ADJACENT BOX BEAM BRIDGE REHABILITATION USING VERY HIGH PERFORMANCE CONCRETE

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ABSTRACT

The objective of this research project was to design a rehabilitation plan for an adjacent box beam bridge with deteriorated joints using Very High Performance Concrete (VHPC). VHPC, a non-proprietary mix developed by the researchers, was chosen as an economical alternative to proprietary Ultra High Performance Concrete (UHPC). The results of extensive material testing of VHPC and grout revealed that VHPC had higher compressive and tensile strengths, a higher modulus of elasticity, gained strength faster, bonded better to precast concrete, was more durable over time, and shrank less than conventional grout. The rehabilitation plan also included blockouts cut into the beams across the joints. A short reinforcing bar was placed in each blockout, and they were filled with VHPC along with the shear key.

The repair method was used to rehabilitate the Buffalo Branch Bridge. Live load tests were performed before and after the rehabilitation to determine if the new connection detail resulted in better load distribution and smaller relative displacements of adjacent beams. Strain and displacement measurements indicated that the soffit beams were more engaged in carrying truck loads after the repair, and relative vertical displacements of adjacent boxes were much smaller.

Keywords: Adjacent Precast Members, Connections, Ultra High Performance Concrete, Experimental Testing, and Strain Compatibility.

INTRODUCTION

MOTIVATION

Adjacent prestressed beam bridges are comprised of either precast box beams or voided slab sections as the superstructure with a deck or topping placed directly on top. The precast members are traditionally connected with a longitudinal shear key filled with grout and transversely tied at intermittent diaphragm locations. This enhances transverse load transfer between neighboring adjacent members. By using precast members, these bridges are fairly simple and can be rapidly constructed. Adjacent box beam bridges are an efficient design for short spans and bridge locations with low vertical clearance requirements. However, over time the traditional grout shear key tends to deteriorate causing reflective cracking to propagate through the wearing surface as shown in Figure 1.

These reflective cracks allow water and corrosive agents, such as deicer salts, to penetrate down into the joints, which, if left uncorrected, can eventually cause the reinforcing and prestressing steel in the precast members to corrode. This leads to the need for bridge repair or replacement well before the end of its anticipated design life, negating the assessed economic value of the adjacent member system. Reflective cracking allowing water to leak through the joint can be seen in Figure 2, which is the underside of the joint in Figure 1.

Figure 1. Reflective cracking

Figure 2. Leaking joint

Due to the problems that arise when reflective cracking appears, the objective of this project is to recommend an alternative connection that can be used to rehabilitate adjacent member bridges. The goal of the improved connection is to increase the service life of the bridge well beyond that obtained by simply replacing the deteriorated shear key with an identical shear key design. Graybeal[1] has suggested replacing the grout shear key with Ultra High Performance Concrete (UHPC). He defines UHPC as “a cementitious composite material composed of an optimized gradation of granular constituents, a water-to-cementitious materials ratio less than 0.25, and a high percentage of discontinuous internal fiber reinforcement. The mechanical properties of UHPC include compressive strength greater than 21.7 ksi and sustained post-cracking tensile strength greater than 0.72 ksi”. Additionally Graybeal asserts that the discontinuous pore structure of UHPC significantly enhances the durability compared to traditional concrete or grout because it reduces the liquid ingress. Graybeal’s recommendation for a UHPC connection of adjacent box beams is to form the members at the precast plant with No. 4 bars extending 4¾ in into the shear key spaced every 8 in. When placed in the field, the overlapping reinforcing steel is spliced together eliminating the need for transverse post-tensioning. The joint is then filled with UHPC instead of grout, ultimately allowing the top flange level of the box beams to act as a continuous slab.

Previous research done at Virginia Tech (VT) by Halbe[2] developed a very similar design compared to Graybeal’s. However, instead of designing the connection exclusively for new construction, the objective was to design a connection that could also be used to rehabilitate existing bridges. In addition to the traditional shear key, it specifies forming a 4 in deep 6 in x 6 in blockout with an exposed stirrup at regular spacings, such as 2 to 3 ft, along the length of each joint. The blockouts are aligned across the joints when the beams are placed adjacent to one another, and a No. 4 splice bar is placed in each blockout, tied to the exposed stirrups. The blockout and shear key are then filled with either UHPC or VT's more economical Very High Performance Concrete (VHPC). The retrofit presented here simplifies construction by recommending the blockout be saw cut into the existing boxes, rather than formed, and by having the reinforcing steel not be in direct contact with a stirrup in the box beam. Using non-contact lap splices eliminates the need to expose the stirrups and allows for the blockouts to be placed without damaging the reinforcing steel in the box beams.

OVERVIEW OF BUFFALO BRANCH BRIDGE

The most recent VDOT visual inspection of the Buffalo Branch Bridge performed in January 2014 stated that water and efflorescence were seeping through the furthest downstream joint for the full length. Evidence of leakage was also seen on the second downstream joint and the furthest upstream joint within 6 ft of the abutments. A picture of the leaking downstream joint is shown in Figure 2. The plan and cross-section views of the bridge are shown in Figure 3 and Figure 4. The 55 ft span bridge consists of nine adjacent box beams with transverse ties at the third points.

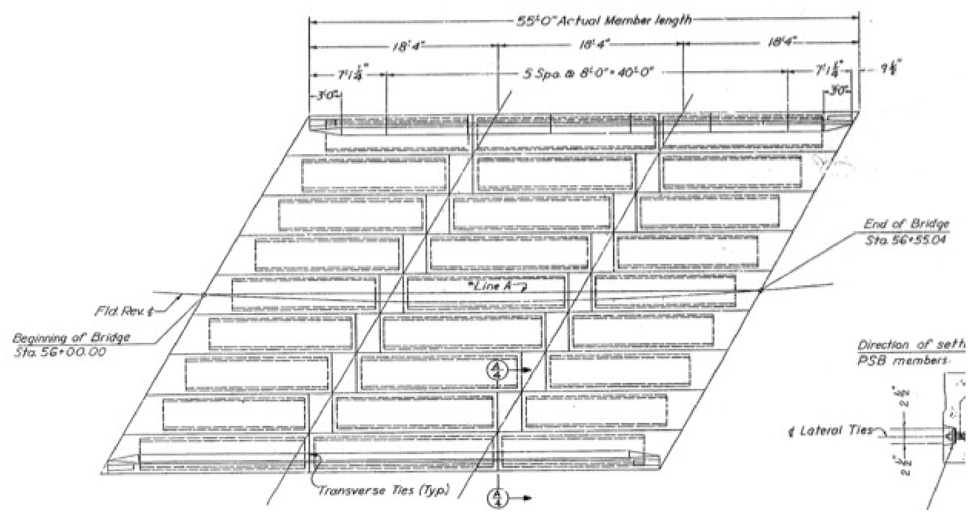

Figure 3. Plan view of Buffalo Branch Bridge

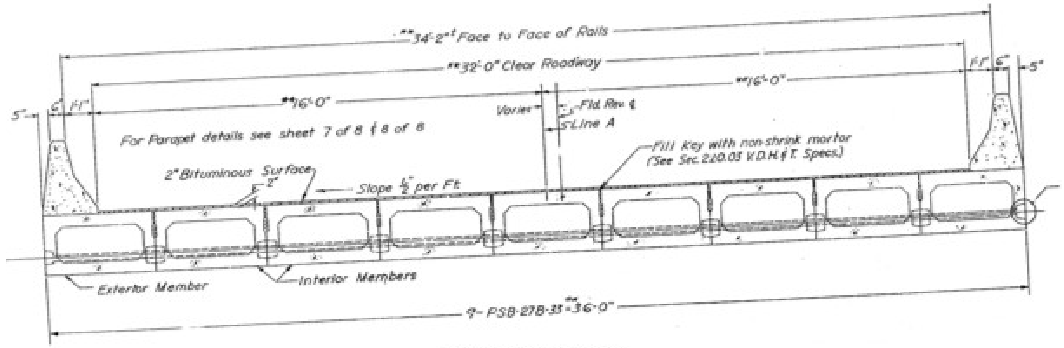

Figure 4. Cross-section view of Buffalo Branch Bridge

A typical 4 ft wide box beam section is shown in Figure 5. The smallest depth that VDOT[3] allows is 27 in, which makes adjacent box beam bridges a favorable option in places with low clearance restrictions. The precast concrete box beams also provide a smooth bottom which allows greater passage of debris under the bridge compared to beam/girder spans, making it an ideal option for heavy debris laden streams/rivers.

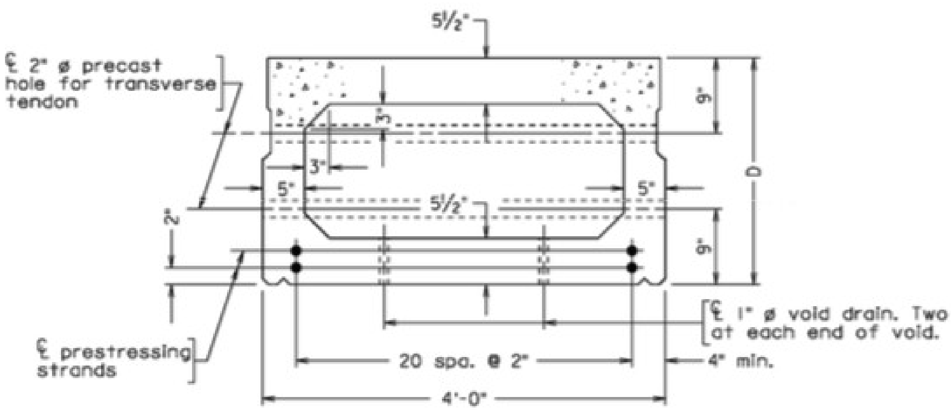

Figure 5. VDOT 4 ft wide box beam (VDOT[3])

The typical connection that VDOT[3] specifies is a partial depth shear key shown in Figure 6. Prior to placing the grout, the shear key is prepared by cleaning, sandblasting, and by creating a saturated surface drying condition. This has been shown to improve the bond between the grout and the precast member.

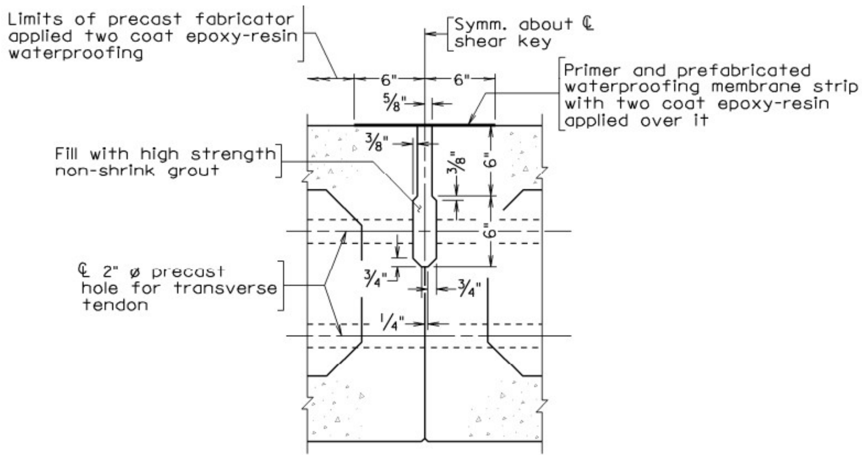

Figure 6. VDOT shear key detail (VDOT[3])

MATERIALS

ULTRA HIGH PERFORMANCE CONCRETE (UHPC) AND VERY HIGH PERFORMANCE CONCRETE (VHPC)

Graybeal[1] recommends using UHPC for field-cast connections between precast bridge members in accelerated bridge construction because it gains strength quickly and will not create weak points in the structure. Another advantage for using UHPC in connections is that the development length required for reinforcing steel is greatly reduced as compared to normal concrete. Because of the superplasticizer in UHPC, it is able to flow efficiently and be self-consolidating while still keeping a low water to cementitious ratio and high strength properties. It should be noted that although UHPC is self-consolidating and can be placed in small connections where normal concrete is not an option, it is still not as fluid as grout, which is the currently accepted material for precast member connections. According to Yuan and Graybeal[5], as of 2013, field-cast UHPC connections had been used in 32 bridges in the United States.

The properties that led Graybeal to recommend using UHPC for field-cast connections are also demonstrated by VHPC, with the added advantage of being more economical. VHPC was originally designed with ¼ in aggregate and 1.2 in long steel fibers and was intended to be used in closure pours. However, when trying to use VHPC in the connections of adjacent member bridges, it was discovered that the aggregate and fibers originally selected were too large to fit in the narrow shear keys. Due to this size restriction, a second VHPC mix was designed with 1/8 in aggregate and ½ in long steel fibers. The original mix with the larger aggregate and fibers was renamed VHPC-Large and the mix with the smaller aggregate and fibers was renamed VHPC-Small.

LAP SPLICES IN UHPC AND VHPC

Halbe et al.[6] reports on tests performed to determine the minimum lap splice length required to fully develop No. 4 bars in UHPC and VHPC-Large. A test method was developed to mimic the lap splice region in adjacent precast members, and splice lengths of 3 in to 6 in were tested.

In the tests, the tension reinforcement in all of the specimens exceeded the yield stress of 60 ksi, therefore, a 3 in lap splice distance was determined to be adequate to yield the steel. However, due to ductility concerns, the recommended lap splice length for a No. 4 reinforcing bar in UHPC or VHPC was 5 in.

MATERIAL TESTS

Material tests were performed on five different mix designs: UHPC, VHPC-Large, VHPC-Small, and a non-shrink grout. Table 1 presents an overview of the tests done on the materials.

While most of tests were performed according to the ASTM standard listed, the test measuring the bond with concrete was slightly modified. The ASTM procedure requires casting a continuous layer on a concrete substrate. Cuts are then made through both the coating and concrete substrate to attach the loading fixture, and a tension force is then applied normal to the test surface. Instead of a continuous layer, the results presented here were obtained by casting 2 in diameter x 1 in tall pucks of VHPC on the precast concrete members. The ASTM procedure was then followed by attaching a loading fixture and applying a tension force normal to the test surface.

Table 1. Material tests overview

Test	Specimen	Material	ASTM Standard	Reference
Compressive Strength	4 in x 8 in Cylinders	UHPC, VHPC-Small VHPC-Large, Grout	C39	ASTM[7]
Compressive Strength	2 in Cubes	VHPC-Small	C109	ASTM[8]
Splitting Tensile Strength	4 in x 8 in Cylinders	UHPC, VHPC-Small VHPC-Large, Grout	C496	ASTM[9]
Modulus of Elasticity	4 in x 8 in Cylinders	UHPC, VHPC-Small VHPC-Large, Grout	C469	ASTM[10]
Bond with Concrete	2 in x 1 in Pucks	VHPC-Small, VHPC-Large, and Grout	D7234	ASTM[11] and Scholz et al.[12]

Durability	3 in x 4 in x 16 in Bars	UHPC, VHPC-Small VHPC-Large, Grout	C666	ASTM[13]
Free Shrinkage	3 in x 3 in x 11 ¼ in Bars	UHPC, VHPC-Small VHPC-Large, Grout	C157	ASTM[14]

BUFFALO BRANCH BRIDGE PRE- AND POST-REPAIR LIVE LOAD TESTS

INSTRUMENTATION

The measurements that were most relevant to compare pre-repair and post-repair behavior were the box beam vertical deflections and longitudinal strains and the relative vertical and horizontal joint movements. The instrumentation was almost identical for the pre-repair test and the post-repair test.

Strains and deflections were recorded at the midspan of each box beam using nine BDI strain transducers and homemade deflectometers. A bonded BDI strain transducer and deflectometer are shown in Figure 7. The gauges were placed to measure the longitudinal strain of the box beam members. Therefore, they are not perpendicular to the midspan line that is drawn with the bridge skew.

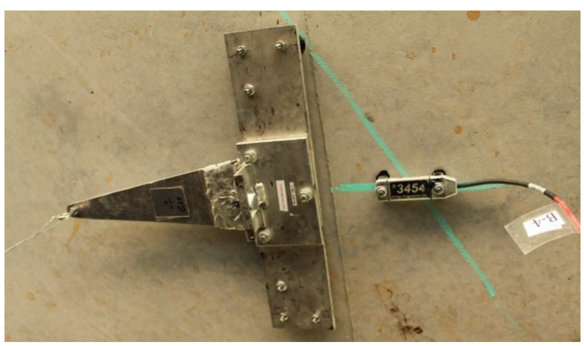

Figure 7. BDI and deflectometer

LVDTs were used to measure the horizontal and vertical relative displacements of adjacent box beam members. The LVDTs were strategically placed at the location of the highest expected relative girder displacements. Horizontally and vertically oriented LVDTs are shown in Figure 8.

Figure 8. Horizontal and Vertical LVDTs

BDI strain transducers and deflectometers were placed at midspan on the bottom of each box beam to measure the longitudinal strain and vertical deflection. Due to the visible signs of deterioration in the two external downstream joints, vertical LVDTs and horizontal LVDTs were placed at midspan and quarterspan, where the most leaking appeared to have occurred, on the two external downstream joints. To be able to compare the results of the relative displacement in deteriorated joints to seemingly undamaged joints, the two external upstream joints were also instrumented with vertical LVDTs and horizontal LVDTs. The instrumentation plan for the pre-repair test is shown in Figure 9 and for the post-repair test is shown in Figure 10. The only difference is that the relative displacements of the joints at the quarter points was not measured in the post-repair tests, because they were negligible in the pre-repair tests.

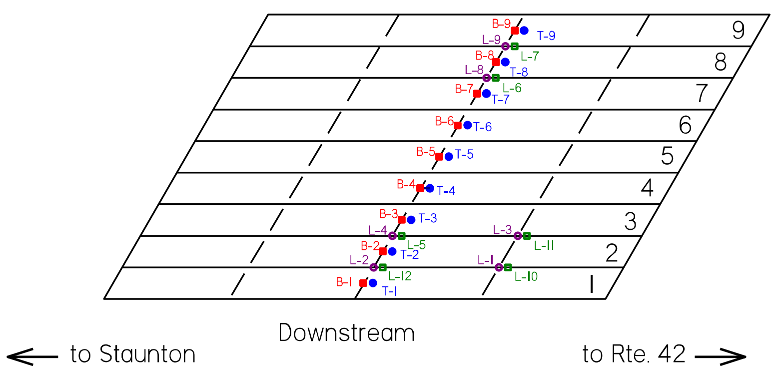

Figure 9. Instrumentation plan for pre-repair test

(T – deflectometer, B – strain gage, and L – LVDT)

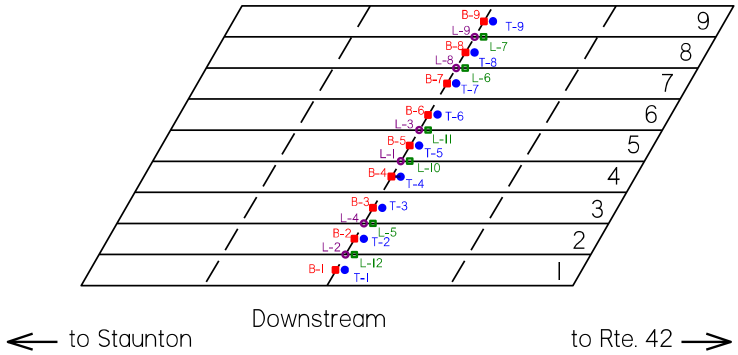

Figure 10. Instrumentation plan for post-repair test
(T – deflectometer, B – strain gage, and L – LVDT)

LOADING PROCEDURE

A total of six quasi-static load cases were run by driving the same loading trucks over the bridge. Each load case was repeated three times. The drivers maintained the slowest possible speed of two to three mph when crossing the bridge. A two truck crossing is shown in Figure 11.

Figure 11. Test trucks crossing for Load Case 3

The load for each test was provided by two VDOT dump trucks loaded to approximately 25 tons each. The measured axle loads for each truck are shown in Figure 12 and Figure 13.

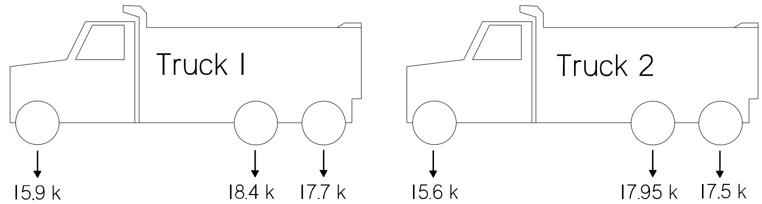

Figure 12. Axle weights of loading trucks for pre-repair tests

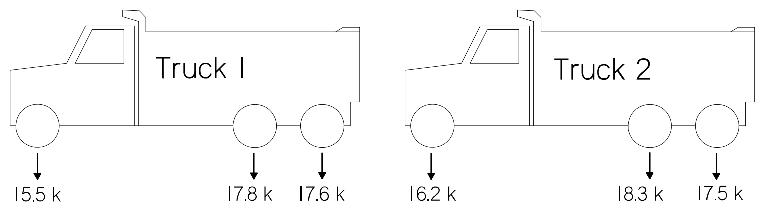

Figure 13. Axle weights of loading trucks for post-repair tests

The two exterior downstream joints, joint 1 located between beam 1 and beam 2 and joint 2 between beam 2 and beam 3, showed the most signs of deterioration. Load case 1 and load case 2 were chosen to maximally load joints 1 and 2 in an attempt to record the largest relative joint displacements experienced. Load case 3 was chosen to obtain the maximum midspan deflections and longitudinal strains in the downstream beams. The first three load cases were mirrored to the upstream side of the bridge in an attempt to gather the same information on the less damaged side of the bridge. All of the load cases are shown in Figure 14.

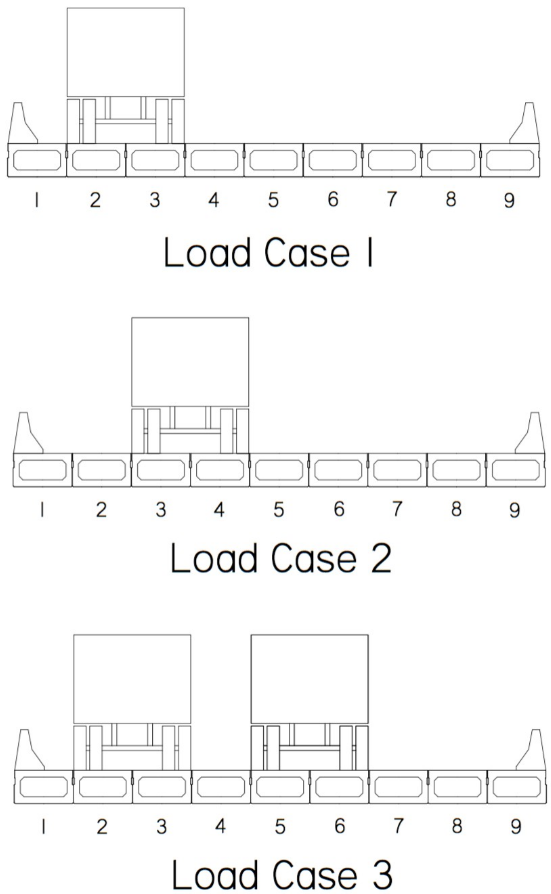

Figure 14. Load cases

BUFFALO BRANCH BRIDGE REHABILITATION

PROPOSED REHABILITATION PLAN

The proposed rehabilitation plan for the Buffalo Branch Bridge is shown in Figure 15. The plan was for all of the joints to be completely cleaned out. The four interior joints were to be replaced with fresh grout and a Kevlar and epoxy topping. The four exterior joints were to be replaced with VHPC and geometric cutouts at the obtuse skew corners were to be added. Figure 16(a) shows the dimensions for the dog bone cutouts to be used in the upstream joints. Figure 16(b) shows the dimensions for the bowtie cutouts to be used in the downstream joints. Laboratory testing was performed on a variety of cutout shapes and the dogbone and bowtie were determined to be the two best alternatives. The depth of the cutouts was to be 4 in.

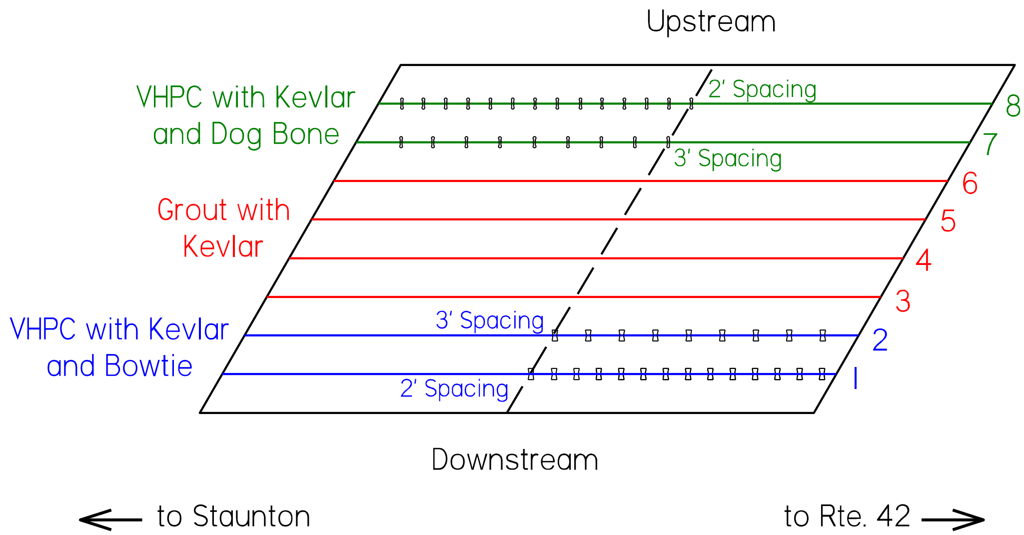

Figure 15. Buffalo Branch Bridge rehabilitation plan

Figure 16. Cut out geometries

To ensure that the placing of the VHPC went smoothly, it was recommended that the contractor acquire two 9 cu ft capacity mortar mixers. Due to the consistency of VHPC, the maximum batch size of 1.5 cu ft of VHPC was recommended for each mixer. The volume of VHPC required for each joint was calculated and converted to number of batches: five for the upstream joints and six for the downstream joints. To assist the contractor in weighing out the materials beforehand, the mix design for each batch was given and is shown in Table 2.

Table 2. Mix design for a 1.5 cu. ft. batch of VHPC-Small

Material	**Amount, lbs**
Fly Ash	13.3
Silica Fume	13.3
½ in Fibers	14.5
Cement	62.3
Sand	74.7
1/8 in Aggregate	36.7
Water	17.7
High Range Water Reducer	650 mL

Retarder	VT to provide as needed

The recommended mixing procedure consisted of mixing the VHPC for 15 minutes and then 5 minutes to place it. To ensure that the VHPC was placed without cold joints forming, the second mixer was recommended to begin the mixing process 10 minutes later.

The 5 step recommended mixing process was:

1. Wet mixer and pour out excess water. Add sand, aggregate, and half of the water and mix for 5 minutes.
2. Add the cement, fly ash, silica fume, and remaining water and mix for 5 minutes.
3. Add the HRWR and mix for 3 minutes. Check the consistency and decide if more HRWR is desired.
4. Add fibers and mix for 2 minutes.
5. Remove VHPC from mixer to place.

REHABILITATION PROCESS

A construction crew, consisting of two traffic flagmen, a supervisor, and six other workers performed the rehabilitation process. Several VDOT and Virginia Tech employees were also present.

Every step of the rehabilitation process was completed in two stages, the downstream half followed by the upstream half, so that traffic could continue to use one lane over the bridge. The rehabilitation process began with removing the asphalt topping to expose the adjacent box beams and the joints. The joints were then cleared and the reinforcing steel was located. In the downstream exterior joints, the deteriorated grout closely resembled sand and could be scooped out barehanded without any effort. However, for the four interior joints, the grout was still securely bonded to the adjacent box beams and was more difficult to remove. While the shear keys were 12 in deep, typically only 4 in of grout was removed.

After the contractor expressed concerns about being able to use the available saw to cut the bowties without cutting through the adjacent reinforcing steel, the rehabilitation plan was modified to only include dog bones. The downstream joints would have dog bones at the same spacing as originally recommended while the upstream exterior joint would be spaced at 4 ft and the upstream interior joint would be filled with VHPC without including any geometric cutouts. To prevent damaging the reinforcing steel within the box beams, a pachometer was used to locate bars on either side of the specified dog bone location. Cutting the dog bones took approximately 20 minutes each and began by cutting both ends with a 3 in core drill. Next, the inside was cut with a saw blade and the whole dog bone was chiseled out. Completed dog bone blockouts are shown in Figure 17.

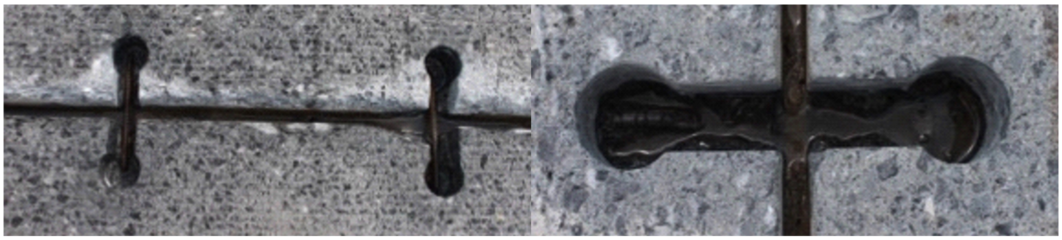

Figure 17. Dog bones

Figure 18(a) shows joint 1 with dog bones spaced at 2 ft and joint 2 with dog bones spaced at 3 ft as specified in the proposed rehabilitation plan except using dog bones instead of bowties. Figure 18(b) shows Joint 7 with only a bowtie at midspan and Joint 8 with dog bones spaced at 4 ft. The joints were sand blasted and sprayed with a hose to create a clean, SSD condition immediately before placing the VHPC.

a) Joints 1 and 2 b) Joints 7 and 8

Figure 18. Final configuration of blockouts

The mixing and placing procedure followed the proposed rehabilitation plan except for only using one mixer. Only one mixer was used because the joints were only cleared 4 in deep instead of all 12 in which required less VHPC. This in turn allowed for the joints to be placed quickly with just one mixer. To do this, a wheelbarrow was filled with the mixed VHPC and dumped at the joint and then shoveled in place. To prevent cold joints from forming where two batches of the VHPC met, it was rodded throughout the section anywhere two batches met.

The VHPC began to set very quickly, as evident by a skin forming on the surface of the joint. Once the VHPC was placed in the entire length of the joint, the joint was covered with wet burlap to provide a moist cure. On day one, the downstream joints were completely placed by 12:00 pm and on day two the upstream joints were completely placed by 1:00 pm. At 5:00 pm each day, the traffic lane traveling over the newly placed joints was opened back up. Therefore, the VHPC was allowed to moist cure for five hours on day one and four hours on day two before traffic was directed back on it. Figure 19 shows the downstream joints that were placed on day one after being driven on from 5:00 pm to the following morning. The joints did not show visual signs of damage due to exposure to traffic this soon after placing the VHPC.

Figure 19. Completed joints

RESULTS AND DISCUSSION

MATERIAL TESTS

A summary of the material test results is presented in Table 3. The UHPC and both VHPC mixes gained strength faster and achieved higher compressive and splitting tensile strengths than the grout. The splitting tensile tests for the UHPC and both VHPC mixes also exhibited post cracking tensile strength where the steel fibers bridged the cracks so that the cylinders continued to take load. Along with the increased strengths, the UHPC and both VHPC mixes also had higher moduli of elasticity than the grout. While the increased compressive and splitting tensile strengths make UHPC and both VHPC mixes a better option than grout, the bond strength with the concrete is the main advantage. Because the deterioration of the grout shear key begins at the bond with the precast concrete member, the increased bond strength could potentially make the joints last significantly longer. The bond strength of the UHPC and both VHPC mixes is much larger than the bond of the grout. The durability as measured with the relative dynamic modulus shows that after 300 cycles all of the mixes are still intact. However, of the four mixes, the grout is the only one that presented scaling. The shrinkage exhibited by the non-shrink grout far exceeded that of the UHPC and both VHPC mixes. The one area where grout was more advantageous than the UHPC and both VHPC mixes was the cost; the grout is slightly more economical up front. However, with the extended lifespan replacing the grout with VHPC could offer, the economic value of the VHPC could surpass the grout in the long run.

Table 3. Material properties summary

Average Material Properties	**Age, days**	**UHPC**	**VHPC-Small**	**VHPC-Large**	**Grout**
Compressive Strength, psi	7	16000	15600	13700	4600
	28	19900	16400	15700	8950
Splitting Tensile Strength, psi	7	1810	1920	1640	621
	28	2410	2140	1920	696
Modulus of Elasticity, ksi	7	7950	6170	5200	3160
	28	8560	6330	5500	3790
Bond with Concrete, psi (sand blasted, SSD)	7	183	102*	190	26
	15	261	N/A	226	17
Relative Dynamic Modulus, %	300 cycles	91	92	95	92
Shrinkage, με	7	511	462	354	724
	28	698	662	561	1347
	92	779	757	673	1680
Cost, $/yd^3	N/A	2000^	830	490	460

*VHPC-Small Bond results were for non-sandblasted, SSD

^Cost estimate for proprietary UHPC, includes engineering with project

The UHPC and both VHPC mixes exhibited higher strengths, better bond, better durability and less shrinkage than the grout. Therefore, these mixes were investigated to be used in the rehabilitation of the Buffalo Branch Bridge. The deciding factor in using the VHPC instead of the UHPC was the cost; VHPC is a more economical option.

BUFFALO BRANCH BRIDGE LIVE LOAD TESTS

Live load tests of the Buffalo Branch Bridge were conducted before and after the repair of the longitudinal joints. Figure 20 through Figure 25 present comparisons of the pre-repair (initial) and post-repair response of the bridge. The plotted lines are the averages of the three truck crossings for each load case. As can be seen in the plots, the pre- and post-repair behaviors are not significantly different, except for the behavior of Beam 1. The worst joint was between Beam 1 and Beam 2, and in pre-repair load cases 1, 2, and 3, which heavily loaded that side of the bridge, the deflections and strains in Beam 1 are smaller in the pre-repair condition than the post-repair condition. This would indicate that post-repair the beam is better tied to the system and carries a larger percentage of the total load. For load cases 4, 5 and 6, there is not a significant difference in behavior because the joints on the heavily loaded side of the bridge were in much better condition pre-repair.

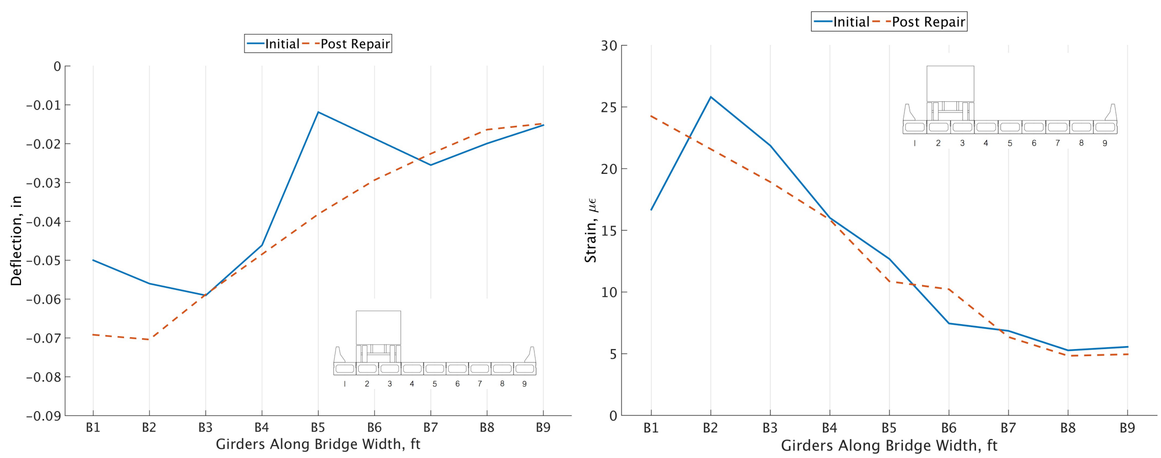

Figure 20. Load case 1 deflection and strain comparisons

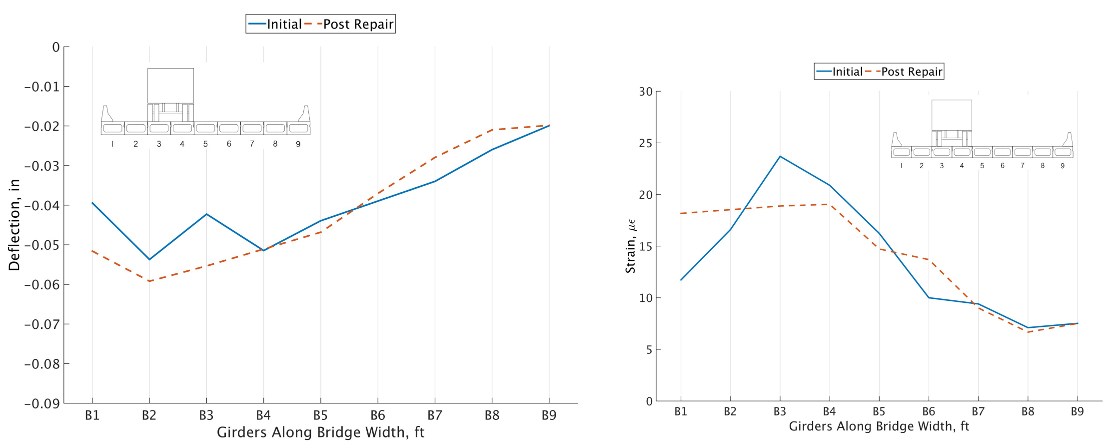

Figure 21. Load case 2 deflection and strain comparisons

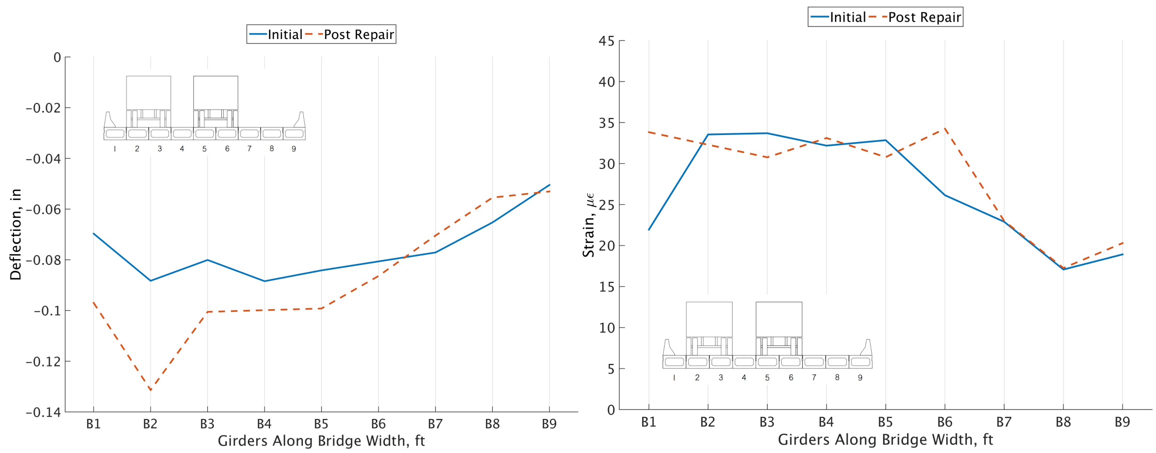

Figure 22. Load case 3 deflection and strain comparisons

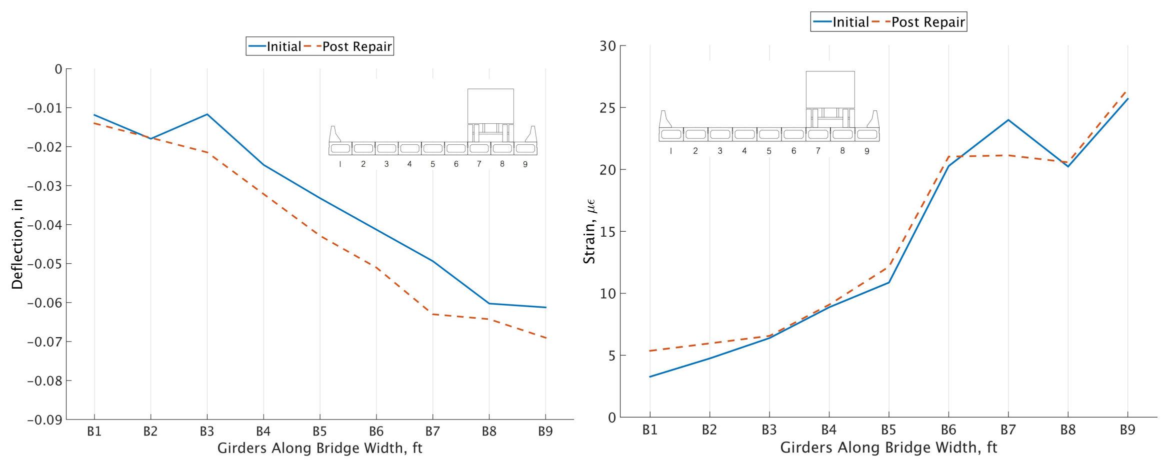

Figure 23. Load case 4 deflection and strain comparisons

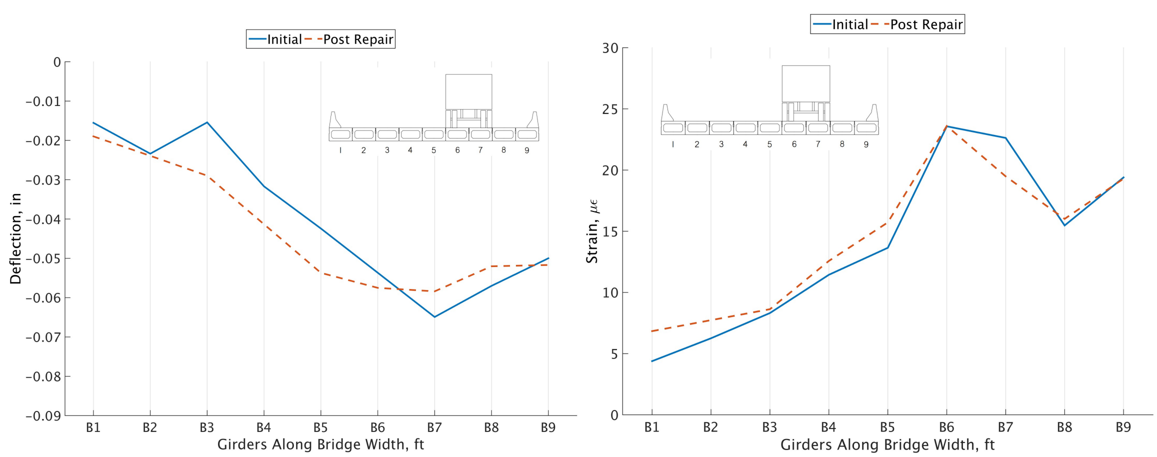

Figure 24. Load case 5 deflection and strain comparisons

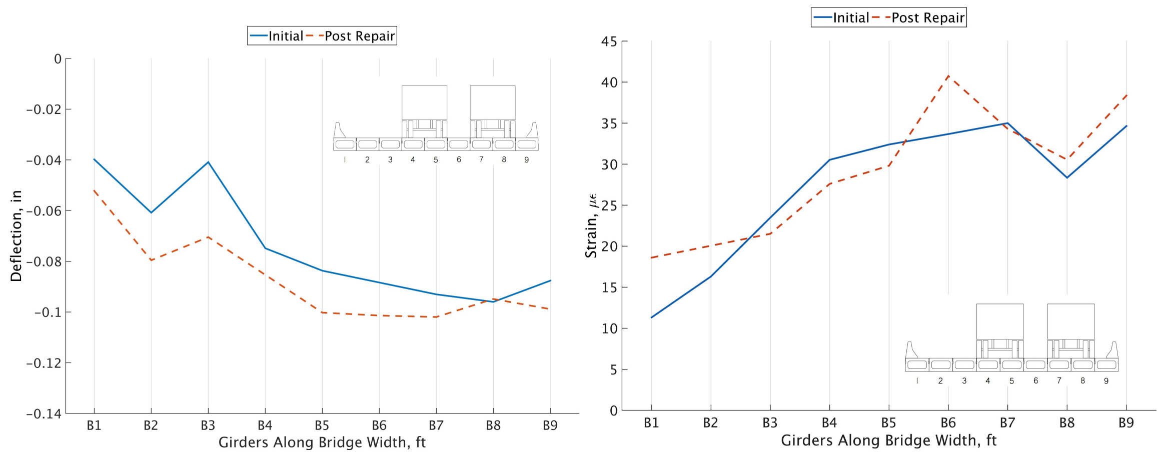

Figure 25. Load case 6 deflection and strain comparisons

The horizontal and vertical relative displacements of adjacent box beam members were measured for each of the three quasi-static runs of the six load cases. The results are presented in Figure 26 and Figure 27. In the figures, the solid lines are used for load cases 1, 2 and 3 which most heavily loaded the downstream side of the bridge. Dashed lines are used for load cases 4, 5 and 6, which most heavily loaded the upstream side of the bridge. Mirror image load cases have the same markers in the figures, for easier comparisons.

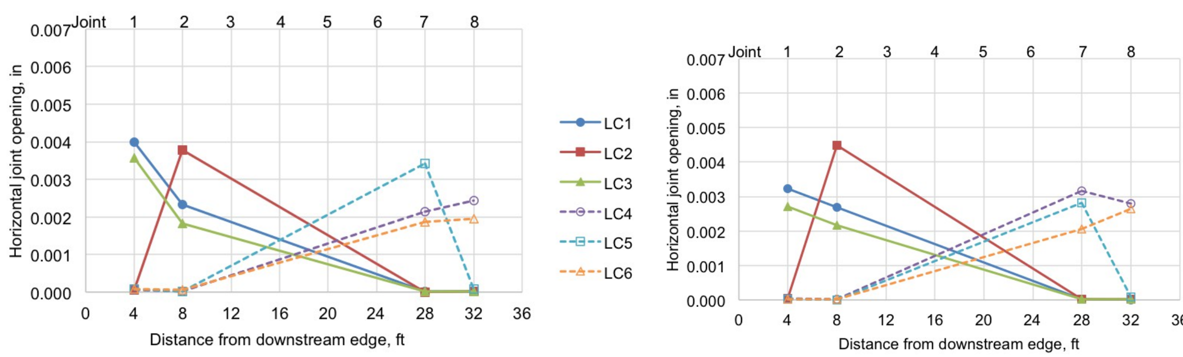

Figure 26. Relative horizontal deflection from pre- and post-repair tests

As observed, the relative horizontal deflections for the two load cases are mirrored as expected; the joints directly under the truck load had higher relative horizontal deflections than those on the other side of the bridge. In the initial test, the downstream deteriorated Joint 1, between Beam 1 and Beam 2, opened about 60 percent more than the upstream joint between Beam 8 and Beam 9, which was in relatively good condition at the time of the initial test. This shows that the deterioration of the downstream joints allowed larger relative horizontal deflection to occur. In Figure 26, it can be seen that the post-repair horizontal deflections are smaller at Joint 1 and more similar to the horizontal deflections on the upstream side of the bridge.

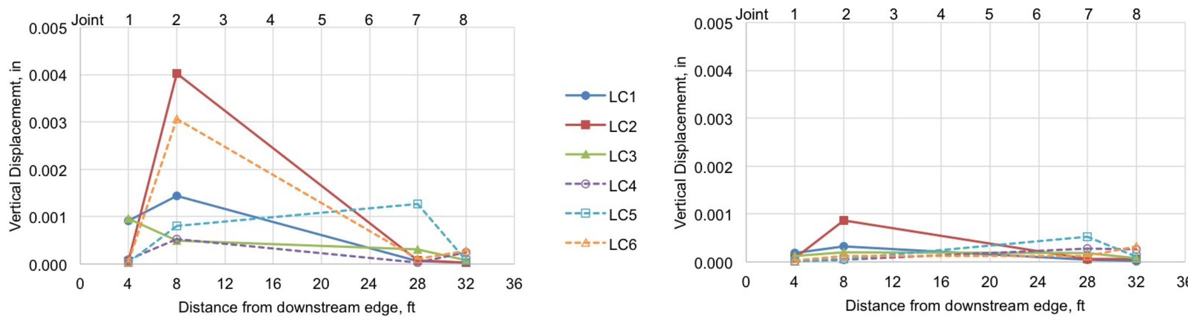

Figure 27. Relative vertical deflection from pre- and post-repair tests

Figure 27 shows that the post-repair relative vertical displacements are much smaller, particularly for the joints on the downstream side of the bridge.

Girder distribution factors (GDFs) determine the fraction of load that each individual girder is designed to carry. Two methods were used to calculate the GDFs of the Buffalo Branch Bridge: the method outlined in the AASHTO LRFD Bridge Design Specifications[15] and the method presented by Collins[16] to determine the GDF based on test results. According to AASHTO, the Buffalo Branch Bridge was classified as a type (g) cross-section which includes precast solid, voided or cellular concrete box with shear keys and with or without transverse post-tensioning and having an integral concrete deck. Table 4 shows the values for the Buffalo Branch Bridge used to obtain the GDFs with AASHTO's.

Table 4. Values used in AASHTO method for Buffalo Branch Bridge

Number of beams	Width of beam	Span length	Moment of inertia	St. Venant torsional inertia	horizontal distance from the centerline of the exterior web of exterior beam at deck level to the interior edge of curb or traffic barrier	Angle of skew
N_b	**b, in**	**L, ft**	**I, in^4**	**J, in^4**	**d_e, ft**	**θ, °**
9	48	55	65941	141060	2.0	30

To calculate the GDF of a skewed bridge based on test results, Collins suggests using

(Eq. 1)

where is the maximum response of the girder, is the number of trucks applying the load, is the number of girders, and is the maximum response of the jth girder. To account for the skew of the bridge, the sum of the maximum responses of all of the girders was used in the denominator.

A comparison of the GDFs obtained using the AASHTO method and the test results is shown in Figure 28 for one design lane loaded and in Figure 29 for two or more design lanes loaded. The maximum GDFs calculated from the live load test data are presented in two groups, the maximum of the seven interior girders and the maximum of the two exterior girders.

The GDFs obtained from the AASHTO method and both live load tests are all similar. The AASHTO method over predicts the GDFs leading to a conservative design for all cases except the interior girders with one truck load pre-repair. The GDFs measured for interior beams with one lane loaded after repair were smaller and the GDFs for the exterior beams were larger. This is another indication that the repair results in better transfer of load to the exterior beams.

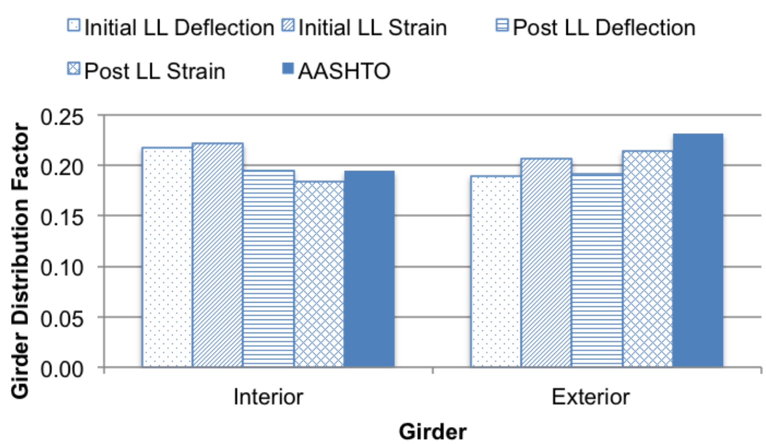

Figure 28. GDF comparison for one design lane loaded

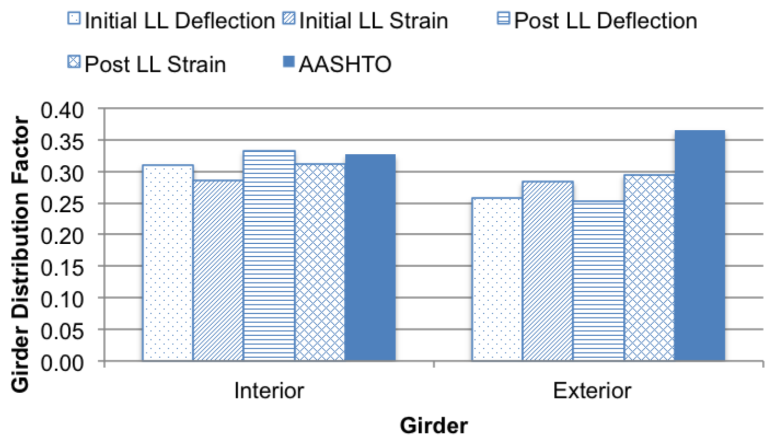

Figure 29. GDF comparison for two lanes loaded

CONCLUSIONS

The objective of this research project was to develop a rehabilitation plan for an adjacent box beam bridge with deteriorated joints using VHPC. First, a material characterization of both VHPC mixes was completed to determine if it was a suitable material to replace the grout used to connect the precast members in adjacent box beam bridges. Second, a pre-repair live load test was performed to characterize the behavior of the bridge with deteriorated joints. Third, the rehabilitation plan was implemented on the Buffalo Branch Bridge and the process was documented. Finally, a live load test was performed on the Buffalo Branch Bridge after the repair to determine the effects of the repair on overall behavior and transverse load distribution.

CONCLUSIONS FROM MATERIAL TESTS

- Grout was the easiest filler material to mix and place.
- The VHPC and UHPC gain strength faster and achieve higher strengths than the grout.
- Due to the steel fibers present, the VHPC and UHPC have high splitting tensile strengths and good post-cracking behavior.
- After seven days, the bond between the VHPC and the precast concrete member was strong enough to fracture the aggregate in the precast concrete member. As opposed to the grout, which developed a bond strength which was too low to remove the paste from the surface of the precast concrete member.
- The durability measured by scaling of the surface of the VHPC and UHPC were significantly better than the durability of the grout.
- The tests performed in this project indicated that the tested grout shrinks more than the other mixtures tested in this investigation.

CONCLUSIONS FROM BUFFALO BRANCH BRIDGE REHABILITATION

- Close coordination with the contractor before the rehabilitation resulted in a smooth operation.
- The bowtie cut-out (see Figure 16(b)) was shown to be more cumbersome and difficult in the field as compared to the dogbone (Figure 16(a)).

CONCLUSIONS FROM BUFFALO BRANCH BRIDGE LIVE LOAD TEST

- The transfer of load to the exterior beam on the downstream side of the bridge was significantly improved after the repair.
- The repair reduced the horizontal and vertical relative displacements of the adjacent boxes, which should result in more durable joints over time.
- The GDFs measured for the Buffalo Branch Bridge compared to the AASHTO GDFs indicate that the bridge was able to better transfer the load across the joints after repair, particularly the transfer of load to the exterior girder was improved.

The repair method presented in this report has already been successfully deployed on the Buffalo Branch Bridge. The method is somewhat more time consuming and costly than the traditional repair, which involves replacing deteriorated grout with the same kind of grout. However, the new repair method should result in a much longer, maintenance free, life span of the bridge. Based on the positive results of the live load testing, this method can be deployed again with no modifications.

Further testing and analysis should be done to determine if the spacing of the cutouts could be increased. It was shown that the 2 ft spacing for joints between exterior and first interior beam, and 3 ft spacing for joints between interior beams decreased the relative displacements and improved the load transfer in the Buffalo Branch Bridge. It is possible that wider spacings could be acceptable, but further testing should be performed to confirm.

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