BRIDGE LOAD RATING UTILIZING EXPERIMENTAL DATA AND FINITE ELEMENT MODELING

by

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ABSTRACT

Nearly one-quarter of Alabama’s bridges are deemed structurally deficient or functionally obsolete. An additional seven percent of Alabama’s bridges were posted bridges in 2010. (Federal Highway Administration, 2011) Accurate bridge load rating can potentially reduce, and even remove, bridge postings throughout the state. Analytical structural bridge models were used to define new load ratings for ALDOT Bridges 005248, 005318, and 012296.

With past methodology, Engineers’ ratings tend to be inaccurate. This is due to the indefinite information in regards to actual traffic loading on bridges. This inaccuracy can lead to over-estimates of bridge safety or on the contrary, excessive conservatism in repairs. Gaining further knowledge on the actual behavior of bridges with the help of analytical models can help reduce inaccuracy in calculation.

The University Transportation Center for Alabama (UTCA) tasked The University of Alabama at Birmingham (UAB) to verify analytical models from which accurate load rating could be obtained. ALDOT performed load testing and calculated ratings for bridges 005248 and 005318. The UAB team assisted ALDOT in gathering strain data for bridge 012296 via the Bridge Weigh-In-Motion (BWIM) technique. Bridge models for 005248 and 005318 proved capable of accurate load rating per data comparison as presented, and 012296 was load rated using Finite Element Modeling.
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1.0 INTRODUCTION

Field data provided was used to calibrate and modify structural models in SAP2000 for representation of actual bridge behavior. The resulting models for the three bridges considered were used for load rating. After appropriate calibration, they can also be used for permit load or overload stress predictions. Additionally, lateral load distribution can be predicted using the calibrated structural models.

In reality, bridges are rarely statically determinate. Complex behavior seen by the reinforced concrete slabs and beam-slab bridges prevents traditional beam analysis and load distribution factoring approaches from accuracy. Geometry and secondary stiffening effects mainly control the behavior of bridges under traffic load. Skew and beam spacing-to-span length ratios largely influence load distribution along the bridge. (Yost, Schulz, & Commander, 2005) End restraints greatly influence the moment seen by the bridge girders, and these restraints will be controlled to mirror actual bridge behavior.

There were three bridges modeled and analyzed for this project. The first bridge modeled was ALDOT Bridge 005248, which crosses the Tallapoosa River and connects SR-49 in Dadeville, AL. The second bridge, ALDOT Bridge 005318 connects State Road 145 and spans the Waxahatchee Creek in Shelby County, AL. The last bridge studied was Bridge 012296, located near the I-459 and SR-150 interchange (Exit 10) in Birmingham, AL. Each bridge was modeled and modified as mentioned above, and the results were recorded.
1.1 OBJECTIVE

The objective of this thesis is to prove that finite element analysis software, such as SAP2000, can be used to model bridges in the state of Alabama for load rating purposes. ALDOT currently uses software such as BRUFEM and VIRTIS to determine load-rating factors for bridges. Load rating values for bridges 005248 and 005318 have been provided along with calculation for comparison with those obtained through finite element analysis software. Once the load rating values calculated from results in SAP2000 are deemed accurate, one can safely assume that the load rating values for Bridge 012296 are precise. ALDOT will then have evidence that other bridges throughout the state can be modeled and calibrated providing an economical means for load rating and permitting. UTCA wishes to use the findings in this report to verify that analytical models can be used as a more accurate load rating technique, potentially reducing or removing bridge postings in Alabama.

1.2 THESIS ORGANIZATION

The thesis has been organized in a manner to effectively guide the reader through the load rating process and findings for each bridge. First, load rating methodology and concepts are explained. Then each bridge model is presented with calibration results versus experimental data provided by ALDOT. All moments and strains obtained through SAP2000 are provided in conjunction with the experimental values via graphical representation. After appropriate calibration evidence is delivered, load-rating values are calculated and compared with ALDOT’s results.
1.3 TASK DESCRIPTION

1.3.1 Task 1 – Data Examination

Visual examination of experimental data (via BWIM and ALDOT) is the first step in qualitative assessment of the structure. Strain data values provide the engineer with a feel for member cross-section definition and response behavior. Other preliminary tasks include noting the peak strain values and girder member properties. With this information, a reasonable initial model can be formed with appropriate parameters and boundary conditions. This step is imperative for an initial evaluation of the structure’s performance and crucial for effective completion of model calibration.

1.3.2 Task 2 – Generate Bridge Model

The finite element analysis program used was SAP2000. Computers and Structures, Inc. provides SAP as a means for structural design and analysis for a variety of buildings, bridges, and other structures. Distribution factors when loading the bridge will not be used in bridge calibration, because the models represent existing structures. The use of distribution factors is reasonable in design, because they provide safety measures for all conditions. (American Association of State Highway and Transportation Officials, 1996) Since the bridges are existing structures, more sophisticated loading measures were used to result in more accurate bridge behavior and load rating factors.

1.3.3 Task 3 – Verification with Experimental Data

Experimental data was gathered from two sources: BWIM data performed by UAB and ALDOT provided values. Values obtained from the SAP model are to be compared with this experimental data to provide visual and statistical information.
1.3.4 Task 4 – Application of Rating Loads

After appropriate calibration of bridge models with experimental data, wheel loads for HS20-44 trucks are applied to the model. Results from loading the model are used to compute the Rating Factors. An accurate model will provide the engineer with accurate bridge response behavior when standard design loads, rating vehicles, or permit loads are applied to the structure. The developed structural model will be used for the following:

- Providing a more accurate structural model
- Be used for load rating and permitting
- Prediction of Lateral Load Distribution
- Enabling of funds to target truly unsafe bridges
- Prove beneficial to maintenance efforts and bridge design

An illustrative process of the approach for bridge modeling and assessment has been provided below.

![Diagram of Integrated Approach to Project Tasks]

Figure 1: Illustration of Integrated Approach to Project Tasks
2.0 LOAD RATING METHODOLOGY

Load rating was determined using the Standard AASHTO “HS” loading method in accordance with the 17th Edition (2002) of Standard Specifications for Highway Bridges. Lane loading was performed using an AASHTO HS20-44 truck in conformance with AASHTO Specifications. The axle loads were distributed into two point loads representing the left and right tires, respectively. The position of the resultant load of two load cases was determined. This resultant load was placed in the center of the bridge, and each lane was loaded accordingly.

The first load case evaluated included only the resultant of one truck per lane. The resultant position of two trucks spaced two feet apart was calculated and placed in the center of the bridge for the second load case. The load case that produced the maximum moment was regarded as controlling for load rating. Each bridge was loaded to result in the largest moment experienced by any girder. This moment was then used in accordance with the AASHTO Standard Specification for load rating. Both Allowable Stress Rating and Load Factor Rating were performed.

AASHTO rating truck HS20-44 axle loads and resultant forces have been included. Provided clearance dimensions for the truck were also used for determination of load positions in the model. All figures representing these locations are included below the following explanation of rating levels and methods.
2.1 RATING LEVELS

Each highway bridge is to be rated at two separate levels. These levels are distinguishable as the Inventory Rating Level (IRL) and the Operating Rating Level (ORL). The Inventory level replicates existing conditions of the bridge, such as material and section deterioration. However, IRL corresponds with initial design level of stresses, and this allows for comparison with capacity for new structures. This provides a live load deemed safe for the existing bridge for future awareness when additional loads are considered. The ORL generally describes a live load that can be considered a maximum permissible load for the bridge to experience. Allowing the bridge to experience this maximum permissible load indefinitely can significantly shorten the life of the bridge. (American Association of State Highway and Transportation Officials, 2011)

2.2 RATING METHODS

Two rating methods were considered in thesis research. These include the Allowable Stress Rating (ASR) and the Load Factor Rating (LFR). Although similar, these two rating methods have significant differences in purpose. The ASR method, also known as the working stress method utilizes actual loads on the members to determine the maximum stresses experienced by each member. These stresses must not surpass the member allowable stress after application of a safety factor. The LFR method uses different factors that are applied to account for uncertainty in load calculations. The rating is performed to ensure that the factored loading does not exceed the strength of the member.
Figure 2: Standard Axle Loads for HS20-44 Truck

Figure 3: Load Resultant Location for Single HS20-44 Truck
Figure 4: Standard Spacing Requirements for HS20-44 Truck
2.3 RATING EQUATION

A general rating equation is used for load rating via the Allowable Stress Method and the Load Factor Method. The general expression is used regardless of the method used for load rating. The general expression is as follows:

Equation 1: General Rating Factor Equation

$$RF = \frac{C - A_1D}{A_2L(1 + I)}$$

Table 1: Rating Factor Equation Coefficient Signification

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>Signification</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF</td>
<td>Rating Factor for live load carrying capacity</td>
</tr>
<tr>
<td>C</td>
<td>The capacity of the member</td>
</tr>
<tr>
<td>D</td>
<td>The dead load effect on the member</td>
</tr>
<tr>
<td>L</td>
<td>The live load effect on the member</td>
</tr>
<tr>
<td>I</td>
<td>The impact factor to be used with live load effect</td>
</tr>
<tr>
<td>A_1</td>
<td>Factor for dead loads (AS=1.0; LF=1.3)</td>
</tr>
<tr>
<td>A_2</td>
<td>Factor for live loads (AS=1.0; LF “IRL”=2.17; LF “ORL”=1.3)</td>
</tr>
</tbody>
</table>

The rating factor is the desired value for comparison with provided information. The capacity of the member, C, is dependent on section and material properties. The dead load effect should be correspondent with the existing conditions of the bridge members. The live load effect can be the result for any of the desired load effects, typically axial force, shear force, bending moment, or axial, shear, or bending stress. The impact factor is in accordance with AASHTO Specifications (1.3 for 005248 and 012296, 1.2553 for 005318).
3.0 BRIDGE ANALYSIS

3.1 ALDOT BRIDGE 005248

3.1.1 Location of Bridge

Bridge 005248 is located in Horseshoe Bend National Historic Military Park near Dadeville, AL. The bridge spans over the Tallapoosa River, connecting strips of State Road 49. The bridge is two lanes wide with both steel and concrete girders. For analysis purposes, the end span with reinforced concrete girders was modeled. The bridge was constructed in 1955 according to the National Bridge Inventory Database (NBI).
3.1.2 Bridge Information

**Bridge Photos**

Figure 6: Photo Along Span for Bridge 005248

Figure 7: Underside View of Bridge 005248
Elevations and Cross Sections

Figure 8: Elevation of 005248 from Original Drawings

Figure 9: Cross Section of 005248 from Original Drawings
Figure 10: Cross Section of Bridge 005248 in AutoCAD

Figure 11: Test Span Girder Section for 005248 in AutoCAD
Sensor Placement

Sensor placement locations were used to determine at which point along the girder moment information should be gathered to compare with experimental results. Moment values at this exact location were converted to strain and compared with field values.

Figure 12: Sensor Location for 005248 Relative to Supports
Figure 13: Sensor Layout Plan View for 005248
3.1.3 Qualitative Assessment of Model

ALDOT had performed an analysis of Bridge 005248 in 2006 with Virtis and BRUFEM. In their analysis, two 3-axle trucks were used for loading on the bridge. The first two axles were spaced at 4’-9.5”, and the spacing between the middle and rear axle was 18’-5.5”. The axle loads in kips for the front, middle, and rear were 30.15, 32.075, and 22.875, respectively. These loads were used in SAP2000 for result comparison and qualitative assessment.

ALDOT performed five separate positioning cases in analysis. They can be found in the figure below. However, for modeling purposes, only sets 1, 3, and 5 were analyzed in SAP2000. Section properties for each girder and distances used in SAP2000 for loading values are tabulated below.

Table 2: Girder Section Properties for 005248

<table>
<thead>
<tr>
<th>Girder</th>
<th>c (in)</th>
<th>(I_x) (in(^4))</th>
<th>(b_f) (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20.68</td>
<td>63400</td>
<td>69.0</td>
</tr>
<tr>
<td>2</td>
<td>22.31</td>
<td>70443</td>
<td>80.0</td>
</tr>
<tr>
<td>3</td>
<td>22.31</td>
<td>70443</td>
<td>80.0</td>
</tr>
<tr>
<td>4</td>
<td>20.56</td>
<td>63400</td>
<td>69.0</td>
</tr>
</tbody>
</table>

The ALDOT calculation set includes a figure that portrays the location of the truck for each loading condition. A screenshot of these positions for each of the five sets has been included below. The distances of the point loads used in the SAP models were also tabulated for reference.
Figure 14: 005248 Truck Positioning Diagram for Analysis

Table 3: Loading Sets for 005248 Model in SAP2000

<table>
<thead>
<tr>
<th></th>
<th>Set 1</th>
<th>Set 3</th>
<th>Set 5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load Distance</td>
<td>Load Distance</td>
<td>Load Distance</td>
</tr>
<tr>
<td></td>
<td>from Top of Model (in)</td>
<td>from Top of Model (in)</td>
<td>from Top of Model (in)</td>
</tr>
<tr>
<td>Truck</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Left</td>
<td>Tire 23.75</td>
<td>Left Tire 40.25</td>
<td>Left Tire 87.25</td>
</tr>
<tr>
<td>Right</td>
<td>98.75</td>
<td>Right Tire 115.25</td>
<td>Right Tire 162.25</td>
</tr>
<tr>
<td>Truck</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Left</td>
<td>Tire 134.00</td>
<td>Left Tire 182.25</td>
<td>Left Tire 198.75</td>
</tr>
<tr>
<td>Right</td>
<td>209.00</td>
<td>Right Tire 257.25</td>
<td>Right Tire 273.75</td>
</tr>
</tbody>
</table>
Figure 15: 005248 SAP Model Strains for Load Set 1
Figure 16: 005248 SAP Model Strains for Load Set 3
Figure 17: 005248 SAP Model Strains for Load Set 5
3.1.4 Modeling of Bridge in SAP2000

The bridge was modeled using SAP2000, developed by Computers and Structures, Inc. Dimensions and section properties for the bridge were obtained from the information given in construction documents and ALDOT analysis results, and the bridge was modeled as a true grid in SAP2000. Section properties were then defined for each frame modeled. All concrete was modeled with a compressive strength of 3000psi. Grade 40 reinforcing steel was defined as rebar material for the model. Screenshots of the model in SAP2000 are included below.

Figure 18: Bridge 005248 Model - Perspective Model

Figure 19: Bridge 005248 Model - 3D Side View
Figure 20: Bridge 005248 Model - Top 2D Frame View
3.1.5 Bridge Model Analysis

*Loading Conditions*

Fourteen test loading conditions were performed with truck increments from six to four feet. The truck loadings were modeled as traveling northbound. The resulting moments were used to develop strain values based on section properties. Wheel loads were modeled as concentrated loads relative to positions for sets 1, 3, and 5.

*Results for Moments and Strains*

For each set, moments were gathered from SAP2000 and converted to strain values comparable to experimental results. Moment values for all four girders were reported, and the results were compiled into tabular form. Moments at each girder are noted in Kip-in, and the strains are without unit.

<table>
<thead>
<tr>
<th>Cond.</th>
<th>Distance Traveled (ft)</th>
<th>Moments via SAP (k-in)</th>
<th>Strains via SAP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Girder 1</td>
<td>Girder 2</td>
</tr>
<tr>
<td>Cond. 1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Cond. 2</td>
<td>177.2</td>
<td>159.5</td>
<td>143.0</td>
</tr>
<tr>
<td>Cond. 3</td>
<td>862.6</td>
<td>869.2</td>
<td>733.2</td>
</tr>
<tr>
<td>Cond. 4</td>
<td>1861.9</td>
<td>2106.3</td>
<td>1676.2</td>
</tr>
<tr>
<td>Cond. 5</td>
<td>1915.6</td>
<td>2156.8</td>
<td>1724.5</td>
</tr>
<tr>
<td>Cond. 6</td>
<td>1619.8</td>
<td>1714.4</td>
<td>1425.9</td>
</tr>
<tr>
<td>Cond. 7</td>
<td>1247.3</td>
<td>1210.5</td>
<td>1064.8</td>
</tr>
<tr>
<td>Cond. 8</td>
<td>1047.2</td>
<td>991.9</td>
<td>887.9</td>
</tr>
<tr>
<td>Cond. 9</td>
<td>985.5</td>
<td>997.3</td>
<td>857.1</td>
</tr>
<tr>
<td>Cond. 10</td>
<td>724.9</td>
<td>784.3</td>
<td>645.7</td>
</tr>
<tr>
<td>Cond. 11</td>
<td>408.2</td>
<td>394.2</td>
<td>349.0</td>
</tr>
<tr>
<td>Cond. 12</td>
<td>203.6</td>
<td>185.0</td>
<td>172.9</td>
</tr>
<tr>
<td>Cond. 13</td>
<td>46.8</td>
<td>42.0</td>
<td>40.6</td>
</tr>
<tr>
<td>Cond. 14</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>
### Table 5: Moment and Strain Values for 005248 Load Set 3

<table>
<thead>
<tr>
<th>Moments via SAP (k-in)</th>
<th>Distance Traveled</th>
<th>Strains via SAP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td>Girder 1</td>
</tr>
<tr>
<td>Cond. 1</td>
<td>0.0</td>
<td>0.00E+00</td>
</tr>
<tr>
<td>Cond. 2</td>
<td>130.7</td>
<td>1.35E-05</td>
</tr>
<tr>
<td>Cond. 3</td>
<td>656.5</td>
<td>6.79E-05</td>
</tr>
<tr>
<td>Cond. 4</td>
<td>1217.5</td>
<td>1.26E-04</td>
</tr>
<tr>
<td>Cond. 5</td>
<td>1502.1</td>
<td>1.55E-04</td>
</tr>
<tr>
<td>Cond. 6</td>
<td>1263.4</td>
<td>1.31E-04</td>
</tr>
<tr>
<td>Cond. 7</td>
<td>966.9</td>
<td>9.99E-05</td>
</tr>
<tr>
<td>Cond. 8</td>
<td>1020.3</td>
<td>1.05E-04</td>
</tr>
<tr>
<td>Cond. 9</td>
<td>769.4</td>
<td>7.95E-05</td>
</tr>
<tr>
<td>Cond. 10</td>
<td>586.6</td>
<td>6.06E-05</td>
</tr>
<tr>
<td>Cond. 11</td>
<td>317.7</td>
<td>3.28E-05</td>
</tr>
<tr>
<td>Cond. 12</td>
<td>160.3</td>
<td>1.66E-05</td>
</tr>
<tr>
<td>Cond. 13</td>
<td>37.0</td>
<td>3.83E-06</td>
</tr>
<tr>
<td>Cond. 14</td>
<td>0.0</td>
<td>0.00E+00</td>
</tr>
</tbody>
</table>

### Table 6: Moment and Strain Values for 005248 Load Set 5

<table>
<thead>
<tr>
<th>Moments via SAP (k-in)</th>
<th>Distance Traveled</th>
<th>Strains via SAP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td>Girder 1</td>
</tr>
<tr>
<td>Cond. 1</td>
<td>0.0</td>
<td>0.00E+00</td>
</tr>
<tr>
<td>Cond. 2</td>
<td>101.0</td>
<td>1.04E-05</td>
</tr>
<tr>
<td>Cond. 3</td>
<td>444.5</td>
<td>4.59E-05</td>
</tr>
<tr>
<td>Cond. 4</td>
<td>718.1</td>
<td>7.42E-05</td>
</tr>
<tr>
<td>Cond. 5</td>
<td>868.3</td>
<td>8.97E-05</td>
</tr>
<tr>
<td>Cond. 6</td>
<td>829.3</td>
<td>8.57E-05</td>
</tr>
<tr>
<td>Cond. 7</td>
<td>731.7</td>
<td>7.56E-05</td>
</tr>
<tr>
<td>Cond. 8</td>
<td>639.5</td>
<td>6.61E-05</td>
</tr>
<tr>
<td>Cond. 9</td>
<td>551.7</td>
<td>5.70E-05</td>
</tr>
<tr>
<td>Cond. 10</td>
<td>372.2</td>
<td>3.85E-05</td>
</tr>
<tr>
<td>Cond. 11</td>
<td>244.3</td>
<td>2.53E-05</td>
</tr>
<tr>
<td>Cond. 12</td>
<td>138.0</td>
<td>1.43E-05</td>
</tr>
<tr>
<td>Cond. 13</td>
<td>33.5</td>
<td>3.46E-06</td>
</tr>
<tr>
<td>Cond. 14</td>
<td>0.0</td>
<td>0.00E+00</td>
</tr>
</tbody>
</table>
3.1.6 Verification with Experimental Data

The strains gathered from SAP2000 were compared with experimental values provided. The strains were graphed accordingly versus the distance that the truck had traveled along the bridge.

It is important to note that with the strain data provided, an accurate Strain vs. Distance graph cannot be created. Strain data was given in units of time, and without knowing if the truck crossed the bridge at a constant speed or stopped momentarily, this time cannot accurately be converted to distance.

For graphing purposes, a constant speed across the bridge for the truck was assumed. A factor was used to convert the time data into distance. The experimental data peaks at a certain time and levels off for a few moments. It is safe to assume that the truck paused in the center of the bridge for accurate static strain data recording.

The dashed lines for the SAP strains versus the distance of the loading points along the bridge simulate a truck crossing the bridge without pause. Therefore, there is no flattening of the graph at the peak strain value obtained. Since the peak strain value experienced by the truck is the most critical for calibration, the graphs generated are sufficient for accurate calibration.

Adjusting boundary conditions is one of the easiest methods of modifying the SAP model. In the field, true pin-roller conditions may not apply. With Bridge 005248, a fixed-roller condition provided the most accurate results. The model was modified until deemed acceptable, and the results were graphed as shown below.
Figure 21: Strain Comparison – Set 1, Girder 1

(0.5\% Difference)

Figure 22: Strain Comparison – Set 1, Girder 2

(0.0\% Difference)
Figure 23: Strain Comparison – Set 1, Girder 3

Figure 24: Strain Comparison – Set 1, Girder 4
Figure 25: Strain Comparison – Set 3, Girder 1

Figure 26: Strain Comparison – Set 3, Girder 2
Figure 27: Strain Comparison – Set 3, Girder 3

Figure 28: Strain Comparison – Set 3, Girder 4
Figure 29: Strain Comparison – Set 5, Girder 1

Figure 30: Strain Comparison – Set 5, Girder 2
Figure 31: Strain Comparison – Set 5, Girder 3

Figure 32: Strain Comparison – Set 5, Girder 4
3.1.7 Load Rating for Bridge 005248

Both the Allowable Stress Method and Load Factor Method were performed in determining the Load Rating Factors for ALDOT Bridge 005248. The first step in determining the load-rating factor for the traditionally reinforced concrete girder bridge was to calculate the section properties for the girders. The table below summarizes the section properties for each girder.

Table 7: Girder Section Properties for Bridge 005248

<table>
<thead>
<tr>
<th></th>
<th>Girder 1</th>
<th>Girder 2</th>
<th>Girder 3</th>
<th>Girder 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_g \text{ (in}^4\text{)}$</td>
<td>63400</td>
<td>70443</td>
<td>70443</td>
<td>63400</td>
</tr>
<tr>
<td>$I_{cr} \text{ (in}^4\text{)}$</td>
<td>40901</td>
<td>47079</td>
<td>47079</td>
<td>40617</td>
</tr>
<tr>
<td>$b \text{ (in)}$</td>
<td>63.00</td>
<td>80.00</td>
<td>80.00</td>
<td>61.00</td>
</tr>
<tr>
<td>$b_w \text{ (in)}$</td>
<td>13.00</td>
<td>13.00</td>
<td>13.00</td>
<td>13.00</td>
</tr>
<tr>
<td>$h_f \text{ (in)}$</td>
<td>6.00</td>
<td>6.00</td>
<td>6.00</td>
<td>6.00</td>
</tr>
<tr>
<td>$A_s \text{ (in}^2\text{)}$</td>
<td>9.36</td>
<td>9.36</td>
<td>9.36</td>
<td>9.36</td>
</tr>
<tr>
<td>$A_v \text{ (in}^2\text{)}$</td>
<td>0.39</td>
<td>0.39</td>
<td>0.39</td>
<td>0.39</td>
</tr>
<tr>
<td>$d \text{ (in)}$</td>
<td>26.00</td>
<td>27.13</td>
<td>27.13</td>
<td>26.00</td>
</tr>
<tr>
<td>$c \text{ for } I_{cr} \text{ (in)}$</td>
<td>7.17</td>
<td>6.64</td>
<td>6.64</td>
<td>7.27</td>
</tr>
<tr>
<td>$f'c \text{ (psi)}$</td>
<td>3000</td>
<td>3000</td>
<td>3000</td>
<td>3000</td>
</tr>
<tr>
<td>$n$</td>
<td>10.00</td>
<td>10.00</td>
<td>10.00</td>
<td>10.00</td>
</tr>
</tbody>
</table>

The girder section properties were used to determine the live and dead load stress effects on the members for use in the general rating equation as discussed in section 2.3. The bridge was loaded using standard HS20-44 trucks in a manner to retrieve the maximum moment and shear that each member would experience when loaded. The resultant of two HS20-44 trucks was assumed in the center of the bridge, and axle loads in the form of point-loads were added accordingly.
The maximum moment and shear values due to the live load only were recorded. Then all live loads were deleted to determine the dead load effect on the structure. The resulting values for each girder are included in the following table.

Table 8: SAP2000 Max Moment and Shear for Bridge 005248

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{L_{\text{max}}}$ (k-in)</td>
<td>2459</td>
<td>2555</td>
<td>2555</td>
<td>2459</td>
</tr>
<tr>
<td>$M_{D_{\text{max}}}$ (k-in)</td>
<td>1828</td>
<td>2088</td>
<td>2081</td>
<td>1806</td>
</tr>
<tr>
<td>$V_{L_{\text{max}}}$ (k)</td>
<td>24.4</td>
<td>25.9</td>
<td>25.9</td>
<td>24.4</td>
</tr>
<tr>
<td>$V_{D_{\text{max}}}$ (k)</td>
<td>18.7</td>
<td>21.3</td>
<td>21.3</td>
<td>18.7</td>
</tr>
</tbody>
</table>

The live and dead load stress effects can be calculated from the values obtained in SAP. The stress effect values will remain constant for the Allowable Stress and Load Factor Methods.

Table 9: Bridge 005248 SAP Stress Effects for Moment Rating

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_{\text{concrete}}$ (psi)</td>
<td>431.14</td>
<td>360.19</td>
<td>360.19</td>
<td>440.08</td>
</tr>
<tr>
<td>$D_{\text{concrete}}$ (psi)</td>
<td>320.39</td>
<td>294.36</td>
<td>293.44</td>
<td>323.09</td>
</tr>
<tr>
<td>$L_{\text{steel}}$ (psi)</td>
<td>11322.01</td>
<td>11760.48</td>
<td>11760.48</td>
<td>11322.01</td>
</tr>
<tr>
<td>$D_{\text{steel}}$ (psi)</td>
<td>8413.72</td>
<td>8586.13</td>
<td>8559.21</td>
<td>8326.99</td>
</tr>
</tbody>
</table>

Table 10: Bridge 005248 SAP Stress Effects for Shear Rating

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>L (psi)</td>
<td>72.64</td>
<td>73.95</td>
<td>73.95</td>
<td>72.56</td>
</tr>
<tr>
<td>D (psi)</td>
<td>55.62</td>
<td>61.00</td>
<td>61.00</td>
<td>55.56</td>
</tr>
</tbody>
</table>

The first rating method performed for Bridge 005248 was the Allowable Stress Method. The Operating and Inventory factors were calculated for the moment experienced by the concrete and the steel, along with the shear in the bridge girders. The
nominal capacity, C, for a reinforced concrete girder bridge in regards to shear and moment rating is dictated by the code. AASHTO allows for the operating nominal capacity of the concrete to be 1900psi, with an inventory capacity of 1200psi. The code also dictates that the nominal capacity for the reinforcing steel (unknown, constructed after 1954) to be 28000psi at the operating level and 20000psi at the inventory level. These values were used in the general rating equation along with other coefficients discussed in section 2.3 to determine the moment rating factors for the bridge.

Table 11: ASR Moment Rating Factors for 005248

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF\textsubscript{conc} (Op)</td>
<td>2.82</td>
<td>3.43</td>
<td>3.43</td>
<td>2.76</td>
</tr>
<tr>
<td>RF\textsubscript{conc} (Inv)</td>
<td>1.57</td>
<td>1.93</td>
<td>1.94</td>
<td>1.53</td>
</tr>
<tr>
<td>RF\textsubscript{steel} (Op)</td>
<td>1.33</td>
<td>1.27</td>
<td>1.27</td>
<td>1.34</td>
</tr>
<tr>
<td>RF\textsubscript{steel} (Inv)</td>
<td>0.79</td>
<td>0.75</td>
<td>0.75</td>
<td>0.79</td>
</tr>
</tbody>
</table>

The allowable shear stress was calculated based on the section properties. AASHTO provides an equation to calculate the nominal shear capacity to be used in the general rating equation. The shear capacity was calculated per the equation provided below, and the shear rating factors for Operating and Inventory levels were determined utilizing the general rating equation. These results were tabulated and are provided.

\textbf{Equation 2: Shear Capacity of Reinforced Concrete Girder}

\[ \nu_s = \frac{f_s A_v}{b_w s} \]
Table 12: ASR Shear Rating Factors for 005248

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF (Op)</td>
<td>2.03</td>
<td>1.99</td>
<td>1.99</td>
<td>2.03</td>
</tr>
<tr>
<td>RF (Inv)</td>
<td>2.29</td>
<td>3.05</td>
<td>3.05</td>
<td>2.29</td>
</tr>
</tbody>
</table>

All of the calculated rating factors were compared to determine the minimum factor. The Allowable Stress Method minimum rating factors for Bridge 005248 were 1.27 and 0.75 for Operating and Inventory levels, respectively.

The second rating method performed for Bridge 005248 was the Load Factor Method. Similarly, the Operating and Inventory factors were calculated for this method. Contrary to dictated allowable stresses for ASR, LFR uses the actual calculated capacities using equations in the AASHTO Specifications as nominal capacities. The moment and shear capacity equations used are as follows:

Equation 3: Moment Capacity for Reinforced Concrete Girders  
\[ M_u = 0.9 \times (A_s f_y) \times (d - \frac{a}{2}) \]

Equation 4: Shear Capacity for Reinforced Concrete Girders  
\[ V_u = V_c + V_s \]

Equation 5: Shear Strength of Concrete for Reinforced Concrete Girders  
\[ V_c = 2\sqrt{f_c} c \times b_w d \]

Equation 6: Shear Strength of Steel for Reinforced Concrete Girders  
\[ V_s = \frac{A_s f_y d}{s} \]

The calculated values for \( M_u \) and \( V_u \) were used as nominal capacities in the general rating equation along with the coefficients for LFR as noted in section 2.3 above. The Load Factor Method minimum rating factors for Bridge 005248 were 1.40 and 0.84.
for Operating and Inventory levels, respectively. The following tables summarize the rating factors calculated for each entity.

Table 13: LFR Moment Rating Factors for 005248

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$RF_{\text{conc}}$ (Op)</td>
<td>1.44</td>
<td>1.40</td>
<td>1.40</td>
<td>1.45</td>
</tr>
<tr>
<td>$RF_{\text{conc}}$ (Inv)</td>
<td>0.86</td>
<td>0.84</td>
<td>0.84</td>
<td>0.87</td>
</tr>
<tr>
<td>$RF_{\text{steel}}$ (Op)</td>
<td>1.52</td>
<td>1.45</td>
<td>1.45</td>
<td>1.52</td>
</tr>
<tr>
<td>$RF_{\text{steel}}$ (Inv)</td>
<td>0.91</td>
<td>0.87</td>
<td>0.87</td>
<td>0.91</td>
</tr>
</tbody>
</table>

Table 14: LFR Shear Rating Factors for 005248

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$RF$ (Op)</td>
<td>1.58</td>
<td>1.41</td>
<td>1.41</td>
<td>1.58</td>
</tr>
<tr>
<td>$RF$ (Inv)</td>
<td>0.95</td>
<td>0.84</td>
<td>0.84</td>
<td>0.95</td>
</tr>
</tbody>
</table>

The structure rating results provided by ALDOT noted that Load Factor methodology was used in determining the controlling rating factor for the bridge. ALDOT determined that the Operating and Inventory Ratings for Bridge 005248 were 1.30 and 0.76, respectively. These load factors were multiplied by the weight of an HS20-44 truck (36 tons) for an allowable weight posting. Only the factors calculated for LFR using values via the SAP model are to be compared to ALDOT’s results. The following table summarizes the comparison of ALDOT and SAP model load rating results.

<table>
<thead>
<tr>
<th></th>
<th>ALDOT LR Factor</th>
<th>SAP LR Factor</th>
<th>ALDOT Rating</th>
<th>SAP Rating</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Operating Level</strong></td>
<td>1.30</td>
<td>1.40</td>
<td>46.8 Tons</td>
<td>50.4 Tons</td>
<td>7.69%</td>
</tr>
<tr>
<td><strong>Inventory Level</strong></td>
<td>0.76</td>
<td>0.84</td>
<td>27.4 Tons</td>
<td>30.2 Tons</td>
<td>10.53%</td>
</tr>
</tbody>
</table>

The SAP model values resulted in higher ratings than ALDOT. The SAP model is producing results based on more accurate bridge behavior; hence, the original rating is considered conservative. The rating is reduced using values obtained in the bridge model.
3.2 ALDOT BRIDGE 005318

3.2.1 Location of Bridge

Bridge 005318 is located in near the Shelby and Chilton County line. The bridge spans over Waxahatchee Creek near the intersection of the Coosa River, connecting strips of State Road 145. Bridge 005318 is a two lane bridge with five sections. The end span with composite steel girders was analyzed in SAP2000. However, for rating purposes, only the steel section properties were used as in the ALDOT analysis. The bridge was designed in July of 1955 according to the National Bridge Inventory Database.

Figure 33: Location of ALDOT Bridge 005318
3.2.2 Bridge Information

*Bridge Photos*

Figure 34: Photo Along Span for Bridge 005318

Figure 35: Satellite Plan View of Bridge 005318
Elevations and Cross Sections

Figure 36: Elevation of 005318 from Original Drawings

Figure 37: Cross Section of 005318 from Original Drawings
Sensor Placement

The test span girder section above was created using the Section Wizard using a similar structural analysis program called STAAD.Pro. The Section Wizard is useful in quickly obtaining section properties when multiple materials and shapes are combined. Moments experienced by each girder were gathered at the location of the sensor as defined by the experimental results from ALDOT. A figure providing the location of the sensor in distance relation to the supports is included below. The following figures were copied from the structure rating results to portray the sensor location and truck positioning for each load set.
Girder Positioning Diagram  Not to Scale

Figure 39: Sensor Location for 005318 Relative to Supports
Figure 40: Sensor Layout Plan View for 005318
3.2.3 Qualitative Assessment of Model

Bridge 005318 was loaded and analyzed in February, 2008 by ALDOT with Virtis and BRUFEM. Two 3-axle trucks were used in five different positions for the initial analysis of Bridge 005318. Similar to the analysis for 005248, the first two axles were spaced at 4’-9.5”, and the spacing between the middle and rear axle was 18’-5.5”. The total loads for the front, middle, and rear axles were 30.15 kips, 32.075 kips, and 22.875 kips, respectively. Axle loads were distributed into two point loads to represent each wheel for SAP2000.

There were five loading positions performed by ALDOT. However, only sets 1, 3, and 5 were modeled in SAP for calibration verification. The table below indicates section properties that were used to convert moment values to strain, and the figure below taken from the initial ALDOT experiment shows the location of each truck in the respective loading set. For simplification, each composite steel girder was assumed to have the same effective width of concrete contributing to the strength of the section.

<table>
<thead>
<tr>
<th>Girder</th>
<th>c (in)</th>
<th>$I_x$ (in$^4$)</th>
<th>$b_f$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>26.99</td>
<td>18723</td>
<td>48.0</td>
</tr>
<tr>
<td>2</td>
<td>26.99</td>
<td>18723</td>
<td>48.0</td>
</tr>
<tr>
<td>3</td>
<td>26.99</td>
<td>18723</td>
<td>48.0</td>
</tr>
<tr>
<td>4</td>
<td>26.99</td>
<td>18723</td>
<td>48.0</td>
</tr>
</tbody>
</table>

The truck-positioning diagram below was copied from the ALDOT results to portray the location of the truck in relation to the bridge curb. These values were used to generate a tabulate the location of the truck point loads in SAP from the top of the model.
Table 16: Loading Sets for 005318 Model in SAP2000

<table>
<thead>
<tr>
<th></th>
<th>Set 1</th>
<th>Set 3</th>
<th>Set 5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Load Distance from Top of</strong></td>
<td><strong>Load Distance from Top of</strong></td>
<td><strong>Load Distance from Top of</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Model (in)</strong></td>
<td><strong>Model (in)</strong></td>
<td><strong>Model (in)</strong></td>
</tr>
<tr>
<td><strong>Truck 1</strong></td>
<td></td>
<td><strong>Truck 1</strong></td>
<td><strong>Truck 1</strong></td>
</tr>
<tr>
<td>Left Tire</td>
<td>20.56</td>
<td>Left Tire</td>
<td>Left Tire</td>
</tr>
<tr>
<td>Right Tire</td>
<td>95.56</td>
<td>Right Tire</td>
<td>Right Tire</td>
</tr>
<tr>
<td><strong>Truck 2</strong></td>
<td><strong>Load Distance from Top of</strong></td>
<td><strong>Load Distance from Top of</strong></td>
<td><strong>Load Distance from Top of</strong></td>
</tr>
<tr>
<td>Model (in)</td>
<td><strong>Model (in)</strong></td>
<td><strong>Model (in)</strong></td>
<td><strong>Model (in)</strong></td>
</tr>
<tr>
<td>Left Tire</td>
<td>140.31</td>
<td>Left Tire</td>
<td>Left Tire</td>
</tr>
<tr>
<td>Right Tire</td>
<td>215.31</td>
<td>Right Tire</td>
<td>Right Tire</td>
</tr>
</tbody>
</table>
Figure 42: 005318 SAP Model Strains for Load Set 1
Figure 43: 005318 SAP Model Strains for Load Set 3
Figure 44: 005318 SAP Model Strains for Load Set 5
3.2.4 Modeling of Bridge in SAP2000

After all critical and girder spacing dimensions were determined from the experimental portfolio provided by ALDOT, Bridge 005318 was then modeled in SAP2000. A true grid condition was used to begin laying out the bridge in the grid system developed. The steel girders were modeled as W36X160 with a six-inch concrete slab above. The concrete was modeled with strength of 3000psi. Screenshots of the model for visualization purposes have been included below.

Figure 45: Bridge 005318 Model - Perspective Model

Figure 46: Bridge 005318 Model - 3D Side View
Figure 47: Bridge 005318 Model - Top 2D Frame View
3.2.5 Bridge Model Analysis

Loading Conditions
Moments were gathered in SAP2000 for thirteen separate loading conditions along the bridge. The trucks were modeled as point loads in eight feet increments along the transverse frames lines of the slab. It is imperative that the slab frames modeled are meshed with the steel girders to effectively generate load distribution. This also allows for composite action to occur.

Results for Moments and Strains
Bridge 005318 is also a four-girder bridge. For each truck loading position along the bridge, moment values were obtained at the location of the strain sensor. All four girders were considered for recording moment values for conversion to strain. The moment values were noted in Kip-in units, and the strains calculated have no unit.

Table 17: Moment and Strain Values for 005318 Load Set 1

<table>
<thead>
<tr>
<th>Condition</th>
<th>Moments via SAP (k-in)</th>
<th>Distance Traveled (ft)</th>
<th>Strains via SAP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Girder 1</td>
<td>Girder 2</td>
<td>Girder 3</td>
</tr>
<tr>
<td>Cond. 1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Cond. 2</td>
<td>1467.9</td>
<td>1231.9</td>
<td>1075.8</td>
</tr>
<tr>
<td>Cond. 3</td>
<td>3455.3</td>
<td>2924.8</td>
<td>2530.8</td>
</tr>
<tr>
<td>Cond. 4</td>
<td>5620.8</td>
<td>4880.6</td>
<td>4150.0</td>
</tr>
<tr>
<td>Cond. 5</td>
<td>8265.4</td>
<td>7555.9</td>
<td>6257.3</td>
</tr>
<tr>
<td>Cond. 6</td>
<td>8708.0</td>
<td>7806.6</td>
<td>6533.1</td>
</tr>
<tr>
<td>Cond. 7</td>
<td>7009.0</td>
<td>6036.3</td>
<td>5178.1</td>
</tr>
<tr>
<td>Cond. 8</td>
<td>6518.0</td>
<td>5798.6</td>
<td>4906.4</td>
</tr>
<tr>
<td>Cond. 9</td>
<td>4188.7</td>
<td>3615.6</td>
<td>3117.2</td>
</tr>
<tr>
<td>Cond. 10</td>
<td>1963.2</td>
<td>1671.6</td>
<td>1453.6</td>
</tr>
<tr>
<td>Cond. 11</td>
<td>1010.4</td>
<td>854.7</td>
<td>750.4</td>
</tr>
<tr>
<td>Cond. 12</td>
<td>380.7</td>
<td>322.0</td>
<td>284.3</td>
</tr>
<tr>
<td>Cond. 13</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>
Table 18: Moment and Strain Values for 005318 Load Set 3

<table>
<thead>
<tr>
<th>Moments via SAP (k-in)</th>
<th>Distance Traveled</th>
<th>Strains via SAP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td>Girder 1</td>
</tr>
<tr>
<td>Cond. 1</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Cond. 2</td>
<td>1246.4</td>
<td>1197.5</td>
</tr>
<tr>
<td>Cond. 3</td>
<td>2927.7</td>
<td>2817.4</td>
</tr>
<tr>
<td>Cond. 4</td>
<td>4763.0</td>
<td>4604.9</td>
</tr>
<tr>
<td>Cond. 5</td>
<td>7049.7</td>
<td>6877.3</td>
</tr>
<tr>
<td>Cond. 6</td>
<td>7410.4</td>
<td>7204.8</td>
</tr>
<tr>
<td>Cond. 7</td>
<td>5950.3</td>
<td>5744.8</td>
</tr>
<tr>
<td>Cond. 8</td>
<td>5567.9</td>
<td>5405.2</td>
</tr>
<tr>
<td>Cond. 9</td>
<td>3571.4</td>
<td>3449.2</td>
</tr>
<tr>
<td>Cond. 10</td>
<td>1672.7</td>
<td>1611.6</td>
</tr>
<tr>
<td>Cond. 11</td>
<td>863.7</td>
<td>831.2</td>
</tr>
<tr>
<td>Cond. 12</td>
<td>326.7</td>
<td>314.3</td>
</tr>
<tr>
<td>Cond. 13</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Table 19: Moment and Strain Values for 005318 Load Set 5

<table>
<thead>
<tr>
<th>Moments via SAP (k-in)</th>
<th>Distance Traveled</th>
<th>Strains via SAP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td>Girder 1</td>
</tr>
<tr>
<td>Cond. 1</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Cond. 2</td>
<td>1067.2</td>
<td>1118.1</td>
</tr>
<tr>
<td>Cond. 3</td>
<td>2473.1</td>
<td>2631.6</td>
</tr>
<tr>
<td>Cond. 4</td>
<td>3918.4</td>
<td>4313.8</td>
</tr>
<tr>
<td>Cond. 5</td>
<td>5563.1</td>
<td>6489.9</td>
</tr>
<tr>
<td>Cond. 6</td>
<td>5945.9</td>
<td>6781.1</td>
</tr>
<tr>
<td>Cond. 7</td>
<td>4958.6</td>
<td>5380.2</td>
</tr>
<tr>
<td>Cond. 8</td>
<td>4547.9</td>
<td>5088.2</td>
</tr>
<tr>
<td>Cond. 9</td>
<td>2998.6</td>
<td>3235.2</td>
</tr>
<tr>
<td>Cond. 10</td>
<td>1421.9</td>
<td>1509.1</td>
</tr>
<tr>
<td>Cond. 11</td>
<td>744.6</td>
<td>778.5</td>
</tr>
<tr>
<td>Cond. 12</td>
<td>284.0</td>
<td>294.7</td>
</tr>
<tr>
<td>Cond. 13</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

51
3.2.6 Verification with Experimental Data

Only the section properties for the steel W-section were used when load rating the steel girder bridge. However, the bridge girder acts in accidental composite action when loaded. Accidental composite action occurs when the strength of the bond between the concrete slab and the beam or the cohesion is larger than the shear force experienced along the connecting plane. This simply means that the concrete slab will aid the steel girder in adding stiffness to the bridge.

Shear studs were not located in the bridge construction documents provided. Shear studs are welded to the top flange of steel beams and the concrete slab is poured along the bridge span. These studs are casted into the concrete slab and provide stiffness and capacity to the steel girder, acting now as a true composite section.

In reality, some composite action will occur when a concrete deck is cast on steel girders. This phenomenon is called accidental composite action as mentioned above. Therefore, when modeled in SAP, composite action can be considered in calibration techniques.

Using the composite section properties was the most efficient way to calibrate the model for actual bridge behavior. The composite girder section properties determined above were used for conversion to strain. Similar to Bridge 005248, only the peak strain values can be used to verify proper calibration. This is due to unknown loading truck behavior in experimental data; hence, accurate conversion of time to distance is impossible. The model was tweaked until the peak strains for SAP and Experimental graphs below matched.
Figure 48: Strain Comparison – Set 1, Girder 1

Figure 49: Strain Comparison – Set 1, Girder 2

(3.6% Difference)

(4.3% Difference)
Figure 50 Strain Comparison – Set 1, Girder 3

Figure 51: Strain Comparison – Set 1, Girder 4
Figure 52: Strain Comparison – Set 3, Girder 1
(8.2% Difference)

Figure 53: Strain Comparison – Set 3, Girder 2
(10.5% Difference)
Figure 54: Strain Comparison – Set 3, Girder 3

Figure 55: Strain Comparison – Set 3, Girder 4
Figure 56: Strain Comparison – Set 5, Girder 1

Figure 57: Strain Comparison – Set 5, Girder 2
Figure 58: Strain Comparison – Set 5, Girder 3

Figure 59: Strain Comparison – Set 5, Girder 4

(9.0% Difference)

(14.9% Difference)
3.2.7 Load Rating for Bridge 005318

Load rating using the Allowable Stress Method and the Load Factor Method were considered in rating Bridge 005318. The American Institute of Steel Construction (AISC) provides section properties for rolled steel members in the AISC Manual. Although the bridge was model to imitate composite action, load rating must use section properties for the rolled W36x160 only. The table below summarizes the necessary properties gathered from the AISC Construction Manual.

<table>
<thead>
<tr>
<th></th>
<th>Girder 1</th>
<th>Girder 2</th>
<th>Girder 3</th>
<th>Girder 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>I&lt;sub&gt;g&lt;/sub&gt; (in&lt;sup&gt;4&lt;/sup&gt;)</td>
<td>9760</td>
<td>9760</td>
<td>9760</td>
<td>9760</td>
</tr>
<tr>
<td>A&lt;sub&gt;s&lt;/sub&gt; (in&lt;sup&gt;2&lt;/sup&gt;)</td>
<td>47.00</td>
<td>47.00</td>
<td>47.00</td>
<td>47.00</td>
</tr>
<tr>
<td>A&lt;sub&gt;v&lt;/sub&gt; (in&lt;sup&gt;2&lt;/sup&gt;)</td>
<td>23.40</td>
<td>23.40</td>
<td>23.40</td>
<td>23.40</td>
</tr>
<tr>
<td>Z&lt;sub&gt;x&lt;/sub&gt; (in&lt;sup&gt;3&lt;/sup&gt;)</td>
<td>624</td>
<td>624</td>
<td>624</td>
<td>624</td>
</tr>
<tr>
<td>S&lt;sub&gt;x&lt;/sub&gt; (in&lt;sup&gt;3&lt;/sup&gt;)</td>
<td>541</td>
<td>541</td>
<td>541</td>
<td>541</td>
</tr>
<tr>
<td>F&lt;sub&gt;y&lt;/sub&gt; (ksi)</td>
<td>33</td>
<td>33</td>
<td>33</td>
<td>33</td>
</tr>
<tr>
<td>Impact, I</td>
<td>1.26</td>
<td>1.26</td>
<td>1.26</td>
<td>1.26</td>
</tr>
</tbody>
</table>

The area of steel contributing to the shear capacity of the girder, A<sub>v</sub>, was calculated by multiplying the thickness of the web by the girder height. The steel yield strength, F<sub>y</sub>, and the Impact factor were both taken directly from the analysis by ALDOT. Using the properties above, the live and dead load stresses that the girder experienced can be calculated. These stresses can be used in the same general load rating equation as above. Placing the resultant of two standard HS20-44 trucks in the center of the bridge produced the maximum moment and shear experienced by the bridge.

Only the truck live load was considered when gathering the maximum moment and strain values from SAP. This was necessary to calculate the stress effect of the live
load only. All truck point loads were deleted from the model and the maximum moment and shear due to the weight of the bridge was recorded. All of the maximum values are included in the table below.

Table 21: SAP2000 Max Moment and Shear for Bridge 005318

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{L_{\text{max}}}$ (k-in)</td>
<td>8676</td>
<td>7800</td>
<td>7800</td>
<td>8676</td>
</tr>
<tr>
<td>$M_{D_{\text{max}}}$ (k-in)</td>
<td>5892</td>
<td>5892</td>
<td>5892</td>
<td>5892</td>
</tr>
<tr>
<td>$V_{L_{\text{max}}}$ (k)</td>
<td>47.1</td>
<td>43.1</td>
<td>43.1</td>
<td>47.1</td>
</tr>
<tr>
<td>$V_{D_{\text{max}}}$ (k)</td>
<td>27.4</td>
<td>27.4</td>
<td>27.4</td>
<td>27.4</td>
</tr>
</tbody>
</table>

The values above were used to calculate the live and dead stress effects on the structure. In this case, the moment values were divided by the section modulus of the W36x160. The live load stress effect includes the impact factor, but the dead load stress has no impact to be accounted for.

Table 22: Bridge 005318 SAP Stress Effects for Moment Rating

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_{\text{steel}}$ (psi)</td>
<td>11322.01</td>
<td>11760.48</td>
<td>11760.48</td>
<td>11322.01</td>
</tr>
<tr>
<td>$D_{\text{steel}}$ (psi)</td>
<td>8413.72</td>
<td>8586.13</td>
<td>8559.21</td>
<td>8326.99</td>
</tr>
</tbody>
</table>

Table 23: Bridge 005318 SAP Stress Effects for Shear Rating

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L$ (psi)</td>
<td>2530</td>
<td>2310</td>
<td>2310</td>
<td>2530</td>
</tr>
<tr>
<td>$D$ (psi)</td>
<td>1170</td>
<td>1170</td>
<td>1170</td>
<td>1170</td>
</tr>
</tbody>
</table>

The Allowable Stress Method was considered first when rating the bridge using the values obtained above. Both the Operating and Inventory level factors were calculated. As for the moment nominal capacity for a steel girder, AASHTO allows for
0.75*F_y at the Operating Level and 0.55*F_y at the Inventory Level. This nominal capacity is utilized in the general rating equation for load rating of the structure.

Table 24: ASR Moment Rating Factors for 005318

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF_{steel} (Op)</td>
<td>0.69</td>
<td>0.77</td>
<td>0.77</td>
<td>0.69</td>
</tr>
<tr>
<td>RF_{steel} (Inv)</td>
<td>0.36</td>
<td>0.36</td>
<td>0.36</td>
<td>0.36</td>
</tr>
</tbody>
</table>

For the Allowable Stress Method, AASHTO also provides nominal stress values to be used in the general equation. The allowable shear stress at the Operating Level is to be 13,500psi, and the allowable stress at the Inventory Level is 11,000psi. These given values were placed in the general rating equation and the following factors were calculated.

Table 25: ASR Shear Rating Factors for 005318

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF (Op)</td>
<td>4.88</td>
<td>5.33</td>
<td>5.33</td>
<td>4.88</td>
</tr>
<tr>
<td>RF (Inv)</td>
<td>3.89</td>
<td>4.25</td>
<td>4.25</td>
<td>3.89</td>
</tr>
</tbody>
</table>

The minimum rating factor for the Operating and Inventory Levels governs the rating of the bridge. The Allowable Stress Method minimum rating factors for Bridge 005318 were 0.69 and 0.36 for Operating and Inventory levels, respectively.

Next, the Operating and Inventory Level ratings for the Load Factor Method were calculated. The nominal capacity coefficients for use in the LFR equations are based on the code dictated capacities of the section. If certain parameters for the section are met and the section is compact, AASHTO allows for one to use the plastic limit state as the section modulus for the steel girder. In the case for Bridge 005318, these parameters were
met and the plastic limit state \( Z_x \) was used in calculating the section moment capacity.

The moment and shear capacity equations used are as follows:

Equation 7: Moment Capacity for Rolled Steel Girders

\[ M_u = Z_x \times F_y \]

Equation 8: Shear Capacity for Rolled Steel Girders

\[ V_u = A_v \times F_y \]

The calculated values for \( M_u \) and \( V_u \) were used as nominal capacities in the general rating equation along with the coefficients for LFR as noted in section 2.3 above.

The Load Factor Method minimum rating factors for Bridge 005318 were 0.91 and 0.55 for Operating and Inventory levels, respectively. The following tables summarize the rating factors calculated for each entity.

Table 26: LFR Moment Rating Factors for 005248

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF\text{steel} (Op)</td>
<td>0.91</td>
<td>1.02</td>
<td>1.02</td>
<td>0.91</td>
</tr>
<tr>
<td>RF\text{steel} (Inv)</td>
<td>0.55</td>
<td>0.61</td>
<td>0.61</td>
<td>0.55</td>
</tr>
</tbody>
</table>

Table 27: LFR Shear Rating Factors for 005248

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF (Op)</td>
<td>9.58</td>
<td>10.47</td>
<td>10.47</td>
<td>9.58</td>
</tr>
<tr>
<td>RF (Inv)</td>
<td>5.74</td>
<td>6.27</td>
<td>6.27</td>
<td>5.74</td>
</tr>
</tbody>
</table>

With large steel bridge girders, moment will most likely control the rating factor for the bridge. This is due to the fact that large W-sections have thick webs that provide adequate shear resistance. ALDOT used the load test rating when determining the rating factor for the bridge. Results versus the SAP model will vary slightly, as only ASR and LFR were used. ALDOT determined that the Operating and Inventory Ratings for Bridge
005318 were 1.19 and 0.59, respectively. These load factors were multiplied by the weight of an HS20-44 truck (36 tons) for an allowable weight posting. Only the factors calculated for LFR using values via the SAP model are to be compared to ALDOT’s results. The following table summarizes the comparison of ALDOT and SAP model load rating results.

<table>
<thead>
<tr>
<th></th>
<th>ALDOT LR Factor</th>
<th>SAP LR Factor</th>
<th>ALDOT Rating</th>
<th>SAP Rating</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operating Level</td>
<td>1.19</td>
<td>0.91</td>
<td>42.9 Tons</td>
<td>32.7 Tons</td>
<td>23.5%</td>
</tr>
<tr>
<td>Inventory Level</td>
<td>0.59</td>
<td>0.55</td>
<td>21.5 Tons</td>
<td>19.8 Tons</td>
<td>6.8%</td>
</tr>
</tbody>
</table>

For the steel girder bridge, the ratings were lower than the ALDOT calculated ratings using the Load Test Method. One reason for the lower postings is that the maximum moment in a girder was achieved at the end girders. When comparing the SAP strain values to the experimental strain values, one will notice that the SAP values represent a less stiff bridge. For a more accurate load rating, the factored difference can be taken account in the load rating factor.

Another place for error lies in the difference in calculations for the load rating factor. In ALDOT’s initial assessment, a 2D line with wheel loads was used to calculate live load moment for the girder. However, SAP realizes and includes the load distribution to nearby girders for a more accurate analysis of the structure. Note that the rating factor for Bridge 005318 at the Operating Level is below 1.0, so bridge posting is necessary.
3.3 ALDOT BRIDGE 012296

3.3.1 Location of Bridge

Bridge 012296 serves as a connector for I-459 over Sulphur Springs Road. The bridge is near the I-459 and SR-150 Interchange, which is noted as Exit 10. The bridge drawings are dated for the fiscal year of 1978, and the bridge was constructed in 1980. A map is included below to display the location of the bridge in relation to Exit 10.

![Map showing location of ALDOT Bridge 012296](image)

Figure 60: Location of ALDOT Bridge 012296

The bridge has ten girders spaced at 7’-1” over a 46’-4” span. The girders are AASHTO Type II for the end spans, including the one load rated via SAP2000. However, the middle span utilizes AASHTO Type III girders.

The Type II girders are three feet in depth, and these specifically have a total of twelve pre-stressing strands with a yield stress of 270,000 psi. Elevations and cross-sections of the bridge girders are included as figures below.
3.3.2 Bridge Information

Bridge Photos

Figure 61: Elevation Photo of Bridge 012296

Figure 62: Lane Details Bridge 012296
Figure 63: Underside Span Photo for Bridge 012296

Figure 64: Elevation and Cross-section of Bridge 012296
Figure 65: Test Span Girder Section for Bridge 012296

Figure 66: Test Span Lane Details for Bridge 012296
Sensor Placement

For the pre-stressed girder bridge, Bridge 012296, there was no set of ALDOT load factor calculations provided. However, strain data gathered from loading the bridge with three separate trucks was given for bridge calibration purposes. Sensor data for each of the three trucks was recorded.

Similar to the other bridges modeled in SAP, determining the correct location of the sensor in relation to the span supports was crucial in calibrating the model. The sensors were placed near mid-span, as this location is expected to experience the maximum moment.

The sensors were placed 22’-0” from the North end support of the bridge. SAP can generate a moment and shear along any frame line at a certain distance from the support. This location was used for calibration of the bridge.

Figure 67: Bridge 012296 Test Span Elevation
Figure 68: Weighing Sensor Locations for Bridge 012296
Figure 69: AMDP Sensor Locations and Lane Dimensions for Bridge 012296
3.3.3 Qualitative Assessment of Model

Bridge 012296 measurements were recorded during the week of March 17, 2008. The test was completed in a five day period from March 18-22. The SiWIM Bridge weigh-in-motion system was calibrated and used for obtaining experimental data. ALDOT trucks were used for experimentation and data collection in the first two days. Gross weights and axel loads of vehicles pulled from traffic were used in the remaining three days of experimentation. These loads were statically weighed by ALDOT, and compared to the values obtained by the SiWIM.

![Figure 70: Girder and Slab Section Dimensions for Bridge 012296](image)

**Table 28: Composite Girder Section Properties for 012296**

<table>
<thead>
<tr>
<th>$I$ ($in^4$)</th>
<th>$f'c$ (psi)</th>
<th>$E$ (ksi)</th>
<th>$c$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>167453</td>
<td>5000</td>
<td>4030508</td>
<td>29.27</td>
</tr>
</tbody>
</table>

**Table 29: AASHTO Type II Beam Section Properties for 012296**

<table>
<thead>
<tr>
<th>$I$ ($in^4$)</th>
<th>$f'c$ (psi)</th>
<th>$E$ (ksi)</th>
<th>$c$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50980</td>
<td>5000</td>
<td>4030508</td>
<td>15.83</td>
</tr>
</tbody>
</table>

71
Table 30: Loading Information for Test Trucks 1, 3, and 4

<table>
<thead>
<tr>
<th>Number</th>
<th>Tag No</th>
<th>US unit</th>
<th>SI unit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Wheel Weight</td>
<td>Axle Group Weight</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Left</td>
<td>Right</td>
</tr>
<tr>
<td>1</td>
<td>1013995</td>
<td>5400</td>
<td>5450</td>
</tr>
<tr>
<td>2</td>
<td>1003303</td>
<td>7350</td>
<td>7900</td>
</tr>
<tr>
<td>3</td>
<td>1003303</td>
<td>8850</td>
<td>9350</td>
</tr>
<tr>
<td>4</td>
<td>1003303</td>
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<td>9650</td>
</tr>
<tr>
<td>5</td>
<td>1003303</td>
<td>8150</td>
<td>8000</td>
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</table>

Weighing time: 03-20-08 10:30AM

<table>
<thead>
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<td>Wheel Weight</td>
<td>Axle Group Weight</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Left</td>
<td>Right</td>
</tr>
<tr>
<td>1</td>
<td>1013995</td>
<td>5550</td>
<td>5750</td>
</tr>
<tr>
<td>2</td>
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<td>8450</td>
</tr>
<tr>
<td>3</td>
<td>1013995</td>
<td>9400</td>
<td>9650</td>
</tr>
</tbody>
</table>

Weighing time: 03-20-08 12:08PM

<table>
<thead>
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<th>Number</th>
<th>Tag No</th>
<th>US unit</th>
<th>SI unit</th>
</tr>
</thead>
<tbody>
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<td>Wheel Weight</td>
<td>Axle Group Weight</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Left</td>
<td>Right</td>
</tr>
<tr>
<td>1</td>
<td>1013995</td>
<td>5250</td>
<td>5450</td>
</tr>
<tr>
<td>2</td>
<td>1013995</td>
<td>7750</td>
<td>8550</td>
</tr>
<tr>
<td>3</td>
<td>1013995</td>
<td>8000</td>
<td>8750</td>
</tr>
</tbody>
</table>

Weighing time: 03-20-08 12:08PM
Figure 71: Experimental Strains for Test Truck 1
Figure 72: Experimental Strains for Test Truck 3
Figure 73: Experimental Strains for Test Truck 4
3.3.4 Modeling of Bridge in SAP2000

Dimensions and section properties for the ALDOT Bridge 012296 were obtained from the information given above, and the bridge was modeled as a true grid in SAP2000. Section properties were then defined for each frame modeled. The concrete girders were modeled at a concrete strength of 5000psi, and slab sections were modeled as 3000psi. Screenshots of the model in SAP2000 are included below.

Figure 74: Bridge 012296 Model - Perspective Model

Figure 75: Bridge 012296 Model - 3D Side View
Figure 76: Bridge 012296 Model - Top 2D Frame View
3.3.5 Bridge Model Analysis

Loading Conditions
For Test Trucks 1, 3, and 4, static loading conditions were determined with the truck moving ten feet headed north each condition. These conditions were modeled and analyzed in SAP2000 to generate a strain curve reflecting the experimental curves. The frames in SAP2000 were loaded using point loads in the respective lanes noted in the experimental results.

Results for Moments and Strains
For each test truck, moments were gathered from SAP2000 and converted to strain to compare to experimental results. There were two girders chosen for comparison, and the results were compiled into tabular form. Moments at each Girder are noted in Kip-in, and the strains are without unit.

Table 31: Moment and Strain Values for 012296 Test Truck 1

| Distance Traveled (ft) | Moments via SAP (k-in) | Strains via SAP | | |
|------------------------|------------------------|----------------|---|
|                        | Girder 4               | Girder 5       | Girder 4 | Girder 5 |
| Cond. 1                | 0.0                    | 0.0            | 0.0     | 0.0      | 0      |
| Cond. 2                | 76.3                   | 69.5           | 3.3E-06 | 3.0E-06  | 10     |
| Cond. 3                | 211.6                  | 221.0          | 9.2E-06 | 9.6E-06  | 20     |
| Cond. 4                | 229.7                  | 227.8          | 1.0E-06 | 9.9E-06  | 30     |
| Cond. 5                | 343.0                  | 320.3          | 1.5E-05 | 1.4E-05  | 40     |
| Cond. 6                | 678.1                  | 755.4          | 2.9E-05 | 3.3E-05  | 50     |
| Cond. 7                | 397.7                  | 400.4          | 1.7E-05 | 1.7E-05  | 60     |
| Cond. 8                | 145.8                  | 131.3          | 6.3E-06 | 5.7E-06  | 70     |
| Cond. 9                | 258.7                  | 244.2          | 1.1E-05 | 1.1E-05  | 80     |
| Cond. 10               | 643.2                  | 687.9          | 2.8E-05 | 3.0E-05  | 90     |
| Cond. 11               | 585.8                  | 638.9          | 2.5E-05 | 2.8E-05  | 100    |
| Cond. 12               | 176.4                  | 168.6          | 7.6E-06 | 7.3E-06  | 110    |
| Cond. 13               | 16.1                   | 14.6           | 7.0E-07 | 6.3E-07  | 120    |
Table 32: Moment and Strain Values for 012296 Test Truck 3

<table>
<thead>
<tr>
<th>Distance Traveled (ft)</th>
<th>Moments via SAP (k-in)</th>
<th>Strains via SAP</th>
<th>Distance Traveled</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Girder 4</td>
<td>Girder 5</td>
<td>Girder 4</td>
</tr>
<tr>
<td>Cond. 1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Cond. 2</td>
<td>73.1</td>
<td>90.4</td>
<td>3.2E-06</td>
</tr>
<tr>
<td>Cond. 3</td>
<td>221.9</td>
<td>169.6</td>
<td>9.6E-06</td>
</tr>
<tr>
<td>Cond. 4</td>
<td>299.2</td>
<td>292.2</td>
<td>1.3E-05</td>
</tr>
<tr>
<td>Cond. 5</td>
<td>591.7</td>
<td>522.7</td>
<td>2.6E-05</td>
</tr>
<tr>
<td>Cond. 6</td>
<td>761.2</td>
<td>502.6</td>
<td>3.3E-05</td>
</tr>
<tr>
<td>Cond. 7</td>
<td>241.1</td>
<td>261.1</td>
<td>1.0E-05</td>
</tr>
<tr>
<td>Cond. 8</td>
<td>185.5</td>
<td>236.0</td>
<td>8.0E-06</td>
</tr>
<tr>
<td>Cond. 9</td>
<td>561.1</td>
<td>477.3</td>
<td>2.4E-05</td>
</tr>
<tr>
<td>Cond. 10</td>
<td>768.2</td>
<td>513.9</td>
<td>3.3E-05</td>
</tr>
<tr>
<td>Cond. 11</td>
<td>242.0</td>
<td>263.1</td>
<td>1.0E-05</td>
</tr>
<tr>
<td>Cond. 12</td>
<td>15.0</td>
<td>18.7</td>
<td>6.5E-07</td>
</tr>
<tr>
<td>Cond. 13</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Table 33: Moment and Strain Values for 012296 Test Truck 4

<table>
<thead>
<tr>
<th>Distance Traveled (ft)</th>
<th>Moments via SAP (k-in)</th>
<th>Strains via SAP</th>
<th>Distance Traveled</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Girder 4</td>
<td>Girder 5</td>
<td>Girder 4</td>
</tr>
<tr>
<td>Cond. 1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Cond. 2</td>
<td>75.6</td>
<td>93.3</td>
<td>3.3E-06</td>
</tr>
<tr>
<td>Cond. 3</td>
<td>250.1</td>
<td>205.9</td>
<td>1.1E-05</td>
</tr>
<tr>
<td>Cond. 4</td>
<td>397.5</td>
<td>403.7</td>
<td>1.7E-05</td>
</tr>
<tr>
<td>Cond. 5</td>
<td>684.5</td>
<td>515.8</td>
<td>3.0E-05</td>
</tr>
<tr>
<td>Cond. 6</td>
<td>521.5</td>
<td>411.0</td>
<td>2.3E-05</td>
</tr>
<tr>
<td>Cond. 7</td>
<td>225.6</td>
<td>271.4</td>
<td>9.8E-06</td>
</tr>
<tr>
<td>Cond. 8</td>
<td>332.9</td>
<td>333.6</td>
<td>1.4E-05</td>
</tr>
<tr>
<td>Cond. 9</td>
<td>616.1</td>
<td>484.3</td>
<td>2.7E-05</td>
</tr>
<tr>
<td>Cond. 10</td>
<td>481.0</td>
<td>380.6</td>
<td>2.1E-05</td>
</tr>
<tr>
<td>Cond. 11</td>
<td>114.3</td>
<td>127.8</td>
<td>5.0E-06</td>
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<tr>
<td>Cond. 12</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Cond. 13</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>
3.3.6 Verification with Experimental Data

The strains calculated using the moments gathered from SAP were compared with experimental values provided. The graphs for Bridge 012296 differ from the previously analyzed bridges. Since this bridge is located on an Interstate, it can be safely assumed that the truck crosses the bridge at a constant speed.

Although strain values are taken at extremely small time increments, distance traveled across the bridge can be shown assuming a constant speed. The strain data was analyzed to find at what point the truck first approaches the bridge. The point at which the strain values begin to rise in the positive direction can be assumed to be the first sign of the truck on the bridge.

A factor was determined using the number of data points remaining and knowing the total span length of the bridge. This factor was multiplied by the time data to convert into distance. Peak strain values remain the crucial part of calibration, however.

In the graphs below, one will see that there is no plateau for strain values gathered from experimental data. The absence of the strain plateau as seen in the previous bridge examples, further proves that the truck crossed the bridge at a constant speed. Strain values gathered from SAP were graphed versus the distance traveled along the bridge, and these graphs were joined for comparison. Peak strain values are the only values of interest. One will notice that the farther away the girder is from the load concentration, the less accurate the strain values are representative of bridge behavior. Since load rating involves gathering maximum values a certain girder experiences, there will be a high load concentration at that girder. Therefore girder moments used for load ratings are precise.
Figure 77: Strain Comparison – Truck 1, Girder 5

Figure 78: Strain Comparison – Truck 1, Girder 4
Figure 79: Strain Comparison – Truck 3, Girder 6

Figure 80: Strain Comparison – Truck 3, Girder 8

(5.4% Difference)

(45.6% Difference)

Strain ($\varepsilon$) vs. Distance Truck Has Traveled (ft)
Figure 81: Strain Comparison – Truck 4, Girder 6

Figure 82: Strain Comparison – Truck 4, Girder 8

(3.4% Difference)

(41.0% Difference)
3.3.7 Load Rating for Bridge 012296

Prestressed girder bridges are to be rated in accordance with the AASHTO Standard Specification at both the Operating and Inventory Levels. However, the Inventory Level must also consider the allowable stresses of the girder entities. Typically, the minimum reinforcement requirements will be met in a prestressed girder bridge. Unlike standard reinforced concrete, there is no strength reduction factor for moment capacity, and the reduction factor for shear is 0.90.

There were six rating equations used in calculating the ratings at Inventory Level. Four of these equations are service limit rating equations using the Allowable Stress Method, while the other two are the LF rating factors for moment and shear. Concrete Compression is to be rated using separate equations. The first concrete compression rating utilizes the section properties for the full composite member, while the second one only considers the Type II Beam properties. All stresses calculated shall use merely the Type II Beam section properties. The following equations were considered:

Equation 9: ASR Concrete Tension Inventory Rating

\[ RF = \frac{6\sqrt{f'_{c}} - (F_{d} + F_{p} + F_{s})}{F_{l}} \]

Equation 10: ASR Concrete Compression Inventory Rating

\[ RF = \frac{0.6f'_{c} - (F_{d} + F_{p} + F_{s})}{F_{l}} \]

Equation 11: ASR Concrete Compression Inventory Rating

\[ RF = \frac{0.4f'_{c} - \frac{1}{2}(F_{d} + F_{p} + F_{s})}{F_{l}} \]
At the Operating Rating Level for prestressed concrete members, there are only three equations that must be considered. The girder was rated for moment and shear capacity, and the prestressing steel tension rating was also calculated. The following equations were used with the section properties to determine these ratings:

Equation 12: ASR Prestressing Steel Tension Inventory Rating
\[ RF = \frac{0.8f^*{y} - (F_d + F_p + F_s)}{F_t} \]

Equation 13: LFR Flexural Strength Inventory Rating
\[ RF = \frac{\phi M_n - (1.3D + S)}{2.17L(1 + I)} \]

Equation 14: LFR Shear Strength Inventory Rating
\[ RF = \frac{\phi V_n - (1.3D + S)}{2.17L(1 + I)} \]

Equation 15: LFR Flexural Strength Operating Rating
\[ RF = \frac{\phi M_n - (1.3D + S)}{1.3L(1 + I)} \]

Equation 16: LFR Shear Strength Operating Rating
\[ RF = \frac{\phi V_n - (1.3D + S)}{1.3L(1 + I)} \]

Equation 17: LFR Prestressing Steel Tension Operating Rating
\[ RF = \frac{0.9f^*{y} - (F_d + F_p + F_s)}{F_t} \]
The service load stresses were first calculated to ensure that the rating factors for concrete tension, concrete compression, and prestressing steel tension were acceptable. Concrete tension was rated using the stresses at the bottom of the beam, and compression was rated using the stresses at the top of the beam. The prestressing steel service rating factor was calculated using parameters at the bottom of the beam, after converting stresses from concrete at the center of gravity of the prestressing strands. The results for the Allowable Stress Inventory Ratings were as follows:

<table>
<thead>
<tr>
<th>Allowable Stress Inventory Ratings</th>
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<tbody>
<tr>
<td><strong>Entity</strong></td>
</tr>
<tr>
<td>Concrete Tension (Bottom)</td>
</tr>
<tr>
<td>Concrete Compression (Top)</td>
</tr>
<tr>
<td>Concrete Compression (Top)</td>
</tr>
<tr>
<td>Prestressing Steel Tension (Bottom)</td>
</tr>
</tbody>
</table>

Inventory Ratings must also be calculated using strength parameters. The flexural and shear rating for the prestressing girders were considered using the calculated capacities for each. The table below summarizes the ratings for flexure and shear.

<table>
<thead>
<tr>
<th>Strength Inventory Ratings</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Entity</strong></td>
</tr>
<tr>
<td>Flexural Rating for Section</td>
</tr>
<tr>
<td>Shear Rating for Section</td>
</tr>
</tbody>
</table>
Operating Ratings for the prestressed girder bridge were also calculated per the Manual for Bridge Evaluation. These ratings also use the calculated capacities for moment and shear, as shown in the rating equations included above for prestressed girders. Ratings were calculated for flexural, shear, and prestressing steel tension and are included in tabular form below.

Table 36: Strength Inventory Ratings for Bridge 012296

<table>
<thead>
<tr>
<th>Strength Operating Ratings</th>
<th>Entity</th>
<th>Rating Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flexural Rating for Section</td>
<td>2.58</td>
</tr>
<tr>
<td></td>
<td>Shear Rating for Section</td>
<td>3.11</td>
</tr>
<tr>
<td></td>
<td>Prestressing Steel Tension</td>
<td>13.93</td>
</tr>
</tbody>
</table>

The controlling ratings for the section are the minimum Inventory and Operating ratings. Bridge 012296 was rated at 1.54 and 2.58 for Inventory and Operating Levels, respectively. The Flexural Rating for each level was determined to be the controlling rating. There were no ALDOT ratings provided for comparison, but NBI notes the bridge to “[meet] currently acceptable standards”. The calculated ratings do not require bridge posting, so they are considered to be accurate. Parameters used in the calculation of all ratings are included in Appendix B.
4.0 CONCLUSION

Two ALDOT bridges were used to prove that accurate load rating for bridges can be prepared using Finite Element Modeling software. SAP2000, developed by Computers and Structures, Inc. was used to model both bridges. Bridge 005248, a traditionally reinforced concrete girder bridge, and Bridge 005318, a steel girder bridge, were provided for comparison. The bridges were modeled and calibrated to equal experimental strain data provided by ALDOT. After calibration, loads for HS20-44 trucks were applied to the models for load rating values to be compared to ALDOT’s calculations.

At locations of directly affected bridge girders, SAP2000 strain values for Bridge 005248 ranged an average of 3.0% from provided experimental values. As for Bridge 005318, there was a difference of 8.8% between the strain values. It was determined that strain accuracy decreases as girder distance from the concentrated loading increases. However, maximum moment is induced in a girder at concentrated loading, and the bridges were deemed to accurately portray actual bridge behavior with percentage differences below 10%. The controlling ratings calculated for Bridge 005248 at the Inventory and Operating Levels were 30.2 tons and 50.7 tons, respectively. The Inventory Rating for 005318 was 19.8 tons, and the Operating Rating was 32.7 tons.

With two bridges of different girder materials calibrated and proven accurate, a third bridge was modeled and calibrated. Experimental strain data was supplied for Bridge 012296 for calibration purposes. The model for the prestressed girder bridge was
also developed in SAP2000. The strain data was compared to the strains obtained from SAP and the model was considered to imitate actual bridge behavior when loaded, with an average range at directly affected girders of 4.1% from experimental values. Rating calculations were performed to obtain Inventory and Operating Level ratings for the bridge. The HS20-44 ratings at Inventory and Operating Levels were calculated to be 55.4 tons and 92.9 tons, respectively. Therefore, the bridge meets currently acceptable standards, as also noted by the NBI database.

Finite Element Modeling was proven to be an accurate method for bridge load rating. FEM has the distinct ability to distribute loads with precision. Using finite element software can be an economical alternative to load rate bridges accurately, and it also allows for simplicity in revising loads for different rating vehicles. However, limitations to economical benefits include the software training required and the behavior research of other rating trucks. As determined by bridge behavior research for Bridges 005248, 005318, and 012296, Finite Element Modeling can provide a more accurate means for load rating, and bridge postings can be reduced or even removed with such analysis.
REFERENCES


APPENDICES

A: LOADING CONDITIONS FOR SAP MODELS
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<th>Condition</th>
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<th>Truck 1</th>
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</thead>
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<td></td>
<td></td>
</tr>
<tr>
<td>Left Tire Load (k)</td>
<td>Right Tire Load (k)</td>
<td>Distance From Line &quot;0&quot; (ft)</td>
<td>Left Tire Load (k)</td>
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<td>15.075</td>
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<td>15.075</td>
</tr>
<tr>
<td><strong>Axel 2</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Left Tire Load (k)</td>
<td>Right Tire Load (k)</td>
<td>Distance From Line &quot;0&quot; (ft)</td>
<td>Left Tire Load (k)</td>
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### Bridge 005318 Loading Conditions

**Condition 13**

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B: CALCULATION PARAMETERS FOR LOAD RATING BRIDGE 012296
### Input Parameters

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<td>Concrete Compressive Strength of Deck</td>
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<td>-989 psi</td>
<td>Unfactored Dead Load Stress at Bottom</td>
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<td>$F_d$ (Top)</td>
<td>1237 psi</td>
<td>Unfactored Dead Load Stress at Top</td>
</tr>
<tr>
<td>$F_p$ (Bottom)</td>
<td>1814 psi</td>
<td>Unfactored Stress due to Prestress Force After Loss at Bot</td>
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<td>$F_p$ (Top)</td>
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<td>Unfactored Stress due to Prestress Force After Loss at Top</td>
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<td>$F_s$ (T&amp;B)</td>
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<td>Unfactored Stress due to Secondary Prestress Forces T&amp;B</td>
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<td>Unfactored Live Load Stress for Comp. Section Inc. Impact</td>
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### Bridge Parameters

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### Rating Moment and Shear Values via SAP2000

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<tr>
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### Concrete Creep Parameters

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### SAP Moment and Shear Results for 1-Truck Resultant

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### SAP Moment and Shear Results for Dead Load Only

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<td><strong>Longitude:</strong></td>
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<tr>
<td><strong>Material Design:</strong></td>
<td>Steel continuous</td>
<td></td>
</tr>
<tr>
<td><strong>Design Construction:</strong></td>
<td>Stringer/Multi-beam or Girder</td>
<td></td>
</tr>
<tr>
<td><strong>Approach Material Design:</strong></td>
<td>Concrete</td>
<td></td>
</tr>
<tr>
<td><strong>Approach Design Construction:</strong></td>
<td>Tee Beam</td>
<td></td>
</tr>
<tr>
<td><strong>Structure Length (m):</strong></td>
<td>263.7</td>
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<tr>
<td><strong>Approach Roadway Width (m):</strong></td>
<td>7.3</td>
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<tr>
<td><strong>Lanes on Structure:</strong></td>
<td>2</td>
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</tr>
<tr>
<td><strong>Average Daily Traffic:</strong></td>
<td>1020</td>
<td></td>
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<tr>
<td><strong>Year of Average Daily Traffic:</strong></td>
<td>2009</td>
<td></td>
</tr>
<tr>
<td><strong>Design Load:</strong></td>
<td>MS 13.5</td>
<td></td>
</tr>
<tr>
<td><strong>Scour:</strong></td>
<td>Bridge foundations determined to be stable for the assessed or calculated scour condition.</td>
<td></td>
</tr>
<tr>
<td><strong>Bridge Railings:</strong></td>
<td>Do not meet currently acceptable standards.</td>
<td></td>
</tr>
<tr>
<td><strong>Historical Significance:</strong></td>
<td>Bridge is not eligible for the National Register of Historic Places.</td>
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</tr>
<tr>
<td><strong># of Spans in Main Structure:</strong></td>
<td>8</td>
<td></td>
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</tbody>
</table>
# of Spans in Approach Structures: 4
Bridge Median: No Median
Structure Flared: No flare
Transitions: Does not meet currently acceptable standards.
Approach Guardrail: Does not meet currently acceptable standards.
Approach Guardrail Ends: Does not meet currently acceptable standards.

Navigation Control: No Navigation Control on waterway (bridge permit not required).
Structure Open?: Open, no restrictions
Deck: Satisfactory Condition
Superstructure: Satisfactory Condition
Substructure: Satisfactory Condition
Structural Evaluation: Equal to present minimum criteria
Sufficiency Rating (%): 68.4
State: AL
NBI Structure Number: 005318
Route Sign Prefix: State Highway
Route Number: 145
Facility Carried: SR 145
Feature Intersected: WAXAHATCHEE CREEK
Location: CHILTON & SHELBY CO LINE
Year Built: 1955
Status: Functionally Obsolete
Record Type: Roadway is carried ON the structure
Level of Service: Mainline roadway
Owner: State Highway Agency
Highway Agency District: 05
Maintenance Responsibility: State Highway Agency
Functional Class: Minor Arterial, Rural
Service On Bridge: Highway
Service Under Bridge: Waterway
Latitude: 33 02 31.69 N
Longitude: 86 35 32.49 W
Material Design: Steel
Design Construction: Stringer/Multi-beam or Girder
Approach Material Design: Concrete
Approach Design Construction: Tee Beam
Structure Length (m): 86.6
Approach Roadway Width (m): 13.4
Lanes on Structure: 2
Average Daily Traffic: 2460
Year of Average Daily Traffic: 2009
Design Load: M 13.5
Scour: Bridge foundations determined to be stable for the assessed or calculated scour condition.
Bridge Railings: Do not meet currently acceptable standards.
Historical Significance: Bridge is not eligible for the National Register of Historic Places.
# of Spans in Main: 3
Structure:

# of Spans in Approach Structures: 2

Bridge Median: No Median
Structure Flared: No flare
Transitions: Meets currently acceptable standards.
Approach Guardrail: Does not meet currently acceptable standards.
Approach Guardrail Ends: Does not meet currently acceptable standards.
Navigation Control: No Navigation Control on waterway (bridge permit not required).
Structure Open?: Open, no restrictions
Deck: Satisfactory Condition
Superstructure: Satisfactory Condition
Substructure: Satisfactory Condition

Structural Evaluation: Somewhat better than minimum adequacy to tolerate being left in place as is

Sufficiency Rating (%): 50.4
<table>
<thead>
<tr>
<th><strong>State:</strong></th>
<th>AL</th>
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<tbody>
<tr>
<td><strong>NBI Structure Number:</strong></td>
<td>012296</td>
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<tr>
<td><strong>Route Sign Prefix:</strong></td>
<td>Interstate</td>
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<tr>
<td><strong>Route Number:</strong></td>
<td>459</td>
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<tr>
<td><strong>Facility Carried:</strong></td>
<td>I 459</td>
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<td><strong>Feature Intersected:</strong></td>
<td>SULPHUR SPRINGS RD</td>
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<tr>
<td><strong>Location:</strong></td>
<td>I459 &amp; SULPHUR SPRINGS RD</td>
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<tr>
<td><strong>Year Built:</strong></td>
<td>1980</td>
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<td><strong>RecordType:</strong></td>
<td>Roadway is carried ON the structure</td>
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<td><strong>Level of Service:</strong></td>
<td>Mainline roadway</td>
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<tr>
<td><strong>Owner:</strong></td>
<td>State Highway Agency</td>
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<tr>
<td><strong>Highway Agency District:</strong></td>
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<tr>
<td><strong>Maintenance Responsibility:</strong></td>
<td>State Highway Agency</td>
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<tr>
<td><strong>Functional Class:</strong></td>
<td>Principal Arterial - Interstate, Urban</td>
</tr>
<tr>
<td><strong>Service On Bridge:</strong></td>
<td>Highway</td>
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<tr>
<td><strong>Service Under Bridge:</strong></td>
<td>Highway, with or without pedestrian</td>
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<tr>
<td><strong>Latitude:</strong></td>
<td>33 21 34.69 N</td>
</tr>
<tr>
<td><strong>Longitude:</strong></td>
<td>86 50 49.41 W</td>
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<td><strong>Material Design:</strong></td>
<td>Prestressed concrete continuous *</td>
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<td><strong>Design Construction:</strong></td>
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<tr>
<td><strong>Approach Material Design:</strong></td>
<td>Prestressed concrete continuous *</td>
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<tr>
<td><strong>Approach Design Construction:</strong></td>
<td>Stringer/Multi-beam or Girder</td>
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<td><strong>Approach Roadway Width (m):</strong></td>
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<td><strong>Lanes on Structure:</strong></td>
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<td><strong>Lanes under Structure:</strong></td>
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<td><strong>Year of Average Daily Traffic:</strong></td>
<td>2009</td>
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<td><strong>Design Load:</strong></td>
<td>MS 18</td>
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<td><strong>Bridge Railings:</strong></td>
<td>Meet currently acceptable standards.</td>
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<td><strong>Historical Significance:</strong></td>
<td>Bridge is not eligible for the National Register of Historic Places.</td>
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<td><strong># of Spans in Main Structure:</strong></td>
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<tr>
<td><strong># of Spans in Approach Structures:</strong></td>
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<tr>
<td><strong>Bridge Median:</strong></td>
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</tr>
<tr>
<td><strong>StructureFlared:</strong></td>
<td>No flare</td>
</tr>
<tr>
<td><strong>Transitions:</strong></td>
<td>Meets currently acceptable standards.</td>
</tr>
<tr>
<td>-----------------</td>
<td>---------------------------------------</td>
</tr>
<tr>
<td><strong>Approach Guardrail:</strong></td>
<td>Meets currently acceptable standards.</td>
</tr>
<tr>
<td><strong>Approach Guardrail Ends:</strong></td>
<td>Does not meet currently acceptable standards.</td>
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<tr>
<td><strong>Navigation Control:</strong></td>
<td>Not Applicable</td>
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<td><strong>Structure Open?:</strong></td>
<td>Open, no restrictions</td>
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<tr>
<td><strong>Deck:</strong></td>
<td>Good Condition</td>
</tr>
<tr>
<td><strong>Superstructure:</strong></td>
<td>Good Condition</td>
</tr>
<tr>
<td><strong>Substructure:</strong></td>
<td>Good Condition</td>
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<tr>
<td><strong>Structural Evaluation:</strong></td>
<td>Equal to present minimum criteria</td>
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<tr>
<td><strong>Sufficiency Rating (%):</strong></td>
<td>90.9</td>
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