Authors' response: *Thank you for the review comments. The question/suggestion to directly test rebar is a good one; however, this testing wasn't done due to resource and scope limitations. Text has been added to the paper to acknowledge this. We also agree with the comment that testing the beams with grouted shear keys would have been preferable. Again, this was not done due to resource and scope limitations. Our hope is that the paper is still useful despite these limitations.*

SERVICE LOAD AND FAILURE TESTS OF SHORT SPAN PRECAST REINFORCED CONCRETE ARCH BEAMS

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ABSTRACT

The South Carolina Department of Transportation (SCDOT) has hundreds of operative short span precast reinforced concrete bridges that utilize "arch" beams. They are so-named because the cross-section includes an arch-shaped void between two stems and below the top flange. The typical age of these bridges is 50 years and plans and specifications no longer exist. To better understand the capacity of these bridges service and strength laboratory tests were conducted. The service load tests were performed on three bolt-connected arch beams to determine transverse load distribution as well as the bolt connection efficacy. Both flexural and shear testing to failure were performed on individual arch beams to determine ultimate capacity. The tested beams were autopsied to determine number, size, and location of the steel reinforcement. The first part of this paper discusses the results from both the service load and failure tests. The second part of this paper explores the flexural and shear capacities of the arch beams through a comparison of the theoretical and experimental capacities.

Keywords: Arch beam, bolt connection efficacy, flexural failure test, shear failure test

INTRODUCTION

The South Carolina Department of Transportation (SCDOT) currently has hundreds of operative bridges that are constructed with precast reinforced concrete arch beams, the cross-section shape is shown in Figure 1. These beams are placed side by side with three evenly spaced bolts connecting the beams together transversely (Figure 2). The arch beams span in the direction of travel and are 25 ft. long, 17 in. deep, 41 in. wide and have two 6 in. webs. In the field these beams could have a non-structural asphalt topping and a grout pocket helping with transverse load distribution.

Figure 1 - Cross Section of Arch Beam

Figure 2 – Bolt Connections between Arch Beams

SCDOT is in the process of load rating every bridge in their inventory. This bridge type was commonly built in the 1960s, and standard plans and specifications no longer exist. Therefore, the concrete and reinforcing steel properties are unknown as well as the configuration and amount of steel reinforcement. This prevents accurate determination of nominal capacity; and hence, load ratings. To aid in the bridge rating process, the following topics were addressed in the testing program:

- Material properties and reinforcement details: Based on typical SCDOT bridge construction practices in the 1960s, concrete compressive strength was assumed to be 4 ksi and steel reinforcement yield stress was assumed to be 40 ksi. The reinforcement details were unknown and determined by autopsy of specimens after load testing.
- Transverse load distribution: The AASHTO LRFD design code defines standard bridge sections and provides associated load distribution equations to guide engineers in design of new bridges and the evaluation of existing ones[1]. However, the arch beam typology is not similar to any of the listed cross sections in AASHTO LRFD. Thus, the load distribution behavior was investigated using two and three beam laboratory set-ups of arch beam bridge spans. In addition, these sets-ups were used to evaluate efficacy of the beam-to-beam bolt connections.
- Shear and flexural strength: Due to the uncertainty of material properties, the unusual shape of the arch beam cross section, and uncertainty of reinforcement details, several beams were tested in the laboratory to determine shear and flexural capacity.

Beams for the test program were salvaged from a demolished bridge that had been in service for decades.

TRANSVERSE LOAD DISTRIBUTIUON AND FLEXURAL TEST METHODOLOGY

The flexural laboratory tests were conducted using a single hydraulic ram rated to 100 tons. Load was applied load through a “spreader” beam in order to create two equal point loads on the arch beam (Figure 3). The service load tests were performed with the webs of adjacent arch beams longitudinally connected using wrench-tight bolts as was done in the field (Figure 4). While the flexural failure test was performed on an individual, unconnected beam. In both cases the arch beams were supported on neoprene bearing pads and had a clear span length of 24 feet.

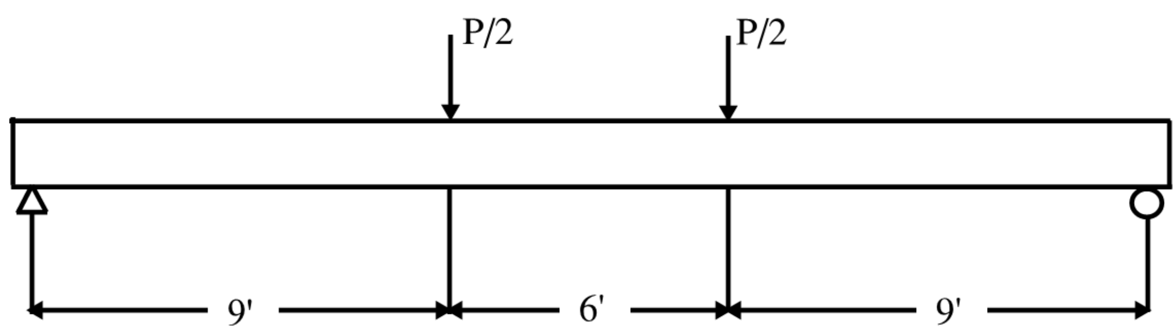

Figure 3 – Free Body Diagram

Figure 4 – Arch Beam Loading Setup

A Bridge Diagnostic Incorporated (BDI) data acquisition system was used during the service load tests. Wire potentiometers (WP) and Linear Variable Differential Transformers (LVDTs) were wirelessly connected to the data acquisition system, which logged data at 10 readings/second. The wire potentiometers were used to measure the vertical displacement of individual arch beams. One potentiometer was placed at the center of each web on the underside of the beam and in order to maximize the measured displacement response all potentiometers were placed at mid-span. Two LVDTs were placed to monitor displacement of the neoprene bearing pads that were supporting the arch beam. This was done in order to correct the mid-span displacement for the effects of bearing pad compression. The transverse locations of the wire potentiometers and LVDTs as well as the naming convention that was used is shown in Figure 5.

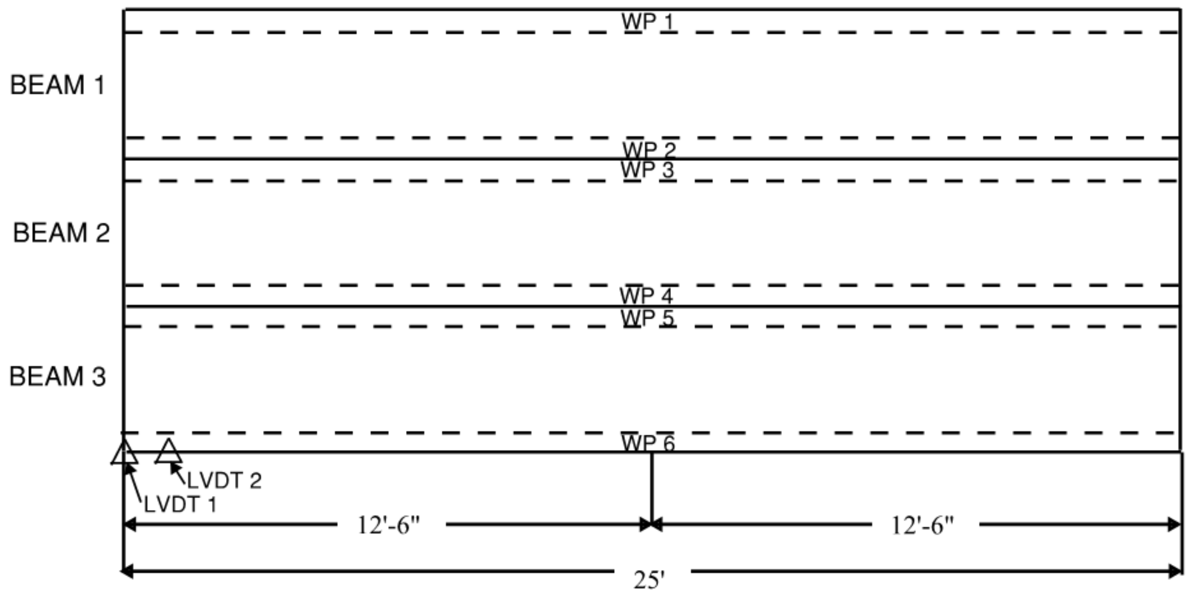

Figure 5 – Testing Equipment Location and Naming Convention

Four different loading locations were used during the service load tests (Figure 6). The first three locations had the hydraulic ram placed at the center of each of the three arch beams. While the fourth location had the hydraulic ram placed above the center of two adjoining beams. These four locations were used to measure the transverse load distribution between adjacent beams as well as evaluate how the bolt connections distributed the load. To measure bolt efficacy, tests were conducted with the bolts wrench-tight, the bolts hand-tight (loose), as well as no bolts at all.

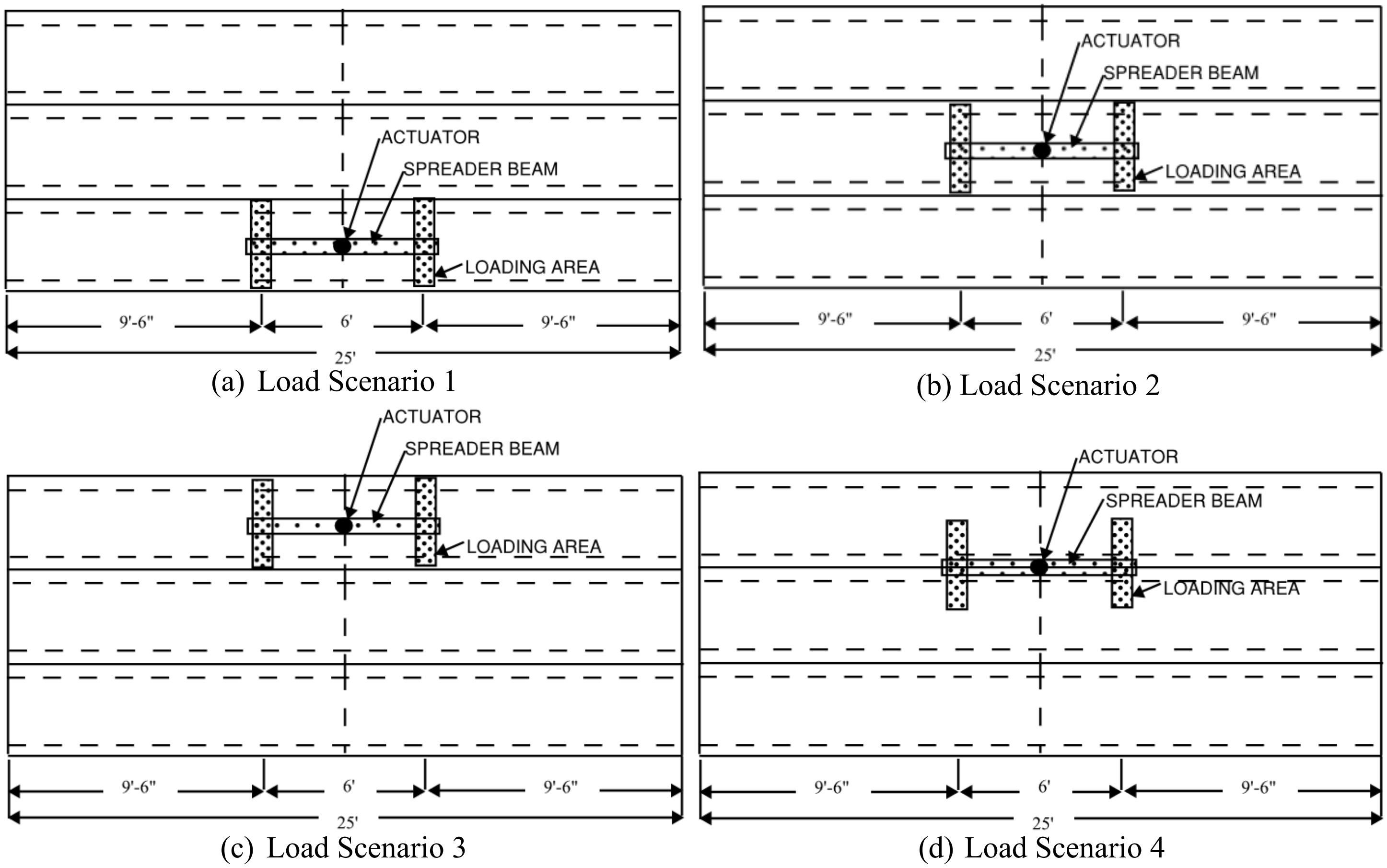

Figure 6 – Service Loading Scenarios

For each service load test the hydraulic ram applied a force in 2 kip increments until it reached 14 kips. This force caused a maximum applied moment of 63.8 kip-ft; the same as the moment in a single arch beam from an H-10 loading, assuming a girder moment distribution factor of 0.5. The assumed distribution factor was based on a single arch beam supporting the full weight of a single wheel line of an H-10 truck. From the test data, load distribution was calculated using Equation 1[2].

$$g_i = \frac{\Delta_i}{\sum_{j=1}^{n} \Delta_j} \qquad \text{Equation 1}$$

Where:

g_i = Distribution of applied load

Δ_i = average experimental displacement of arch beam i under a particular load

Δ_j = average experimental displacement in all arch beams, including beam i, under the same particular load

One of the arch beams from the service tests was then afterwards used to determine flexural capacity. For the flexural failure test a National Instruments (NI) data acquisition system was used. A pressure gauge was connected to the hydraulic jack in order to measure the pressure, which was used to determine the applied load. Wire potentiometers were used during this test to measure vertical deflection. The wire potentiometers and pressure gauge were connected to a data acquisition system, which logged 10 readings/second. The wire potentiometers were located in the same position as the previously mentioned BDI wire potentiometers (WP 5 and WP 6). The flexural failure test had the same load and boundary conditions as shown in Figure 3.

During the flexural failure test the load was increased by 10 kip increments until the hydraulic ram reached an applied load of 70 kips. After this the increment was 5 kips until the arch beam failed. Failure for reinforced concrete under flexural loading was identified by crushing on the compression (top) face concrete.

SHEAR TEST METHODOLOGY

Shear capacity is always a critical consideration for bridges, but particularly for bridges without plans[3]. Accordingly, a shear test to failure was conducted on a single 25 ft. arch beam. Force was applied using a single hydraulic ram rated for 300 kips. The test frame supporting the load was composed of a single I-beam oriented transversely relative to the test specimen, with support beams above and below to transfer the load to the vertical support structure which was bolted to the laboratory strong floor (Figure 7).

Figure 7 - Load configuration

The test specimen was supported on concrete blocks at both ends, with an 8 in. wide steel bearing plates and neoprene bearing pads between the blocks and specimen (Figure 8). The unsupported length was 23 ft. 8 in. Load was applied at 34 in. from the support, a distance equal to twice the arch beam height.

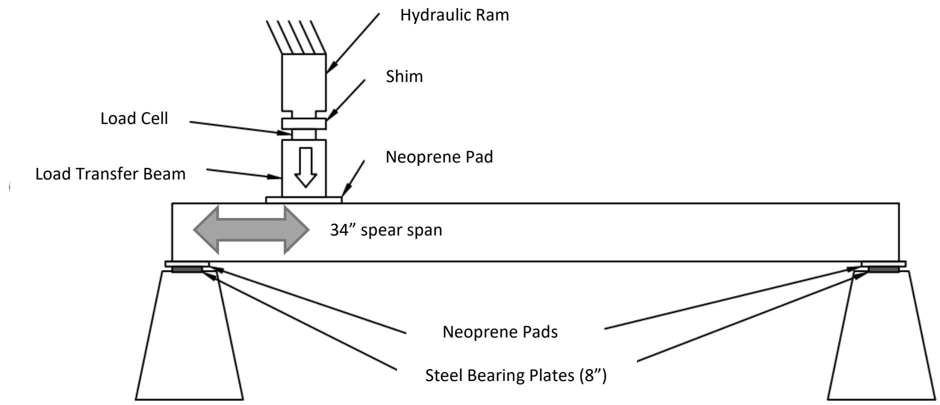

Figure 8 - Test specimen support and load conditions

Instrumentation for this test included two 100 kip load cells and six LVDTs (Figure 9). Data was collected with a Vishay Micro-Measurements system at a sampling rate of 10 Hz. Two load cells were used in parallel to distribute the load, as the anticipated failure load exceeded the capacity of an individual load cell. The summed value from these load cells compared favorably to the pressure output from the hydraulic pump. Load rate was controlled manually.

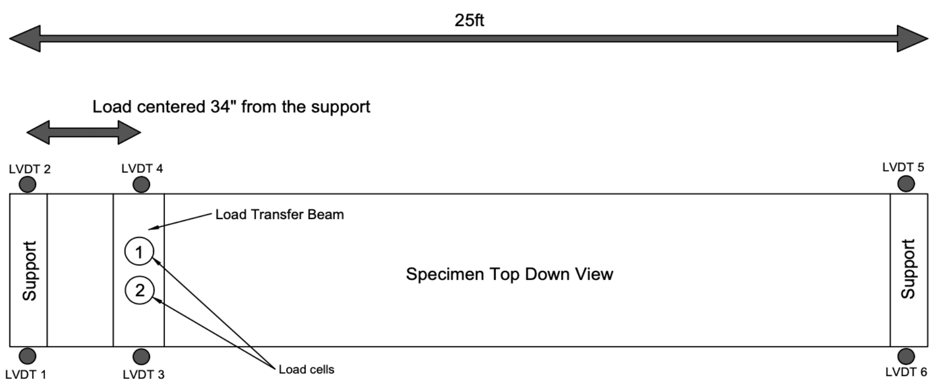

Figure 9 - Sensor configuration

TEST RESULTS

TRANSVERSE LOAD DISTRIBUTION

The bolts connecting adjacent arch beams for the first round of service load tests were wrench-tight. This was thought to be the best-case scenario for bolt tightness for bridges of this age. Figure 10 shows example data collected from a service load test, in this case scenario 1 (Figure 6a).

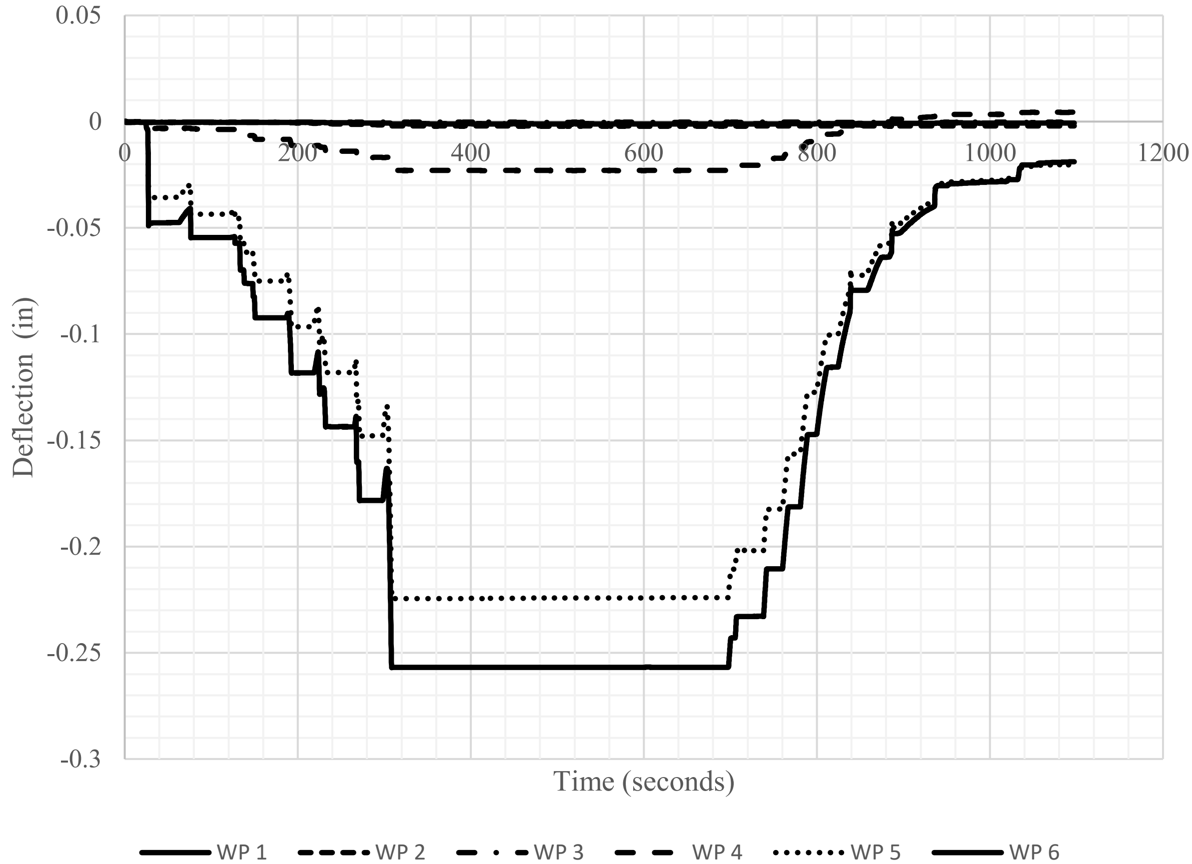

Figure 10 – Data Collection from Wire Potentiometers during Load Scenario 1

From the plot it is noticeable that the changes in displacement correspond to the two kip increments of the applied load. It is also evident that the two wire potentiometers on loaded beam 3 (WP 5 and WP 6) experience the largest vertical displacement. However, WP 5 is slightly less than WP 6 because of the bolts connecting beams 2 and 3 together. The maximum deflections in WP 4, WP 5, and WP 6 are approximately 0.02 inches, 0.22 inches, and 0.26 inches, respectively. The deflection of WP 4 is about 10% of that in WP 5 indicating that some load is shared between beams 2 and 3.

Figure 11 presents the maximum average displacement from each beam during load scenarios one through four. The figure demonstrates that all three beams behaved similarly when they directly received load; the directly loaded beams received most of the load and had the greatest displacement, with some of the load being distributed to beams adjacent to the loaded one. For load scenario four beams 1 and 2 were directly loaded and experienced similar magnitudes of displacement. This indicates more uniform load distribution between the directly loaded beams, with a small portion of the load going into the unloaded beam (beam 3).

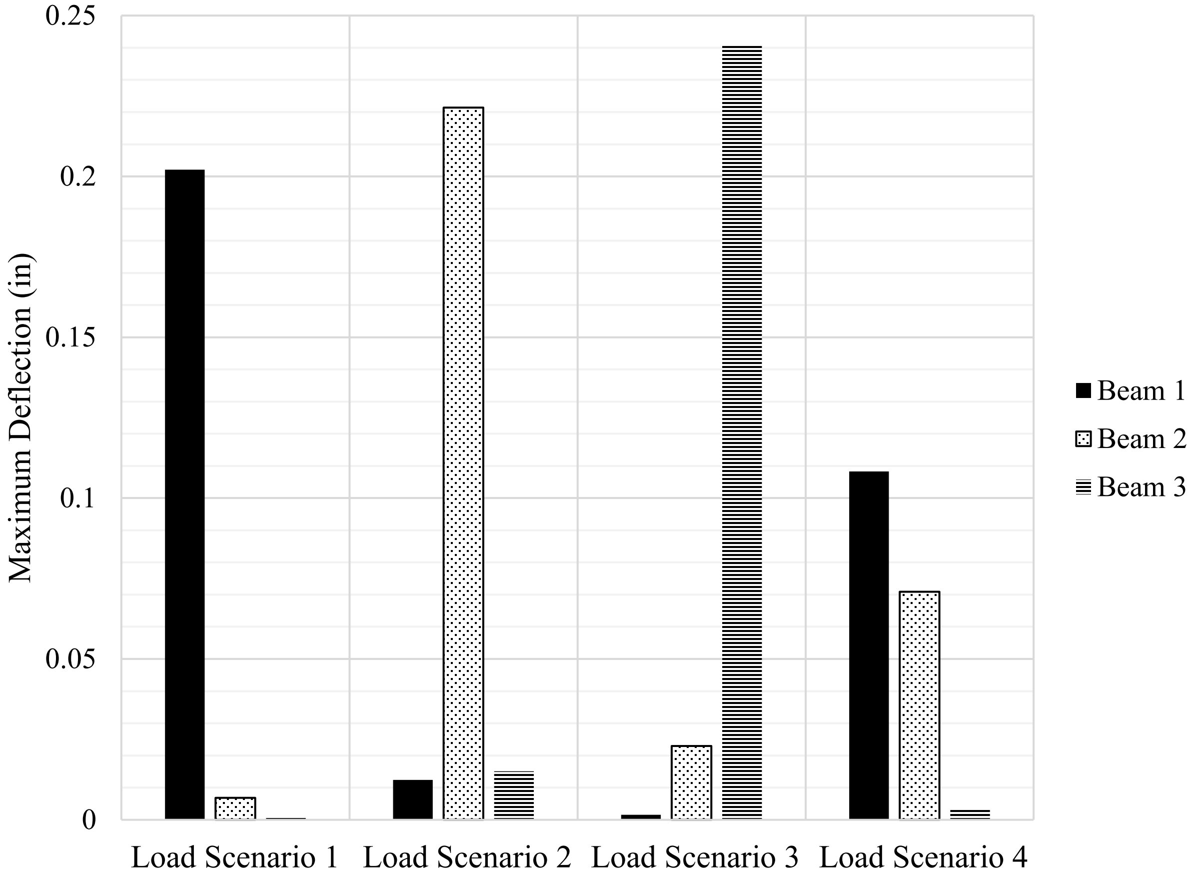

Figure 11 – Maximum Deflections of each Beam for all Service Load Scenarios

The service load tests verified that the assumed distribution factor of 0.5 is reasonable and conservative as a minimal amount of load was distributed transversely to adjacent arch beams. This means that the arch beams tended to act as single entities and not as a cohesive unit.

CONNECTION EFFICACY

Connection efficacy is defined as the ability of the connection between beams to transfer load. In this case bolts form the connections that are intended to transfer loads between the adjacent arch beams. A second round of service load tests was conducted to study the effects of bolt condition on connection efficacy. Each test in the second round was performed with a 14 kip load similar to the first rounds of service load tests. Recall that this loading represents an assumed distribution factor of 0.5 or one wheel line of an H-10 truck load per arch beam. The three bolt configurations tested in the second round: wrench-tight, hand-tight (loose), and no bolts present. This was done to investigate the relationship between bolt tightness and connection performance (i.e., transverse load distribution).

Figure 12 compares the load deflection response of the directly loaded beam for each of the different bolt configurations. The directly loaded beam exhibited approximate linear elastic behavior during the loading. When the applied moment was equal to that of an H-10 truck (M_{H10} on the figure) the deflection reduced slightly as the connection went from no bolts present, to loosened bolts, to tightened bolts, respectively. This was expected behavior as the presence of bolts and tightening increased the amount of load transferred to adjacent beams that did not have direct load. When the bolts were not present the loaded beam had to support the full load, and thus had relatively greater deflection.

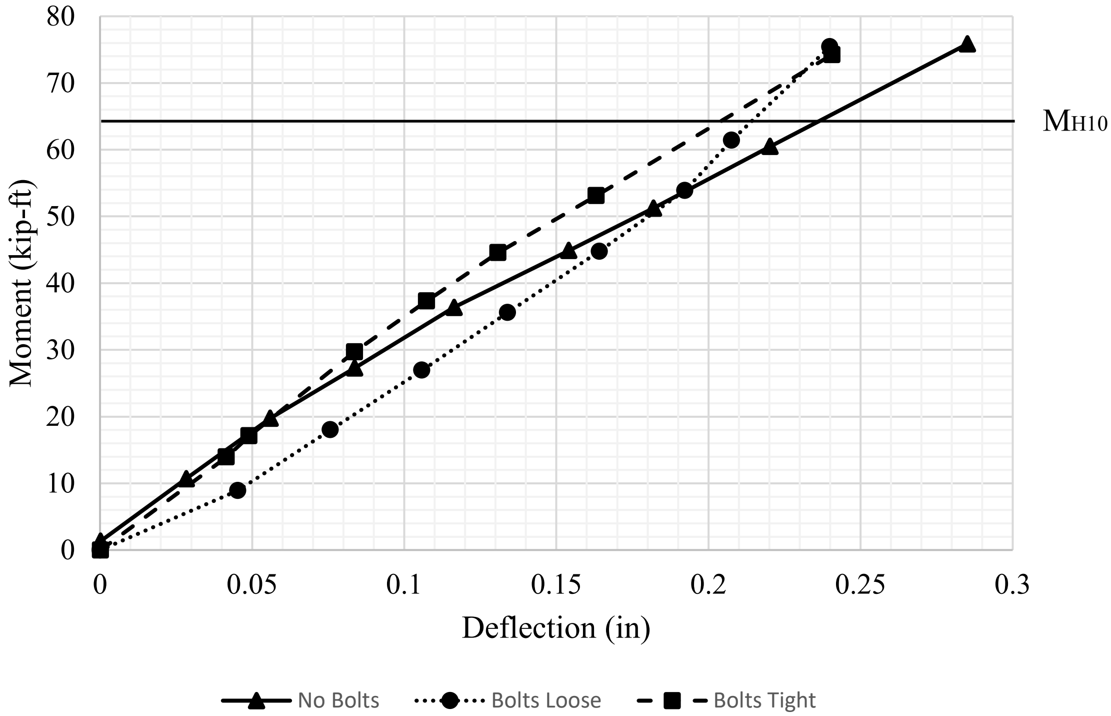

Figure 12 – Moment vs. Mid-Span Deflection with Different Bolt Configurations

Figure 13 is a graphical representation of the distribution of the applied load for the different bolt configurations. Distribution of load was quantified using Equation 1. In practice the arch beams have a grout pocket between adjacent girders, however, the beams were delivered without grout and none was added for the laboratory tests. Therefore, the distribution of the applied load observed in the lab tests is conservative and gives an upper bound (worst case) for the distribution between the arch beams. A single beam condition has no bolts present and all of the applied load is supported by the single arch beam (i.e., distribution of applied load = 1.0). When the bolts were installed but loose, 98% of the load was supported by the directly loaded beam. When bolts were installed and tightened, the directly loaded beam supported 96% of the load. The service load test with loosened bolts was performed on only two connected beams due to the third beam carrying less than 1% of the load during the tightened bolts test. Based on the service load tests, it is concluded the bolt connections do little to provided transverse load distribution.

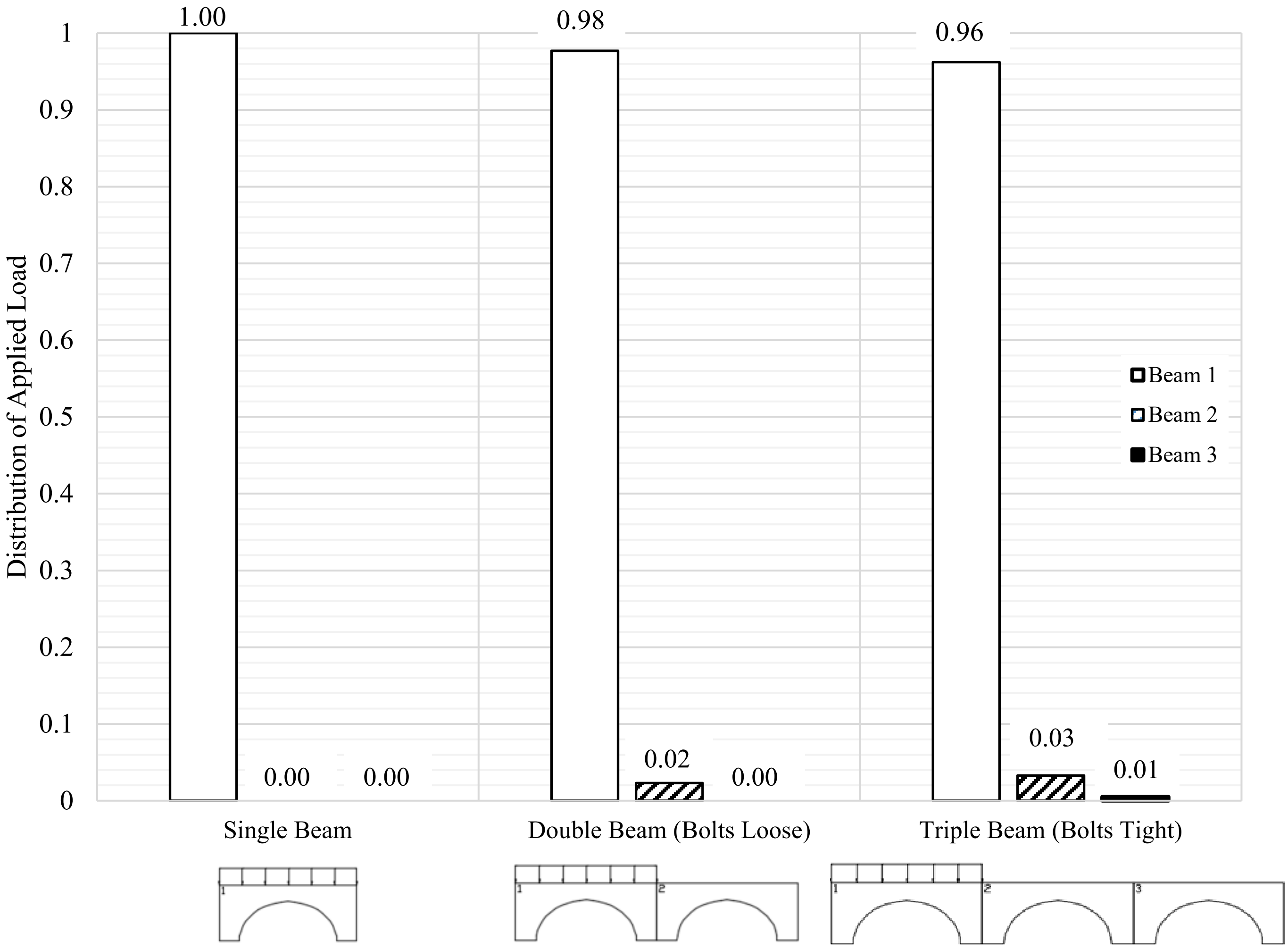

Figure 13 – Distribution of Applied Load with Different Bolt Configurations

FLEXURAL STRENGTH TEST

Flexural failure occurred at around 597 kip-ft. At failure the arch beam had undergone significant deformation (Figure 14) and had many flexural cracks (Figure 15). At the peak moment the concrete at the top of the member crushed in compression (Figure 16) and the reinforcement near the top of the beam buckled Figure 17.

Figure 14 – Deformation at Failure

Figure 15 – Cracking near Load Point

Figure 16 – Top Flange Concrete Compression Failure

Figure 17 – Buckled Compression Steel

Applied moment vs. mid-span deflection from the flexural failure test is shown in Figure 18. Analysis and observation prior to testing showed that the arch beam had already experienced cracking due to previous loadings, self-weight and traffic. From the plot it is noticeable that the beam exhibited approximate linear-elastic behavior until about 493 kip-ft; this is likely the point at which the steel reinforcement began to yield. Once the steel had yielded the slope of the moment versus deflection plot was constant with deflection increasing significantly until the concrete crushed and the top compression steel buckled. Failure occurred at a total moment (self-weight plus applied) of 597 kip-ft. For comparison purposes, Figure 18 includes horizontal lines representing different theoretical calculations. The horizontal lines present represent 1) the calculated self-weight moment of 37.5 kip-ft, 2) the service load moment caused by one half of an H-10 loading, self-weight and a two inch wearing surface of 150 kip-ft, 3) the ultimate moment for an H10 strength I limit state (232 kip-ft), 4) the ultimate moment for an H15 strength I limit state (290 kip-ft), 5) the calculated nominal moment capacity of 311 kip-ft. The nominal capacity was calculated using the reinforcement sizes and placement from the autopsy. The next section will discuss the large difference between tested nominal strength and calculated strength and design moments.

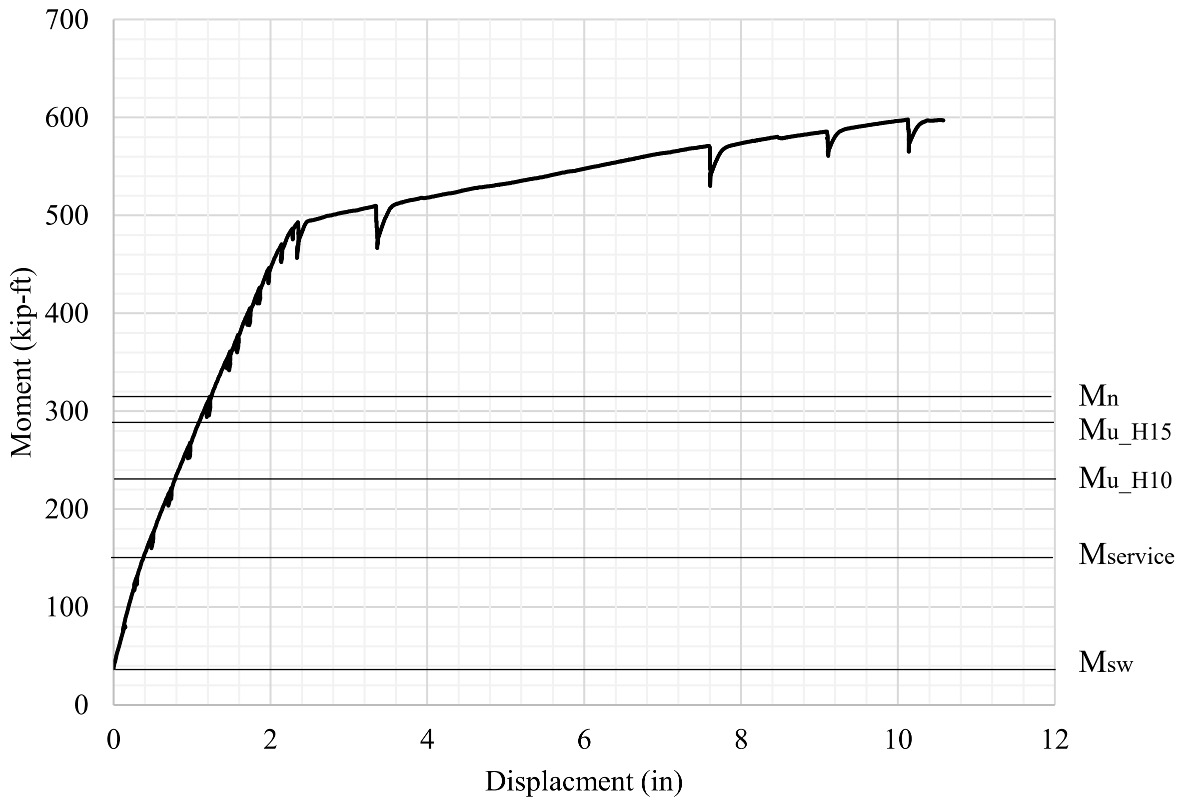

Figure 18 – Moment vs. Mid-span Displacement Plot

SHEAR STRENGTH TEST

The failure load for the arch beam was 117 kip, which corresponded to a peak shear force of 108 kip at the support nearest to the load point. The failure mode is shown in Figure 19. Shear and displacement data for the shear test is shown in Figure 20. The displacement data presented in Figure 21 is from LVDT 3 and 4 (Figure 9). Shear behavior was relatively linear with minimal cracking to shear force until approximately 50 kips. At this level of load noticeable cracking occurred and continued until ultimate shear capacity was reached. Shear failure occurred in only one web.

Figure 19 - Photograph of failure mode

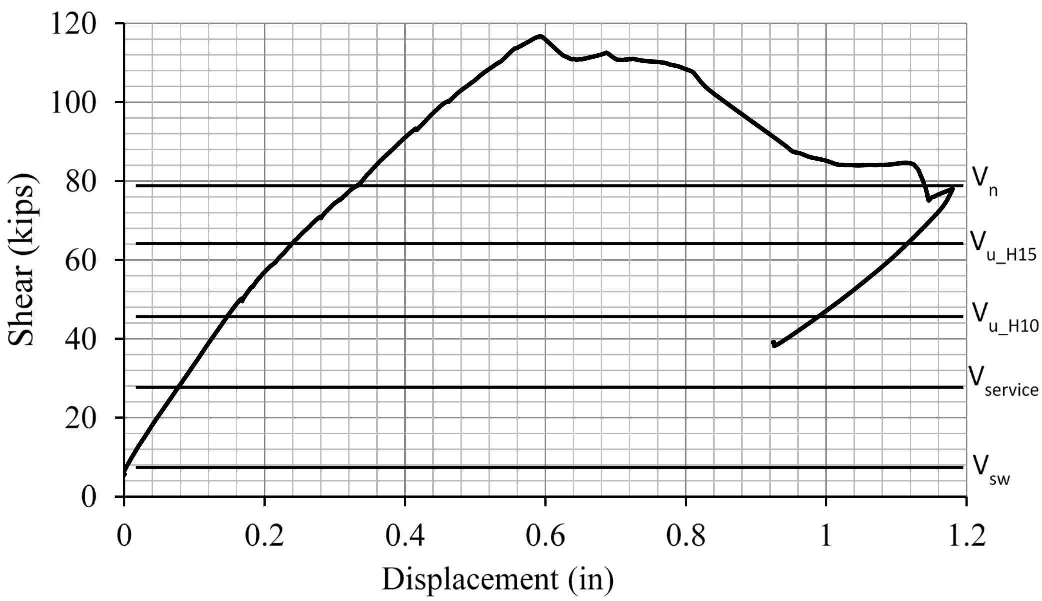

Figure 20 – Shear vs. Displacement Plot

For comparison purposes Figure 20 includes horizontal lines representing 1) the calculated self-weight shear, 2) the service load shear caused by one half of an H-10 loading, self-weight and a two inch wearing surface, 3) the ultimate shear for an H10 strength I limit state, 4) the ultimate shear for an H15 strength I limit state, 5) the calculated nominal shear capacity (approximately 80 kips).

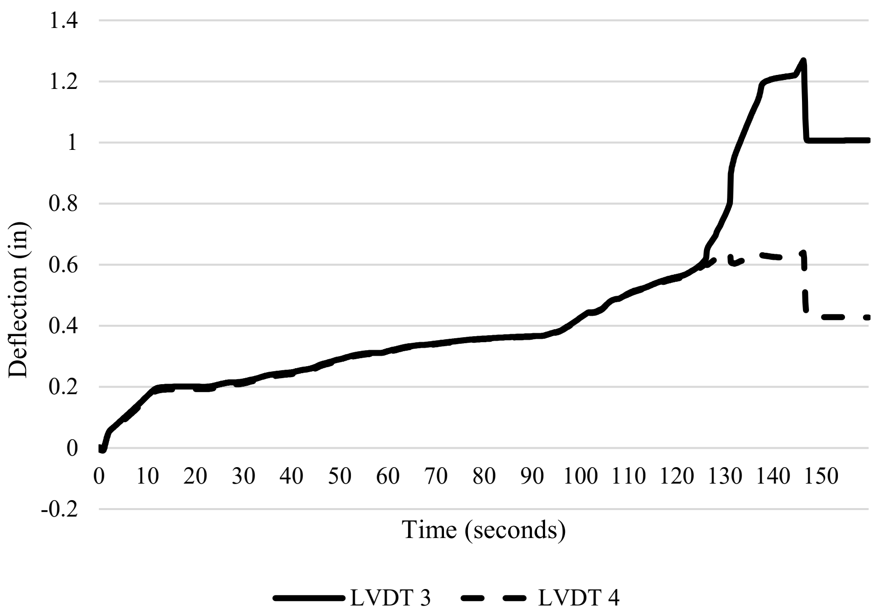

Figure 21 - Deflection vs. Time

After shear failure the load and displacement were asymmetric, resulting in noticeable differences on the different sides of the specimen (Figure 21) as the load approached peak shear capacity. Displacement on both sides of the beam was approximately 0.6 in immediately prior to shear failure and peak load.

AUTOPSY RESULTS

Once the arch beams were tested to failure, a jackhammer was used to further expose the reinforcement by breaking away the concrete. Once the reinforcement was sufficiently uncovered, a tape measure was used to measure the spacing between reinforcements and a caliper was used to measure the diameter of the reinforcement. The findings are shown in Figure 22 and Figure 23.

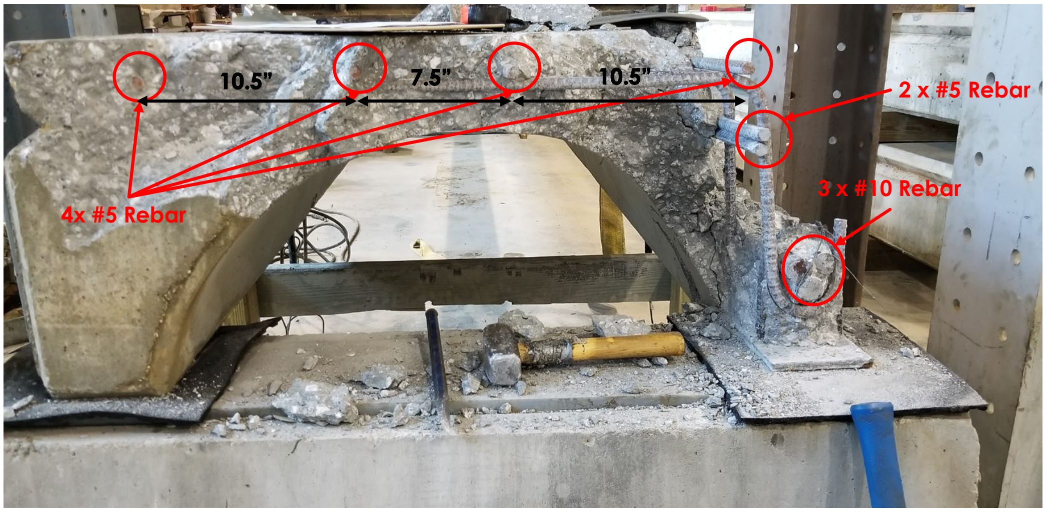

Figure 22 – Rebar Size and Spacing found in Arch Beam

Figure 23 – Autopsy Showing Reinforcement Pattern and Bearing Plate

FACTORS CONTRIBUTING TO EXPERIMENTAL CAPACITY

In this section of the paper possible explanations for the differences between tested and calculated nominal flexural and shear strength capacities will be discussed. The most plausible reasons for the differences are:

1. Concrete compression strength higher than the assumed 4 ksi
2. Steel yield strength larger than the assumed 40 ksi
3. Confinement of concrete compression zone by shear reinforcement
4. Horizontal thrust from bearing restraints during shear and flexural tests

In order to understand if the concrete properties were higher than assumed, six cores were taken from two arch beam specimens. Each core had a diameter of 2.8 in. and a length of 5 in. The length-to-diameter ratio was 1.8. Table 1 shows the measured and corrected compressive strengths of the samples. The measured compressive strength was almost 1.9 times the design compressive strength (4,000 psi). The reason for this increased strength is not clear but follows a similar trend to that recently reported by a bridge consultant who is also evaluating South Carolina bridges[4]. The nominal compressive strength (7,360 psi) was calculated by considering the average strength of five specimens minus 1.65 times the standard deviation.

Table 1 - Concrete compression test results

Diameter (in.)	Length (in.)	L/D	Failure Load (lb.)	Measured f'c (psi)
2.8	5	1.8	48,240	7,834
2.8	5	1.8	47,000	7,633
2.8	5	1.8	47,150	7,657
2.8	5	1.8	45,350	7,365
2.8	5	1.8	48,640	7,956
			Average	7,689
			SD	200.7
			1.65 x SD	331.2
			Corrected f'c (psi)	7,360

SD = Standard deviation (population)
L = length
D = diameter

The nominal flexural and shear capacities were re-calculated with different combinations of concrete compressive strength (4 ksi and 7.5 ksi) and steel yield stress (40 ksi and 60 ksi). While direct testing of rebar samples would likely have been insightful, testing was not conducted due to limited resources and scope. Comparisons of experimental and nominal moment capacities were used as a direct measure. Based on typical construction practices of the 1960s 40 ksi was assumed and based on today's typical material standards 60 ksi was used to see if the nominal moment capacity would become closer to the lab tested flexural capacity. Table 2 shows the calculated nominal moment capacity for the four combinations of concrete compressive strength and steel yield stress. Table 3 presents nominal shear capacities for the same combinations of materials properties. Shear capacities were calculated using the AASHTO LRFD[5] simplified procedure with $\beta = 2$ and $\theta = 45^0$. Comparing the experimental results with the theoretical results, the arch beam material properties resulting in the closest match with the experiments are 7.5 ksi concrete compressive strength and 60 ksi steel yield stress. Thus, the material properties of the beams appear to be greater than those that would otherwise be conservatively assumed based on the vintage of the arch beams.

Table 2 - Nominal flexural capacity

Expected Combination	Concrete Compressive strength (ksi)	Steel yield stress (ksi)	Nominal Moment Capacity (kip-ft)
1	4	40	313
2	4	60	446
3	7.5	40	339
4	7.5	60	479

Table 3 - Nominal shear capacity

Expected Combination	Concrete Compressive Strength (ksi)	Stirrup yield Stress (ksi)	Nominal Shear Capacity (kips)
1	4	40	78.4
2	4	60	82.8
3	7.5	40	87.0
4	7.5	60	115

Increased flexural strength due to the confining effects of the transverse reinforcement is another potential factor that could have caused the higher-than-expected moment capacity. The arch beam was found to have #5 at 5.5 in. intervals throughout the entire span. However, it is understood that extra confinement does more to increase the ductility of the concrete beams but does little to increase flexural strength[6].

While the boundary conditions were intended to behave as pin and roller (Figure 3), horizontal "thrust" reactions were present at both reactions during the flexural failure test. This is based on the observation of the steel beam supports below the arch beam "leaned" outward due to the thrust applied through the bearing. Thin neoprene bearing pads were placed between the arch beam and the supporting steel beam which partially, but not completely, mitigated the thrust reaction. The implication is that the thrust led to a partial arching action in the beam which tends to increase the stiffness and capacity relative to a beam on true pin-roller supports. Although this action would have contributed to the experimental flexural stiffness and capacity, the boundary conditions in the lab were likely more compliant as those in the field. Arch beams of this vintage do not have neoprene pads in the field and are also restrained by abutments and adjacent spans. Thus boundary conditions in the field are likely to have greater impact on flexural capacity and stiffness than boundary conditions in the lab tests.

SUMMARY AND CONCLUSIONS

Precast reinforced concrete arch beams were tested under service and ultimate conditions. The beams were in service for approximately 50 years prior to testing. Plans and specifications for the arch beams no longer exist. Service load tests were conducted on systems of one, two, and three beams to evaluate load distribution through the three bolt-connections which joined adjacent beams. Failure tests were performed on individual beams in order to investigate the flexural capacity and shear capacity. Autopsies were performed on specimens to determine the steel reinforcement details. The conclusions presented below are specific to the tested specimens and may not be generally applicable of all arch beams in South Carolina.

The following are conclusions based on the test programs:

- Transverse load distribution between the tested adjacent girders was found to be negligible. The presence of connection bolts (even tight or loose) had no significant effect (less than 4%) on transverse load distribution.
- The tested flexural capacity of the arch beam was 597 kip-ft. Based on the vintage of the beams the assumed material properties would be a steel yield stress of 40 ksi and concrete compressive strength of 4 ksi. The values result in a calculated nominal moment capacity of 311 kip-ft. The large difference between these values suggests that the material properties – particularly the reinforcement yield stress – were greater than the assumed values.
- The tested shear capacity of the arch beam was 108 kips. For comparison, the calculated nominal shear capacity was 78 kip, assuming 40 ksi yield stress and concrete compressive strength of 4 ksi.
- The arch beams had a significant amount of reinforcement, including (3) No. 10 bars longitudinal at the bottom of each web, (8) No. 5 longitudinal bars in or near the top flange, and No. 5 stirrups at 6 in. spacing.
- The compressive strength of the concrete cores, after adjusting for standard deviation, was approximately 7,400 psi.

ACKNOWLEDGEMENTS

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