelines, 2012). Section type “g” is considered appropriate for the hollow and solid slab spans used in the HRB (Filosa, 2017).

When calculating AASHTO LRFD DFMs for the Hanging Rock Creek Bridge (HRB) NEXT-D girders, the percent difference decreases from using section type “k” to using section type “i” for interior girder DFM design is 24.8% and 17.9% for one design.
lane loaded and two design lanes loaded respectively. This difference leads to the reduction in calculated design moments for NEXT-D girders when considering AASHTO LRFD section type “i” vs “k”. Often moment controls girder designs, thus, if it compares reasonably to experimental DFMs, section type “i” would be the best section to consider when calculating AASHTO LRFD DFMs since it would lead to the most economic designs.

Design interior DFM for section type “i” or “k” can be calculated using equations 1 and 2 (for one or two or more lanes), respectively. These equations can be found in the AASHTO LRFD (Table 4.6.2.2b-1). Equations 1 and 2 are considered applicable to type “i” because the girders are assumed to be “sufficiently connected to act as a unit” (AASHTO LRFD, 2014). When considering section type “k”, the section properties of a single stem and the average stem spacing are considered in the calculation of DFMs. Additionally, the calculated DFM is doubled because both stems are assigned to each girder. Conversely, when considering section type “i”, the section properties of both stems of the beam and the beam spacing itself are considered in the calculation of DFMs. The final DFM is not doubled when considering section type “i”. Example DFM calculations for the NEXT-D span are shown in Appendix B. Examples are given for both type “i” and “k”.

For one design lane loaded:

\[
GDFM_{i1} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12.0 L t_s^2} \right)^{0.1}
\]  
Equation 1

For two or more design lanes loaded:

\[
GDFM_{i2} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12.0 L t_s^3} \right)^{0.1}
\]  
Equation 2

The variables in equations 1 and 2 are defined below (AASHTO 2014):
\[K_g = n(I_{bs} + A_{bs} + e_g^2)\]
\[n = \frac{E_{Beam}}{E_{Slab}}\]
\[K_g = \text{longitudinal stiffness parameter for the composite girder (in}^4\text{)}\]
\[S = \text{girder spacing (ft)}\]
\[L = \text{bridge span length (ft)}\]
\[t_s = \text{depth of the concrete slab (in)}\]
\[n = \text{modular ratio between the girder material and deck material}\]
\[e_g = \text{distance between the centers of gravity of the stems and flange (in)}\]
\[A_{bs} = \text{cross sectional area of the stems (in}^2\text{)}\]
\[E_{Beam} = \text{elastic modulus of the NEXT beam (ksi)}\]
\[E_{Slab} = \text{elastic modulus of the deck slab (ksi)}\]
I_{bs} = \text{moment of inertia of the stems (in}^4)\\
GDFM_{i1} = \text{distribution factor for moment for one design lane loaded}\\
GDFM_{i2} = \text{distribution factor for moment for two or more design lanes loaded}\\

AASHTO LRFD does not provide equations for determining exterior girder DFMs considering section types “i” or “k”. Instead, for calculating exterior girder DFMs for one and two design lanes loaded, the lever rule is used. The lever rule, described in AASHTO 4.6.2.2.2, is applied by assigning hinges where each interior girder supports the deck and treating the deck as simply supported. Next, moments are summed about an interior support (girder) to find the reaction at the considered exterior support (girder). When using the lever rule, the multiple presence factors outlined in AASHTO LRFD 3.6.1.2-1 must be considered to calculate the factored moments. The exception for calculating exterior DFMs is considering section type “k” with two design lanes loaded if the system uses three girders, in which equation three may be used.

\[ g = e g_{\text{interior}} \]  \hspace{1cm} \text{Equation 3}

Where:
g = \text{exterior moment distribution factor for two or more design lanes loaded}\\
g_{\text{interior}} = \text{moment distribution factor for an interior girder}\\
e = 0.77 + \frac{d_e}{9.1}\\
d_e = \text{horizontal distance from the centerline of the exterior web of exterior beam to the interior edge of the curb or traffic barrier}

The interior moment distribution factors for box beams are determined from the equations for the type “g” section. Interior girder moment distribution factor for the box girders can be calculated using equations 4 and 5 from AASHTO LRFD (Table 4.6.2.2.2b-1).

For one design lane loaded:

\[ GDFM_{i1} = k \left( \frac{b}{33.3L} \right)^{0.5} \left( \frac{I}{J} \right)^{0.25} \]  \hspace{1cm} \text{Equation 4}

Where:
\[ k = 2.5(N_p)^{-0.2} \geq 1.5 \]

For two or more design lanes loaded:

\[ GDFM_{i2} = k \left( \frac{b}{305} \right)^{0.6} \left( \frac{b}{12.0L} \right)^{0.2} \left( \frac{I}{J} \right)^{0.06} \]  \hspace{1cm} \text{Equation 5}

The variables in equations 4 and 5 are defined below:
J = St. Venant’s torsional inertia (in$^4$)\\
L = bridge span length (ft)
b = width of beam (in)
I = moment of inertia of beam (in$^4$)
$N_b$ = number of girders
$GDFM_i^1$ = distribution factor for moment for one design lane loaded
$GDFM_i^2$ = distribution factor for moment for two or more design lanes loaded

The exterior moment distribution factors for the type “g” section are determined using the equations 6 and 7. For one design lane loaded:

$$g = e g_{\text{interior}}$$ \hspace{1cm} \textbf{Equation 6}

Where:
$g$ = moment distribution factor
$g_{\text{interior}}$ = moment distribution factor for an interior girder
$e = 1.125 + d_e/30 \geq 1.0$
$d_e$ = horizontal distance from the centerline of the exterior web of exterior beam to the interior edge of the curb or traffic barrier

For two or more design lanes loaded:

$$g = e g_{\text{interior}}$$ \hspace{1cm} \textbf{Equation 7}

Where:
$g$ = moment distribution factor
$g_{\text{interior}}$ = moment distribution factor for an interior girder
$e = 1.04 + d_e/25 \geq 1.0$
$d_e$ = horizontal distance from the centerline of the exterior web of exterior beam to the interior edge of the curb or traffic barrier

Distribution factors have evolved significantly from the early 1990s to the factors in the AASHTO LRFD design code today. The evolution of these factors is highly influenced by experimental research which aids in the validation of these distribution factors and refinement of the finite element models that helped develop the factors.

Originally, equation 8 was used to determine distribution factors where “S” was the girder spacing, “D” was a constant dependent on the bridge type and material, and “g” was thought of as the number of lanes the girder carries (Barker, 2007). Compared to today’s equations, this equation is quite simple.

$$g = S/D$$ \hspace{1cm} \textbf{Equation 8}

Through finite element analyses conducted as a part of NCHRP project 12-26, Zoakie et al. (1991) and Nowak (1993) determined that the simple formula presented in equation 8 was not consistently accurate for certain bridge configurations. Through the
same project, Zoakie researched additional formulas which considered more parameters, and developed methods for determining distribution factors that provide the basis for the current AASHTO LRFD distribution factors as discussed above. Research included many computer analyses and comparing those results to experimental field analyses. The most accurate of the computer analyses were further developed to identify the parameters that most strongly influenced load distribution. Finally, many rigorous finite element analyses (FEAs) determined the distribution factors utilized in the code today (Barker, 2007).

**Experimental Determination of Moment Distribution Factors**

Although AASHTO LRFD distribution factors have now far advanced, the need to experimentally test distribution factors (whether it is just a routine examination of how a bridge responds to the code factors, an examination of how a bridge behaves after a retrofit, or an examination of how a new type of bridge component behaves) strongly persists. Several methods have been used to conduct experimental tests for determining distribution factors. Once common method for determining distribution factors for moment is to use experimentally measured bending strains. To maximize bending strain measurements, gauges are commonly placed on or near the bottom of girders near midspan.

The work reported by Idriss and Laing (2010) is one example of experimentally determined distribution factors. Live load monitoring and testing was conducted on one span of the I-25 Bridge at the Dona Ana exit in Las Cruces, New Mexico. The superstructure consisted of prestressed concrete girders with sensors embedded (during fabrication of the girders) in the top and bottom flanges. The bridge was monitored for two years (beginning at the time of prestress release) for several purposes, one of which was to determine live load distribution factors. An optical fiber sensor system was used to measure strain during regular traffic loading and live load testing. Moment distribution factors were determined from the measured strains using equation 9 (Idriss, 2010). The same equation is used to process data in the current report.

\[
GDF_i = \frac{ES_i \varepsilon_i}{\sum_{j=1}^{k} ES_j \varepsilon_j} \quad \text{Equation 9}
\]

Where:
- \( GDF \) = girder distribution factor
- \( E \) = girder elastic modulus
- \( S \) = girder, section modulus
- \( \varepsilon \) = bottom flange strain at midspan
i=ith beam
k=number of beams

RESEARCH METHOD

The Hanging Rock Creek Bridge (HRB) is a four simple span bridge which contains one NEXT-D girder span, one solid slab girder span, and two voided slab girder spans. For each girder type, the longitudinal joints (commonly called shear keys) connecting the girders and providing transverse bridge integrity are filled with UHPC. Three methods of evaluation were utilized to assess the behavior of the span containing NEXT D girders: material property documentation, visual inspection, and live load (LL) testing. Live load (LL) tests were performed in July of 2017, and January, May, and October of 2018. All live loads testing is described in detail by Filosa (2017) and Hess (2018). Additionally, testing of constituent materials is reported by Filosa. Visual inspection results are reported by Hess and summarized in the Appendix.

UHPC

A summary of the materials tests conducted in the project is provided in Table 1. The UHPC mix design was Ductal by LaFarge, and contains Ductal ready mix (cement, silica fume, and sand), high range water reducer, and steel fibers. A common Ductal UHPC mix design can be found below in table 2. The mixing of the UHPC was handled by Lafarge employees.

Table 1: Material Test Summary

<table>
<thead>
<tr>
<th>Test</th>
<th>Specimen</th>
<th>ASTM Standard</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength</td>
<td>2 in. Cubes</td>
<td>C109</td>
<td>ASTM (2013)</td>
</tr>
<tr>
<td>Splitting Tensile Strength</td>
<td>3. in. x 6 in. Cylinders</td>
<td>C496</td>
<td>ASTM (2011)</td>
</tr>
<tr>
<td>Pull-off Test</td>
<td>2 in x 2.5 in Cylinders</td>
<td>C1583</td>
<td>ASTM (2013)</td>
</tr>
</tbody>
</table>
Table 2: Typical Ductal UHPC Mix Design

<table>
<thead>
<tr>
<th>Constituent</th>
<th>lb/yd³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ductal Premix</td>
<td>3700</td>
</tr>
<tr>
<td>Water</td>
<td>219</td>
</tr>
<tr>
<td>Premia 150 (Superplasticizer)</td>
<td>51</td>
</tr>
<tr>
<td>1/2 in. Steel Fibers (2%)</td>
<td>263</td>
</tr>
<tr>
<td>w/cm</td>
<td>0.06</td>
</tr>
</tbody>
</table>

Test Methods

Compressive strength was determined by testing 2 in. x 2 in. x 2 in. cubes. The cubes were tested in groups of five at ages of 28 days, 90 days, and 6 months. Split tensile strength was determined by testing 3 in. diameter x 6 in. cylinders. The cylinders were also tested in groups of five at ages of 28 days, 90 days and 6 months, respectively.

Bond strength of the UHPC to the precast concrete was measured by using pull-off tests. Figure 10 is a picture of the test specimen after the UHPC had been cast. The specimen consisted of a 6 in. thick precast base slab with 1.5 in. of UHPC cite cast on top. The precast base slab was built at the same precast facility and using the same concrete as the bridge girders. The specimen cured for 28 days before being transported to the bridge site. At the bridge site, 1.5” of UHPC topping was placed on top of the precast base slab. The UHPC was cast using the same batch as the shear keys. After allowing the UHPC to cure the specimen was transported to Clemson University for the pull-off tests.
Multiple 2 in. diameter by 2 in. deep cored holes were drilled in the test specimen. Using a high strength epoxy and dedicated pull-off testing equipment, the 2 in. diameter UHPC overlay was pulled off of the concrete slab. Four failure modes can occur in pull-off tests: a) Failure in the substrate (precast concrete), b) Bond failure at the precast concrete to UHPC connection, c) Failure in the UHPC, and d) Failure at the epoxy bond. These four failure modes are illustrated in figure 11 below.
Visual Inspection

Visual inspections of the HRB were conducted to assess and document the bridge condition. Visual inspections were conducted at approximately six month intervals beginning with an inspection at the time of the bridge’s opening and concluding about 16 months later. During the final assessment, a photo survey of the bridge was organized to provide visual documentation of the bridge health. The photo survey is presented in Appendix A.

Live Load Tests

Four LL tests were performed on the HRB with the dates of purpose of each shown in Table 3.
Table 3: Live Load Test Summary

<table>
<thead>
<tr>
<th>Live Load Test Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Number</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>3</td>
</tr>
</tbody>
</table>

Strain Transducers and Linear Variable Displacement Transformers

The instrumentation used during the LL tests consisted of strain transducers and linear variable displacement transformers (LVDT) purchased from and calibrated by Bridge Diagnostics Incorporated (BDI). LVDTs were utilized to measure the relative horizontal displacements of the girders. The LVDTs were attached near girder transverse joints so that the LVDT arm stretched over the transverse joint and made contact with a wood block that was glued to the adjoining girder. In this way, the LVDTs measured the horizontal displacement of a girder relative to its adjoining girder and were used to indicate if the UHPC joints cracked or debonded from the NEXT D girders under load. The LVDTs were attached longitudinally at mid-span on the NEXT-D span. A close-up of an attached LVDT is shown in figure 12.
Strain transducers were attached to the bottom surfaces of girders at mid-span to measure the longitudinal girder bending strain. The bending strain data collected allowed the researchers to calculate girder DFMs and evaluate transverse integrity of the NEXT-D span. Figure 13 show a strain transducer attached to the underside of a NEXT-D girder. Strain transducers were also utilized to measure shear strain, and for this purpose were located closer to the supports and placed near the web-flange intersection in a rosette orientation.
Figure 13: Strain transducer attached to a NEXT-D girder

**Instrumentation Location and Orientation.** During the LL tests Linear Variable Displacement Transformers (LVDT) were attached at selected transverse joints at mid-span on the HRB. The typical locations of the LVDTs used are presented in figure 14.

![LVDT locations on the NEXT-D span for the LL tests 3 and 4](image)

Figure 14: LVDT locations on the NEXT-D span for the LL tests 3 and 4

Strain transducers (ST) were attached at mid-span of the NEXT-D girders during LL tests 1, 3, and 4. The location and typical orientations of the strain transducers on the NEXT-D span are presented in figures 15 and 16.
The first LL (July 2017) test indicated that approximately 5% of the load was distributed to the girder furthest away from the applied LL in all load configurations, therefore, it was deemed acceptable to not instrument girder 6 in future LL tests. By considering the load on the girder furthest from the applied load as negligible, conservative DFMs were calculated. Additionally, it was evident from the first LL test that the unloaded girders displayed a linear trend of transverse load distribution. For this reason, during future LL tests, a web in girder 5 and a web in girder 4 was not instrumented, and instead the bending strain experienced by these webs was linearly interpolated. By using minimal strain transducers during the DFM LL test, moment and shear distribution LL tests could be conducted simultaneously.
Data Acquisition System. The LVDTs and strain transducers were the only instrumentation connected to the LL DAQ system. These instruments were connected to 4-channel nodes, which wirelessly sent the measured displacements and strains to a base station located onsite. A laptop on which the measured data could be accessed and saved was remotely connected to the base station.

Truck Placement and Girder Horizontal Displacements

Truck positions and instrumentation were designed to create and measure “worst case” horizontal movement of the longitudinal bridge joints. For all four LL tests, horizontal movements across the UHPC/girder interface were measured using LVDTs. The loading used in each LL test was a standard SCDOT three axle dump truck loaded to a total weight of approximately 24 tons. In figure 21 typical truck axle weights and dimensions are provided.

![Typical Truck weight and dimensions](image)

(a)

(b)

Figure 21: Typical Truck weight (a) and dimensions (b)

Transverse truck orientations on the HRB span was selected to cause critical effects at the UHPC/girder joint. Figure 22 displays typical transverse truck locations used. For each of the truck orientations, an axle straddles a girder and effectively places maximum load directly over two joints.
Figure 22: Transverse Truck Locations

Truck Placement and Distribution Factor for Moment

Trucks placements were designed to evaluate load distribution under multiple load scenarios. The placements were designed with consideration of the primary objective of this research, specifically to experimentally determine moment distribution factors (DFMs) for the NEXT-D span of the HRB. DFMs of the NEXT-D beams were determined using data from the strain transducers attached during the LL tests 1, 3 and 4. The approximate weight and dimensions of the trucks used during LL testing was presented earlier (figure 21). Figure 23 shows the transverse truck orientations used to create maximum strains in subject girders, and, therefore, to give the largest DFMs.
Figure 23: Truck locations for DFM and DFV calibration LL tests
RESULTS

Material testing and four live load tests were performed on the Hanging Rock Creek Bridge (HRB) to evaluate the health of the bridge’s shear key joints, calculate experimental distribution factors for moment (DFM), and evaluate the method for calculating distribution factors for shear (DFV). Following is a discussion of the results of testing.

Material Tests Results

Materials tests were conducted to document the properties of the UHPC including compressive, tensile, and bond strengths. These properties were compared to results from other researchers and were used to draw general conclusions about the quality of the UHPC used in the shear keys.

Compressive Strength Test

Results of compressive strength tests of the UHPC used in the NEXT-D girder shear keys are shown in table 4.

<table>
<thead>
<tr>
<th>Test Age</th>
<th>Average Compressive Strength (ksi)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>28 days</td>
<td>20.8</td>
<td>23.3%</td>
</tr>
<tr>
<td>90 days</td>
<td>17.4</td>
<td>21.4%</td>
</tr>
<tr>
<td>6 months</td>
<td>22.5</td>
<td>6.7%</td>
</tr>
</tbody>
</table>

Graybeal states that for a concrete to be considered UHPC, it must display compressive strengths greater than or equal to 21.7 ksi (Graybeal and Russell, 2013). Specimens that were tested at 28 days had high variability relative to each other, but the average compressive strength was near Graybeal’s criteria. At 90 days, the specimens had high variability and for an unknown reason the average compressive decreased significantly from the 28 day compressive strength. A possible reason for the low 90 day compressive strength was the small sample size of only 4 cubes. At 6 months, the average compressive strength exceeded Graybeal’s compressive strength criteria by 3.5%, and the coefficient of variation was much lower. Based on the tests at 6 months, the UHPC used in the Hanging Rock Creek Bridge possesses adequate compressive strength to be classified as UHPC per Graybeal.
Splitting Tensile Test

Results of the UHPC splitting tensile strength tests performed at 28 days, 90 days, and 6 months are shown in table 5.

Table 5: UHPC Tensile Strength Test Results

<table>
<thead>
<tr>
<th>Test Age</th>
<th>Average Tensile Strength (ksi)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>28 days</td>
<td>3.32</td>
<td>6.9%</td>
</tr>
<tr>
<td>90 days</td>
<td>3.05</td>
<td>8.8%</td>
</tr>
<tr>
<td>6 months</td>
<td>3.51</td>
<td>9.3%</td>
</tr>
</tbody>
</table>

Graybeal also provides a tensile strength limitation for characterizing UHPC. Specifically, for a concrete to be considered UHPC it must display tensile strengths greater than or equal to 0.72 ksi (Graybeal and Russell, 2013). Average test results at 28 days, 90 days and 6 months exceeded this criterion. Because the average tensile strength did not increase significantly between the first and last tests, it is believed that the UHPC reached its full tensile strength before it was 28 days old.

Pull-Off Tests

Pull-off tests were conducted 6 months after the UHPC was cast. Results are shown in table 32. Of the 8 specimens, 3 had failures at the UHPC to concrete interface while the remaining 5 had failures at the steel disc to epoxy interface. Results were inconsistent. Failures in the UHPC to precast interface had a bond stress at failure of 120 psi to 235 psi, while failures in the steel disc epoxy interface ranged from 200 to 300 psi. The results could illustrate one or more of the following: First, it is possible that that test specimen was not prepared uniformly. By intentionally roughening the precast surface, the UHPC layer theoretically could have a stronger bond. It is possible that the surface was not uniformly rough, which would contribute to certain sections of the specimen having greater bond strength than others. Second, some of the specimens were tested near the perimeter of the specimen while others were tested towards the center. It is possible that those specimens in the center of the specimen, which are surrounded by more concrete and UHPC, could have had a stronger bond strength that those at the perimeter.

The average bond stress at failure of specimens that failed at the UHPC-to-concrete interface was 200 psi and is referred to as the Minimum Average Bond Strength in Table 6. Although the pull off test results were not consistent, they are close to results of other researchers. Table 7 contains UHPC bond strength results from 2 different Virginia Tech researchers (Halbe, 2014 & Joyce, 2014). It is important to note that the bond strength test
results for UHPC used in the HRB are similar bond strengths reported by Virginia Tech. Also, the superiority of the UHPC-to-concrete bond in comparison with the grout-to-concrete bond is noted. According to Joyce (2014) bond strength between UHPC and concrete is an order of magnitude greater than bond strength between traditionally used grout and concrete. The implication is that UHPC-to-concrete interfaces at shear keys are less likely to crack than grout-to-concrete shear keys.

Table 6: UHPC Pull-Off Test Results

<table>
<thead>
<tr>
<th>Specimen #</th>
<th>Failure Stress (psi)</th>
<th>Failure Mode</th>
<th>Bond Strength (psi)</th>
<th>Minimum Average Bond Strength (psi)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>362</td>
<td>Epoxy Interface</td>
<td>&gt; 362</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>120</td>
<td>UHPC to Concrete Interface</td>
<td>= 120</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>235</td>
<td>UHPC to Concrete Interface</td>
<td>= 235</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>177</td>
<td>UHPC to Concrete Interface</td>
<td>= 177</td>
<td>200</td>
<td>31.8%</td>
</tr>
<tr>
<td>5</td>
<td>300</td>
<td>Epoxy Interface</td>
<td>&gt; 300</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>231</td>
<td>Epoxy Interface</td>
<td>&gt; 231</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>200</td>
<td>Epoxy Interface</td>
<td>&gt; 200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>230</td>
<td>Epoxy Interface</td>
<td>&gt; 230</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 7: Virginia Tech UHPC Pull-Off Test Results

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Specimen ID (Type)</th>
<th>Specimen Age</th>
<th>Failure Stress (psi)</th>
<th>Number of Specimens Tested</th>
</tr>
</thead>
<tbody>
<tr>
<td>Halbe</td>
<td>1 (UHPC)</td>
<td>13 Days</td>
<td>204</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>2 (UHPC)</td>
<td>13 Days</td>
<td>93</td>
<td>6</td>
</tr>
<tr>
<td>Joyce</td>
<td>3 (UHPC)</td>
<td>7 Days</td>
<td>180</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>4 (UHPC)</td>
<td>12 Days</td>
<td>240</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>5 (UHPC)</td>
<td>15 Days</td>
<td>260</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td><strong>Average UHPC Failure Stress = 195 psi</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6 (Grout)</td>
<td>7 Days</td>
<td>25</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>7 (Grout)</td>
<td>15 Days</td>
<td>20</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td><strong>Average Grout Failure Stress = 22.5 psi</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Transverse Displacement Results

Relative horizontal displacements were measured at select shear keys on the NEXT-D span to evaluate the health of the NEXT-D shear keys. Cracking or other degradation at the shear key can be identified through significant displacement at one interface of a key relative to the displacement measured at the adjacent interface (on the same joint), and/or displacements that are significantly larger than those measured at other interfaces. Relatively low displacements are an indication of a sound uncracked interface. Linear Variable Displacement Transformers (LVDT) were placed to measure interface displacements at five locations for LL test 3 and six LVDT locations were selected for LL test 4. During visual inspection of the bridge deck just before both load tests, no reflective cracking effervescence underneath the bridge was evident on the NEXT-D span. Thus, reflective cracking did not influence which NEXT-D joints were selected for instrumentation.

Figures 24 and 25 show typical LVDT measurements of horizontal movement across the girder-to-joint interfaces. Positive values in the figures indicate tensile movement or “pulling apart” across the interface. The LVDT data are labeled based on the adjacent girder numbers and the side of the interface. For example, “J 1-2 W” was placed at the west interface of the shear key between girders G1 and G2. West is toward the right in the truck placement diagram shown in the figure.
Figure 24: Girder-shear key interface horizontal displacements from truck location 1 during LL test 4 load
A comparison of maximum horizontal displacements from shear key interfaces during LL testing is presented in figure 26. The comparison indicates that the overall maximum relative horizontal tensile displacement was measured during the LL test 1 at J 1-2.
The maximum displacements at each joint generally decrease through the progression of the tests. The exception is at joint 3-4W where the displacement during LL test 4 is greater than the displacements measured during LL tests 1 or 3. No significant increase in displacement (an increase by a factor of 4 or greater) in consecutive LL tests is reported at any of the joints instrumented, and all maximum displacements are equal to or less than one thousandth of an inch. Finally, none of the joints displayed displacement behavior significantly greater (factor of 5 or greater) than the other joints. These observations tend to suggest that shear keys are performing well and did not experience significant damage.

As stated earlier in the report, longitudinal bars were inadvertently left out the shear keys between girders 1 and 2 and between girders 2 and 3. No discernable difference in transverse behavior between shear keys with and without longitudinal bars was observed.

The solid black line on Figure 26 is the lower bound displacement value for which cracking may occur at the key-girder interface. In short, it is the displacement that corresponds to the tensile strength of the UHPC being exceeded. This value was determined considering a lower bound failure stress of precast-UHPC bonds and the elastic modulus of the precast girders. From the aforementioned variables, a lower bound “cracking strain” (crack at the bond) was determined using mechanics principles and the corresponding lower bound “cracking displacement” was calculated considering an

Figure 26: Comparison of maximum relative horizontal displacements during LL tests 1, 3, and 4
estimated gauge length of the LVDTs. The lower bound stress required to cause failure of the precast-UHPC bond (~230 psi) was determined based on multiple experimental results (Filosa, 2017; Halbe, 2014; Joyce, 2014; Sheng, 2013).

Based on the displacement data, the NEXT-D system is performing well. Measured displacements are generally at or above the calculated cracking displacement, however, this displacement is a lower bound. Small cracks at the bond may have formed, but there is no confirmation of cracks developing or existing cracks propagating since the displacement data has only decreased or insignificantly increased over time. Additionally, from visual inspection of the NEXT-D span (see photo survey in the appendix), the research team saw no signs of cracking at the precast-UHPC bond. Thus, the system is assumed to be performing as designed. Regarding the patchwork on the NEXT-D span, the structural behavior at the precast-UHPC joints has not been influenced by the patchwork since no significant changes in the displacement data has occurred.

DFM Test Results

DFMs were calculated using the strain data collected during live load testing. The calculation processes of experimentally measured DFMs and AASHTO LRFD based DFMs were presented earlier. This section compares the experimental and code-based DFMs for the NEXT-D span. In this section positive strain indicates a tensile strain and negative strain indicates compressive strain.

Bending Strain Data Results

Typical bending strain data is shown in figure 27. Bending strain data were collected from strain transducers placed on the bottom of the girder webs. Figure 27 is from the load truck crossing close to the guardrail; hence the larger strains were measured at the webs closest edge (figure inset shows truck position). Each curve follows the strain data collected by a specific strain transducer and is denoted with the girder web to which the strain transducer was attached. For example, G1-W is the strain transducer placed on the western web of girder 1. West is to the right in the inset drawing in figure 27. There is one strain transducer attached to each web meaning that two strain transducers were attached to each girder (since each girder possesses two webs).
Experimental DFM$s for each NEXT-D girder web were calculated using equation 13, outlined in the literature review. The calculated DFM$s for a particular girder were determined by summing the DFM values of the associated webs. In this manner a girder DFM was determined for each of the six NEXT-D girders employed on the HRB. On a subject NEXT-D girder, the maximum bending strain measured on one of its webs during LL testing may not occur simultaneously with the maximum bending strain measured on its other web during LL testing, thus both time independent and time dependent calculation procedures were considered for calculating the girder DFM$s. The worst case (higher DFM) resulted from the time independent approach; these values are reported in in Figures 28 and 29. Horizontal lines shown on the figures represent the AASHTO calculated DFM$s for type “i” and “k” bridge superstructures.

**Figure 27: Bending strain collected during LL test 3 considering load case**
Figure 28: Comparison of single truck experimental DFMs and AASHTO DFMs

Figure 29: Comparison of double truck experimental DFMs and AASHTO DFMs
Across the three live load tests the exterior and interior DFM for the single truck loading experienced little change. The interior DFM for the double truck loading experienced small change as well, however, a 20% change occurred in the reported experimental exterior DFM between LL test 1 and LL test 3 (little change occurred between LL tests 3 and 4). The change in the exterior DFM is likely because opposite transverse sides of the NEXT-D span were tested during the double truck crossing of LL tests 1 and 3. During LL test 1 both sides of the bridge for the single-truck loading were considered and the experimental exterior DFMs reported differed by 0.06. Thus, it is considered reasonable that for the double truck loading the experimental exterior DFMs differed by 0.1 between LL tests 1 and 3.

The experimental DFMs for the single truck load cases were always less than the AASHTO LRFD DFMs for typologies “i” and “k”. For the double truck load cases, the interior DFM calculated using standard bridge typology “i” was less than or equal to the experimental DFM calculated for this same load scenario and girder location. DFMs calculated using standard bridge typology “k” were always greater than the respective experimental DFMs calculated. Thus, considering the HRB is just a little over one year old and has experienced little degradation, it is prudent to design future NEXT-D bridges similar to the HRB with design DFMs calculated for bridge typology “k”.

CONCLUSIONS AND RECOMMENDATIONS

The conclusions of this report are split into two parts. The first part considers the transverse durability of the shear keys between NEXT-D girders. The second part considers the experimental distribution factors for moment (DFM) for NEXT-D spans and how they compare to the theoretical DFMs based on AASTHO LRFD. Recommendations are presented after the conclusions.

Transverse Durability

Conclusions regarding the transverse durability of the NEXT-D span is presented below.

- The transverse displacement responses were similar across load tests. In all tests the shear key interfaces experienced very little residual relative horizontal displacement, and the displacement data displayed clearly when the trucks’ front and back axles passed over the LVDT locations. LVDTs close to the applied load experienced tensile displacements whereas those away from the applied load measured compressive displacements. This indicates the bottom surface of the interfaces can undergo both tensile and compressive strains due to bridge traffic.
Maximum measured displacements at the shear key interfaces were generally at or above the calculated lower-bound displacement for cracking. However, significant cracking does not appear to be occurring since the maximum displacements measured tended to decrease throughout the progression of LL tests. Furthermore, the maximum relative horizontal displacements of each interface do not differ by a significant factor (always less than a factor of 4). Finally, no reflective cracking was observed on the NEXT-D span. For these reasons it is concluded that little deterioration has occurred in the NEXT-D shear keys on the HRB.

**DFM Conclusions**

The following conclusions regard the experimental and theoretical DFMs determined on the NEXT-D span. These conclusions are based on the results of LL tests 3 and 4 and reference the results from LL test 1.

- Experimental DFMs for a given girder varied very little across the different testing dates. The maximum absolute change in single-truck loading cases was 0.06. For side-by-side truck and exterior girders the maximum change was 0.1. Values never changed by more than 0.03 in consecutive LL tests.

- In case of exterior girders and side-by-side truck loading, the difference in DFM from different tests may be attributed a change in side of the bridge being tested. The eastern side of the HRB was tested during one test date and resulted in larger experimental DFMs than the western side of the bridge which was tested later. The small differences in magnitude of the experimental DFMs between the two LL tests indicate that the bridge has not changed its behavior in dispersing LL transversely.

- Both the “k” and “i” section typologies have code-calculated DFMs greater than the experimentally measured DFMs for all but one case. For the interior girder side-by-side truck loading, the theoretical DFM determined from section type “i” is equal to the experimental DFM determined during LL test 1 and less than the experimental DFMs determined in LL tests 3 and 4. Thus, section typology “i” is not strictly conservative for the design of NEXT-D beams. Section typology “k” was conservative in comparison to all experimentally determined DFM.

**Recommendations**

Considering the above conclusions, recommendations are as follows:

- The dimensions and detailing of the NEXT D girders performed well during the duration of the test program, and, therefore, no modification to either is recommended.
• SCDOT should consider NEXT-D girders for future short- to medium-span bridges. There was no visible sign of cracking at any of the shear keys on the NEXT-D span.

• When designing NEXT-D spans similar to that utilized on the HRB, AASHTO LRFD typology “k” should be considered for calculating moment distribution factors.

• Longer span lengths (>40 feet) should be tested to evaluate the suitability of NEXT-D girders for longer spans. PCI (Precast/Prestressed Concrete Institute) reports practical span lengths of up to 60 feet for a bridge superstructure containing 28 inch deep NEXT D girders.

• NEXT D girders are recommended for testing on more heavily trafficked bridges (in excess of 3,000 vehicles/day).

• UHPC is recommended for shear keys for future NEXT-D spans. The NEXT-D UHPC shear keys have undergone little deterioration since the girder relative horizontal displacements and DFMds did not significantly change over one and a half years. Additionally, there were no visual signs of joint deterioration on the NEXT-D span.
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“Rebuilding our Roads.” South Carolina Department of Transportation. 31 May 2018.

APPENDIX: September 2018 Photo Survey

Bridge Deck Photo Survey (all deck photos facing south)

- G6
- G5
- G4
- G3
- G2
- G1

= Patchwork

= Transverse Abutment Crack

NEXT-D Span

Photo 1

Photo 2
Solid Slab Span

= Longitudinal Reflective Crack

= Transverse Abutment Crack
Bridge Underside Photo Survey

Photo 15: NEXT-D girder 6 facing north

Photo 16: NEXT-D girder 6 facing south

Photo 17: NEXT-D joint between girders 5 and 6

Photo 18: Solid slab girder 14 facing south
Photo 19: Solid slab girder 14 facing north  
Photo 20: Solid slab joint between girders 13 and 14