The Influence of Temperature Changes on Bridge Structural Behavior

B.R. Brenner[1], M. Sanayei[2], E.S. Bell[3], P.L. Rosenstrauch[4], E.J. Pheifer[5], W.A. Marr[6]

[1] Vice President, Fay, Spofford & Thorndike, LLC, Burlington, MA; Professor of the Practice, Tufts University, Medford, MA; email: brian.brenner@tufts.edu

[2] Professor, Department of Civil and Environmental Engineering, Tufts University, Medford, MA; email: masoud.sanayei@tufts.edu

[3] Associate Professor, Department of Civil Engineering, University of New Hampshire, Durham, NH; email: erin.bell@unh.edu

[4] Graduate Student, Department of Civil and Environmental Engineering, Tufts University, Medford, MA; email: paul.rosenstrauch@tufts.edu

[5] Structural Engineer, Fay, Spofford & Thorndike, LLC, Burlington, MA; email: epheifer@fstinc.com

[6] President, Geocomp Corporation, Boxborough, MA; email: wam@geocomp.com

ABSTRACT

In 2009, an instrumentation system was installed on the Little Mystic Truss, one of the two truss spans that comprise part of the Maurice J. Tobin Memorial Bridge connecting Charlestown and Chelsea, Massachusetts. The structure was the subject of extensive three dimensional finite element modeling. To assist in model verification, truck load tests were performed and strains were measured. The models were calibrated using the measurements. Researchers have subsequently evaluated the measured response from the instrumentation system to gain further understanding of the bridge's structural behavior. This article describes the evaluation of the data, with a discussion of behavior due to temperature effects. This evaluation is one of the research goals involved in furthering the development of bridge structural health monitoring systems that integrates the structural model, monitoring system, and visual inspection reports.

INTRODUCTION

The effect of temperature change on bridge structural behavior can be significant. The structural response associated with a change in temperature over the course of a year can be of the same magnitude as the structural response caused by live loading (Santini-Bell and Sipple, 2009 and Sanayei et al., 2011).

In order to demonstrate the significance of temperature loading, the authors conducted a study on the effects of temperature changes on a long span truss bridge.

In a previous study, as part of a design verification project this bridge was instrumented with a system including strain and temperature sensors. Also in this previous study, the authors developed a finite element model of this bridge, which was then calibrated from load test data. This instrumentation system and calibrated model provide background data that is the basis of the discussion presented in this paper.

Coordinating instrumentation and modeling give engineers an increased ability to understand the behavior of the bridge and while it is in-service. Engineers may better understand how a bridge behaves in the field, versus how it was designed to behave. With refinement and calibration, the structural model may be used as a baseline for how the bridge is expected to behave. Deviations between the model's response and the measured data can indicate that a condition has changed on the bridge. When the cause of the change has been identified, engineers may be able to better direct maintenance work. This process when combined with information from a visual inspection may provide engineers with additional objective information about bridge performance that may streamline and improve the bridge maintenance effort. At a time when many bridges in the United states are approaching the end of their design lives, the authors hope that such techniques will be used in the future in addressing long term bridge maintenance, rehabilitation, and replacement.

An important requirement for calibration and verification of a baseline structural model is the application of controlled load tests. Vehicular load tests on an active bridge can be disruptive and expensive. However, variation in temperature is, in effect, a load case, but one that occurs with no impact to traffic. If it is possible to accurately measure the performance of a bridge by measuring its temperature field, then long term structural health monitoring of bridges becomes more practical.

Changes in temperature happen on a daily, seasonal and annual basis, and directly affect bridge structural behavior. In this paper, the authors focus on the daily and seasonal trends in temperature and strain in a long span truss bridge. With further work, the authors hope to gain a better understanding of how changes in temperature affect long span truss bridge behavior, and also to further refine the baseline calibrated model for use in maintenance.

BACKGROUND

Changes in temperature may have a variety of effects on a single structural element depending on the element's support conditions. If a simply supported beam is subjected to a uniform change in temperature, the change will cause either elongation or shortening in the element depending whether the temperature change is positive or negative. Mathematically, it is stated as in the following equation given by Hibbeler (2008):

$$\Delta\epsilon = \alpha\Delta T$$

If a constrained element is subjected to a change in temperature, significant stresses may build up in the element. A member that is neither fully free nor fully fixed at its end will elongate or contract, as well as develop stresses (Usmani et al., 2001). Gradients throughout the depth of the section will cause thermal bowing (Usmani et al., 2001).

Large scale structures exhibit similar behavior when they are subjected to changes in temperature. A determinate structure will expand or contract, possibly in many directions, but the strains generated by the change in temperature will not cause stress in the structural elements. The strains caused by the changes in temperature in an indeterminate structure, however, may be comparable to the live load due to traffic (Catbas, 2008). The manner in which the deformation occurs also depends on the structure's support conditions. Thermal bowing throughout the structure may result from a temperature change if some parts of the structure are less constrained than others.

Changes in temperature are rarely uniform, especially on long span bridges. The change in temperature usually happens over time, and the temperature load case acts more like a temperature gradient. An experiment performed by Bishnoi and Uomoto showed that gradients more than local temperature changes control the changes in strain of a structural element (Bishnoi and Uomoto, 2008). In a three dimensional (3D) structure such as a bridge, changes in strain in one location will be related not only to the applied change in temperature at that location, but also due to the changes in temperature occurring at many other locations on the bridge.

The strains caused by changes in temperature are important for bridge performance. Temperature-induced strains were measured on the Commodore Barry Bridge in Pennsylvania by N. Catbas (2008). This is a cantilevered truss bridge with a simply supported truss span between the interior cantilever arms. The vertical members at the ends of the simply supported span were designed to act as tensile hanger elements. Vibrating wire strain gauges as well as ambient temperature sensors were placed on the hangers, which are wide-flange beams, to monitor changes in strain due to temperature. The strains followed trends; on one flange, the strains followed the changes in temperature while on the other flange, the strains followed the inverse of the change in temperature. This means that the change in temperature caused a bending effect in the hanger (Catbas, 2008). When the load rating was recomputed to include the temperature effect, it was significantly lower than the load rating calculated without considering the temperature effects (Catbas, 2008).

Changes in temperature cause deflections and rotations in bridges. This was shown through experiments performed on two instrumented bridges in Iowa (Wipf, 1990). Tilt meters and ambient temperature sensors were placed on one pier of the Karl King Bridge and on one pier the Black Hawk Bridge. The longitudinal direction of both bridges is from east to west, and transverse direction is north to south. Data was gathered for one year. The data showed patterns for a daily variation and seasonal variation. On a daily time scale, the instrumented bridge pier recorded an easterly tilt during the early morning. As the temperature rose throughout the day, however, the pier tilted west. The pier rotated east overnight to begin the next day with an easterly tilt. Over the year, both instrumented piers experienced a net tilt west as time progressed from winter to summer, and experienced a net tilt east as time progressed from the summer to winter. Both the daily and seasonal trends can be explained by the change in temperature; easterly tilts are associated with a drop in temperature while westerly tilts are associated with an increase in temperature for these two bridges.

Many types of behavior were described above, from expansion and contraction due to uniform temperature change, to bowing due to gradients and tilting caused by daily and seasonal variation. Strain measurements can provide an insight into these bridge structural behaviors due to temperature changes. In a long span bridge, the authors expect to see a combination of these behaviors acting together. The Maurice J. Tobin Bridge, which has been instrumented since 2009, provides a basis for the study of bridge behavior due to temperature change.

TOBIN BRIDGE STUDY

Tobin Overview. The Maurice J. Tobin Memorial Bridge is 2 ¼ miles long and is located just north of the city of Boston, Massachusetts. It spans the Mystic River and connects Charlestown and Chelsea. It is the longest bridge in Massachusetts. The bridge has two decks for northbound and southbound traffic and has a total of six travel lanes. The bridge was constructed between 1948 and 1950. The bridge consists of two spans; the Little Mystic Truss (LMT), which is a Warren truss and the subject of this research, and the cantilevered Big Mystic Truss. The main truss chords are built up box-sections consisting of steel plates that have been riveted together and connected with gusset plates (Pheifer, 2010). The LMT (Figures 1 to 3) spans four hundred forty feet, and the distance between the centerlines of the main LMT trusses is forty-six feet.

Figure 1. The Little Mystic Truss

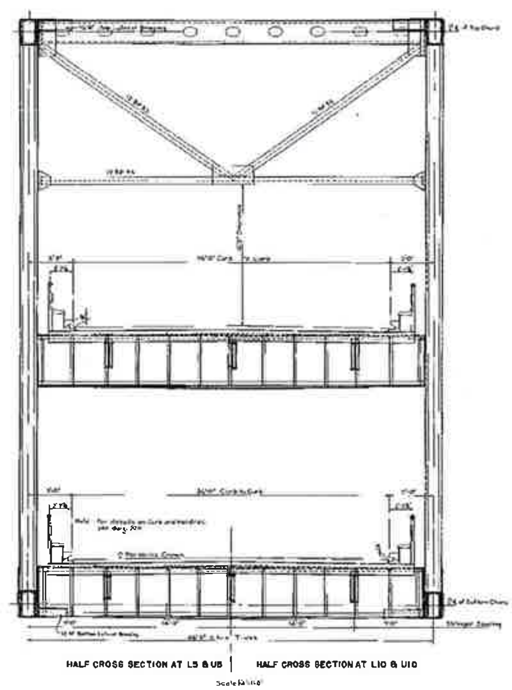

Figure 2. Section view of LMT

Instrumentation. In 2008, a team led by engineers at Fay, Spofford, and Thorndike Inc. (FST) were selected for a contract to develop a finite element model (FE model) of the Tobin Bridge. Researchers at Tufts University, the University of New Hampshire, FST, and Geocomp Corporation carried out this research. In 2009, the LMT was instrumented with various electronic sensors for the purpose of collecting structural response and calibrating the FE model. Table 1 shows the instruments that were installed on the LMT:

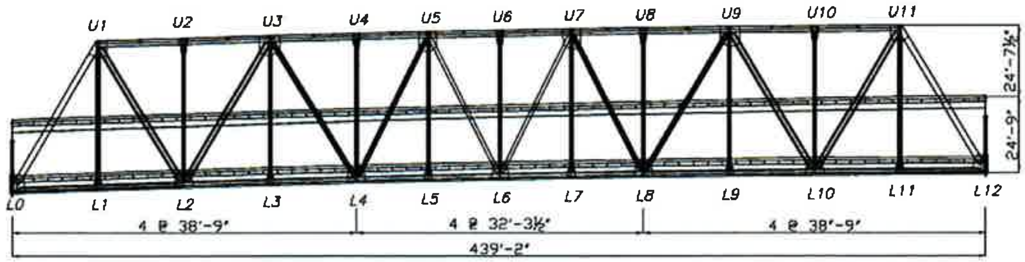

Figure 3. East elevation of LMT

Table 1. Instrument types and quantities on the Little Mystic Span

Instrument Type	Number of Instruments
Strain Gauge – Uniaxial	72
Strain Gauge Rosettes	36
Tiltmeters	2
Accelerometers	18
Temperature Sensors	6

The strain gauges were installed on lower chords located near midspan on the east side, diagonal and vertical members on the east side, and on some lower chords located near midspan on the west side. Because these instrumented members are box-sections, groups of four gauges were installed at a pre-determined distance from the joints at each end of the member; one gauge was installed per side for a total of 8 strain gauges per member.

The strain rosettes were installed on the LMT's gusset plates. Each rosette consists of three strain gauges arranged at a zero, forty-five, and ninety degree angle respectively. Accelerometers were installed on the main chords of the truss. The thermistors were installed at three locations; one on each side of the south pier, one on the east and west main truss at quarter span, and one on the east and west main truss at midspan. The tiltmeters were installed on the two bridge shoes on the south pier. The data collected by the instruments are processed through the data acquisition system and are transmitted wirelessly to Geocomp's iSite Central™ database where researchers and engineers can access this data by simply using a web browser. Data collection began in October of 2009. Most data was recorded at a frequency of one reading per hour, although at some periods, data was recorded at the rate of one reading per minute.

The development of the LMT FE model was based on the original design drawings. Figure 2, above, is an AutoCAD drawing of the LMT. Dimensions and member sizes were imported from AutoCAD into SAP2000 using the AutoLISP and Visual Basic programming languages to create the FE model (Sanayei et al., 2010). Figure 3, above, is a section cut of the LMT from the original design drawings.

Load Test. On November 10th, 2009, the research team conducted a full scale load test of the LMT. The LMT was closed and cleared of traffic. Two trucks, each loaded with approximately 35,000 pounds of sand were driven onto the LMT in a pre-determined pattern. The trucks stopped at midspan for approximately 20 seconds. Each truck was tracked with an automated motorized total station (AMTS) to record their position. During this time, data was collected at a rate of 1 Hz. The strain gauges at midspan registered a change in strain of six microstrains; this value was then used by Pheifer (2010) to calibrate the LMT FE model. To account for the initial strains present in the strain gauges, the change in strain was computed over the time period of the load test.

Calibration of the LMT FE model was governed by how well the strains that were output from the model matched the strains that were recorded during the test. The truck loading was treated as a load case in the FE model by inputting the wheel weights as point loads on the joints of the bridge. After the initial comparison, the model was updated by Pheifer (2010) to include both concrete decks and the piers. The member stiffnesses were reduced to account for the rivet hand-holes. The joints were fixed to account for rigidity of the gusset plate connections (Brenner, 2010). The final calibrated model matches the strains in the bridge members at the locations of the strain gauges due to the truck loading.

Using Temperature as a Load Case. The authors are currently seeking to understand the behavior of the LMT further by studying the LMT's reaction to changes in temperature. The instrumentation system installed on the LMT provides data concerning how the bridge responds to temperature changes, and the LMT FE model provides the expected behavior. Changes in temperature can be input into a finite element program as a load case. Because the changing temperature is recorded by the thermistors installed on the LMT, there is no need to organize an expensive load test to obtain the load case.

Members of determinate structures may freely expand and contract due to changes in temperature. However, the LMT is an indeterminate structure with support conditions that are neither fully unrestrained nor fully fixed in the longitudinal direction. The bridge shoes on either side of the LMT behave like pins (Pheifer, 2010). The shoes support the LMT at its base near the lower chords as opposed to the neutral axis of the bridge in bending. Additionally the main chords are continuous and the vertical and diagonal members are connected to the top and bottom chords with stiff gusset plates that act as rigid connections. For these reasons, this indeterminate truss will experience additional stresses due to temperature changes. The authors expect to observe bending induced deflections from thermal bowing in the truss due to placement of the bridge supports at the bottom chord level. If an increase in temperature occurs, the piers will resist the horizontal expansion of the bottom chords. As a result, members in the top chord will expand more than the bottom chord members and the bridge will bow upwards. The opposite behavior is expected for a negative change in temperature. Over the course of a year the authors expected to observe the upwards and downwards bowing on a daily and seasonal variation in strain that could be compared to the results of the LMT FE model.

TEMPERATURE INDUCED STRAINS

Daily and Seasonal Temperature Variation. Ambient temperature varies over the period of twenty-four hours. During the day, temperatures are higher than they are at night, and over a year, the temperature is much higher in the summer than it is in the winter. In Figure 4, temperature versus time is graphed for two time periods of three days. One period occurs in the summer and the other period is in the winter. These time periods were chosen because there is an eighty degree Fahrenheit temperature change between the maximum and minimum recorded temperatures. These are typical extreme diurnal temperature changes observed on the LMT.

According to Figure 4, the temperature change over the course of twenty-four hours is 20 degrees on average in this figure, and can be as much as 30 degrees. The maximum daily temperature occurs later in the day in the summer than in the winter because the sun shines for longer. The temperature begins to rise earlier in the summer than it does in the winter, again, because the sun rises earlier. Based on this information the authors expect to see distinct patterns in the daily and seasonal strain variation.

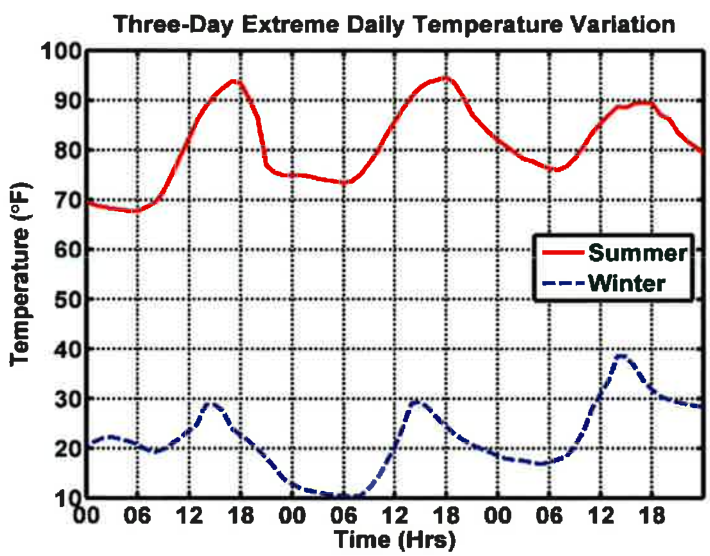

Figure 4. Temperature Variation Recorded on LMT

Daily and Seasonal Strain Variation. Changes in temperature in an indeterminate structure such as the LMT will cause strain tendencies that lead to mechanical forces and stresses in the members that are constrained from free movement. Figure 5, shows the strain variation over the same two three-day periods as those in Figure 4. The data from Figure 5 was recorded by strain gauge SG-L4L5-E-05. This gauge is located on the top of lower chord L4-L5 on the east side of the LMT. The strain variation shown in Figure 5 illustrates the daily and seasonal change in strain.

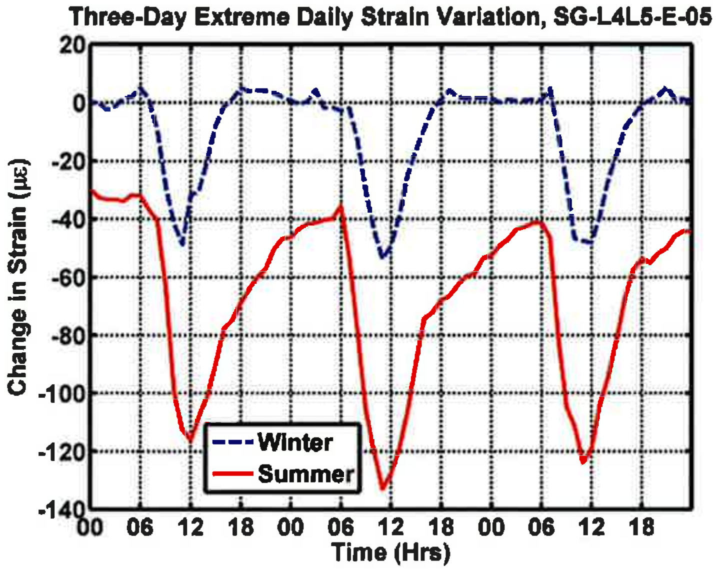

Figure 5. Daily and seasonal strain variation on girder L4-L5

The sign convention is important when interpreting graphs of strain. Because the strain gauges have an initial reading, the measured strain is not the strain in the structural member and a gauge reading of zero strain does not mean that the member is unstressed. Rather, the change in strain between two points in time must be computed to gain an insight into the strain in the bridge members. In Figure 5, the first recorded point in the winter time period was taken to be the initial reading, and the change in strain between all recordings was computed. Loads that cause the strain to decrease in value are compressive loads, while loads that cause the strain to increase in value are tensile loads.

Both the strain patterns in the winter and summer for this gauge show consistent daily variation. During both periods, the compressive strains increase while the temperature is rising, and tensile strains increase when the temperature falls. This same behavior is present in the long term. Higher summer temperature cause measurements that are more compressive, while lower winter temperatures cause measurements that are more tensile. Figure 6 contains a graph of strain and temperature versus time for one year where the strain long term strain trends may be seen. The trends in Figure 5 continue to hold. The strains grow in compressive magnitude between the winter to the summer, and decrease in compressive magnitude between the summer to the winter. From the maximum, or most tensile, strain in this winter period to the minimum, or most compressive, strain in this summer period, the difference is 138.2 microstrains.

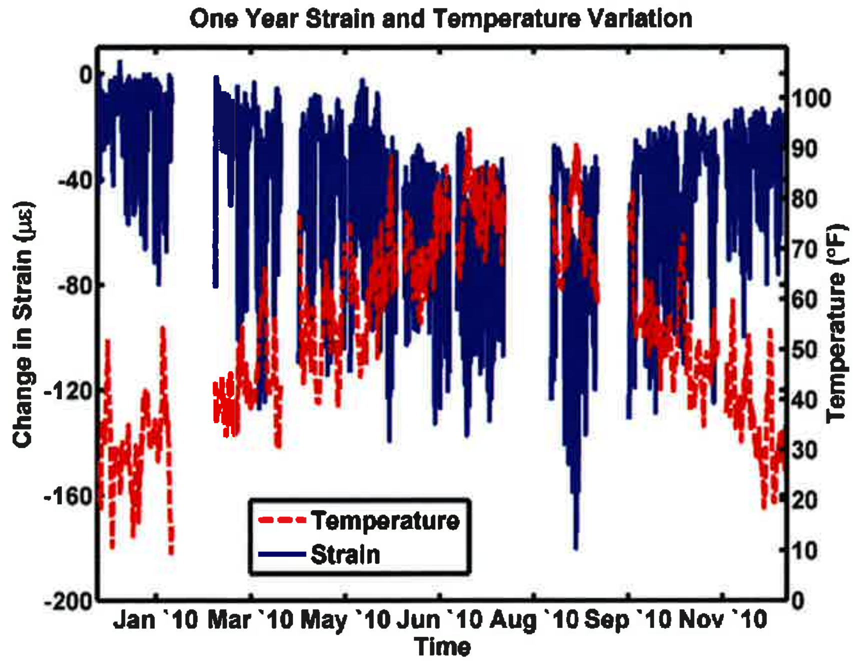

Figure 6. Long term temperature and strain variation

The LMT FE model may explain the change in strain associated with changes in temperature through the corresponding deflected shapes. Figure 6 shows that the change in temperature between the maximum and minimum is approximately eighty degrees Fahrenheit, a reasonable value for Boston. To demonstrate the behavior associated with a positive and negative temperature change, the authors applied a positive and negative change in temperature of forty degrees Fahrenheit to the LMT FE model from an unstressed state. The deflected shapes associated with the two temperature changes are shown in Figure 7a and Figure 7b below.

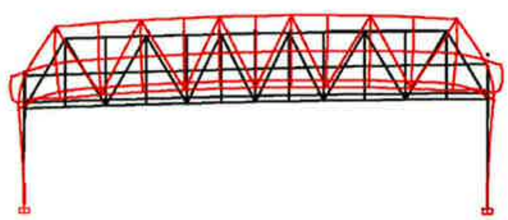

Figure 7a. Deflected shape from positive temperature change

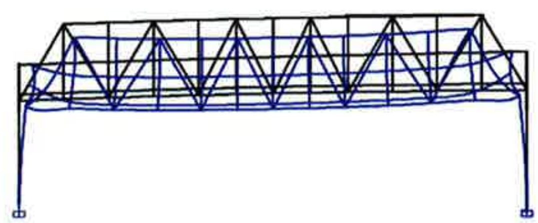

Figure 7b. Deflected shape from negative temperature change

The deflected shapes are consistent with the daily and seasonal change in strain trends that were recorded by strain gauge SG-L4L5-E05. Figure 7a shows that the model deflects upwards under a positive temperature change, causing the lower chord to go into compression. Figure 7b shows that the lower chord goes into tension because of the downwards deflection associated with the negative change in temperature. The magnitude of the deflection for each case is 0.81 inches, which means that the deflection for an eighty degree Fahrenheit is 1.62 inches.

The FE model has the ability to output member axial forces and internal moments. Based on the section properties of the LMT, the strains may be computed by summing the axial and bending strains as shown in the following equation.

$$\epsilon = \frac{1}{E}\left(\pm\frac{P}{A} \pm \frac{M_y z}{I_y} \pm \frac{M_z y}{I_z}\right)$$

To compute the expected strain at the location of SG-L4L5-E-05, which is on the top of girder L4-L5 east, the authors did not include a term for the z-direction moment. This is because SG-L4L5-E-05 was installed on the centerline of the member, and so bending around this axis would not cause normal strains at the centerline.

Based on the output from the FE model, the authors found that the expected change in microstrain in girder L4-L5 was 162.4 microstrains. Compared to the 138.2 microstrains given by the strain gauge, the FE model has an eighteen percent difference for this location. It is important to note that there are a wide degree of changes in temperature and changes in strain, therefore, slight variability in the magnitudes of deflection and strain are to be expected. This represents a reasonable match given the complexity of the LMT, although future work includes matching these strains more closely than eighteen percent difference.

Comparison to AASHTO Truck Loading. The load cases for which bridges are designed are governed by AASHTO (AASHTO 1.3, 2007). Traffic loading according to AASHTO 3.6.1.2.4 is taken to be 0.64 kips per linear foot per lane. The design weight of the design truck load is seventy-two kips according to AASHTO figure 3.6.1.2.2-1. Together, the lane load and truck load comprise the total AASHTO vehicular live load. The authors applied the lane load to all six lanes and the total truck load at six locations, one in each lane, at midspan of the LMT FE model. Since the LMT has six lanes, the authors applied a 35 percent load reduction factor to both the lane and truck loads based on the multiple presence factor from AASHTO table 3.6.1.1.2-1. The truck load was then increased by thirty-three percent to account for impact (AASHTO Table 3.6.2.1-1, 2007).

Figure 8, shows the deflection based on the AASHTO loading described previously. At midspan, the deflection is 1.54 inches according to the LMT FE model. This value is five percent less than the deflection due to an eighty degree temperature change. The strain predicted by the LMT FE model at the location of strain gauge SG-L4L5-E-05 on top of girder L4-L5 east is 78 microstrains, which is forty-eight percent of the strain caused by an eighty degree temperature change. Based on these values, a typical seasonal temperature change may cause substantial deflections and strains as compared to typical design live loads.

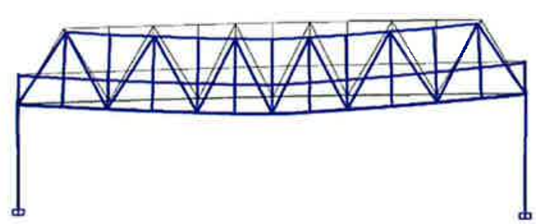

Figure 8. Deflection due to AASHTO loading

CONCLUSION

In this research, the authors used data from an instrumented, long span truss bridge, and a calibrated finite element model to predict how the bridge would react to a change in temperature. The authors found that increases in temperature cause an upwards deflection and compressive strains in the lower chord of this bridge, while decreases in temperature caused downwards deflection and tensile strains in the lower chord. The strains predicted by the calibrated model matched those that were given by the instrumentation system with fifteen percent error. Although the match between the instrumentation data and the model data is acceptable, the authors feel that the match could be improved with further research.

The LMT FE model is based on the original design drawings. Over the bridge's service life, changes in the bridge's sections due to normal use have been documented by inspectors. The next step in this research project is to update the model based on the inspection reports. To improve the accuracy of the LMT FE model output, accuracy is needed regarding the FE model inputs, namely, the temperature load case. Full knowledge of the full temperature load case is not possible, as that would require knowing the temperature at every point on the bridge. Yet, it is possible to input temperature gradients. Future work includes increasing the number of thermistors on the LMT so that the load case is more thoroughly defined. With these improvements to the LMT FE model and with further monitoring, the authors hope to gain a deeper insight into the structural behavior of the LMT and long span truss bridges due to changes in temperature.

A fully calibrated model may be used as part of a structural health monitoring system. Structural health monitoring is a technology that may be used to understand structural behavior for objective load rating. SHM can also be used for finite element model calibration that can assist engineers in approaching the problem of an aging infrastructure system. Maintenance of the US infrastructure is vital the US economy and homeland security and therefore is a major issue due to its age and current state of disrepair (AASHTO, 2008). This research suggests that it is possible to match strains predicted by a finite element model with strains from an instrumentation system. Such a calibrated model, when used in conjunction with visual inspection data, provides an expanded and objective view of bridge structural behavior. This view will help guide engineers in objectively making decisions related to maintenance based on discrepancies between the behavior predicted by a calibrated model and instrumentation data .With such a structural health monitoring instrumentation system, maintenance may be guided in a way that is economical and objective for many bridges.

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