Structural Health Monitoring of
HAN-30-0295 Highway Bridge

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for the
Ohio Department of Transportation
Office of Research and Development

and the
U.S. Department of Transportation
Federal Highway Administration

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**Abstract**

HAN-30-0295 (SFN 3201112) Highway Bridge is located in Hancock County, approximately 4 miles north of Ada, Ohio. The bridge is a two-lane highway bridge carrying State Route OH-235 over US-30. The superstructure is of a deck-and-girder type consisting of five precast prestressed concrete I-girders. The bridge consists of two simple spans of approximately 121 ft, made continuous at the intermediate pier. Cracking of the concrete diaphragms at the intermediate pier location was observed.

The primary objective of the research was to assist the ODOT District One Office in conducting structural health monitoring of the HAN-30-0295 Bridge. Specifically, the objectives were to: (i) Develop an understanding of the possible causes of the problem and the mechanisms at work; (ii) Conduct temperature differential measurements across the depth of the girders; (iii) Conduct a diagnostic load test using a dump truck while monitoring strains in the girders to obtain information on the physical behavior of the structure including continuity for live loads, strength of the girders, fixity at the abutments and distribution of the live load; (iv) Develop an analytical (finite element) model of the bridge; and (v) Use data obtained from objectives (iii) and (iv) to perform load rating of the structure.

The net effect of creep and shrinkage can lead to positive moment at the pier. In addition, positive temperature gradients can create positive moment at the piers. Often these effects are not taken into account explicitly by the designer, as was the case in this particular bridge. The results from this study show that temperature effects alone could create a moment exceeding 1.2M_{cr}, which is sufficient to cause the type of cracking seen. The results from the diagnostic load test show that the beams in service have higher strength than designed. Results also indicate that there is significant continuity developed at the abutments which lowers the maximum positive moment induced in the beams from live load. The recommendation for the studied bridge is that no remedial action is necessary. Recommendations for future bridge designs are to require consideration of thermal and creep and shrinkage effects by design engineers; utilize bond breakers between girders and diaphragms; and delay the pouring of the deck to allow for more differential shrinkage to take place.

**Key Words**

Structural Health Monitoring, Diaphragm Cracking, Prestressed Concrete Girders

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Prepared in cooperation with the Ohio Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration
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1. INTRODUCTION

1.1 Problem Statement

HAN-30-0295 (SFN 3201112) Highway Bridge is located in Hancock County, approximately 4 miles (6 km) north of Ada, Ohio. The bridge is a two-lane highway bridge carrying State Route OH-235 over US-30. The interchange between these two highways is also located at the bridge. In 2004, the estimated total average annual daily traffic (AADT) using the bridge was 4600 [1]. The superstructure is of a deck-and-girder type consisting of five precast prestressed concrete I-girders. The bridge consists of two simple spans of approximately 121-ft (36.8 m), made continuous at the intermediate pier (see Fig. 1).

The bridge falls under the jurisdiction of Ohio Department of Transportation (ODOT) District One. They are responsible for regular inspection and load rating of the bridge. Engineers from the District One Office observed cracking of the concrete diaphragms at the intermediate pier location. They subsequently submitted a request for a research study that would include structural health monitoring of the bridge.

Researchers from Ohio Northern University (ONU) traveled to the field site in July 2006, to conduct a preliminary assessment to assist in developing a research proposal. The team observed severe spalling of the concrete at the bottom of the diaphragm at every interface between the diaphragms and the girders. Large pieces of spalled-off concrete were seen on the ground. On the north face of the diaphragms, rebars had been exposed indicating that the depth of spalling must be 2 to 3 inches (5 to 7.6 cm) in some cases. On the south face of the diaphragms, cracking is also present, but has not yet reached the same severity as the north face. These damage conditions are documented in Fig. 2.

Figure 1 Superstructure of HAN-30-0295 Highway Bridge.
Figure 2 (a) pieces of concrete fallen on the ground; (b) diaphragm cracking present at every girder-diaphragm interface; (c) diaphragm cracking on the south face of the diaphragms; and (d) spalling on the north face.

1.2 Research Objectives

The primary objective of the research is to assist the District One Office in conducting structural health monitoring of the HAN-30-0295 Bridge. Specifically, the objectives are to:

i. Develop an understanding of the possible causes of the problem and the mechanisms at work.

ii. Conduct temperature differential measurements across the depth of the girders, since temperature effects were thought to be a major contributing factor from the outset.

iii. Conduct a diagnostic load test using a dump truck while monitoring strains in the girders. Test results would provide useful information on the physical behavior of the structure including continuity for live loads, strength of the girders, fixity at the abutments and distribution of the live load.
iv. Develop an analytical (finite element) model of the bridge.
v. Use data obtained from objectives (iii) and (iv) to perform load rating of the structure.

2. BACKGROUND AND LITERATURE REVIEW

2.1 Background

Several instances of diaphragm cracking in precast prestressed concrete girder bridges that have been made continuous at the piers have been reported in the literature. The connection details at the interior pier location in such construction continue to be a focus of research. Under gravity loads, such as live loads, negative moments are expected to develop over the pier location. The negative moment connection is usually made through the reinforced concrete deck. Due to temperature, creep and shrinkage effects, however, there is a tendency for positive moments to develop at the pier locations. Without an adequate positive moment connection, cracking might develop.

“Continuous-for-live-load” prestressed/precast concrete bridges are used widely in the United States. Girders started to be made continuous in the early 1960’s. The largest loads these bridges are expected to carry are live loads; therefore, these bridges get the name, “continuous-for-live-load.” The concept is demonstrated in Fig. 3. In constructing a continuous-for-live-load precast prestressed bridge, the girders are installed first and act as simple spans for supporting self-weight and the weight of the deck. The diaphragm and deck may or may not be poured at the same time as the deck. After the deck and diaphragm have hardened the structure behaves as continuous for additional loads including live loads. Maintenance costs associated with deck joints can be avoiding using this design. In addition, the positive moment in the girders is reduced and the seismic performance may be better.

Figure 3: Continuous-for-live-load Connection
2.2 Literature Review

A series of PCA investigations detailed the effects of time-dependent effects of creep and shrinkage. These can be summarized with the aid of Fig. 4 [2]. Creep effects on the parts of the concrete in compression will cause the parts to compress. Since the bottom has larger compression due to the prestress, this leads to positive moment in the connection (Fig. 4 a). Since the girders are older than the deck, the rate of shrinkage of the deck is more than that of the girder causing negative moment at the connection (Fig. 4 b).

![Figure 4: (a) Creep in girder causes positive moment in the connection, and (b) Differential shrinkage cause negative moment in the connection](image)

Barr et al. studied the effects of temperature variations on precast/prestressed concrete girders [3]. They found that service temperature gradients may cause up to 61% increase in bottom tension in continuous bridges. Designers often neglect the thermal effects without serious consequences. Reasons for this may include: (i) allowable tension stress is less than true cracking stress; (ii) conservatism in live load distribution factors and other provisions in AASHTO [4]; and (iii) temporary cracking will occur only under full live load.

A detailed study was conducted by the University of Missouri-Columbia and the University of Missouri-Rolla for the Missouri Department of Transportation to investigate the causes of precast I-girder cracking [5]. They concluded that vertical cracks at girder ends, diaphragm spalling, and girders pulling out of diaphragms were attributed to service temperature loading and continuity detailing used. Options to consider would be to (i) provided unbonded joint between diaphragm and bent beam, (ii) allow for construction joint in midheight in the diaphragm, and (iii) provide bond breakers so the girders may slide freely in and out of the diaphragm. They found that thermal stresses could be on the order of 0.3 to 1.3 times the dead and live load stresses. They recommended that designers should be required to evaluate the thermal effects.
A major research project sponsored by the National Cooperative Highway Research Program [NCHRP] and led by the University of Cincinnati studied different positive moment connections [6]. The positive moment connections studied included (i) bent bars (Fig. 5); (ii) bent bars with beams embedded in diaphragm; (iii) bent prestressing strands (Fig. 6); and (iv) bent strands with beams embedded in diaphragm. They recommended that positive moment connections should be proportioned to resist the moments caused by creep and shrinkage, temperature, and live load effects. Typically these are proportioned to provide 1.2 times the cracking moment, $M_{cr}$. If the analysis shows that more than 1.2 $M_{cr}$ is required, then steps should be taken to decrease the amount of positive moment. The easiest way to do this is to specify a minimum age at the formation of continuity which will allow for some of the girder creep and shrinkage to occur before continuity is created. At present there are no specifications for designing the bent strand connection, however recommendations were made in the report.

![Figure 5: Bent bar connection at diaphragm](image)

![Figure 6: Bent strands connection at diaphragm](image)

### 2.3 Details of Studied Bridge

An examination of the drawings for HAN-30-0295 Bridge shows that negative moment connection is made through deck reinforcement only. For positive moments, bent strand connection is employed. Four prestressing strands are specified to be extended 1ft-4in (40.6 cm) into the diaphragm and bent. This detail is shown in Fig. 7. An examination of the design
computations does not show the determination of the connection capacity or embedment length. Also, the design computations do not appear to take into account the thermal, creep and shrinkage effects. Based on the results of the previous body of work done in this field, this seems to be the major possibility for the damage seen in the diaphragms.

Figure 7: Details of the positive moment connection.

An investigation of the diaphragm to beam connection is shown in Fig. 8. The diaphragm is stopped short of the bent beam as recommended by the Missouri study. As can be seen the six # 5 rebars running through the webs of the beam provide a rigid connection. This connection most likely causes distress of the diaphragm by not allowing the girders to pull away from the diaphragms.

Figure 8: Details of diaphragm-to-beam connection.
3. ANALYTICAL INVESTIGATION

3.1 Methodology

Analytical evaluation of the bridge was conducted with the main purpose of assisting ODOT District One with the load rating of the HAN 30-0295 Bridge, and providing the framework for incorporating the results of the load testing. Maximum dead and live load moments were established, section properties were determined, and load rating was performed for various sections of the bridge. Finite element analysis (FEA) was conducted to determine more accurately the live load distribution effects. The FEA also allowed investigation of the effect of the diaphragm on load carrying capacity of the girders, as well as moments induced in the diaphragms. The load ratings were revised utilizing the results of the FEA.

3.2 Maximum Loads

Dead Loads:

The bridge is assumed to consist of two simply supported spans of 121 ft (36.8 m) for resisting the self-weight of the girders, the slab weight, and the concrete haunch weight as shown in Fig. 9.

The maximum moment was determined for both the 4/10\textsuperscript{th} point (M104) and the midspan (M105). The moment at any distance $x$ can be calculated from:

$$M = \frac{wx}{2} (L - x)$$

For the case of additional loads applied after the diaphragm and deck are poured, the bridge is thought to behave as two-span continuous as shown in Fig. 10. This situation is used to determine the moments for superimposed dead loads such as the barrier weights and future wearing surface loads.
The positive moment was calculated at both the 4/10\textsuperscript{th} point (M104) and midspan (M105). The positive moment at any distance $x$ is calculated from:

$$M = \frac{wx}{16} 7L-8x - \frac{wL}{16} x$$

The negative moment over the pier is calculated as:

$$M = \frac{wL^2}{8}$$

Live Loads:

The primary live load vehicle used for load rating purposes is the AASHTO HS-20 truck. The truck is illustrated schematically in Fig. 11. The front axle is an 8k (36 kN) load with a fixed axle spacing of 14 ft (4.3 m). The middle and rear axle are 32k (144 kN) each with a variable axle spacing of 14-30 ft (4.3-9.1 m); however for all of the cases investigated herein, the 14 ft (9.1 m) spacing controls.

The live load analysis can easily be accomplished with the use of software such as SAP 2000. For illustration, the most critical locations are discussed below. Maximum positive moment due to live load typically occurs around the 4/10\textsuperscript{th} point, whereas maximum positive moment due to dead load occurs around the midspan. Since it is not clear which section actually controls, both points were investigated in this study. If the influence line is plotted for the
moment M104, it is clear that the maximum value will occur when the middle axle is placed at the 4/10\textsuperscript{th} point and the other axles placed as shown in Fig. 12. Once the location of the truck is known, from continuous beam analysis, M104 is found to be 1542 k-ft (2091 kN-m).

![Image](image1.png)

*Figure 12: Influence line and determination of maximum value of M104 due to HS-20 truck.*

Similarly, from Fig. 13, the maximum moment at the midspan, M105 is found to be 1512 k-ft (2050 kN-m).

![Image](image2.png)

*Figure 13: Influence line and determination of maximum value of M105 due to HS-20 truck.*

The maximum negative moment for both dead loads and live loads will occur over the middle pier. As illustrated in Fig. 14, the maximum negative moment over the piers, M200 is found to be -814.8 k-ft (-1105 kN-m).
Live load analysis was also performed for the four Ohio Legal Loads as defined in the Bridge Design Manual [7]. The definition of the Ohio Legal Loads is shown in Fig. 15.

The maximum positive moment (M104) and maximum negative moment (M200) created by the Ohio Legal Loads was determined using SAP 2000 and summarized in Table 1.
Table 1: Maximum positive and negative moments due to various live loads.

<table>
<thead>
<tr>
<th>Load Cases</th>
<th>Max. Positive Moment</th>
<th>Max. Negative Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>OH2F1</td>
<td>845 kip-ft (1146 kN-m)</td>
<td>-415 kip-ft (-563 kN-m)</td>
</tr>
<tr>
<td>OH3F1</td>
<td>1270 kip-ft (1722 kN-m)</td>
<td>-633 kip-ft (-858 kN-m)</td>
</tr>
<tr>
<td>OH4F1</td>
<td>1462 kip-ft (1982 kN-m)</td>
<td>-740 kip-ft (-1003 kN-m)</td>
</tr>
<tr>
<td>OH5C1</td>
<td>1674 kip-ft (2270 kN-m)</td>
<td>-980 kip-ft (-1329 kN-m)</td>
</tr>
<tr>
<td>HS20-44</td>
<td>1542 kip-ft (2091 kN-m)</td>
<td>-815 kip-ft (-1105 kN-m)</td>
</tr>
</tbody>
</table>

3.3 Section Properties

The points investigated for maximum positive moment were the 4/10th point and midspan, and the maximum negative moment was over the pier. For analysis purposes, the relevant cross-sectional properties had to be determined. The prestress harping profile is determined from the drawings and shown in Fig. 16. Since the dimension “C” in the drawing is 24.22 ft (7.4 m), it follows that the prestress profile at the 4/10th point (x = 48.4 ft (14.8 m)) is the same as the profile at midspan (x = 60.5 ft (18.4 m)).

The typical cross-section of the beam at the 4/10th point and midspan is shown in Fig. 17. Some of the relevant dimensions are: overall depth of 59 in (150 cm), top flange width of 48 in (123 cm), bottom flange width of 13 in (33 cm) and bottom flange height of 8 in (20 cm). The prestressing consists of sixty ½ in (1.3 cm) diameter low relaxation strands with an ultimate strength of 270 ksi (1.9 GPa). The spacing between rows of prestressing tendons is 2 in (5 cm).

The positive moment section properties of both the non-composite and composite sections were then determined using a combination of AutoCAD and hand calculations (Appendix B). The composite section properties included the contribution of the deck concrete and reinforcement and the difference in material between the deck and girder. The drawings show the slab thickness to be 8.5 in (21.6 cm); however, design calculations often include a 1 in (2.54 cm) sacrificial layer. For the purpose of this research, a thickness of 8.5 in (21.6 cm) was assumed. After completing the diagnostic load test, the actual composite section properties were found to be higher than expected, and revised properties were calculated. The section properties are given in Fig. 18.

The negative moment section properties of the composite section were determined using hand calculations. These are shown in Fig. 19. It should be noted that a discrepancy was observed. According to the drawings for the bridge, the reinforcement shown over the piers in the deck are 50 #6 bars and 55 #4 bars at the top and 55 #5 bars in the bottom. This gives a total of roughly 9.2 in$^2$ (59.4 cm$^2$) of steel per beam over the effective width. The design computations, however called for a total of 19.2 in$^2$ (123.9 cm$^2$) of steel. This fact becomes significant when performing the load rating as discussed later on.
Figure 16: Prestressing tendon profile.

Figure 17: Typical cross-section of girder at midspan.
b_b = 13 in (33 cm), t_s = 8.5 in (21.6 cm), b_1 = 48 in (121.9 cm), haunch = 2 in (5.1 cm)

<table>
<thead>
<tr>
<th></th>
<th>Non-composite section</th>
<th>Composite section</th>
<th>Composite (based on test)</th>
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<tbody>
<tr>
<td>y_t, in (m)</td>
<td>28.08 (0.71)</td>
<td>13.08 (0.33)</td>
<td>10.34 (0.26)</td>
</tr>
<tr>
<td>y_b, in (m)</td>
<td>30.92 (0.79)</td>
<td>45.92 (1.17)</td>
<td>48.66 (1.24)</td>
</tr>
<tr>
<td>I, in^4 (m^4)</td>
<td>410848 (0.171)</td>
<td>868818 (0.362)</td>
<td>953567 (0.397)</td>
</tr>
<tr>
<td>A, in^2 (m^2)</td>
<td>882 (0.569)</td>
<td>1567 (1.011)</td>
<td>1827 (1.179)</td>
</tr>
<tr>
<td>b_e, in (m)</td>
<td>NA</td>
<td>97 (2.46)</td>
<td>108 (2.74)</td>
</tr>
</tbody>
</table>

Figure 18: Positive moment section properties

<table>
<thead>
<tr>
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<th>Composite section</th>
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<tr>
<td>y_t, in (m)</td>
<td>26.46 (0.67)</td>
</tr>
<tr>
<td>y_b, in (m)</td>
<td>32.54 (0.83)</td>
</tr>
<tr>
<td>I, in^4 (m^4)</td>
<td>460643 (0.192)</td>
</tr>
<tr>
<td>A, in^2 (m^2)</td>
<td>925 (0.597)</td>
</tr>
<tr>
<td>b_e, in (m)</td>
<td>97 (2.46)</td>
</tr>
</tbody>
</table>

Figure 19: Negative moment section properties
3.4 Finite Element Analysis.

The AASHTO distribution factor for live load is widely recognized to be overly conservative. A more realistic distribution was necessary if results were expected to match with any experimental results. More accurate models were obtained using finite element analysis. A finite element model using shell elements to model the deck and beam elements for the girders was implemented. A difficult task to accomplish is to provide continuity between the deck and girders which is often done by adding rigid links. For this reason it was desirable to use the software ETABS. Although this is primarily a program for building analysis and design, it has template models simulating floor slabs attached to floor beams, which is analogous to the bridge deck on girder situation. The cross-section of the girder was drawn in to the program using Section Designer as shown in Fig. 20. Parametric variation of the prestressing tendons is not allowed, hence they were assumed to be straight. The structural deck thickness of 8.5 in (21.6 cm) was specified as well as material properties for both the beam and slab concrete. These included compressive strengths and moduli of elasticity. The diaphragm section was modeled as a rectangular cross section 59 in (150 cm) deep and 9 in (23 cm) wide. The diaphragm and slab were assigned 4500 psi (31 MPa) concrete while the girders were assigned 7000 psi (48 MPa) concrete. Reinforcement was applied at the top and bottom of the slab in both directions using a layered shell element.

![Figure 20: Beam cross-section used for finite element analysis](image)

In order to model the loading from the HS-20 truck, the loads on each wheel were applied as a pressure over the tire contact area. Tire contact areas were determined according to pavement analysis and design procedures [8]. Assuming the tire pressure to be 80 psi (0.55 MPa), for the middle and rear axle, assuming dual wheels as shown in Fig. 21, the contact area is determined from:

\[
A_c = \frac{F}{p} = \frac{8000 \text{lb}}{80 \text{ psi}} = 100 \text{in}^2
\]
Figure 21: Wheel loads for middle and rear axle of HS-20

The dimensions of an equivalent rectangular area for the tire contact area are illustrated in Fig. 22.

Figure 22: Equivalent rectangular tire contact area

The dimension, L can be found from:

\[ L = \sqrt{\frac{A}{0.5227}} = \sqrt{\frac{100}{0.5227}} = 13.83\text{in} \]

This yields a longitudinal dimension of 12.05 in (30.6 cm) and transverse dimension of 8.30 in (21 cm).

\[ 0.8712L = 0.8712 \times 13.83 = 12.05\text{in} \]
\[ 0.6L = 0.6 \times 13.83 = 8.30\text{in} \]

In order to make it simpler to input into the FEA model, it was decided to use an area of 12 in (30.5 cm) by 16 in (40.6 cm) with an applied pressure of 0.0833 ksi (12 ksf) (575 kPa) as depicted in Fig. 23.
Similar calculations are made for the front axle assuming only single wheels.

$$A_c = \frac{F}{p} = \frac{4000 \text{ lb}}{80 \text{ psi}} = 50 \text{ in}^2$$

$$L = \sqrt{\frac{A_c}{0.5227}} = \sqrt{\frac{50}{0.5227}} = 9.78 \text{ in}$$

$$0.8712L = 0.8712 \times 9.78 = 8.52 \text{ in}$$

$$0.6L = 0.6 \times 9.78 = 5.87 \text{ in}$$

For simplification, an area of 8 in (20.3 cm) by 6 in (15.2 cm) was used with an applied pressure of 0.0833 ksi (12 ksf) (575 kPa). This is illustrated in Fig. 24.
Two different transverse positions of the truck were investigated as shown in Fig. 25. The first position T1 placed the truck 2 ft (0.6 m) away from the edge of the barrier. This makes the total distance from the centreline of the bridge to the center of gravity of the truck to be 15 ft (4.6 m). This position tended to create the largest moments on the exterior girder. The second position of the truck T2 was used to simulate the effect of a truck following the designated traffic lanes. The truck was positioned 3 ft (0.9 m) away from the centreline of the bridge, making the distance to the center of gravity to be 6 ft (1.8 m).

![Figure 25: Transverse positioning of trucks for FEA](image)

A comparison was made on the effect of refining the mesh for the bridge deck. A coarser mesh consisting of about 2200 shell elements was compared to a finer mesh with about 3200 elements. The longitudinal position of the truck was to maximize M104 (see Fig. 12). The meshing difference can be seen by comparing Figs. 26 and 27. The location of the trucks can also be seen in the figures as the meshing needed to be done very finely in order to accurately place the tire contact areas. The results from the two simulations are shown in Table 2. The percentage difference in distribution factor is seen to be negligible especially for the most heavily loaded girders; hence the additional computation time needed for the refined mesh was not warranted and all subsequent analyses used the coarser mesh of 2200 elements.

**Table 2: Distribution factors obtained from coarse and fine meshes**

<table>
<thead>
<tr>
<th>Girder</th>
<th>Mid-axle at 104 – Transverse Position T1</th>
<th>Coarse Mesh</th>
<th>Fine Mesh</th>
<th>% change in DF</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M104 (k-ft)</td>
<td>DF</td>
<td>M104 (k-ft)</td>
<td>DF</td>
</tr>
<tr>
<td>G1</td>
<td>592.533</td>
<td>0.387</td>
<td>592.699</td>
<td>0.388</td>
</tr>
<tr>
<td>G2</td>
<td>546.189</td>
<td>0.357</td>
<td>540.146</td>
<td>0.354</td>
</tr>
<tr>
<td>G3</td>
<td>260.878</td>
<td>0.170</td>
<td>264.431</td>
<td>0.173</td>
</tr>
<tr>
<td>G4</td>
<td>112.975</td>
<td>0.074</td>
<td>108.469</td>
<td>0.071</td>
</tr>
<tr>
<td>G5</td>
<td>17.751</td>
<td>0.012</td>
<td>20.738</td>
<td>0.014</td>
</tr>
<tr>
<td>Total</td>
<td>1530.326</td>
<td>1.000</td>
<td>1526.483</td>
<td>1.000</td>
</tr>
</tbody>
</table>
Figure 26: Fine meshing of deck area for FEA

Figure 27: Coarse meshing of deck area for FEA
Figure 28: Deflected shape from truck at transverse position T2 (for M104)

Figure 29: Deflected shape from truck at transverse position T1 (for M104)
Figure 30: Moment diagrams from truck at transverse position T2 (for M104)

Figure 31: Moment diagrams from truck at transverse position T1 (for M104)
Three different loading situations were analyzed. For the case of maximum positive moment at the 4/10\textsuperscript{th} point, the truck was positioned longitudinally to produce the maximum M104 for two different transverse positions T1 and T2. It can be seen from the deflected shapes of Fig. 28 and 29 that the position T1 loads the exterior girder more heavily. Moment diagrams are then obtained for each of the beams. Again the difference between positions T1 and T2 can be seen by comparing the moment diagrams of Figs. 30 and 31. The third loading situation placed the truck longitudinally to maximize the negative moment, M200 over the interior pier (see Fig. 14). This was only done for transverse position T2. The results from all three loading scenarios are summarized in Table 3.

The largest moments occurred on the exterior girder from position T1. Position T2, which is more depictive of actual expected traffic produced the largest distribution factor on an interior girder of 0.327 for M104 and 0.333 for negative moment. For comparison, the AASHTO specified live load distribution factor is 0.82, and hence can be seen to be overly conservative. Errors are produced from such effects as inexact placement of the loading in the FEA, and secondary effects. One way of checking the accuracy of the results is to check the sum of moments for all five beams against the moment that would be produced in a single beam as determined from an exact analysis. In all cases, the percentage accuracy proved to be excellent.

<table>
<thead>
<tr>
<th>Girder</th>
<th>Mid-axle at 104 Transverse position T1</th>
<th>Mid-axle at 104 Transverse position T2</th>
<th>Mid-axle at 106 Transverse position T2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M104 (k-ft)</td>
<td>DF</td>
<td>M104 (k-ft)</td>
</tr>
<tr>
<td>G1</td>
<td>590.340</td>
<td>0.386</td>
<td>338.580</td>
</tr>
<tr>
<td>G2</td>
<td>542.840</td>
<td>0.355</td>
<td>487.580</td>
</tr>
<tr>
<td>G3</td>
<td>264.010</td>
<td>0.173</td>
<td>399.110</td>
</tr>
<tr>
<td>G4</td>
<td>114.080</td>
<td>0.074</td>
<td>204.900</td>
</tr>
<tr>
<td>G5</td>
<td>18.500</td>
<td>0.012</td>
<td>81.630</td>
</tr>
<tr>
<td>Total</td>
<td>1529.770</td>
<td>1.000</td>
<td>1511.800</td>
</tr>
<tr>
<td>Exact M</td>
<td>1541.98</td>
<td></td>
<td>1541.98</td>
</tr>
<tr>
<td>% Accuracy</td>
<td>99.2</td>
<td></td>
<td>98.0</td>
</tr>
</tbody>
</table>

3.5 Effect of Diaphragm

Once the FEA model was established, the stiffness contribution of the diaphragm to the vertical load carrying capacity of the bridge could be investigated in more detail. Four different scenarios were investigated:

(i) Diaphragm rigidly connected to beams. As discussed earlier, the original connection details of the diaphragm to beam could be considered fixed.

(ii) Diaphragm pinned to beams. This was accomplished by releasing the rotational restraint at both ends of the diaphragms. As the concrete begins to spall and crack such as what is exhibited in HAN 30-0295 Bridge, the rotational stiffness of the diaphragm will be decreased probably down to partial fixity. The extreme situation would be for no rotational restraint.
(iii) Diaphragm removed. In this scenario the diaphragm was deleted from the FEA, however the girders and deck were left with their continuity over the middle support. The moment diagrams for M104 and truck position T2 for this situation is shown in Fig. 32.

(iv) Diaphragm removed and no continuity. Finally, the case of diaphragm removed and loss of continuity in the girders but not the deck was analyzed by removing the rotational restraints at the ends of the girders.

Figure 32: Moment diagrams from truck at transverse position T2 (for M104) with diaphragm removed

The results of the first three scenarios of diaphragm stiffness variation are given in Table 4. As can be seen the effect of diaphragm is negligible.

<table>
<thead>
<tr>
<th>Girder</th>
<th>Mid-axle at 104 – Transverse Position T2</th>
<th>Diaphragm rigid connection</th>
<th>Diaphragm pinned</th>
<th>Diaphragm removed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M104 (k-ft)</td>
<td>DF</td>
<td>M104 (k-ft)</td>
<td>DF</td>
</tr>
<tr>
<td>G1</td>
<td>338.580</td>
<td>0.224</td>
<td>338.580</td>
<td>0.224</td>
</tr>
<tr>
<td>G2</td>
<td>487.580</td>
<td>0.323</td>
<td>487.580</td>
<td>0.323</td>
</tr>
<tr>
<td>G3</td>
<td>399.110</td>
<td>0.264</td>
<td>399.170</td>
<td>0.264</td>
</tr>
<tr>
<td>G4</td>
<td>204.900</td>
<td>0.135</td>
<td>204.890</td>
<td>0.135</td>
</tr>
<tr>
<td>G5</td>
<td>81.630</td>
<td>0.054</td>
<td>81.600</td>
<td>0.054</td>
</tr>
<tr>
<td>Total</td>
<td>1511.800</td>
<td>1.000</td>
<td>1511.820</td>
<td>1.000</td>
</tr>
</tbody>
</table>
In addition, the moment diagrams for the diaphragm were plotted for the case of diaphragm rigid end connections and pinned connections. These are shown in Fig. 33. The maximum moment experienced in the case of rigid connections was about 6 k-ft (8.1 kN-m) which was reduced to less than 4 k-ft (5.4 kN-m) for the pinned case. In either case, the diaphragm reinforcement should be sufficient to handle these moments without damage.

Figure 33: Moment diagrams for diaphragm sections with (a) rigid connections, and (b) pinned connections.
3.6 Load Ratings

Load rating was performed for maximum positive moment at midspan and 4/10th point and maximum negative moment at the pier. The detailed calculations are given in Appendix A. The procedures of [9] were followed. For the live loading, the distribution factor was taken to be the AASHTO distribution factor of 0.82. The rating vehicle was the HS-20. The results are summarized in Table 5. The inventory rating corresponds to the safe load that the bridge can carry without any damage; while the operational rating represents the maximum allowable live load. Note that for the case of negative moment M200, a discrepancy was observed between the design calculations and the drawings for the bridge. The drawings show only 9.2 in² (59 cm²) of steel for negative moment over the piers while the design computations use 19.2 in² (124 cm²). As the calculations show, if the amount of steel is only 10 in² (64.5 cm²) then the bridge inventory rating is less than HS-20. Load rating was also done assuming no continuity at the piers, that is, two simply supported spans. This situation would be relevant for example if removal and/or replacement of the diaphragm were ever to be considered. As can be seen in this situation, the allowable stress in tension would be exceeded.

Table 5. Load rating of HAN 30-0295 Bridge using AASHTO distribution factors

<table>
<thead>
<tr>
<th>Method</th>
<th>M105</th>
<th>M104</th>
<th>M200 (w/ 9.2 in² of steel)</th>
<th>M200 (w/ 19.2 in² of steel)</th>
<th>M105 (assume no continuity)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored Load</td>
<td>RF&lt;sub&gt;IN&lt;/sub&gt;</td>
<td>2.21</td>
<td>2.22</td>
<td>0.7</td>
<td>2.08</td>
</tr>
<tr>
<td></td>
<td>RF&lt;sub&gt;OP&lt;/sub&gt;</td>
<td>3.69</td>
<td>3.63</td>
<td>1.16</td>
<td>3.48</td>
</tr>
<tr>
<td>Allowable Stress</td>
<td>RF&lt;sub&gt;IN&lt;/sub&gt;</td>
<td>1.05</td>
<td>1.07</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The rating factors above are conservative because they use the AASHTO distribution factors and theoretical section properties. The load ratings were recalculated using the results of the FEA and the section properties calculated from the experiments. The results are given in Table 6. All of the ratings are seen to be satisfactory in this case, including the case of only 9.2 in² of steel for negative moment.

Table 6. Load rating of HAN 30-0295 Bridge using FEA distribution factors

<table>
<thead>
<tr>
<th>Method</th>
<th>M105</th>
<th>M104</th>
<th>M200 (w/ 9.2 in² of steel)</th>
<th>M200 (w/ 19.2 in² of steel)</th>
<th>M105 (assume no continuity)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored Load</td>
<td>LL DF</td>
<td>0.327</td>
<td>0.327</td>
<td>0.333</td>
<td>0.333</td>
</tr>
<tr>
<td></td>
<td>RF&lt;sub&gt;IN&lt;/sub&gt;</td>
<td>5.55</td>
<td>5.45</td>
<td>1.75</td>
<td>5.23</td>
</tr>
<tr>
<td></td>
<td>RF&lt;sub&gt;OP&lt;/sub&gt;</td>
<td>9.26</td>
<td>9.10</td>
<td>2.92</td>
<td>8.73</td>
</tr>
</tbody>
</table>

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4. TEMPERATURE, AND CREEP AND SHRINKAGE EFFECTS

4.1 Creep and Shrinkage

An analysis of creep and shrinkage effects was performed with the aid of the software RMCalc available from the Washington State Department of Transportation Bridge Software website. The development of the program is outlined in NCHRP Report 322 [10] and [11]. The program is based on the Construction Technologies Laboratories (CTL) method. Over time, the continuity of a continuous bridge may be reduced by the effect of creep in the girders. The girder concrete creeps under the prestressing force causing the girder to deflect upwards. This creates positive moment at the pier connection.

Fortunately, creep effects are offset by differential shrinkage between the girder concrete and deck concrete. The deck concrete is younger than the girder concrete. When the deck is poured, the girder concrete has already had time to shrink, whereas the shrinkage of the deck concrete has yet to occur. On a continuous bridge this results in negative restraint moment at the pier.

Certain assumptions were made in obtaining the results from RMCalc. The relative humidity was 77%. For the concrete deck, loading age was 7 days, volume-to-surface ratio was 7.5 in (19 cm), concrete slump was 4 in (10 cm), cement content was 600 lb/yd$^3$ (356 kg/m$^3$), and air content was 2%. For the girders, the loading age was 21 days, volume-to-surface (area-to-perimeter) ratio was $882/215 = 4.1$ in (10.4 cm), concrete slump was 5 in (13 cm), cement content was 658 lb/yd$^3$ (390 kg/m$^3$), and air content was 2%. Additional dead load of 4.7 psf (225 Pa) was applied for the barriers, girder age at prestress release was 21 days, girder age at continuity and time deck is in place was 28 days.

The results are shown in Fig. 34. Calculations were performed up to 1825 days (5 years) time. A positive restraint moment of 1233 k-ft (1671 kN-m) was found for the pier connection.

In general, the older a girder is before continuity is established, the lower the positive restraint moments will be, because less creep and less shrinkage remain to develop in the girder. Less remaining creep results in lower positive restraint moments due to creep. Less remaining girder shrinkage results in larger differential shrinkage between the deck concrete and girder concrete, which translates to larger negative restraint moments due to shrinkage. The combined effect is a lower positive restraint moment (less positive or more negative), which leads to greater continuity.
4.2 AASHTO Temperature Gradient

In a continuous bridge, a positive temperature gradient across the depth of the girder will cause an upward deflection resulting in positive moment over the pier. The AASHTO specified temperature gradient \([4]\) was applied to the composite section. Fig. 35 shows half of the cross-section of a girder and the AASHTO temperature gradient across the depth.

Consider a beam restrained at both ends and subjected to a positive temperature gradient across the depth. The beam will have the tendency to deflect upwards and positive moment will be created over the pier. The strain across the depth is given by

\[
\varepsilon_y = \alpha T y
\]

Assuming linear elastic behaviour, the stress can be related by Hooke’s Law to strain:

\[
\sigma = E \varepsilon_y = E \alpha T y
\]
Figure 35. AASHTO temperature gradient applied over depth of composite beam.

Table 7. Formulas for areas of sections of the beam and AASHTO temperature gradient

<table>
<thead>
<tr>
<th>Y Range</th>
<th>Area</th>
<th>Temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-8</td>
<td>( A = 2 \int_{0}^{12.5} dy )</td>
<td>( T = 0 )</td>
</tr>
<tr>
<td>8-17</td>
<td>( A = 2 \int_{8}^{20.5} dy )</td>
<td>( T = 0 )</td>
</tr>
<tr>
<td>17-49</td>
<td>( A = 2 \int_{17}^{3.5} dy )</td>
<td>( T = 0 )</td>
</tr>
<tr>
<td>49-52</td>
<td>( A = 2 \int_{49}^{y-45.5} dy )</td>
<td>( T = 0 )</td>
</tr>
<tr>
<td>52-52.5</td>
<td>( A = 2 \int_{52}^{y-45.5} \left( \frac{35}{6} y - \frac{1775}{6} \right) dy )</td>
<td>( T = 0 )</td>
</tr>
<tr>
<td>52.5-55</td>
<td>( A = 2 \int_{52.5}^{y-45.5} \left( \frac{35}{6} y - \frac{1775}{6} \right) dy )</td>
<td>( T = \frac{11}{12} y - \frac{385}{8} )</td>
</tr>
<tr>
<td>55-59</td>
<td>( A = 2 \int_{55}^{24} dy )</td>
<td>( T = \frac{11}{12} y - \frac{385}{8} )</td>
</tr>
<tr>
<td>59-61</td>
<td>( A = 0 )</td>
<td>( T = \frac{11}{12} y - \frac{385}{8} )</td>
</tr>
<tr>
<td>61-64.5</td>
<td>( A = 2 \int_{61}^{38.9} dy )</td>
<td>( T = \frac{11}{12} y - \frac{385}{8} )</td>
</tr>
<tr>
<td>64.5-69.5</td>
<td>( A = 2 \int_{64.5}^{38.9} dy )</td>
<td>( T = 6 y - 376 )</td>
</tr>
</tbody>
</table>
The force acting on a differential element, \( dA \) on the beam is

\[
dF = \sigma dA
\]

And the moment created by this differential element is

\[
dM = y\sigma dA
\]

Therefore the total thermal moment is found from

\[
M_t = \int y\sigma dA
\]

Table 7 summarizes the equations for areas of different sections of the composite beam, together with the corresponding equation for the AASHTO temperature gradient. For the beam with AASHTO temperature gradient this yields a total thermal moment given by

\[
M_t = 2E\alpha \left[ \int_{0.255}^{0.55} y \left( \frac{35}{6} y - \frac{1775}{6} \right) dy + \int_{0.55}^{0.69} y \left( 24 \left( \frac{11}{12} y - \frac{385}{8} \right) - 6y - 376 \right) dy \right]
\]

Assuming \( E = 5072 \text{ ksi} \) (35 GPa) and \( \alpha = 5.7 \times 10^{-6} \text{/F} \) (10x10^{-6} \text{/C}), this results in \( M_t = 25,874 \text{ k-in or 2156 k-ft (2915 kN-m)} \). Moment distribution on a two span continuous bridge shows the final end moment at the pier to be 1.5\( M_t \), however, based on the results of the diagnostic load test, the bridge is found to behave as if it were fixed at the ends. Thus, the final end moment is taken to be \( M_t = 2156 \text{ k-ft (2915 kN-m)} \).

**4.3 Temperature Gradient from Field Test**

Temperature profiles across both an interior and an exterior girder were obtained over the course of two typical summer days (74°F (23°C) and 80°F (27°C)). The largest temperature differential occurred in the afternoon. The more critical temperature profile was captured on the exterior girder. The obtained critical temperature gradient for the exterior girder is shown in Fig. 36. It can be seen that the lower half of the beam actually experiences slight negative temperature gradient. The overall thermal moment obtained is not as severe as the AASHTO profile. The equations for temperature gradient and areas are displayed in Table 8. Utilizing the same procedure as described earlier, the maximum thermal moment \( M_t = 1478 \text{ k-ft (2004 kN-m)} \).

The creep and shrinkage moment after 5 years as discussed in the previous section was estimated at 1233 k-ft (1671 kN-m). The moment due to temperature differential was 1478 k-ft (2004 kN-m), giving a total of 2711 k-ft (3676 kN-m). The barrier loads (0.188 k/ft) (14.6 N/mm) provide a permanent opposing moment of 344 k-ft (466 kN-m) at the pier, thus the total positive moment at the pier from creep plus critical temperature gradient may be as high as 2367 k-ft (3209 kN-m).
Figure 36. Recorded critical temperature gradient over depth of exterior beam.

Table 8. Formulas for areas of sections of the beam and recorded temperature gradient

<table>
<thead>
<tr>
<th>Y Range</th>
<th>Area</th>
<th>Temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-8</td>
<td>$A = 2 \int_{0}^{8} 12.5 , dy$</td>
<td>$T = - \left( \frac{-2.106}{29.5} y + 2.106 \right)$</td>
</tr>
<tr>
<td>8-17</td>
<td>$A = 2 \int_{8}^{17} 20.5 - y , dy$</td>
<td>$T = - \left( \frac{-2.106}{29.5} y + 2.106 \right)$</td>
</tr>
<tr>
<td>17-29.5</td>
<td>$A = 2 \int_{17}^{29.5} 3.5 , dy$</td>
<td>$T = - \left( \frac{-2.106}{29.5} y + 2.106 \right)$</td>
</tr>
<tr>
<td>29.5-49</td>
<td>$A = 2 \int_{29.5}^{49} 3.5 , dy$</td>
<td>$T = \frac{0.875}{24.5} y - 1.053571$</td>
</tr>
<tr>
<td>49-52</td>
<td>$A = 2 \int_{49}^{52} y - 45.5 , dy$</td>
<td>$T = \frac{0.875}{24.5} y - 1.053571$</td>
</tr>
<tr>
<td>52-54</td>
<td>$A = 2 \int_{52}^{54} \frac{35}{6} y - \frac{1775}{6} , dy$</td>
<td>$T = \frac{0.875}{24.5} y - 1.053571$</td>
</tr>
<tr>
<td>54-55</td>
<td>$A = 2 \int_{54}^{55} \frac{35}{6} y - \frac{1775}{6} , dy$</td>
<td>$T = \frac{0.869}{7} y - 5.82871$</td>
</tr>
<tr>
<td>55-59</td>
<td>$A = 2 \int_{55}^{59} 24 , dy$</td>
<td>$T = \frac{0.869}{7} y - 5.82871$</td>
</tr>
<tr>
<td>59-61</td>
<td>$A = 0$</td>
<td>$T = \frac{0.869}{7} y - 5.82871$</td>
</tr>
<tr>
<td>61-69.5</td>
<td>$A = 2 \int_{61}^{69.5} 38.9 , dy$</td>
<td>$T = \frac{23.637}{8.5} y - 167.886$</td>
</tr>
</tbody>
</table>
The cracking moment can be calculated over the piers. For the negative moment section, the distance from the centroid to the bottom of the girder is 32.54 in (82.7 cm) and to the top of the deck is 36.96 in (93.9 cm), therefore the top controls. The rupture modulus is calculated from

\[ f_r = 7.5 \sqrt{f_c'} = 7.5\sqrt{4500} = 503 \text{ psi} \]

The cracking moment is then calculated as

\[ M_{cr} = f_r \frac{I_g}{y_i} \frac{0.503}{36.96} = 6269 \text{ k-in} = 522 \text{ k-ft} \]

The magnitude of \( 1.2M_{cr} \) therefore is 627 k-ft (850 kN-m). The total positive moment from creep and shrinkage and temperature effects may be as high as 2367 k-ft (3209 kN-m) and this significantly exceeds \( 1.2M_{cr} \) therefore it is very likely that this may lead to the cracking of the diaphragm that is observed. It is impractical to design a positive moment connection to resist this amount of positive moment. Steps taken to reduce the amount of positive moment are more likely to be effective. Even if continuity is totally lost in the bridge, that is two simple spans are created, the beams may be found adequate to support the live load.

**5. FIELD TESTING AND RESULTS**

**5.1 Setup**

Field testing was performed on August 25, 2008. Temperature sensors were left in place and removed after 24 hours. Ohio Department of Transportation District One provided a loaded dump truck and traffic control to close SR 235 during the test. Access to the bottom of the girders was obtained using a scissor lift from off the shoulder and no closure of US 30 was required see Figs. 37 and 38.

Eight strain transducers with cable lengths of 15 to 20 ft (4.6 to 6.1 m) length were attached to two wireless battery-powered nodes. These nodes in turn communicated wirelessly which in turn communicated with a laptop computer. The longitudinal and transverse positions of the truck were noted. The truck moved at crawl speed (5 mph or 440 ft/min (2.2 m/s)). The data collection rate was maintained at 30 Hz. Gauges were applied to the members using super glue on tabs. The wireless nodes must be mounted on the cross frames using clamps. A total of eight temperature sensors were utilized to record temperature profiles over a 24-hr period. The sensors collected data every 15 minutes. The relative locations of the strain gauges (e.g. B1580) and temperature sensors (e.g. T6) are shown in Fig. 39.
Figure 37. Installation of strain gauges and temperature sensors.

Figure 38. Diagnostic load test with loaded dump truck.
A three-axle dump truck with the configuration shown in Fig. 40 with a total weight of 23.75 tons (211 kN) was utilized. With the middle axle at 104 and the truck in the same direction as the HS-20, the maximum moment was expected to be 1055 k-ft (1430 kN-m). This compares to 1542 k-ft (2091 kN-m) produced by an HS-20 truck. That is, the dump truck would produce about 68.4% of the moment created by an HS-20 truck.
5.2 Results

The results obtained from the temperature sensors between 1:30 pm and 1:30 pm the following day are shown in Figs. 41 and 42. The peak temperature differentials were recorded around 1:30 pm each afternoon. The corresponding ambient temperatures were 74°F (23°C) and 80°F (27°C). The larger temperature differential occurred on the second day, and the gradient for the exterior girder was slightly more critical than that on the interior girder, thus this gradient was used for analysis. The details of the analysis are given in section 4.3.

*Figure 41. Temperatures recorded across exterior girder G1.*
The strain measurements obtained from all girders is shown in Figure 43. Some amount of data manipulation was required. It was observed that several of the gauges exhibited linear drift due to temperature effects because of the large difference in thermal inertias. This was obvious from the fact that the gauges showed readings even when the truck was not on the bridge, and did not return to zero at the end of the test. The data were corrected by subtracting out the amount of drift. Secondly, fluctuations in readings made it difficult in some cases to determine the peaks exactly. Thus, a running average of 21 data points was calculated to smooth out the curves. The resulting strain diagrams are shown in Fig. 44.

In order to illustrate the behaviour in detail, the strain readings obtained over exterior girder G1 are shown in Fig. 45. The maximum negative moment at M104 occurred exactly when the middle axle was at the midpoint of the second span i.e. M205 as expected. This fact clearly indicates that the bridge is behaving as continuous for live load. The maximum positive moment occurred exactly when the middle axle was over the 4/10th point as expected. From this peak point the neutral axis location could be determined for girders G1, G2, and G3. The results are given in Table 9.
Figure 43. Strains recorded during diagnostic load test.
Figure 44. Strains corrected for drift and averaged.
The average neutral axis location for the three girders was 48.88 in (124.2 cm). This is shifted upwards from the theoretical neutral axis of 45.92 in (116.6 cm). In other words, the beams are stronger than predicted. The contribution to composite action from the deck is larger than the theoretical. Possible reasons for this may include higher compressive strengths in the deck, thicker deck, and larger effective width. An estimated moment of inertia to match more closely with experimental results was calculated by assuming the deck strength to be 7000 psi (48.3 MPa) instead of 4500 psi (31 MPa) and effective width to be 108 in (2.7 m) instead of 97 in (2.5 m). This yields a neutral axis location of 48.66 in (123.6 cm) and a moment of inertia of 953,567 in⁴ (0.372 m⁴). This is an increase over the theoretical moment of inertia of 868,818 in⁴ (0.362 m⁴).

The amount of moment in each beam was then calculated using the maximum tensile strain. The strain at a distance $y$ from the neutral axis is related to the moment by

$$\text{Strain} = \frac{M}{I} y$$

where $M$ is the moment and $I$ is the moment of inertia.
This equation can be used to calculate moment, assuming $E = 5072$ ksi (35 GPa), $I = 953,567$ in$^4$ (0.372 m$^4$), and the distance to the centroid from the bottom to be 48.85 in (124 cm). The results are shown in Table 10. The distribution factors were also calculated and compared to those obtained from the finite element analysis.

### Table 10. Calculated Moments and Distribution Factors.

<table>
<thead>
<tr>
<th>Girder</th>
<th>Max tensile strain (με)</th>
<th>M104, k-ft (kN-m)</th>
<th>DF</th>
<th>FEA DF</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>15.61458</td>
<td>128.8 (174.6)</td>
<td>0.235</td>
<td>0.224</td>
</tr>
<tr>
<td>G2</td>
<td>17.13679</td>
<td>141.4 (191.7)</td>
<td>0.258</td>
<td>0.323</td>
</tr>
<tr>
<td>G3</td>
<td>19.83722</td>
<td>163.7 (221.9)</td>
<td>0.298</td>
<td>0.264</td>
</tr>
<tr>
<td>G4</td>
<td>9.393863</td>
<td>77.5 (105.1)</td>
<td>0.141</td>
<td>0.135</td>
</tr>
<tr>
<td>G5</td>
<td>4.515093</td>
<td>37.3 (50.6)</td>
<td>0.068</td>
<td>0.054</td>
</tr>
</tbody>
</table>

Another very useful bit of information can be obtained from the results in Table 10. The sum of the moments from all the girders gives a total moment of 548.7 k-ft (744 kN-m). The theoretical moment from the dump truck if the ends of the bridge were pinned would be 1055.1 k-ft (1430 kN-m). If the ends are fixed however, the theoretical moment would be 568.8 k-ft (771 kN-m). Thus, it is quite clear (96.5% accuracy) that the abutments are acting like fixed restraints rather than pinned which means the maximum positive moment is reduced.

### 6. CONCLUSIONS AND RECOMMENDATIONS

It is believed that the reason for the diaphragm cracking is excessive positive moment developed over the pier, and the failure to account for these moments in the design of the bridge. An estimated positive creep and shrinkage moment after 5 years of 1233 k-ft (1671 kN-m) was obtained. The moment due to measured temperature differential on a typical warm sunny day was 1478 k-ft (2004 kN-m), giving a total of 2711 k-ft (3675 kN-m). The barrier loads (0.188 k/ft) (14.6 N/mm) provide a permanent opposing moment of 344 k-ft (466 kN-m) at the pier, thus the total net positive moment at the pier from creep plus critical temperature gradient may be as high as 2367 k-ft (3209 kN-m). This significantly exceeds the magnitude of $1.2M_{cr}$ which is 627 k-ft (850 kN-m). Since the diaphragms are rigidly connected to the beams they may be expected to experience some cracking from these effects.

Results from the experimental load testing indicate that (i) there is continuity across the middle pier for live loads, (ii) the composite beams are stronger than theoretical (moment of inertia of 953,567 in$^4$ (0.397 m$^4$) versus 868,818 in$^4$ (0.362 m$^4$)), and (iii) the bridge behaves as if the connections to the abutments are fixed rather than pinned as assumed in the design, considerably lowering the amount of positive moment within the beams.

Results from the finite element analysis indicate that the bridge could continue to adequately support live loads even if the diaphragm became severely damaged (changing from rigid to flexible connection or even being removed completely). Load rating results were all...
satisfactory, even in the case of loss of continuity, that is, the bridge behaving as two simple spans.

Since the diaphragm cracking does not seem to present any serious structural safety concern, no remedial action is recommended. Recommendations for future bridge designs are to require consideration of thermal and creep and shrinkage effects by design engineers; utilize bond breakers between girders and diaphragms; use older age girders so that much of the creep and shrinkage will have already taken place; and delay the pouring of the deck to allow for more differential shrinkage to take place.
7. REFERENCES

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APPENDIX A

Load Rating Procedure
## Materials and Conditions

<table>
<thead>
<tr>
<th>Given</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of traffic lanes</td>
<td>2</td>
</tr>
<tr>
<td>Span Length, L</td>
<td>121 ft</td>
</tr>
<tr>
<td>Bridge Width</td>
<td>43 ft</td>
</tr>
<tr>
<td>Structural slab thickness, t_s</td>
<td>7.5   in.</td>
</tr>
<tr>
<td>Total slab thickness</td>
<td>8.5   in.</td>
</tr>
<tr>
<td>Future wearing surface</td>
<td>60 psf</td>
</tr>
<tr>
<td>Parapet weight</td>
<td>470 plf</td>
</tr>
<tr>
<td>Specified concrete strength of beam, f_c'</td>
<td>7 ksi</td>
</tr>
<tr>
<td>Modulus of elasticity of beam concrete, E_c</td>
<td>5072.2 ksi</td>
</tr>
<tr>
<td>Specified concrete strength at transfer(beam), f_c'</td>
<td>6300 psi</td>
</tr>
<tr>
<td>Modulus of elasticity of beam concrete at transfer, E_c</td>
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<td>Specified concrete strength of deck, f_c'</td>
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<td>Modulus of elasticity of deck concrete, E_c</td>
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<td>Unit weight of concrete for beams and deck, w_c</td>
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</tr>
<tr>
<td>Allowable tensile stress at service (midspan)</td>
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</tr>
<tr>
<td>Prestressing strand strength, f_s'</td>
<td>270 ksi</td>
</tr>
<tr>
<td>Modulus of elasticity of strand, E_s</td>
<td>29000 ksi</td>
</tr>
<tr>
<td>Area of .5 in. dia. Prestressing strand</td>
<td>0.167 in.²</td>
</tr>
<tr>
<td>Initial prestress, f_{si}</td>
<td>202.5 ksi</td>
</tr>
<tr>
<td>Initial prestress force/strand, R_j</td>
<td>33.82 kips</td>
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<td>b_e</td>
<td>8.08 ft</td>
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<tr>
<td>b_t</td>
<td>4 ft</td>
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<tr>
<td>slab thickness</td>
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<td>haunch thickness</td>
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<td># of barriers</td>
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<tr>
<td># of beams</td>
<td>5</td>
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<tr>
<td>Unit weight of future wearing surface</td>
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<td>RH</td>
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<td># of prestressing strands</td>
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<td></td>
<td>Non-composite section</td>
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<td>-------------------------</td>
<td>------------------------</td>
</tr>
<tr>
<td>$y_c$, in</td>
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<tr>
<td>$y_{bc}$, in</td>
<td>30.92</td>
</tr>
<tr>
<td>$I$, in$^4$</td>
<td>410848</td>
</tr>
<tr>
<td>$A$, in$^2$</td>
<td>882</td>
</tr>
</tbody>
</table>
Positive Moment (M105 – midspan)

Beam Self Weight Moment

\[ M_s = \frac{wL^2}{8} = \frac{A_{\text{concrete}}L^2}{8} = \left( \frac{\text{882in}^2}{\text{144in}^2} \times \frac{\text{ft}^2}{\text{150pcf} \times 12^2 \text{ft}^2} \right) \times \text{1681.43k-ft} \]

Slab and Haunch Moment

\[ M_s = \frac{wL^2}{8} = \frac{9.0 \text{ft} \times \left( 8.5 \text{in} \times \frac{\text{ft}}{12\text{in}} \right) + 4 \text{ft} \times \left( 2\text{in} \times \frac{1\text{ft}}{12\text{in}} \right)}{8} \times \text{1933.07k-ft} \]

Barrier Moment

\[ \frac{470\text{lb} \times 2}{w = \frac{\text{ft}}{5}} = 0.188 \text{k/ft} \]

\[ M_b = \frac{wx}{16} (7L - 8x) - \frac{wL}{16} x = \frac{0.188 \times 60.5}{16} (7 \times 121 - 8 \times 60.5) - \frac{0.188 \times 121}{16} \times 60.5 = 172 \text{k-ft} \]

Future Wearing Surface

\[ w = \frac{0.06\text{ksf} \times 40 \text{ft}}{5} = 0.48 \text{k/ft} \]

\[ M_{ws} = \frac{wx}{16} (7L - 8x) - \frac{wL}{16} x = \frac{0.48 \times 60.5}{16} (7 \times 121 - 8 \times 60.5) - \frac{0.48 \times 121}{16} \times 60.5 = 439 \text{k-ft} \]

Total Dead Load

\[ M_d = 1681.43 + 1933.07 + 172 + 439 = 4225.5 \text{k-ft} \]

Live Load Positive Moment

\[ I = \frac{50}{125 + L} \leq 0.3 \quad \text{[STD 3-1]} \]

\[ I = \frac{50}{125 + 121} = 0.20 \]
Maximum Wheel Load Moment (HS-20)

\[ M_{WL-HS-20} = \frac{1512}{2} = 756k - ft \]

AASHTO Wheel Load Distribution Factor

\[ WDF = \frac{S}{5.5} = \frac{9}{5.5} = 1.64 \quad [STD \ Table \ 3.23.1] \]

Live Load Moment per Beam

\[ M_{LL,I} = WDF \times M_{WL-HS-20} (1 + I) = 1.64 \times 756 \times 1.2 = 1487.8k - ft \]

Pre-Stress Losses

Initial Prestress Force Immediately After Transfer \quad [STD \ 9.16.2.1.2]

\[ P_i = 60 \times 0.1670in^2 \times 270 \frac{kips}{in^2} \times 0.69 = 1867kips \]

Eccentricity of Prestress Force

\[ e = y_b - y_{bs} = 30.92 - 7.47 = 23.45in \]

\[ y_{bs} = \frac{\bar{A}}{A} \sum \frac{P_i e^2}{I} - M_g \frac{e}{I} \]

\[ f_{cir} = \frac{1867kips}{882in^2} + \frac{1867kips \times 23.45in^2}{410848in^4} - \frac{1681.43k - ft \times 12 \frac{in}{ft} \times 23.45in}{410848in^4} = +3.46ksi \ (C) \]

\[ f_{cds} = \frac{-M_s e}{I} - \frac{(M_b + M_{bs})(y_{bc} - y_{bs})}{I_c} \]

\[ f_{cds} = \frac{-1933.07k - ft \times 23.45 \times 12}{410848in^4} - \frac{172 + 439 \times 12 \times 45.92 - 7.47}{868818} = -1.35ksi \ (T) \]
Elastic Shortening Loss

\[ ES = \frac{E_x \cdot f_{cir}}{E_{ci}} = \frac{29000 ksi \cdot 3.46 ksi}{4812 ksi} = 20.85 ksi \]

Shrinkage Loss (assume RH=70%)

\[ SH = 17 - 0.15 \cdot RH = 17 - (0.15 \cdot 70) = 6.5 ksi \]

Creep Loss

\[ CR_c = 12 \cdot f_{cir} - 7 \cdot f_{cds} = 12 \cdot 3.46 \cdot 7 \cdot 1.35 = 32.07 ksi \]

Relaxation Loss

\[ CR_r = 5 - 0.1 \cdot ES - \left[ 0.05 \cdot SH + CR_c \right] = 5 - 0.1 \cdot 20.85 ksi - \left[ 0.05 \cdot 6.5 ksi + 32.07 ksi \right] = 0.99 ksi \]

Total Prestress Losses

\[ ES + SH + CR_c + CR_r = 20.85 + 6.5 + 32.07 + 0.99 = 60.41 ksi \]

Effective Final Prestress

\[ f_{se} = 0.75 \cdot 270 - 60.41 = 142.09 ksi \]

Effective Final Prestress Force

\[ P_{se} = 60 \cdot 1.167 \cdot 142.09 = 1424 kips \]

Stresses and Strengths

Total Tensile Stress at Service-Inventory Rating

Dead load Stress on Non-Composite section

\[ f_{NDL} = - \frac{M_g + M_s \cdot y_b}{I} = \frac{-1681.43 k - ft + 1933.07 k - ft \cdot 30.92 in \cdot 12}{410848 in^4} = -3.26 ksi \quad (T) \]

Dead Load Stress on Composite Section

\[ f_{CDL} = - \frac{M_b + M_{wc} \cdot y_{bc}}{I_c} = \frac{-172 k - ft + 439 k - ft \cdot 45.92 \cdot 12}{86818} = -0.388 ksi \quad (T) \]
Live Load Stress on Composite Section

\[ f_{LL+1} = \frac{-M_{LL+1}y_c}{I_e} = \frac{-1487.8k - ft \times 12 \times 45.92}{868818} = -0.94 ksi (T) \]

Stress from Prestress Force

\[ f_{pe} = \frac{P_{se}}{A} + \frac{P_{se} * e * y_b}{I} = \\frac{1424 kips}{82in^2} + \frac{1424 kips \times 23.45 in \times 30.92 in}{410848in^4} = +4.13 ksi (C) \]

Total Tensile Stress at Service

\[ f_{total} = f_{pe} - f_{NDL} + f_{CDL} - f_{LL+1} = 4.13 - 3.26 + .388 - 0.94 = -0.458 ksi (T) \]

\[ f_{allow} = -6\sqrt{f_c} = -0.502 ksi \]

\[ R.F_{IN} = \frac{f_{allow}}{f_{total}} = \frac{0.502}{.458} = 1.10 \]

Design Flexural Strength-Operating Rating

\[ M_u = 1.3 \ M_d + 1.67M_{LL+1} = 1.3 \ 4225.5 + 1.67 \times 1487.8 = 8723 k - ft \]

\[ d = 59 + 9.5 - 7.47 = 61.03 in \]

\[ f_{su}^* = f_s \left[ 1 - \left( \frac{\gamma_s^*}{\beta_i} \right) \left( \frac{p^* f_s}{f_c} \right) \right] \quad [STD 9-17] \]

\[ f_{su}^* = 270 \left[ 1 - \left( \frac{0.28}{0.825} \right) \left( \frac{0.1670 \times 60}{108 \times 61.03} \right) \left( \frac{270}{4.5} \right) \right] = 261.6 ksi \]

\[ a = \frac{A_s f_{su}^*}{0.85bf_c} = \frac{0.167 \times 60 \times 261.6}{.85 \times 108 \times 4.5} = 6.345 in \]

\[ M_n = A_s f_{su}^* \left( d - \frac{a}{2} \right) = 60 \times 0.167 \times 261.6 \times \left( 61.03 - \frac{6.345}{2} \right) \left( \frac{1}{12} \right) = 12638 k - ft \]

\[ R.F_{OP} = \frac{M_n}{M_u} = \frac{12638 k - ft}{8723 k - ft} = 1.45 \]
Evaluation Guide Specification Rating

Operating and Inventory Ratings using the Factored Load Method

\[ R.F_{OP} = \frac{\phi M_n - 1.3 M_d}{1.3 M_{LL+I}} = \frac{1.0 \times 12638 - 1.3 \times 4225.5}{1.3 \times 1487.8} = 3.69 \]

\[ R.F_{IN} = \frac{\phi M_n - 1.3 M_d}{1.3 \times 1.67 \times M_{LL+I}} = \frac{1.0 \times 12638 - 1.3 \times 4225.5}{1.3 \times 1.67 \times 1487.8} = 2.21 \]

Inventory Rating Using the Allowable Stress Method

\[ R.F_{IN} = \frac{f_{pe} - f_{DL} - f_{allow}}{f_{LL+I}} = \frac{4.13 - 3.26 + .388 + .502}{0.94} = 1.05 \]

Positive Moment (M104 – 4/10th point)

Beam Self Weight Moment

\[ M_g = \frac{w x}{2} \left( L - x \right) = \frac{0.91875 \times 48.4}{2} \left( 121 - 48.4 \right) = 1614.17k - ft \]

Slab and Haunch Moment

\[ M_s = \frac{w x}{2} \left( L - x \right) = \frac{1.056 \times 48.4}{2} \left( 121 - 48.4 \right) = 1855.31k - ft \]

Barrier Moment

\[ M_b = \frac{w x}{16} \left( 7L - 8x \right) - \frac{w L}{16} x = \frac{0.188 \times 48.4}{16} \left( 7 \times 121 - 8 \times 48.4 \right) - \frac{0.188 \times 121}{16} \times 48.4 = 252k - ft \]

Future Wearing Surface

\[ M_b = \frac{w x}{16} \left( 7L - 8x \right) - \frac{w L}{16} x = \frac{0.48 \times 48.4}{16} \left( 7 \times 121 - 8 \times 48.4 \right) - \frac{0.48 \times 121}{16} \times 48.4 = 492k - ft \]

Total Dead Load

\[ M_d = 1614.17 + 1855.31 + 252 + 492 = 4213.48k - ft \]
Live Load Positive Moment

\[ I = \frac{50}{125 + 121} = 0.20 \]

Maximum Wheel Load Moment (HS-20)

\[ M_{WL-HS20} = \frac{1542}{2} = 771\,k - ft \]

Live Load Moment per Beam

\[ M_{LL,I} = WDF \times M_{WL-HS20}(1 + I) = 1.64 \times 771 \times 1.2 = 1517.3\,k - ft \]

Pre-Stress Losses

Initial Prestress Force Immediately After Transfer [STD 9.16.2.1.2]

\[ P_{si} = 1867\,kips \]

Eccentricity of Prestress Force

\[ e = y_b - y_{bs} = 30.92 - 7.47 = 23.45\,in \]

\[ f_{cir} = \frac{P_{si}}{A} + \frac{P_{si} \times e^2}{I} - \frac{M_g \times e}{I} \]

\[ f_{cir} = \frac{1867\,kips}{882in^2} + \frac{1867\,kips \times 23.45in^2}{40848in^4} - \frac{1614.17\,k - ft \times 12\,in \times 23.45in}{410848in^4} = +3.51\,ksi \text{ (C)} \]

\[ f_{eds} = \frac{-M_s \times e}{I} - \frac{(M_s + M_{ws})(y_{bc} - y_{bs})}{I_e} \]

\[ f_{eds} = \frac{-1855.31\,k - ft \times 23.45in \times 12}{410848in^4} - \frac{252 + 492 \times 12 \times 45.92 - 7.47}{868818in^4} = -1.66\,ksi \text{ (T)} \]

Effective Final Prestress

\[ f_{se} = 0.75 \times 270 - 60.41 = 142.09\,ksi \]

Effective Final Prestress Force

\[ P_{se} = 60 \times 1.167 \times 142.09 = 1424\,kips \]
Stresses and Strengths

Total Tensile Stress at Service-Inventory Rating

Dead load Stress on Non-Composite section

\[
f_{NDL} = - \frac{M_g + M_s}{I} y_b = - \frac{-1614.17k-ft + 1855.13k-ft \times 30.92in \times 12}{410848in^4} = -3.13ksi \text{ (T)}
\]

Dead Load Stress on Composite Section

\[
f_{CDL} = - \frac{M_b + M_{wr}}{I_c} y_{bc} = - \frac{-252k-ft + 492k-ft \times 45.92in \times 12}{868818in^4} = -0.47ksi \text{ (T)}
\]

Live Load Stress on Composite Section

\[
f_{LL+I} = - \frac{M_{LL+I} y_{bc}}{I_c} = - \frac{-1517.3k-ft \times 12 \times 45.92in}{868818in^4} = -0.962ksi \text{ (T)}
\]

Stress from Prestress Force

\[
f_{pe} = \frac{P_{se}}{A} + \frac{P_{se} \times e \times y_b}{I} = \frac{1424kips}{882in^2} + \frac{1424kips \times 23.45in \times 30.92in}{410848in^4} = +4.13ksi \text{ (C)}
\]

Total Tensile Stress at Service

\[
f_{total} = f_{pe} - f_{NDL} + f_{CDL} - f_{LL+I} = 4.13 - 3.13 + 0.47 - 0.962 = -0.432ksi \text{ (T)}
\]

\[
f_{allow} = -6\sqrt{f'_{c}} = -0.502ksi
\]

\[
R.F_{IN} = \frac{f_{allow}}{f_{total}} = \frac{0.502}{-0.432} = 1.16
\]

Design Flexural Strength-Operating Rating

\[
M_u = 1.3 \ M_d + 1.67M_{LL+I} = 1.3 \times 4213.48 + 1.67 \times 1517.3 = 8772k-ft
\]

\[
d = 59 + 9.5 - 7.47 = 61.03in
\]

\[
f_{su}^* = f_y \left[ 1 - \left( \frac{\gamma^*}{\beta_1} \right) \left( \frac{p^* f_s}{f'_{c}} \right) \right] \text{ [STD 9-17]}
\]
\[ f_{sw}^* = 270 \left[ 1 - \left( \frac{0.28}{0.825} \right) \left( \frac{0.1670 \times 60}{108 \times 61.03} \right) \left( \frac{270}{4.5} \right) \right] = 261.6 ksi \]

\[ a = \frac{A_f f_{sw}^*}{0.85 bf_c} = \frac{0.167 \times 60 \times 261.6}{0.85 \times 108 \times 4.5} = 6.345 \text{in} \]

\[ M_n = A_f f_{sw}^* \left( d - \frac{a}{2} \right) = 60 \times 0.167 \times 261.6 \times \left( 61.03 - \frac{6.345}{2} \right) \times \left( \frac{1}{12} \right) = 12638 k - ft \]

\[ R.F_{\text{OP}} = \frac{M_n}{M_u} = \frac{12638 k - ft}{8772 k - ft} = 1.44 \]

**Evaluation Guide Specification Rating**

**Operating and Inventory Ratings using the Factored Load Method**

\[ R.F_{\text{OP}} = \phi M_n - 1.3 M_d \]

\[ R.F_{\text{IN}} = \phi M_n - 1.3 M_d \]

\[ = 1.0 \times 12638 - 1.3 \times 4213.48 \]

\[ = 1.3 \times 1517.3 \]

\[ = 3.63 \]

\[ = 2.22 \]

**Inventory Rating Using the Allowable Stress Method**

\[ R.F_{\text{IN}} = \frac{f_{pe} - f_{DL} - f_{allow}}{f_{LL+I}} = \frac{4.13 - 3.13 + .47 + .502}{0.962} = 1.07 \]

**Negative Moment (M200 – middle piers)**

**Beam Moment**

\[ M_s = 0 k - ft \]

**Slab Moment**

\[ M_s = 0 k - ft \]
Barrier Moment

\[ M_b = 2 \frac{wl^2}{16} = \frac{wL^2}{8} = -344.06k - ft \]

Future Wearing Surface

\[ M_{FWS} = \frac{0.6 \times 40 \times 121^2}{8 \times 5} = -878.46k - ft \]

Total Dead Load

\[ M_d = 344.06 + 878.46 = -1222.52k - ft \]

Live Load Negative Moment

\[ I = 0.20 \]

AASHTO Wheel Load Distribution Factor

\[ WDF = \frac{S}{5.5} = \frac{9}{5.5} = 1.64 \]

Maximum Wheel-Load Moment

\[ M_{WL-20} = -814.8 / 2 = -407.4k - ft \]

Live Load Moment per Beam

\[ M_{LL-I} = WDF * M_{WL-20} (1 + I) = 1.64 * 407.4 * 1.2 = -801.8k - ft \]

Stresses and Strength

Design Flexural Strength-Operating Rating (Assuming 10.01 in² of steel)

\[ M_u = 1.3 \, M_d + 1.67M_{LL-I} = 1.3 \, 1222.52 + 1.67 * 407.4 = -2473.7k - ft \]

Average \( d = (66 + 62.5) / 2 = 64.25 \)

\[ A_s f_y = 0.85 f_c' A_c \]

\[ A_c = \frac{10.01(60)}{0.85(7)} = 100.94in^2 \]
By observation the depth of compression will be in the bottom flange

\[ a = \frac{100.94 \text{in}^2}{25''} = 4.04'' \]

\[ M_n = A_f f_y \left( d - \frac{a}{2} \right) = 10.01 \times 60 \times \left( 64.25 \text{in} - \frac{4.04}{2} \text{in} \right) = 37375k \text{in} = 3115k - \text{ft} \]

\[ RF_{op} = \frac{M_n}{M_a} = \frac{3115k - \text{ft}}{2473.7k - \text{ft}} = 1.26 \]

**Evaluation Guide Specification Rating**

**Operating and Inventory Ratings using the Factored Load Method**

\[ R.F_{op} = \frac{\phi M_n - 1.3 M_d}{1.3 M_{LL-I}} = \frac{0.9 \times 3115 - 1.3 \times 1222.52}{1.3 \times 801.8} = 1.16 \]

\[ R.F_{IN} = \frac{\phi M_n - 1.3 M_d}{1.3 \times 1.67 M_{LL-I}} = \frac{0.9 \times 3115 - 1.3 \times 1222.52}{1.3 \times 1.67 \times 801.8} = 0.70 \]

**Design Flexural Strength-Operating Rating (Assuming 19.2 in\(^2\) of steel)**

\[ M_n = 1.3 M_d + 1.67 M_{LL-I} = 1.3 \times 1222.52 + 1.67 \times 407.4 = -2473.7k - \text{ft} \]

**Average \(d\) = \((66 + 62.5) / 2\) = 64.25**

\[ A_f f_y = 0.85 f_y' A_c \]

\[ A_c = \frac{19.2(60)}{0.85(7)} = 193.6 \text{in}^2 \]

By observation the depth of compression will be in the bottom flange

\[ a = \frac{193.6 \text{in}^2}{25''} = 7.74'' \]

\[ M_n = A_f f_y \left( d - \frac{a}{2} \right) = 19.2 \times 60 \times \left( 64.25 \text{in} - \frac{7.74}{2} \text{in} \right) = 69558k \text{in} = 5796.5k - \text{ft} \]
\[ RF_{\text{op}} = \frac{M_a}{M_u} = \frac{5796.5 - ft}{2473.7 - ft} = 2.34 \]

**Evaluation Guide Specification Rating**

**Operating and Inventory Ratings using the Factored Load Method**

\[ RF_{\text{op}} = \frac{\phi M_n - 1.3 M_d}{1.3 M_L + L} = \frac{0.9 \times 5796.5 - 1.3 \times 1222.52}{1.3 \times 801.8} = 3.48 \]

\[ RF_{\text{in}} = \frac{\phi M_n - 1.3 M_d}{1.3 \times 1.67 \times M_L + L} = \frac{0.9 \times 5796.5 - 1.3 \times 1222.52}{1.3 \times 1.67 \times 801.8} = 2.08 \]

**Positive Moment (M105 – assuming no continuity)**

**Beam Self Weight Moment**

\[ M_s = \frac{wL^2}{8} = \frac{A \gamma_{\text{conc}} L^2}{8} = \frac{\left( 882 \text{in}^2 \frac{\text{ft}^2}{144 \text{in}^2} \right) \times 150 \text{pcf}}{8} \times 121^2 \text{ft}^2 = 1681.43k - ft \]

**Slab and Haunch Moment**

\[ M_s = \frac{wL^2}{8} = \frac{9.0 \text{ft} \times \left( 8.5 \text{in} \times \frac{\text{ft}}{12 \text{in}} \right) + 4 \text{ft} \times \left( 2 \text{in} \times \frac{1 \text{ft}}{12 \text{in}} \right) \times 150 \text{pcf} \times 21 \text{ft}^2}{8} = 1933.07k - ft \]

**Barrier Moment**

\[ w = \frac{\frac{470 \text{lb}}{\text{ft}}}{5} = 0.188 \text{k/ft} \]

\[ M_b = \frac{wL^2}{8} = \frac{0.188 \times 121^2}{8} = 344.06k - ft \]


**Future Wearing Surface**

\[ w = \frac{0.06 \text{ksf} \times 40 \text{ft}}{5} = 0.48k \text{ / ft} \]

\[ M_{ws} = \frac{wL^2}{8} = \frac{0.48 \times 121^2}{8} = 878.46k - \text{ft} \]

**Total Dead Load**

\[ M_d = 1681.43 + 1933.07 + 344.06 + 878.46 = 4837k - \text{ft} \]

**Live Load Positive Moment**

\[ I = \frac{50}{125 + L} \leq 0.3 \quad [\text{STD 3-1}] \]

\[ I = \frac{50}{125 + 121} = 0.20 \]

**Maximum Wheel Load Moment (HS-20)**

\[ M_{WL-HS20} = 1898/2 = 949k - \text{ft} \]

**AASHTO Wheel Load Distribution Factor**

\[ WDF = \frac{S}{5.5} = \frac{9}{5.5} = 1.64 \quad [\text{STD Table 3.23.1}] \]

**Live Load Moment per Beam**

\[ M_{LL,1} = WDF \times M_{WL-HS20}(1 + I) = 1.64 \times 949 \times 1.2 = 1867.6k - \text{ft} \]

**Pre-Stress Losses**

Initial Prestress Force Immediately After Transfer \quad [\text{STD 9.16.2.1.2}]

\[ P_s = 60 \times 1.670 \text{in}^2 \times 270 \frac{\text{kips}}{\text{in}^2} \times 0.69 = 1867 \text{kips} \]

**Eccentricity of Prestress Force**

\[ e = y_b - y_{bs} = 30.92 - 7.47 = 23.45 \text{in} \]
\[ f_{cr} = \frac{P_{si}}{A} + \frac{P_{si} \cdot e^2}{I} - \frac{M_g \cdot e}{I} \]

\[ f_{cr} = \frac{1867 \text{kips}}{882 \text{in}^2} + \frac{1867 \text{kips} \cdot 23.45 \text{in}^2}{410848 \text{in}^4} - \frac{1681.43 \text{ksi} \cdot 12 \text{in}^2 \cdot 23.45 \text{in}}{410848 \text{in}^4} = +3.46 \text{ksi (C)} \]

\[ f_{cds} = -\frac{M_s \cdot e}{I} - \frac{(M_k + M_w)(y_k - y_s)}{I_c} \]

\[ f_{cds} = -\frac{1933.07 \text{kips} \cdot 23.45 \text{in} \cdot 12}{410848 \text{in}^4} - \frac{344.06 + 878.46 \cdot 12 \cdot 45.92 - 7.47}{868818 \text{in}^4} = -1.97 \text{ksi (T)} \]

**Elastic Shortening Loss**

\[ ES = \frac{E_s \cdot f_{cr}}{E_{ci}} = \frac{29000 \text{ksi}}{4812 \text{ksi}} \cdot 3.46 \text{ksi} = 20.85 \text{ksi} \]

**Shrinkage Loss (assume RH=70%)**

\[ SH = 17 - 0.15 \cdot RH = 17 - (0.15 \cdot 70) = 6.5 \text{ksi} \]

**Creep Loss**

\[ CR_c = 12 \cdot f_{cr} - 7 \cdot f_{cds} = 12 \cdot 3.46 - 7 \cdot 1.97 = 27.73 \text{ksi} \]

**Relaxation Loss**

\[ CR_s = 5 - 0.1 \cdot ES \cdot \left[ 0.05 \cdot SH + CR_c \right] = 5 - 0.1 \cdot 20.85 \text{ksi} \cdot \left[ 0.05 \cdot 6.5 \text{ksi} + 27.73 \text{ksi} \right] = 1.20 \text{ksi} \]

**Total Prestress Losses**

\[ ES + SH + CR_c + CR_s = 20.85 + 6.5 + 27.73 + 1.20 = 56.28 \text{ksi} \]

**Effective Final Prestress**

\[ f_{se} = 0.75 \cdot 270 = 165 \text{ksi} \]

**Effective Final Prestress Force**

\[ P_{se} = 60 \cdot 0.167 \cdot 146.22 = 1465 \text{ kips} \]
Stresses and Strengths

Total Tensile Stress at Service-Inventory Rating

Dead load Stress on Non-Composite section

\[ f_{\text{NDL}} = -\frac{M_g + M_s}{I} y_b = -\frac{1681.43k - ft + 1933.07k - ft \times 30.92\text{in} \times 12}{410848\text{in}^4} = -3.26\text{ksi} \ (T) \]

Dead Load Stress on Composite Section

\[ f_{\text{CDL}} = -\frac{M_b + M_{\text{wr}}}{I_c} y_{\text{bc}} = -\frac{344.06k - ft + 878.46k - ft \times 45.92\text{in} \times 12}{868818\text{in}^4} = -0.775\text{ksi} \ (T) \]

Live Load Stress on Composite Section

\[ f_{\text{LL+I}} = -\frac{M_{\text{LL+I}}}{I_c} y_{\text{bc}} = -\frac{1867.6k - ft \times 12 \times 45.92\text{in}}{868818\text{in}^4} = -1.18\text{ksi} \ (T) \]

Stress from Prestress Force

\[ f_{\text{pe}} = \frac{P_e}{A} + \frac{P_e \times e \times y_b}{I} = \frac{1465\text{kips}}{882\text{in}^2} + \frac{1465\text{kips} \times 23.45\text{in} \times 30.92\text{in}}{410848\text{in}^4} = +4.25\text{ksi} \ (C) \]

Total Tensile Stress at Service

\[ f_{\text{total}} = f_{\text{pe}} - f_{\text{NDL}} + f_{\text{CDL}} - f_{\text{LL+I}} = 4.25 - 3.26 + .775 - 1.18 = -0.965\text{ksi} \ (T) \]

\[ f_{\text{allow}} = -6\sqrt{f_{\text{c}'}} = -0.502\text{ksi} \]

\[ R.F. = \frac{f_{\text{allow}}}{f_{\text{total}}} = \frac{0.502}{0.965} = 0.52 \]

Design Flexural Strength-Operating Rating

\[ M_d = 1.3 \ M_d + 1.67M_{\text{LL+I}} = 1.3 \times 4837 + 1.67 \times 1867.6 = 10343k - ft \]

\[ d = 59 + 9.5 - 7.47 = 61.03\text{in} \]

\[ f_{\text{su}} = f_y \left[ 1 - \left( \frac{f_{\text{pe}}}{f_{\text{c}'}} \right) \right] \quad [\text{STD 9-17}] \]

57
\[ f_{su}^* = 270 \left[ 1 - \left( \frac{0.28}{0.825} \right) \left( \frac{0.1670 \times 60}{108 \times 61.03} \right) \left( \frac{270}{4.5} \right) \right] = 261.6 \text{ksi} \]

\[ a = \frac{A_s f_{su}^*}{0.85 b f_c} = \frac{0.167 \times 60 \times 261.6}{0.85 \times 108 \times 4.5} = 6.345 \text{in} \]

\[ M_n = A_s f_{su}^* \left( d - \frac{a}{2} \right) = 60 \times 0.167 \times 261.6 \times \left( 61.03 - \frac{6.345}{2} \right) \times \left( \frac{1}{12} \right) = 12638 \text{ k-ft} \]

\[ R.F_{\text{op}} = \frac{M_n}{M_a} = \frac{12638 \text{ k-ft}}{10343 \text{ k-ft}} = 1.22 \]

**Evaluation Guide Specification Rating**

**Operating and Inventory Ratings using the Factored Load Method**

\[ R.F_{\text{op}} = \frac{\phi M_n - 1.3 M_d}{1.3 M_{LL+I}} = \frac{1.0 \times 12638 - 1.3 \times 4837}{1.3 \times 1867.6} = 2.62 \]

\[ R.F_{\text{in}} = \frac{\phi M_n - 1.3 M_d}{1.3 \times 1.67 \times M_{LL+I}} = \frac{1.0 \times 12638 - 1.3 \times 4837}{1.3 \times 1.67 \times 1867.6} = 1.57 \]

**Inventory Rating Using the Allowable Stress Method**

\[ R.F_{\text{in}} = \frac{f_{pE} - f_{DL} - f_{allow}}{f_{LL+I}} = \frac{4.25 - 3.26 + 775 + 0.502}{1.18} = 0.61 \]

**Positive Moment (M105 – w/ FEA DF)**

Assume Live load DF of 0.327

\[ M_{LL+I} = 0.327 \times 1512 \times 1.2 = 593.3 \text{ k-ft} \]

**Operating and Inventory Ratings using the Factored Load Method**

\[ R.F_{\text{op}} = \frac{\phi M_n - 1.3 M_d}{1.3 M_{LL+I}} = \frac{1.0 \times 12638 - 1.3 \times 4225.5}{1.3 \times 593.3} = 9.26 \]
\[ R.F_{IN} = \frac{\phi M_n - 1.3M_d}{1.3*1.67* M_{LL+I}} = \frac{1.0*12638 - 1.3*4225.5}{1.3*1.67*593.3} = 5.55 \]

**Positive Moment (M104 – w/ FEA DF)**

Assume Live load DF of 0.327

\[ M_{LL+I} = 0.327*1542*1.2 = 605.1k \text{ ft} \]

**Operating and Inventory Ratings using the Factored Load Method**

\[ R.F_{OP} = \frac{\phi M_n - 1.3M_d}{1.3M_{LL+I}} = \frac{1.0*12638 - 1.3*4213.48}{1.3*605.1} = 9.10 \]

\[ R.F_{IN} = \frac{\phi M_n - 1.3M_d}{1.3*1.67* M_{LL+I}} = \frac{1.0*12638 - 1.3*4213.48}{1.3*1.67*605.1} = 5.45 \]

**Negative Moment (M200 – w/ 10 in² steel and FEA DF)**

Assume Live load DF of 0.333

\[ M_{LL+I} = 0.327*814.8*1.2 = -319.7k \text{ ft} \]

**Operating and Inventory Ratings using the Factored Load Method**

\[ R.F_{OP} = \frac{\phi M_n - 1.3M_d}{1.3M_{LL+I}} = \frac{0.9*3115 - 1.3*1222.52}{1.3*319.7} = 2.92 \]

\[ R.F_{IN} = \frac{\phi M_n - 1.3M_d}{1.3*1.67* M_{LL+I}} = \frac{0.9*3115 - 1.3*1222.52}{1.3*1.67*319.7} = 1.75 \]

**Negative Moment (M200 – w/ 19.2 in² steel and FEA DF)**

Assume Live load DF of 0.333

\[ M_{LL+I} = 0.327*814.8*1.2 = -319.7k \text{ ft} \]
Operating and Inventory Ratings using the Factored Load Method

\[
R.F_{OP} = \frac{\phi M_n - 1.3M_d}{1.3M_{LL+I}} = \frac{0.9 \times 5796.5 - 1.3 \times 1222.52}{1.3 \times 319.7} = 8.73
\]

\[
R.F_{IN} = \frac{\phi M_n - 1.3M_d}{1.3 \times 1.67 \times M_{LL+I}} = \frac{0.9 \times 5796.5 - 1.3 \times 1222.52}{1.3 \times 1.67 \times 319.7} = 5.23
\]

Positive Moment (M105 – w/ FEA DF and assuming no continuity)

Assume Live load DF of 0.327

\[M_{LL+I} = 0.327 \times 1512 \times 1.2 = 593.3k - ft\]

Operating and Inventory Ratings using the Factored Load Method

\[
R.F_{OP} = \frac{\phi M_n - 1.3M_d}{1.3M_{LL+I}} = \frac{1.0 \times 12638 - 1.3 \times 4837}{1.3 \times 593.3} = 8.23
\]

\[
R.F_{IN} = \frac{\phi M_n - 1.3M_d}{1.3 \times 1.67 \times M_{LL+I}} = \frac{1.0 \times 12638 - 1.3 \times 4837}{1.3 \times 1.67 \times 593.3} = 4.93
\]
Appendix B
Section Property Calculations
| fc' girder | 7000 psi | Eg | 5072 ksi | nrebar | 5.72 be | 97 in |
| fc' deck | 4500 psi | Ed | 4067 ksi | ndeck | 0.80 ts | 7.5 in |

**Girder + 77.6 in Deck 7.5 in thick**

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| fc' girder | 7000 psi | Eg | 5072 ksi | nrebar | 5.72 be | 97 in |
| fc' deck | 4500 psi | Ed | 4067 ksi | ndeck | 0.80 ts | 7.5 in |

**Girder + 77.6 in Deck 8.5 in thick**

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| fc' girder | 7000 psi | Eg | 5072 ksi | nrebar | 5.72 be | 108 in |
| fc' deck | 7000 psi | Ed | 5072 ksi | ndeck | 1.00 ts | 8.5 in |

**Girder + 108 in Deck 8.5 in thick**

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| fc' girder | 7000 psi | Eg | 5072 ksi | nrebar | 5.72 be | 97 in |
| fc' deck | 4500 psi | Ed | 4067 ksi | ndeck | 0.80 ts | 8.5 in |
Girder + 77.6 in Deck 8.5 in thick (negative moments) over pier

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Title: Structural Health Monitoring of HAN-30-0295 Highway Bridge

State Job Number: 134329
PID Number:
Research Agency: Ohio Northern University
Researcher(s): Dr. Farhad Reza
Technical Liaison(s): Mike Loeffler, Steve Reichenbach
Research Manager:
Sponsor(s):
Study Start Date: February 2007
Study Completion Date: December 2008
Study Duration: 22 months
Study Cost: $10,000
Study Funding Type:

STATEMENT OF NEED:
HAN-30-0295 (SFN 3201112) Highway Bridge is located in Hancock County, approximately 4 miles north of Ada, Ohio. The bridge is a two-lane highway bridge carrying State Route OH-235 over US-30. The superstructure is of a deck-and-girder type consisting of five precast prestressed concrete I-girders. The bridge consists of two simple spans of approximately 121-ft, made continuous at the intermediate pier. The bridge falls under the jurisdiction of Ohio Department of Transportation (ODOT) District One. They are responsible for regular inspection and load rating of the bridge. Engineers from the District One Office observed cracking of the concrete diaphragms at the intermediate pier location. They subsequently submitted a request for a research study that would include structural health monitoring of the bridge.

RESEARCH OBJECTIVES:
The primary objective of the research was to assist the District One Office in conducting structural health monitoring of the HAN-30-0295 Bridge. Specifically, the objectives were to:
1. Develop an understanding of the possible causes of the problem and the mechanisms at work.
2. Conduct temperature differential measurements across the depth of the girders, since temperature effects were thought to be a major factor from the outset.
3. Conduct a diagnostic load test using a dump truck while monitoring strains in the girders. Test results would provide useful information on the physical behavior of the structure including continuity for live loads, strength of the girders, fixity at the abutments and distribution of the live load.
4. Develop an analytical (finite element) model of the bridge.
5. Use data obtained from objectives (iii) and (iv) to perform load rating of the structure.

RESEARCH TASKS:
- Review the relevant literature, bridge plans, and bridge design calculations.
- Perform load rating of the structure using typical assumptions of the (AASHTO) Factored Load Method.
- Develop a finite element model of the bridge. Use the model to test various scenarios e.g. change in diaphragm stiffness, and loss of continuity.
- Estimate creep and shrinkage moments for the girders.
- Measure temperature gradients across the depth of girders. Estimate thermal moments in the girders due to both the measured gradient and the AASHTO specified gradient.
- Conduct a diagnostic load test using a dump truck while monitoring strains in the girders. Use the data to interpret the physical behavior of the structure, and update the load ratings and finite element model.
- Write the final report

**RESEARCH DELIVERABLES:**
- A final report and executive summary

**RESEARCH RECOMMENDATIONS:**
Simple span prestressed concrete beams made continuous are popular because the elimination of a middle joint reduces maintenance problems, and the maximum positive moment in the span is reduced. Negative moments develop at the pier due to live loads and are taken into account by the designers by placing reinforcement in the deck.

However, the net effect of creep and shrinkage can lead to positive moment at the pier. In addition, positive temperature gradients can create positive moment at the piers. Often these effects are not taken into account explicitly by the designer, as was the case in this particular bridge. The results from this study show that temperature effects alone could create a moment exceeding $1.2M_{cr}$ which is sufficient to cause the type of cracking seen.

The results from the diagnostic load test show that the beams in service have higher strength than designed. This most likely is due to greater strength contribution from the deck than anticipated for example because of thicker deck, stronger concrete, and larger effective flange. Results also indicate that there is significant continuity developed at the abutments which lowers the maximum positive moment induced in the beams from live load.

The recommendation for the studied bridge is that no remedial action is necessary. Recommendations for future bridge designs are to require consideration of thermal and creep and shrinkage effects by design engineers; utilize bond breakers between girders and diaphragms; and delay the pouring of the deck to allow for more differential shrinkage to take place.

**PROJECT PANEL COMMENTS:**

**IMPLEMENTATION STEPS & TIME FRAME:**
The author believes that as a first start to reducing such problems in the future, it could be specified that design engineers consider creep, shrinkage, and temperature effects for these types of bridges. These specifications could be added in the ODOT Bridge Design Manual. If ODOT wishes to be more particular in the specifications, then reference could be made to relevant guides e.g. NCHRP Reports 322 and 519. It should be noted that since this is not current practice in the bridge design industry, a sufficient amount of time (e.g. four years) should be allowed for bridge designers to become more conversant with these issues.

More research is needed in this area. Of particular interest would be to conduct surveys across the different states to identify bridges that have been successful in reducing diaphragm cracking. It would be interesting to find whether there are bridges constructed with techniques such as breaking the bond at the girder-diaphragm interface, and determining if the techniques were effective.

ODOT could also sponsor one or two “pilot-project” bridges, where the designers have considered the effects of creep, shrinkage, and temperature and may have incorporated innovative features in the design.

**EXPECTED BENEFITS:**
Bridge owners can still continue to enjoy the benefits of prestressed girder bridges made continuous (e.g. in the elimination of the middle joint), and yet eliminate the drawbacks of having a relatively new bridge experience diaphragm cracking and spalling. These disadvantages include questions which may arise on the integrity of the structure at the time of inspection, poor aesthetic appearance, and negative perception if detected by the public.

EXPECTED RISKS, OBSTACLES, & STRATEGIES TO OVERCOME THEM:
There is a slight risk that designers will resort to designing simply supported spans or even find an alternative to prestressed concrete; however, this risk can be mitigated by more conversation and exchange of information between ODOT and the bridge design industry e.g. through OTEC Conference.

OTHER ODOT OFFICES AFFECTED BY THE CHANGE:
All district offices overseeing construction of new bridges of this type.

PROGRESS REPORTING & TIME FRAME:

TECHNOLOGY TRANSFER METHODS TO BE USED:
- The final report of this research will be available online at the ODOT webpage.
- A presentation at a conference e.g. OTEC is proposed.

IMPLEMENTATION COST & SOURCE OF FUNDING:
The costs associated with adding specifications to the Bridge Design Manual, funding additional research, and implementing pilot projects would have to be determined by the concerned parties within ODOT.

Approved By: (attached additional sheets if necessary)

Office Administrator(s):

Signature: ______________________ Office: ___________ Date: ______________

Signature: ______________________ Office: ___________ Date: ______________

Division Deputy Director(s):

Signature: ______________________ Office: ___________ Date: ______________

Signature: ______________________ Office: ___________ Date: ______________
Structural Health Monitoring of HAN-30-0295 Highway Bridge

Start Date: February 2007
Duration: 22 months
Completion Date: December 2008
Report Date: September 2008
State Job Number: 134239
Report Number: ST/SS/XX-XXX

Funding: $10,000
Principal Investigators:
Farhad Reza, Ph.D., P.E.

ODOT Contacts:
Technical:
Mike Loeffler, P.E.
Steve Reichenbach, P.E.

Administrative:
Monique R. Evans, P.E.
Administrator, R&D
614-728-6048

Problem

HAN-30-0295 (SFN 3201112) Highway Bridge is located in Hancock County, approximately 4 miles north of Ada, Ohio. The bridge is a two-lane highway bridge carrying State Route OH-235 over US-30. The superstructure is of a deck-and-girder type consisting of five precast prestressed concrete I-girders. The bridge consists of two simple spans of approximately 121-ft, made continuous at the intermediate pier. The bridge falls under the jurisdiction of Ohio Department of Transportation (ODOT) District One. They are responsible for regular inspection and load rating of the bridge. Engineers from the District One Office observed cracking of the concrete diaphragms at the intermediate pier location. They subsequently submitted a request for a research study that would include structural health monitoring of the bridge.

Objectives

The primary objective of the research was to assist the District One Office in conducting structural health monitoring of the HAN-30-0295 Bridge. Specifically, the objectives were to:
i. Develop an understanding of the possible causes of the problem and the mechanisms at work.
ii. Conduct temperature differential measurements across the depth of the girders, since temperature effects were thought to be a major factor from the outset.
iii. Conduct a diagnostic load test using a dump truck while monitoring strains in the girders. Test results would provide useful information on the
physical behavior of the structure including continuity for live loads, strength of the girders, fixity at the abutments and distribution of the live load.
iv. Develop an analytical (finite element) model of the bridge.
v. Use data obtained from objectives (iii) and (iv) to perform load rating of the structure.

Description

A literature review was performed. Finite element models of the bridge were developed to study the behavior of the bridge both under HS-20 as well as the dump truck. This model was also useful for checking the effect of diaphragm-beam connections ranging from fixed to pinned, and having no diaphragm on the live load capacity. Additionally the magnitude of moments developed in the diaphragm was estimated.

Temperature sensors were attached to both an interior and exterior girder and data was collected for more than 24 hours over the course of two typical summer days (74°F and 80°F). The largest gradient was used to calculate the positive thermal moment over the middle pier. AASHTO temperature profiles were also investigated.

A diagnostic load test was performed using a dump truck of known axle configurations and weights, and strain measurements were taken at the location of maximum positive moment (4/10th point).

Conclusions & Recommendations

Simple span prestressed concrete beams made continuous are popular because the elimination of a middle joint reduces maintenance problems, and the maximum positive moment in the span is reduced. Negative moments develop at the pier due to live loads and are taken into account by the designers by placing reinforcement in the deck.

However, the net effect of creep and shrinkage can lead to positive moment at the pier. In addition, positive temperature gradients can create positive moment at the piers. Often these effects are not taken into account explicitly by the designer, as was the case in this particular bridge. The results from this study show that temperature effects alone could create a moment exceeding $1.2M_{cr}$ which is sufficient to cause the type of cracking seen.

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Implementation Potential

The results of the study could be used as the basis for further research leading to development of standards requiring consideration of positive moment in design including temperature, creep and shrinkage effects; design of the positive moment connection; and design of diaphragm-to-beam connections.