LOAD TESTING FOR BRIDGE RATING:
DEAN’S MILL OVER HANNACROIS CREEK

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Dean’s Mill Road Over Hannacrois Creek

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ABSTRACT

The report discusses testing and load rating of the bridge carrying Dean’s Mill Road over the Hannacrois Creek in Greene County, New York (County Bridge BIN 3201350). The bridge was built in 1961, and consists of five 70-ft long post-tensioned bulb-T beams connected by 8-inch wide closure pours. No documents or plans for the structure are available. The bridge deck is topped with an asphalt overlay for a riding surface. It is a single span structure, has two traffic lanes, and an AADT of approximately 650 vehicles. In 1970, the structure was load posted for 12 tons. Absence of the bridge plans discouraged the County Engineers’ evaluation of the structure to increase the 12 ton posting. Pressed by the public’s demand to accommodate school bus traffic on the bridge, the County approached the New York State Department of Transportation (NYSDOT) on how to respond to the pressing demand. The County agreed to a load testing plan proposed by the Transportation Research and Development Bureau (TR&DB) of the NYSDOT. The plan was based on investigating actual behavior of the structure under controlled truck loading. The bridge was instrumented and load tested using trucks of known weights and configurations positioned at specified locations on the deck, to gradually increase their load effect on the structure. The load testing results gave actual stiffness of the bridge beams and revealed the level of fixity at the bridge abutments. Prior to the load testing, the beam geometry was determined and the bridge structure was analyzed using the 1961 AASHTO specifications. The analysis was based on the assumption that the structure was designed to meet the specifications’ requirements regarding satisfying initial and final stresses. This analysis provided estimates for the initial and final post-tensioning forces, and the eccentricity and cross sectional area of the post-tensioning steel. Using this information, the beams’ ultimate and cracking moments, and a safe/threshold moment to be applied during the testing were determined. Utilizing the test results, the bridge load rating at the inventory and operating levels was performed using the ASSHTO load factor method, for both H-20 and HS-20 trucks.
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I. INTRODUCTION

A. GENERAL

Bridge rating is routinely conducted by load rating engineers to evaluate and update load carrying capacities of bridges. The rating process combines the knowledge learned during inspection cycles, which accounts for in-service condition, with analysis to determine two levels of load rating: Inventory and Operating. The inventory rating level generally corresponds to the customary design level of stresses, while reflecting deterioration in existing bridge condition, and represents a live load that can safely use the structure for an indefinite period of time. The operating load rating generally describes the maximum permissible live load to which a structure may be subjected. Using these ratings, transportation agencies often apply their own policies to determine a structure’s safe load carrying capacity, which may result in allowing legal traffic on the bridge, posting a load limit on the structure, or restricting the bridge to certain types of permit loads. Most of the time there is a high degree of confidence in the parameter which influence the load carrying capacity evaluation (such as section properties of structural members, materials strengths, and physical characteristics) to conduct an analytical rating. However, occasionally, these parameters cannot be adequately estimated/accounted for and a test-based rating is warranted for a more realistic appraisal of the structures’ load carrying capacity. In this report, load testing was conducted to determine a test-based rating of a simple span, post-tensioned bulb-T structure with no plans on record. The report discusses the preliminary analysis conducted to determine safe loads that can be applied during the testing, the plans developed to instrument and test the structure, the results from the load testing, and how these results are used in a rating analysis.

B. BACKGROUND

In the summer of 2004, the Transportation Research and Development Bureau (TR&DB) at the New York State Department of Transportation (NYSDOT) launched a program for load testing of bridges for load rating. Requests to test a number of structures came from Region 8 and the Main office Structures Division. In October 2004, TR&DB was contacted by Region 1 engineers regarding the possibility of load testing a county bridge which was load posted for 12-tons and has no plans on record. The structure (Figures 1 and 2) carries Dean’s Mill Road over the Hannacrois Creek near the Town of New Baltimore in Greene County, New York (BIN 3201350). The bridge was built in 1961 and is made of 5 post-tensioned concrete bulb-T beams with 8-in. wide closure pours between the beams and an asphalt overlay riding surface.
In 1970, a load posting of 12-tons was placed on the bridge. No bridge plans or documentation for the load posting is on file. Since no information on the post-tensioning forces was available, a preliminary analysis to estimate the capacity of the beams was performed using the 1961 edition AASHTO Standard Specifications for Highway Bridges (1). Among the important revisions in this edition were the sections on Prestressed Concrete which were based largely on the report of the Joint ASCE-ACI Committee on Prestressed Concrete of 1958 which had been used as Tentative Specifications for two years.

Instrumentation and load test plans were first developed, and the bridge was instrumented on January 31 and February 1, 2005, and load tested on February 8, 2005.
II. PRELIMINARY ANALYSIS

The analysis of the bridge structure conducted prior to the load testing is discussed in this chapter. The goal from this analysis was: 1) to identify the beams’ design parameters, 2) to estimate ultimate and cracking moment capacities, 3) to determine the maximum load that can be safely applied during the testing and 4) estimate expected strains under this load. Identification of the design parameters was important to determine the stresses that will be used in the load rating equations. Ultimate and cracking moments are needed to determine the safe load to be applied and the strain induced by this load is an important trigger to watch for during the testing.

A. SECTION PROPERTIES

In the absence of the bridge plans, the County Engineers had to rely on field measurements of the structure’s critical dimensions shown in Figure 3. Using these dimensions, ANSYS finite element modeling and analysis software was used to determine the section properties for a typical interior beam (Figure 4). These properties were assumed to apply for the fascia beams, ignoring the parapet contribution to the section properties.
Figure 3. Field-measured dimensions shown on bridge section.
Figure 4. Section properties of the bulb tee beams.

B. BEAM ANALYSIS

A typical beam was analyzed using a Mathcad program. The analysis was performed assuming that the beam was designed to satisfy the 1961 AASHTO requirements on initial and final stresses (1). Section properties were calculated based on the data in Figure 4. Dead load moment was calculated using the weight of the beam, closure pours, guide rails, and the asphalt overlay (Total dead load moment $M_{DL} = 522 \text{ kip-ft}$). Live load moment was calculated assuming the bridge was designed for an AASHTO HS-20 or H-20 loading, using a distribution factor of $S/5$, where $S$ is the beam spacing (4 ft-4 in.), and an impact factor of 1.26 (respective design moments, $M_{HS-20} = 535.9$ or $M_{H-20} = 384.9 \text{ kip-ft}$) (1,2,3).
B.1. ASSUMPTIONS

The following assumptions were made in the analysis:

1. Compressive strength of concrete at 28 days $f'_c = 5000$ psi.
2. Compressive strength of concrete at time of initial prestress $f'_{ci} = 4000$ psi.
3. Modulus of elasticity of concrete $E_c = 57000 \sqrt{f'_c}$.
5. Nominal yield point stress of prestressing steel at 1% extension $f_{sy} = 192$ ksi.
6. Modulus of elasticity of steel $E_s = 29000$ ksi.

B.2. ALLOWABLE STRESSES

The following allowable stresses were used in the program:

1. Temporary stresses in the steel before losses due to creep and shrinkage = $0.7 f'_s$.
2. Steel stress at design load (after losses) = the smaller of $0.6 f'_s$ or $0.8 f_{sy}$.
3. Temporary compressive stresses in concrete before losses due to creep and shrinkage = $0.55 f'_{ci}$.
4. Temporary tensile stresses in concrete before losses due to creep and shrinkage = $3 \sqrt{f'_{ci}}$.
5. Concrete compressive stress after losses have occurred = $0.4 f'_c$.
6. Concrete tensile stress after losses have occurred = 0.
7. Concrete cracking stress $f_{cracking} = 7.5 \sqrt{f'_{ci}}$.

B.3. PRESTRESS LOSSES

Prestress losses were calculated based on:

1. Post-tensioning steel stress losses due to all causes except friction = 25 ksi.
2. Post-tensioning steel stress losses due to friction = $0.15 \times 0.7 f'_s$.

B.4. ULTIMATE MOMENT CAPACITY

Ultimate moment capacity of the beam was calculated by first confirming that the neutral axis was located inside the top flange. This is usually the case when the flange thickness calculated using $t_{flange} = 1.4 \frac{d \rho f'_{su}}{f'_c}$ is greater than the actual flange thickness $t$ ($t = 6.5$ in.).

The following equations:

$$M_u = A_{sr} f_{su} d (1 - 0.6 \frac{A_{sr} f_{su}}{b'd f'_c}) + 0.85 f'_{c}(b - b') t (d - 0.5t)$$  \hspace{1cm} (1)

where
\[ A_{sr} = A_s - A_{sf} \]  
(2)

is the steel area required to develop the ultimate compressive strength of the web.

\[ A_f = 0.85 f'_c \frac{(b - b')t}{f_c'} \]  
(3)

is the steel area required to develop the ultimate capacity of the overhanging portion of the flange. The steel stress \( f_{su} \) is estimated using the following equation

\[ f_{su} = f_s' \left(1 - 0.5 \frac{O_f}{f_c'} \right) \]  
(4)

which is also subject to the condition that the effective prestress after losses is not less than 0.5 \( f_c' \).

B.5. CRACKING MOMENT

Cracking moment was calculated using:

\[ M_{Cracking} = S_b \left[ P_e \left( \frac{1}{A} + \frac{e}{S_b} \right) + f_{Cracking} \right] \]  
(5)

where \( S_b \) is the section modulus for beam bottom, \( P_e \) is the effective prestress force, \( A \) is the beam cross section area, \( e \) is the post-tensioning steel eccentricity, and \( f_{Cracking} \) is previously defined.

B.6. MAXIMUM AND MINIMUM STEEL PERCENTAGES

The maximum percentage of steel used in the analysis was 0.3 percent and the minimum percentage was calculated using:

\[ \rho_{min} = \frac{A_{sr}}{b'd} \frac{f_{su}}{f_c'} \]  
(6)

B.7. FEASIBLE SOLUTIONS FOR INITIAL PRESTRESS FORCE & ECCENTRICITY

The analysis consisted of determining feasible solutions for the reciprocal of the initial prestressing force and eccentricity using a linear programming approach. In this approach, the four inequalities relating the reciprocal of the initial post-tensioning force and eccentricity were first formulated, based on satisfying the code requirements for midspan top and bottom stresses under initial and service load conditions, and then plotted to determine the region of feasible solutions. The four inequalities were formulated as follows:
where \( P_1(e) \) and \( P_4(e) \), respectively, were based on satisfaction of midspan bottom and top stresses under initial prestressing force and before prestressing losses take place. \( P_3(e) \) and \( P_4(e) \), respectively, were based on satisfaction of midspan bottom and top stresses under final prestressing force and after prestressing losses take place. \( e \) is the prestressing steel eccentricity from the center of the beam, \( A \) is the beam cross sectional area, and the rest of the variables have been previously defined. The resulting feasible solutions utilizing the above equations are given by the regions inscribed by the four lines in Figures 5 and 6, for the AASHTO HS-20 and H-20 truck loadings, respectively. The solutions in these figures are also bound by the physical constraint that the eccentricity \( e \) cannot exceed 18.68 in. (the distance from the NA to the beam’s bottom minus 3 in. cover). Two solutions were investigated to account for the possibility that the bridge was originally designed for either of the two AASHTO loads \((1,2,3,4,5)\).

Any combination of initial prestress force and eccentricity \((P_i \text{ and } e)\) falling in the feasible solution region should result in midspan stresses meeting the allowable stress limits of Section B3. Using any such combination to calculate the stresses \( f_{bi}, f_{ti}, f_{tf}, \) and \( f_{bf} \) in Equation 8 below at midspan should result in stresses meeting the requirements of Section B3 on the respective stresses \( f_{bi}, f_{ti}, f_{tf}, \) and \( f_{bf} \). This is demonstrated for two sample \( P_i \) and \( e \) combinations (one from each solution) in Appendix A.
Figure 5. Feasible solution for AASHTO HS-20 loading.

Figure 6. Feasible solution for AASHTO H-20 loading.
Two initial prestress force and eccentricity combinations were selected from each of the solutions in Figures 5 and 6. Analysis was then performed for each of those combinations to determine effective prestress force $P_e$, area of steel $A_s$, ultimate moment $M_u$, factored dead load and live load moments $M_{Fact}$, cracking moment $M_{cr}$, the moment available for testing $M_{Test}$ and its resulting midspan stress $f_{Test}$, and the maximum deflection to be expected during the test $\Delta$. The results of this analysis are shown in Table 1. Maximum deflections were calculated for a concentrated midspan load calculated using the test moment and assuming simply supported end conditions. From this analysis the following observations can be made:

1. As expected, most economical designs, in terms of area of steel, include combinations of low prestressing forces and large eccentricities. One of those combinations was probably used in the bridge design.
2. All combinations resulted in ultimate moments exceeding factored dead load and live load moments. This makes the graphical solution for initial force and eccentricity combinations plausible.
3. The lowest moment available for testing is $6.88 \times 10^3$ (573 kip-ft). This is the moment that would crack a beam if it was fully applied during the testing.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>HS-20</th>
<th>H-20</th>
</tr>
</thead>
<tbody>
<tr>
<td>$1/P_i \times 10^4$ (1/kip)</td>
<td>High P/T</td>
<td>Low P/T</td>
</tr>
<tr>
<td>$P_i$ (kip)</td>
<td>7.38</td>
<td>13.96</td>
</tr>
<tr>
<td>$e$ (in.)</td>
<td>1355.00</td>
<td>716.50</td>
</tr>
<tr>
<td>$P_e$ (kip)</td>
<td>7.10</td>
<td>18.7</td>
</tr>
<tr>
<td>$A_s$ (in.$^2$)</td>
<td>949.86</td>
<td>502.27</td>
</tr>
<tr>
<td>$M_u \times 10^4$ (kip-in.)</td>
<td>8.07</td>
<td>4.27</td>
</tr>
<tr>
<td>$M_{Fact} \times 10^4$ (kip-in.)</td>
<td>2.65</td>
<td>3.14</td>
</tr>
<tr>
<td>$M_{cr} \times 10^4$ (kip-in.)</td>
<td>2.55</td>
<td>2.55</td>
</tr>
<tr>
<td>$M_{Test} \times 10^4$ (kip-in.)</td>
<td>1.55</td>
<td>1.52</td>
</tr>
<tr>
<td>$f_{Test}$ (ksi)</td>
<td>9.21</td>
<td>8.90</td>
</tr>
<tr>
<td>$\Delta$ (in.)</td>
<td>2.17</td>
<td>2.09</td>
</tr>
</tbody>
</table>

Table 1. Beam analysis for selected $P_i$ and $e$ combinations.
In addition to the above observations, it was also noted that the code requirements on maximum and minimum prestressing steel percentages were satisfied for all four $P_i$ and $e$ combinations.
III. INSTRUMENTATION AND LOAD TEST PLANS

A. INSTRUMENTATION PLAN

The bridge was instrumented at various locations to determine 1) midspan bottom flange stresses for all 5 beams, 2) level of fixity at the ends of all beams, 3) neutral axis location for the two fascia beams (Beams 1 and 5) and one interior beam (Beams 3), and 4) deflections at midspan of Beams 1 and 5 (fascia beams), and Beam 3.

BDI gages (manufactured by Bridge Diagnostics, Inc.) were used in the instrumentation. Three deflection gages were also planned for midspans of Beam 3, 1, 3, and 5 but were not mounted because of logistics problems, described below. A general purpose strain gage measurement system "System 6000" (manufactured by the Measurement Group) was employed for data acquisition. The complete instrumentation plan, including the location of BDI strain gages and proposed LVDT deflection gages is shown in Figure 7.

At the bridge crossing, the Hannacrois Creek is too deep to use ladders or waders to mount instrumentation at the beams midspan locations. Additionally, vertical clearance of the bridge is low. The low clearance combined with the 12-ton posting made the use of a snooper truck impossible. A decision was made to wait for the river to freeze and instrument the bridge from the ice surface. The thickness of the ice was monitored with an ice drill, and on January 31 and February 1, 2005, TR&DB personnel mounted and wired 20 BDI strain gages (Figure 8). On the day of the testing (February 8, 2005), warm weather melted the ice on the creek and instrumenting the midspans of 3 girders with deflections gages (LVDTs) was not possible (Figure 8).

![Figure 7. Original instrumentation plan (LVDT sensors not used in testing).]
Figure 8. Mounting strain gages at midspan and a photo of the melted ice one week later.

B. TRUCK WEIGHTS AND DIMENSIONS

Prior to the testing, the vehicles were weighed and measured. When loading the vehicles with sand, care was taken to evenly distribute the load in the backs of the trucks. Two trucks were used in the testing – a 12-ton vehicle and a 15-ton vehicle. The loading of the 12-ton truck was increased to 15 tons for the second half of the testing. Dimensions and wheel loads are shown in Figure 9, axle and gross vehicle weights are summarized in Table 2, and a photo of a test vehicle is shown in Figure 10. Each of the 15 ton trucks closely resembles an AASHTO H-15 truck (1,2,3).
Figure 9. Test Vehicle dimensions and wheel loads.

Test Vehicle   | Front Axle Wt. (kips) | Rear Axle Wt. (kips) | Gross Vehicle Wt. (kips) |
---------------|-----------------------|-----------------------|--------------------------|
12-ton Truck   | 8.78                  | 15.36                 | 24.14                    |
15-ton Truck   | 11.30                 | 18.50                 | 29.80                    |
12-ton Raised to 15-ton | 9.28                | 20.56                 | 29.84                    |

Table 2. Test vehicle axle and gross weights.
Figure 10. Test vehicle moving into position.

C. LOAD TEST PLAN

The test plan is divided into four phases based on the loading scenario. Prior to the testing, each scenario was analyzed assuming the structure to be simply supported to determine expected strains levels during the testing. The structure will be loaded incrementally, by positioning the test trucks in a manner that it would gradually increase midspan moment on the structure, while continually monitoring the strain gage readings. The following is a full description of the load test plan phases.

C.1. Phase 1

The first portion of the testing focuses one axle line of a 12-ton (the current posted weight limit) vehicle on each of the structure’s 5 beams. The vehicle is to cross the bridge at crawl speed, stopping for readings when the rear axle is at each sixth point of the bridge (5 readings per load sequence). All crossings begin at the south side of the structure. This phase is expected to reveal relative differences among the beams based on their response to similar loads. Lines indicating the beam centerlines and positions to place the rear axle were painted on the bridge prior to the test date (Figure 11). Illustrations of the truck paths are shown in Figures 12 and 13. The beams will be tested in the order, Beam 1 – Beam 5. After each beam has been loaded, peak strain readings for each beam will be compared to the estimated safe strain and a decision will accordingly be made to continue with Phase 2 of the testing or not. At the end of this phase testing, the 12-ton truck will return to a maintenance yard and the gross weight of the vehicle will be increased to 15-tons.
Figure 11. Test grid layout and 12-ton truck on a beam line.

Figure 12. Wheel line locations for Phase 1 (Not to scale).
Figure 12. Wheel line locations for Phase 1 continued (Not to scale).

Figure 13. Phase 1 testing. A 12-ton vehicle with a wheel line on Beam 1 is shown stopping at marked locations on the bridge. This is repeated for each beam.

C.2. Phase 2

Phase 2 of the testing uses a 15-ton vehicle which is planned to travel across the bridge at crawl speed stopping with the rear axle at the bridge’s six points for readings. The same vehicle will cross in the Northbound lane first and then in the Southbound lane. Both crossings begin on the
south side of the structure. This phase will provide information on the structure’s response for a load slightly higher than the posted. The truck paths are shown in Figures 14 and 15. Strain data will be evaluated at each position and if the results are determined to be acceptable, the testing will be continued to Phase 3.

C.3. Phase 3

In Phase 3, two 15-ton trucks will be used. The two vehicles will follow each other across the bridge at crawl speeds, without stopping, keeping the following distance between the trucks to approximately 15 feet. The testing will be repeated in the Northbound first and then in the Southbound lanes. Both crossings to begin at the South end of the bridge. This phase will increase the loading on the structure by about 25 percent more than that used in Phase 2. Truck positions for Phase 3 are shown in Figures 14 and 16. Strains will be continuously monitored to determine if the testing should be continued to Phase 4.

Figure 14. 15-ton vehicle load paths.
Figure 15. Phase 2 testing. A 15-ton vehicle traveling North in the Southbound lane is shown stopping at marked locations on the bridge. This testing was repeated for the Northbound lane.

Figure 16. Phase 3 testing. Two 15-ton vehicle crawling North in the Southbound lane approximately 15' apart. The vehicles do not stop on the bridge. This testing was repeated for the Northbound lane.

C.4. Phase 4

The two 15-ton vehicles will be placed back to back on the bridge. Testing will begin on the Northbound lane with the rear axle of each vehicle at a sixth point of the bridge. Strains will be monitored at each step during the test, and if found to be acceptable, the trucks will be brought
closer together to the 2/6 span markings, increasing the load effect of the bridge. If the strain at all steps are deemed to be acceptable, the trucks will be brought to a final back-to–back position at midspan to produce the maximum loading that will be applied during the testing. The Phase 4 back-to-back configuration will be repeated in the southbound lane. This phase of the testing will produce the highest loading on the structure, because the limited bridge width does not allow for a side by side testing. The truck positions in this phase are shown in Figure 17.

![Figure 17. Two 15-ton vehicles back-to-back. This testing was repeated in the Northbound lane.](image)

**D. ESTIMATED LOAD EFFECTS**

Before the testing, the axle spacing for the test trucks was measured (164 in.) and the axle weights had to be estimated. For the 12-ton vehicle, a steering axle weight of 8 kips and a rear axle weight of 16 kips were assumed, and for the 12-ton truck, a steering axle weight of 10 kips and a rear axle weight of 20 kips were assumed. Using a simply supported finite element beam model, gross moments were calculated for each load case and then reduced to the beam level using the AASHTO distribution factor of 0.43. A modulus of elasticity was calculated for concrete based on a 5000 psi compressive strength. Using the estimated modulus, the section properties in Figure 4, and the estimated load effects, midspan strain for each load case in the testing were obtained as shown in Table 3.
<table>
<thead>
<tr>
<th>Test Truck</th>
<th>Rear Axle Position</th>
<th>Predicted Bridge Moment (kip-ft)</th>
<th>Predicted Beam Moment (kip-ft)</th>
<th>Predicted Strain (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12-ton</td>
<td>1/6 Point</td>
<td>197</td>
<td>85</td>
<td>59</td>
</tr>
<tr>
<td>12-ton</td>
<td>2/6 Point</td>
<td>319</td>
<td>137</td>
<td>96</td>
</tr>
<tr>
<td>12-ton</td>
<td>3/6 Point</td>
<td>365</td>
<td>157</td>
<td>110</td>
</tr>
<tr>
<td>12-ton</td>
<td>4/6 Point</td>
<td>223</td>
<td>96</td>
<td>67</td>
</tr>
<tr>
<td>12-ton</td>
<td>5/6 Point</td>
<td>93</td>
<td>40</td>
<td>28</td>
</tr>
<tr>
<td>15-ton</td>
<td>1/6 Point</td>
<td>246</td>
<td>106</td>
<td>74</td>
</tr>
<tr>
<td>15-ton</td>
<td>2/6 Point</td>
<td>398</td>
<td>171</td>
<td>120</td>
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<tr>
<td>15-ton</td>
<td>3/6 Point</td>
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<td>137</td>
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<tr>
<td>15-ton</td>
<td>4/6 Point</td>
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<td>123</td>
<td>86</td>
</tr>
<tr>
<td>15-ton</td>
<td>5/6 Point</td>
<td>117</td>
<td>50</td>
<td>35</td>
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<tr>
<td>2 15-ton</td>
<td>Following</td>
<td>620</td>
<td>267</td>
<td>187</td>
</tr>
<tr>
<td>2 15-ton Back-to-Back</td>
<td>1/6 Point</td>
<td>230</td>
<td>99</td>
<td>69</td>
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<tr>
<td>2 15-ton Back-to-Back</td>
<td>2/6 Point</td>
<td>568</td>
<td>244</td>
<td>171</td>
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<tr>
<td>2 15-ton Back-to-Back</td>
<td>Midspan</td>
<td>763</td>
<td>328</td>
<td>230</td>
</tr>
</tbody>
</table>

Table 3. Estimated midspan moments and strains on beams during the testing, assuming simply supported end conditions.
IV. ANALYSIS OF THE TEST RESULTS

A. LOAD DISTRIBUTION

A.1. Phase 1

The testing in this phase included a 12-ton truck aligning a wheel line with each of the five bridge beams. Figure 18 shows the midspan strains recorded at the midspan point during each truck crossing. In most paths, the beams intentionally being loaded had the peak strain values. The exception is Path 3. In this passing, Beam 4 had the highest peak strain. For the Paths 1, 2, and 3 crossings, the driver’s side wheel line was placed directly on the beam. For the Paths 4 and 5 crossings, the passenger side wheel load was placed on the beam. Therefore, the Path 3 and Path 4 crossings apply similar loads on Beams 3 and 4. This can be seen in the Phase 1 test plan in Figure 12. The peak strain in this portion of the testing was 50 µε, which is significantly lower than the 110 µε predicted in the preliminary analysis.

The percentage of the total moment on the bridge due to the test trucks is calculated by dividing each midspan gage reading by the sum of the five midspan gage readings, assuming all the beams to have similar section modulii (\(4\)).

\[
M_{pi} = \frac{S_{xi} \varepsilon_i}{\sum S_{xj} \varepsilon_j}
\]  

where \(M_{pi}\) is the moment percentage carried by any beam \(i\), and \(S_{xi}\) and \(\varepsilon_i\) are the beam’s section modulus and strain. The percentages of the total truck moment carried by each of the beams during the truck crossings at the midspan point are shown in Table 4 and Figure 19. As already noted, the similarity of Paths 3 and 4 crossings is reflected in the figure. Figures 20 and 21 were generated to investigate symmetry in load distribution among the beams. The figures show comparisons between beams on opposite sides of the structure during Paths 1 and 5, and Paths 2 and 4 crossings, respectively. The almost identical distribution percentages carried by each pair of beams on opposite sides of the bridge are indications of similarity in load distribution across the joints for the compared paths crossings. This is further illustrated in Table 5 which shows the percentage of the load distributed between any two beams across a longitudinal joint, assuming that 100 percent of the load is carried by the two beams. After an on-site review of the low strains and the observed good load distribution, a decision was made to continue with the next phase of the testing.
Figure 18. Midspan strain readings for the 12-ton crossings.

<table>
<thead>
<tr>
<th>Path Number</th>
<th>Beam Number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>42.5</td>
</tr>
<tr>
<td>2</td>
<td>15.4</td>
</tr>
<tr>
<td>3</td>
<td>7.4</td>
</tr>
<tr>
<td>4</td>
<td>10.4</td>
</tr>
<tr>
<td>5</td>
<td>4.8</td>
</tr>
</tbody>
</table>

Table 4. Distribution percentages for the 12-ton crossings.
Figure 19. Distribution percentages for the 12-ton crossings.

Figure 20. Comparing Paths 1 and 5 distribution percentages for the 12-ton crossings.
Figure 21. Comparing Paths 2 and 4 distribution percentages for the 12-ton crossings.

Table 5. Distribution percentages across longitudinal joints.
A.2. Phase 2

The 15-ton truck traveled North twice on each of the upstream and downstream lanes of the structure in this phase. Figures 22 and 23, respectively, show the maximum midspan strains recorded during each of the truck crossings and the beam moment distribution percentages. The figures show that the two paths on each lane gave almost identical results. The relatively low strains on Beam 1 during the Upstream 2 crossing can be attributed to a minor shift in the truck position away from Beam 1 during the crossing. The percent distributions in Figure 23 again illustrate symmetric behavior of the structure and confirm the conclusions made in the previous section regarding this issue. Again, after an on-site review of peak strain readings and the good load distribution between the bridge beams, a decision was made to continue with the next phase of the testing.

[Figure 22. Midspan strain readings for the 15-ton crossings.]
A.3. Phase 3

This phase of the testing involved two 15-ton trucks following each other at crawl speed without stopping on the structure. The test was repeated on the upstream and downstream lanes, and the time histories for the midspan gages for the two lanes are shown in Figures 24 and 25, respectively. Beam moment distribution percentages are obtained in Figure 26 using peak strains and again in Figures 27 and 28 using continuous representations. From these figures, Beam 1 is clearly the most loaded beam during the upstream lane crossing, while Beams 4 and 5 were the most loaded beams during the downstream crossing. There are noticeable differences when comparing the time histories and distribution percentages of the upstream and downstream following load cases. The strains under comparable loading measured on Beam 1 are higher than those measured on Beam 5. The highest load percentage calculated from this phase of the testing was also for Beam 1. Several factors could be responsible for these differences. First, the position of the trucks in the lane could have varied: The vehicles could have been very close to the curb near Beam 1 during the upstream crossing and further away from Beam 5 curb during the downstream crossing. Higher strain readings are sometimes indicative of a low section modulus or poor load distribution; however, these are unlikely based on the results of the previous phases. The joints on the sides of both fascia beams, along the closure pours, showed deterioration that allows for water seepage and formation of icicles (Figure 29). Based on the low strain readings and good load distribution, the testing continued with positioning the two 15-ton trucks back-to-back as planned in Phase 4 testing.
Figure 24. Two 15-ton trucks following each other on the upstream lane of the bridge.

Figure 25. Two 15-ton trucks following each other on the downstream lane of the bridge.
Figure 26. Load distribution percentages for 2 15-ton trucks following each other.

Figure 27. Continuous representation of load distribution percentages for 2 15-ton trucks following each other on the upstream lane.
Figure 28. Continuous representation of load distribution percentages for 2 15-ton trucks following each other on the downstream lane.

Figure 29. Icicles from water seeping between beams.
A.4. Phase 4

This phase of the testing included loading of the structure on the upstream and downstream lanes, one lane at a time, with two trucks positioned back-to-back and moved in steps towards the bridge midspan. Midspan strain results are shown in Figures 30 and 31 for the upstream and downstream lanes, respectively. The heaviest load case, “Mid” with the rear axles approximately 5 feet from midspan, places 763 kip-ft in a lane which is greater than an H-20 lane loading for a 66 ft span (clear span). The peak reading under this loading was 117 µε and again, it was on Beam 1. Load distribution patterns for all the back-to-back load cases can be seen in Figure 32. The patterns are similar to those found in the earlier testing. Notice how the load percentages carried by Beam 1 increase with load under the upstream loadings. This increase was not seen on Beam 5 during the downstream loading, and was most likely caused by a shift in the truck positions away from the Beam 5 curb.

![Figure 30. Upstream lane back-to-back midspan strain results.](image-url)
Figure 31. Upstream lane back-to-back midspan strain results.

Figure 32. Load distribution percentages for all 15-ton truck back-to-back positions.
The strain results for the maximum loading position during this phase testing were used to generate the comparative load percentages plots of Figures 33 and 34, comparing midspan to those recorded at the beam ends. The end span percentages are generally within 10 percent of those obtained for the midspan location.

![Figure 33](image_url)

**Figure 33.** Load distribution percentages: Midspan versus ends for 15-ton truck back-to-back at maximum loading position on upstream lane.
B. NEUTRAL AXIS LOCATION

Gages were placed on the bottom flange at all midspan beam locations and additional top flange gages were placed on Beams 1, 3, and 5. With the distance between the top and bottom gages known (29.5 in.), the neutral axis of the beam can be calculated using the formula:

$$NA = 29.5 \times \frac{|\varepsilon_{\text{top}}|}{|\varepsilon_{\text{top}}| + |\varepsilon_{\text{bot}}|}$$

(10)

where NA is the neutral axis measured from the bottom of the beam in inches, and $\varepsilon_{\text{top}}$ and $\varepsilon_{\text{bot}}$ are the top and bottom strains, respectively. The neutral axis of the beam estimated based on the beam’s dimensions is 21.668 inches from the bottom. Using the data recorded from the static loadings the average neutral axis locations for Beams 1, 3 and 5 were calculated in Table 6. Using the six back-to-back position results, the neutral axis locations were also calculated for Beams 1, 3, and 5 in Figure 35.
Beam Number & Average Midspan Neutral Axis Location

<table>
<thead>
<tr>
<th>Beam Number</th>
<th>Table 6. Average midspan neutral axis locations (Measured from the bottom of the beam)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 and 3</td>
<td>22.8 in</td>
</tr>
<tr>
<td>5</td>
<td>19.5 in</td>
</tr>
</tbody>
</table>

Figure 35. Neutral axis location under back-to-back loading.

In the preliminary analysis, using the moment of inertia (92182 in$^4$) and the location of the neutral axis (21.668 in) in Figure 4, the bottom section modulus of the beams was estimated at 4254 in$^3$. Using the neutral axis locations in Table 6, new test-based section properties were obtained for Beams 1, 3, and 5 as shown in Table 7. From this table it appears that Beam 5 is about 10 percent weaker than the other two beams, which was not apparent from the previous results. Since the beams are similar in geometry, deterioration of the top part of the beam would be a likely explanation of the reduced stiffness and lowered neutral axis location.

<table>
<thead>
<tr>
<th>Beam Number</th>
<th>$y_{bot}$</th>
<th>$y_{top}$</th>
<th>I</th>
<th>$S_{bot}$</th>
<th>$S_{top}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 and 3</td>
<td>22.8 in</td>
<td></td>
<td>96048</td>
<td>4240</td>
<td>7196</td>
</tr>
<tr>
<td>5</td>
<td>19.5 in</td>
<td></td>
<td>76332</td>
<td>3835</td>
<td>4743</td>
</tr>
</tbody>
</table>

Table 7. Test-based beam section properties.
Utilizing the strains recorded during the testing and the section modulus $S_{bot}$ from the above table, together with the elastic modulus for concrete ($E = 4030$ ksi for $f_c' = 5000$ psi), the actual moment on a beam can be determined. For example, for the highest recorded strain during the testing (117 $\mu$e recorded during Phase 4 on Beam 1) the actual moment on Beam 1 was about 164 kip-ft. This moment is about 50 percent of that estimated in Table 3.

C. END FIXITY INVESTIGATION

This was first investigated by studying how the recorded strains on each of the beams related to the truck moments on the structure assuming simply supported and fixed end conditions. The results of this investigation in Appendix B show that the beams were acting more like being fixed at their ends than being simply supported. This encouraged further investigation to determine the level of fixity at the beam’s ends based on satisfying the following statics equation

$$M_{SS} = M_c + \frac{M_N + M_S}{2}$$

(11)

Where $M_{SS}$ is the simply supported midspan moment, $M_c$ is the midspan moment, $M_N$ and $M_S$ are the north and south end moments. For any of the beams, the simply supported midspan moment can be obtained and compared to the simply supported truck moment. A one-to-one relationship between the two moments would generally result if the midspan and end moments were accurately determined. Usually, midspan moment can be obtained with great accuracy, however, end moments cannot be easily obtained because the end gage locations and the actual point of fixity may not coincide. Therefore, it would be necessary to adjust the end moments as shown in Figures 36 to 41. The adjustment in these figures was performed by distributing the moment difference (midspan simply supported truck moment - $M_{SS}$) to the ends in the proportion of their measured strains. For example, if the south and north end strains were 5 and 10 $\mu$e, 33 and 67 percent of the adjustment moment would, respectively, be distributed to the south and north ends of the beam.

Figures 42 and 43, respectively, show the adjusted moments plotted against the completely fixed moments, respectively, for the north and south ends. From these plots it can be concluded that the bridge experienced 90 and 76 percent fixity at the south and north abutments, respectively.
Figure 36. Beam 1 simply supported truck moment versus statics test-based moment.

Figure 37. Beam 2 simply supported truck moment versus statics test-based moment.
Figure 38. Beam 3 simply supported truck moment versus statics test-based moment.

Figure 39. Beam 4 simply supported truck moment versus statics test-based moment.
Figure 40. Beam 5 simply supported truck moment versus statics test-based moment.

Figure 41. All Beams simply supported truck moment versus statics test-based moment.
Figure 42. North end all Beams moment: Adjusted versus completely fixed.

Figure 43. South end all Beams moment: Adjusted versus completely fixed.
V. LOAD RATING ANALYSIS

A. SUMMARY OF TEST FINDINGS

A test-based procedure for load rating of the bridge structure is proposed in this chapter. Before this procedure is introduced, the following summary of the important findings, on the bridge behavior revealed by the load tests is presented.

1. Transverse load distribution based on Phases 1 to 4 tests indicated that:

   a. Beams 1 and 5, fascia beams, carried over 40 percent of the load when a truck wheel line was applied directly on top of the beams during Paths 1 and 5 crossings. The highest theoretical midspan simply supported moments on those beams were about 365x0.42 = 153 kip-ft under Phase 1 and 763x0.47 = 358 kip-ft under Phase 4 testing. The higher of the two moments is about 62 percent of the moment estimated to crack the beam in Table 1 (573 kip-ft). These two moments may be compared with those estimated in Table 3 as 157 and 328 kip-ft, which were calculated assuming simply supported end conditions and the AASHTO distribution factors.

   b. From Phase 2 results, it can be seen that when both the upstream and downstream lanes are similarly loaded, each of the fascia beams is expected to carry about 40 percent the total load on the structure. Assuming that it was feasible to repeat the 2 15-ton Back-to-Back test in both lanes simultaneously, this would have resulted in a moment of about 763 x 0.59 = 450 kip-ft in the fascia beams. This moment would still be lower than the 573 kip-ft moment estimated to cause beam cracking.

   c. Although the longitudinal joints between Beams 1 and 2, and Beams 4 and 5 showed clear signs of deterioration and leaking, the joints are still effective in transmitting traffic loads.

2. From the transverse load distribution analysis, it was clear that under normal traffic the fascia beams would be stressed most.

3. The section modulus determined based on the neutral axis gage readings was very close to that estimated theoretically (4254 in.³) using the beam geometry.

4. The beams section modulii calculated assuming fixity at the abutments were much closer to those calculated based on actual strain measurements, indicating a high degree of fixity at the abutments. Although the bridge was most likely designed as a simply supported structure, the beams supports at the abutments were in fact almost fully restrained (76 and 90 percent fixity estimated at the North and south ends of the bridge). This end restraint is attributed to the type of construction (the ends of the beams are cast into a large block at the abutment), and thus can be deemed reliable in the proposed load rating analysis.
B. PROPOSED TEST-BASED LOAD RATING

The proposed test-based load factor rating analysis is summarized in Table 8. Based on the findings presented in the previous section, the following decisions were made:

1. To calculate live load moment using the AASHTO distribution factor (S/5, giving an equivalent percentage of about 43 percent) instead of that determined based on the test results (about 47 percent). The higher test distribution percentage was calculated by combining crossings to derive a percentage based on two lanes being loaded. The narrow bridge width does not allow for the presence of more than one large truck on the bridge. The AASHTO distribution factor agrees well with the field measured distribution percentage with one lane loaded (Beam 1 in Figures 19 and 26) \((1,2,3,4)\).

2. To calculate stresses, based on a typical section modulus the theoretical section modulus, which is close to that based on the test results.

3. To conservatively ignore fixity in the rating analysis because there is uncertainty in the bridge design and fixity assumptions may not remain valid at higher loads.

The results in Tables 8 and 9 show the inventory rating factors, operating rating factors and tonnage calculated using the AASHTO LFD method, including impact \((1,2,3,4)\). (See Appendix C for details of the rating analysis). Note that HS-20 and H-20 rating trucks were assumed in the analysis. Each loading scenario considers the two possibilities of the bridge being designed using “High P/T” and “Low P/T” (High and Low Post-tensioning forces as explained earlier).

From Table 9, the governing inventory rating is about 24.6 tons and the governing operating rating is about 40.8 tons.

<table>
<thead>
<tr>
<th>Rating</th>
<th>Parameter</th>
<th>HS-20</th>
<th>H-20</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>High P/T</td>
<td>Low P/T</td>
</tr>
<tr>
<td>Inventory</td>
<td>Concrete in Tension</td>
<td>1.35</td>
<td>1.31</td>
</tr>
<tr>
<td></td>
<td>Concrete in Compression</td>
<td>2.21</td>
<td>3.31</td>
</tr>
<tr>
<td></td>
<td>Concrete in Compression</td>
<td>1.70</td>
<td>2.25</td>
</tr>
<tr>
<td></td>
<td>Flexure Strength</td>
<td>1.12</td>
<td>1.44</td>
</tr>
<tr>
<td>Operating</td>
<td>Flexure Strength</td>
<td>1.88</td>
<td>2.41</td>
</tr>
</tbody>
</table>

Table 8. Test-based LFD load rating factor results including impact.

<table>
<thead>
<tr>
<th>Rating</th>
<th>Parameter</th>
<th>HS-20</th>
<th>H-20</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>High P/T</td>
<td>Low P/T</td>
</tr>
<tr>
<td>Inventory</td>
<td>Concrete in Tension</td>
<td>48.6</td>
<td>47.2</td>
</tr>
<tr>
<td></td>
<td>Concrete in Compression</td>
<td>79.6</td>
<td>119.2</td>
</tr>
<tr>
<td></td>
<td>Concrete in Compression</td>
<td>61.2</td>
<td>81.0</td>
</tr>
<tr>
<td></td>
<td>Flexure Strength</td>
<td>40.3</td>
<td>51.8</td>
</tr>
<tr>
<td>Operating</td>
<td>Flexure Strength</td>
<td>67.7</td>
<td>86.8</td>
</tr>
</tbody>
</table>

Table 9. Test-based LFD load rating tonnage results.
VI. CONCLUSIONS AND RECOMMENDATIONS

The major test and analysis results findings presented in this report can be summarized in the following:

1. Load distribution on the structure was excellent in comparison with that of the estimated AASHTO equation.

2. The theoretical beam section modulus was verified using results based on strain readings from gages mounted to determine neutral axis locations and again from moment versus strain results.

3. The testing revealed that the structure was about 90 percent fixed at the South end and about 76 percent fixed at the north end.

4. AASHTO LFD load rating was performed using a conservative approach, assuming the structure to be simply supported, the actual AASHTO distribution factor, and the theoretical section modulus, which was very close to that based on the test results.

5. The recommended H inventory and operating ratings are 24.6 and 40.8 tons, respectively.

6. The recommended HS inventory and operating ratings are 40.3 and 67.7 tons, respectively

Based on these test results, a 15-ton school bus does not pose any hazard of damaging the structure.
ACKNOWLEDGEMENTS

Instrumentation and data collection for this project was conducted by George Schongar and Harry Greenberg of the Transportation Research and Development Bureau. The coordination and assistance of Stan Wase and the Greene County Highway Department staff is gratefully acknowledged. The County is also acknowledged for providing the test vehicles and controlling traffic during the testing. The authors are thankful for the valuable review comments and assistance they received from Scott Lagace and Brian McCaffrey of the Bridge Safety Assurance Unit of the NYSDOT Structures Division.
REFERENCES


2. “New York State Department of Transportation Standard Specifications for Highway Bridges,” and provisions in effect as of April 2001, New York State Department of Transportation.


APPENDIX A

H-20 SOLUTION CHECKS:

1. Initial and Final Stresses at the Lower P/T:

\[ e_{c_{\text{Low}}} = 18.68 \text{ in} \]

\[ P_1 := \frac{1}{P_3(e_{c_{\text{Low}}})} \]

\[ P_1 = 614.328 \text{ kip} \]

\[ f_{bi_{\text{-}}} := \frac{P_1}{A} + \frac{P_1 \cdot e_{c_{\text{Low}}}}{S_b} - \frac{M_{DL}}{S_b} \]

Comp. Bot. \( f_{bi_{\text{-}}} = 2.176 \text{ ksi} \)

\[ f_{bi_{\text{-}}} = 2.338 \text{ ksi} \]

\[ f_{ti_{\text{-}}} := \frac{P_1}{A} + \frac{P_1 \cdot e_{c_{\text{Low}}}}{S_t} - \frac{M_{DL}}{S_t} \]

Tension Top \( f_{ti_{\text{-}}} = -0.141 \text{ ksi} \)

\[ f_{ti_{\text{-}}} = 0.196 \text{ ksi} \]

\[ f_{bf_{\text{-}}} := \frac{\kappa \cdot P_1}{A} - \frac{\kappa \cdot P_1 \cdot e_{c_{\text{Low}}}}{S_b} + \frac{M_{Tot}}{S_b} \]

Tension Bot. \( f_{bf_{\text{-}}} = 0 \text{ ksi} \)

\[ f_{bf_{\text{-}}} = 0 \text{ ksi} \]

\[ f_{tf_{\text{-}}} := \frac{\kappa \cdot P_1}{A} - \frac{\kappa \cdot P_1 \cdot e_{c_{\text{Low}}}}{S_t} + \frac{M_{Tot}}{S_t} \]

Comp. Top \( f_{tf_{\text{-}}} = 1.108 \text{ ksi} \)

\[ f_{tf_{\text{-}}} = 2 \text{ ksi} \]

2. Initial and Final Stresses at the Higher P/T:

\[ e_{c_{\text{High}}} = 8.0 \text{ in} \]

\[ P_1 := \frac{1}{P_3(e_{c_{\text{High}}})} \]

\[ P_1 = 1.111 \times 10^3 \text{ kip} \]

\[ f_{bi_{\text{-}}} := \frac{P_1}{A} + \frac{P_1 \cdot e_{c_{\text{High}}}}{S_b} - \frac{M_{DL}}{S_b} \]

Comp. Bot. \( f_{bi_{\text{-}}} = 2.338 \text{ ksi} \)

\[ f_{bi_{\text{-}}} = 2.338 \text{ ksi} \]

\[ f_{ti_{\text{-}}} := \frac{P_1}{A} + \frac{P_1 \cdot e_{c_{\text{High}}}}{S_t} - \frac{M_{DL}}{S_t} \]

Tension Top \( f_{ti_{\text{-}}} = -1.312 \text{ ksi} \)

\[ f_{ti_{\text{-}}} = 0.196 \text{ ksi} \]

\[ f_{bf_{\text{-}}} := \frac{\kappa \cdot P_1}{A} - \frac{\kappa \cdot P_1 \cdot e_{c_{\text{High}}}}{S_b} + \frac{M_{Tot}}{S_b} \]

Tension Bot. \( f_{bf_{\text{-}}} = -0.113 \text{ ksi} \)

\[ f_{bf_{\text{-}}} = 0 \text{ ksi} \]

\[ f_{tf_{\text{-}}} := \frac{\kappa \cdot P_1}{A} - \frac{\kappa \cdot P_1 \cdot e_{c_{\text{High}}}}{S_t} + \frac{M_{Tot}}{S_t} \]

Comp. Top \( f_{tf_{\text{-}}} = 1.929 \text{ ksi} \)

\[ f_{tf_{\text{-}}} = 2 \text{ ksi} \]
HS-20  SOLUTION CHECKS:

1. Initial and Final Stresses at the Lower P/T:

   \[ e_{\text{Low}} = 18.68 \text{ in} \]
   \[ P_{1} = \frac{1}{P_{3}(e_{\text{Low}})} \]
   \[ P_{1} = 716.473 \text{ kip} \]

   \[ f_{\text{bi}} = \frac{P_{1} + P_{1} e_{\text{Low}}}{A} - \frac{M_{DL}}{S_{b}} \quad \text{Comp. Bot.} \]
   \[ f_{\text{bi}} = 2.783 \text{ ksi} \quad f_{\text{bi}} = 2.888 \text{ ksi} \]

   \[ f_{\text{ti}} = -\frac{P_{1} + P_{1} e_{\text{Low}}}{A} - \frac{M_{DL}}{S_{t}} \quad \text{Tension Top} \]
   \[ f_{\text{ti}} = -1.442 \times 10^{-3} \text{ ksi} \quad f_{\text{ti}} = 0.217 \text{ ksi} \]

   \[ f_{\text{bf}} = -\frac{\kappa P_{1}}{A} - \frac{\kappa P_{1} e_{\text{Low}}}{S_{b}} + \frac{M_{Total}}{S_{b}} \quad \text{Tension Bot.} \]
   \[ f_{\text{bf}} = 0 \text{ ksi} \quad f_{\text{bf}} = 0 \text{ ksi} \]

   \[ f_{\text{tf}} = -\frac{\kappa P_{1}}{A} - \frac{\kappa P_{1} e_{\text{Low}}}{S_{t}} + \frac{M_{Total}}{S_{t}} \quad \text{Comp. Top} \]
   \[ f_{\text{tf}} = 1.292 \text{ ksi} \quad f_{\text{tf}} = 2.4 \text{ ksi} \]

2. Initial and Final Stresses at the Higher P/T:

   \[ e_{\text{High}} = 7.10 \text{ in} \]
   \[ P_{1} = \frac{1}{P_{1}(e_{\text{High}})} \]
   \[ P_{1} = 1.355 \times 10^{3} \text{ kip} \]

   \[ f_{\text{bi}} = \frac{P_{1} + P_{1} e_{\text{High}}}{A} - \frac{M_{DL}}{S_{b}} \quad \text{Comp. Bot.} \]
   \[ f_{\text{bi}} = 2.887 \text{ ksi} \quad f_{\text{bi}} = 2.888 \text{ ksi} \]

   \[ f_{\text{ti}} = -\frac{P_{1} + P_{1} e_{\text{High}}}{A} - \frac{M_{DL}}{S_{t}} \quad \text{Tension Top} \]
   \[ f_{\text{ti}} = -1.576 \text{ ksi} \quad f_{\text{ti}} = 0.217 \text{ ksi} \]

   \[ f_{\text{bf}} = -\frac{\kappa P_{1}}{A} - \frac{\kappa P_{1} e_{\text{High}}}{S_{b}} + \frac{M_{Total}}{S_{b}} \quad \text{Tension Bot.} \]
   \[ f_{\text{bf}} = -0.073 \text{ ksi} \quad f_{\text{bf}} = 0 \text{ ksi} \]

   \[ f_{\text{tf}} = -\frac{\kappa P_{1}}{A} - \frac{\kappa P_{1} e_{\text{High}}}{S_{t}} + \frac{M_{Total}}{S_{t}} \quad \text{Comp. Top} \]
   \[ f_{\text{tf}} = 2.396 \text{ ksi} \quad f_{\text{tf}} = 2.4 \text{ ksi} \]
APPENDIX B

Using a finite element model of a beam, simply supported and fixed-fixed total midspan moments were calculated for each static loading (Phases 1, 2, and 4 results). A moment on a beam was determined using the total midspan moment calculated for each load case and the beam’s load distribution percentage. The simply supported and fixed-fixed individual beam moments were then plotted against the measured strains as shown in Figures B1 to B5. These linear plots provide slopes equal to the beam moment over strain. By manipulating the equation:

\[
\frac{\text{Moment}}{S_s} = \varepsilon \times E \quad \text{to} \quad S_s = \frac{\text{Moment}}{\varepsilon \times E}
\]

where \( S_s \) is the slope of the plotted line. \( \text{(12)} \)

Using an estimated modulus of elasticity (E) of 4030 ksi and the slope of the line known in the figure, the section modulus for each beam can be obtained. The section modulus calculated based on the slopes of the lines for simply supported and fixed end conditions in the figures are summarized in Table B1. These values can be compared with those estimated based on the neutral axis calculations in Table 3 (about 4000 in.\(^3\)). From this comparison it is clear that the beams were acting more like being fixed at their ends than being simply supported. The impact of this is very significant; for example, the highest strain measured during the testing was 117 \( \mu \varepsilon \) would have been nearly doubled (230 \( \mu \varepsilon \)) if the beams were not experiencing end fixity.

<table>
<thead>
<tr>
<th>Beam Number</th>
<th>Simply Supported Section Modulus (in(^3))</th>
<th>Fixed-Fixed Section Modulus (in(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9071</td>
<td>3373</td>
</tr>
<tr>
<td>2</td>
<td>9704</td>
<td>3596</td>
</tr>
<tr>
<td>3</td>
<td>10090</td>
<td>3811</td>
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<td>4</td>
<td>10496</td>
<td>3918</td>
</tr>
<tr>
<td>5</td>
<td>10785</td>
<td>4032</td>
</tr>
</tbody>
</table>

Table B1. Simply supported and fixed-fixed beam section modulii.
Figure B1. Beam 1 moment versus strain.

Figure B2. Beam 2 moment versus strain.
Figure B3. Beam 3 moment versus strain.

Figure B4. Beam 4 moment versus strain.
Figure B5. Beam 5 moment versus strain.
APPENDIX C

Load Rating Analysis for HS-20 High P/T:

Inventory Rating:

\[ f_c = 6 \text{ ksi} \quad f_{ys} = 192 \text{ ksi} \]
\[ M_u = 2.653 \times 10^4 \text{ kip-in} \quad M_{DL} = 6.264 \times 10^3 \text{ kip-in} \quad M_{LL} = 6.431 \times 10^3 \text{ kip-in} \]
\[ P_e = 950.113 \text{ kip} \quad \epsilon_c = 7.1 \text{ in} \]
\[ A = 646.1 \text{ in}^2 \quad S_b = 4.254 \times 10^3 \text{ in}^3 \quad S_t = 6.432 \times 10^3 \text{ in}^3 \]
\[ F_d := \frac{M_{DL}}{S_b} \quad F_p := P_e \left( \frac{1}{A} + \frac{\epsilon_c}{S_b} \right) \quad F_l := \frac{M_{LL}}{S_b} \]

\[ RF_{Inv} := \frac{6 \cdot f_c \frac{1000}{1000} + (\epsilon_c + F_p)}{F_l} \]
\[ RF_{Inv} = 1.355 \quad \text{Concrete in Tension (1)} \]

\[ F_d := \frac{M_{DL}}{S_t} \quad F_p := P_e \left( \frac{1}{A} - \frac{\epsilon_c}{S_t} \right) \quad F_l := \frac{M_{LL}}{S_t} \]

\[ RF_{Inv} := \frac{0.6 \cdot f_c - (F_d + F_p)}{F_l} \]
\[ RF_{Inv} = 2.205 \quad \text{Concrete in Compression (2)} \]

\[ F_d := \frac{M_{DL}}{S_t} \quad F_p := P_e \left( \frac{1}{A} - \frac{\epsilon_c}{S_t} \right) \quad F_l := \frac{M_{LL}}{S_t} \]

\[ RF_{Inv} := \frac{0.4 \cdot f_c - 0.5(f_d + F_p)}{F_l} \]
\[ RF_{Inv} = 1.703 \quad \text{Concrete in Compression (3)} \]

\[ F_d := \frac{M_{DL}}{S_b} \quad F_p := P_e \left( \frac{1}{A} + \frac{\epsilon_c}{S_b} \right) \quad F_l := \frac{M_{LL}}{S_b} \]

\[ RF_{Inv} := \frac{0.8 \cdot f_{ys} - (F_d + F_p)}{F_l} \]
\[ RF_{Inv} = 98.611 \quad \text{Prestressing Steel in tension (4)} \]

\[ RF_{Inv} := \frac{0.9 \cdot M_u - (1.3 \cdot M_{DL})}{2.17 \cdot M_{LL}} \]
\[ RF_{Inv} = 1.127 \quad \text{Flexure Strength (5)} \]

Operating Rating:

\[ RF_{Opr} := \frac{0.9 \cdot M_u - (1.3 \cdot M_{DL})}{1.3 \cdot M_{LL}} \]
\[ RF_{Opr} = 1.882 \quad \text{Flexure Strength (1)} \]
Load Rating Analysis for HS-20 Low P/T:

**Inventory Rating:**

\[ f_c = 6 \text{ ksi} \quad f_{ys} = 192 \text{ ksi} \]

\[ M_u = 3.14 \times 10^4 \text{ kip} - \text{in} \quad M_{DL} = 6.264 \times 10^3 \text{ kip} - \text{in} \quad M_{LL} = 6.431 \times 10^3 \text{ kip} - \text{in} \]

\[ P_e = 502.403 \text{ kip} \quad ec = 18.68 \text{ in} \]

\[ A = 646.1 \text{ in}^2 \quad S_b = 4.254 \times 10^3 \text{ in}^3 \quad S_t = 6.432 \times 10^3 \text{ in}^3 \]

\[ F_d := \frac{M_{DL}}{S_b} \quad F_P := P_e \left( \frac{1 + ec}{S_b} \right) \quad F_1 := \frac{M_{LL}}{S_b} \]

\[ RF_{Inv} := \frac{6 \cdot \sqrt{f_c \cdot 1000}}{1000} + \left( -F_d + F_P \right) \]

\[ RF_{Inv} = 1.307 \quad \text{Concrete in Tension (1)} \]

\[ F_d := \frac{M_{DL}}{S_t} \quad F_P := P_e \left( \frac{1 - ec}{S_t} \right) \quad F_1 := \frac{M_{LL}}{S_t} \]

\[ RF_{Inv} := \frac{0.6 \cdot f_c - (F_d + F_P)}{F_1} \]

\[ RF_{Inv} = 3.308 \quad \text{Concrete in Compression (2)} \]

\[ F_d := \frac{M_{DL}}{S_t} \quad F_P := P_e \left( \frac{1 - ec}{S_t} \right) \quad F_1 := \frac{M_{LL}}{S_t} \]

\[ RF_{Inv} := \frac{0.4 \cdot f_c - 0.5(F_d + F_P)}{F_1} \]

\[ RF_{Inv} = 2.254 \quad \text{Concrete in Compression (3)} \]

\[ F_d := \frac{M_{DL}}{S_b} \quad F_P := P_e \left( \frac{1 + ec}{S_b} \right) \quad F_1 := \frac{M_{LL}}{S_b} \]

\[ RF_{Inv} := \frac{0.8 \cdot f_{ys} - (F_d + F_P)}{F_1} \]

\[ RF_{Inv} = 98.659 \quad \text{Prestressing Steel in tension (4)} \]

\[ RF_{Inv} := \frac{0.9 \cdot M_u - (1.3 \cdot M_{DL})}{2.17 \cdot M_{LL}} \]

\[ RF_{Inv} = 1.441 \quad \text{Flexure Strength (5)} \]

**Operating Rating:**

\[ RF_{Opr} := \frac{0.9 \cdot M_u - (1.3 \cdot M_{DL})}{1.3 \cdot M_{LL}} \]

\[ RF_{Opr} = 2.406 \quad \text{Flexure Strength (1)} \]
Load Rating Analysis for H-20 High P/T:

Inventory Rating:

\[ f_c = 5 \text{ ksi} \quad f_{ys} = 192 \text{ ksi} \]
\[ M_u = 2.268 \times 10^4 \text{ kip in} \quad M_{DL} = 6.264 \times 10^3 \text{ kip in} \quad M_{LL} = 4.619 \times 10^3 \text{ kip in} \]
\[ P_e = 779.023 \text{ kip} \quad e_c = 7.75 \text{ in} \]
\[ A = 646.1 \text{ in}^2 \quad S_b = 4.254 \times 10^{-3} \text{ in}^3 \quad S_t = 6.432 \times 10^{-3} \text{ in}^3 \]

\[ F_d := \frac{M_{DL}}{S_b} \quad F_p := P_e \left( \frac{1}{A} + \frac{e_c}{S_b} \right) \quad F_1 := \frac{M_{LL}}{S_b} \]

\[ \text{RF}_{Inv} := \frac{6 \cdot \frac{f_c}{1000} + \left( -F_d + F_p \right)}{F_1} \quad \text{RF}_{Inv} = 1.452 \quad \text{Concrete in Tension (1)} \]

\[ F_d := \frac{M_{DL}}{S_t} \quad F_p := P_e \left( \frac{1}{A} - \frac{e_c}{S_t} \right) \quad F_1 := \frac{M_{LL}}{S_t} \]

\[ \text{RF}_{Inv} := \frac{0.6 \cdot f_c - (F_d + F_p)}{F_1} \quad \text{RF}_{Inv} = 2.45 \quad \text{Concrete in Compression (2)} \]

\[ F_d := \frac{M_{DL}}{S_t} \quad F_p := P_e \left( \frac{1}{A} - \frac{e_c}{S_t} \right) \quad F_1 := \frac{M_{LL}}{S_t} \]

\[ \text{RF}_{Inv} := \frac{0.4 \cdot f_c - 0.5(F_d + F_p)}{F_1} \quad \text{RF}_{Inv} = 1.921 \quad \text{Concrete in Compression (3)} \]

\[ F_d := \frac{M_{DL}}{S_b} \quad F_p := P_e \left( \frac{1}{A} + \frac{e_c}{S_b} \right) \quad F_1 := \frac{M_{LL}}{S_b} \]

\[ \text{RF}_{Inv} := \frac{0.8 \cdot f_{ys} - (F_d + F_p)}{F_1} \quad \text{RF}_{Inv} = 137.695 \quad \text{Prestressing Steel in tension (4)} \]

\[ \text{RF}_{Inv} := \frac{0.9 \cdot M_u - (1.3 \cdot M_{DL})}{2.17 \cdot M_{LL}} \quad \text{RF}_{Inv} = 1.225 \quad \text{Flexure Strength (5)} \]

Operating Rating:

\[ \text{RF}_{Opr} := \frac{0.9 \cdot M_u - (1.3 \cdot M_{DL})}{1.3 \cdot M_{LL}} \quad \text{RF}_{Opr} = 2.044 \quad \text{Flexure Strength (1)} \]
Load Rating Analysis for H-20 Low P/T:

Inventory Rating:

\[ f_c = 5 \text{ ksi}; \quad f_{ys} = 192 \text{ ksi} \]

\[ M_u = 2.616 \times 10^4 \text{ kip in}; \quad M_{DL} = 6.264 \times 10^3 \text{ kip in}; \quad M_{LL} = 4.619 \times 10^3 \text{ kip in} \]

\[ P_e = 430.811 \text{ kip}; \quad e_c = 18.68 \text{ in} \]

\[ A = 646.1 \text{ in}^2; \quad S_b = 4.254 \times 10^3 \text{ in}^3; \quad S_t = 6.432 \times 10^3 \text{ in}^3 \]

\[ F_d := \frac{M_{DL}}{S_b}; \quad F_P := P_e \left( \frac{1 + ec}{A} \right); \quad F_I := \frac{M_{LL}}{S_b} \]

\[ RF_{Inv} := \frac{6 \cdot \frac{P_e \cdot 1000}{1000} - (F_d + F_P)}{F_I}; \quad RF_{Inv} = 1.391 \quad \text{Concrete in Tension (1)} \]

\[ RF_{Inv} := \frac{0.6 \cdot f_c - (F_d + F_P)}{F_I}; \quad RF_{Inv} = 3.635 \quad \text{Concrete in Compression (2)} \]

\[ RF_{Inv} := \frac{0.4 \cdot f_c - 0.5(F_d + F_P)}{F_I}; \quad RF_{Inv} = 2.514 \quad \text{Concrete in Compression (3)} \]

\[ RF_{Inv} := \frac{0.8 \cdot f_{ys} - (F_d + F_P)}{F_I}; \quad RF_{Inv} = 137.756 \quad \text{Prestressing Steel in tension (4)} \]

\[ RF_{Inv} := \frac{0.9 \cdot M_u - (1.3 \cdot M_{DL})}{2.17 \cdot M_{LL}}; \quad RF_{Inv} = 1.537 \quad \text{Flexure Strength (5)} \]

Operating Rating:

\[ RF_{Opr} := \frac{0.9 \cdot M_u - (1.3 \cdot M_{DL})}{1.3 \cdot M_{LL}}; \quad RF_{Opr} = 2.565 \quad \text{Flexure Strength (1)} \]