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STRUCTURAL HEALTH MONITORING SYSTEM OPTIMIZATION FOR A BRIDGE

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ABSTRACT

This paper provides the reader with a methodology for installing and implementation an innovative structural health monitoring system (SHM) for providing better safety and asset management tools for a critically important structure in the State of Alaska. The proposed SHM was designed to assist in evaluating structural integrity and, serviceability, and to provide reliable information for changing structural response, etc. of monitored bridges.

Based on a SAP2000 (Computers and Structures) finite element model's moving load analysis, modal analysis results and field inspection, a system and sensor layout plan was developed and used to establish a bridge SHM system for the Klehini River Bridge and was used to identify the things that are needed when monitoring Alaskan type bridges. The system includes a preferred sensor layout, system integrator and instrumentation suitable for Alaska's remote locations with harsh weather.

A variety of sensors were proposed to measure and monitor structural and environmental conditions to assist in the evaluation of the performance of the Klehini River Bridge. The proposed system is capable of providing reliable information for evaluating the structural health condition. It can also be used to improve safe performance of this bridge. As a new safety and management tool, this SHM system will complement traditional bridge inspection methods. Implementation of an effective monitoring system will likely result in a reduction in inspection manpower, early detection of deterioration/damage, development of optimum inspection cycle and repair schedules before deterioration/damage grows to a condition where major repairs are required.

KEYWORDS

Structural health monitoring, bridge health monitoring, sensor priority.

INTRODUCTION

Bridges in Alaska are routinely subjected to harsh weather conditions such as extremely cold temperatures, large amounts of snow fall, wind, seismic events these structures are often located in remote areas. Maintenance, rehabilitation and replacement of these bridges in a cost effective manner depend critically on reliable inspection and condition assessment. Inspections of these bridges are both costly and time consuming. Compared with other states in the nation, bridge monitoring in Alaska is more needed but also more challenging. This is partially due to the harsh weather conditions and issues related to remoteness. For example, power is not always available at the bridge site and thereby this causes special challenges in data retrieval and reliable data communication from remote sites. To address these challenges, the overall objectives for this study were to establish a SHM system based on available knowledge and technologies for bridges in cold, harsh environments. The SHM shall provide guidelines for implementation of the SHM program. Using this system, we proposed to instrument the Klehini River Bridge to monitor its structural response to active traffic loading and to evaluate its structural condition in real time. Development and implementation of a real-time SHM program for the Klehini River Bridge should greatly enhance the ADOT&PF Bridge Section's ability to safely manage this bridge during its service life.

BRIDGE DESCRIPTION

The Klehini River Bridge is located on the Porcupine Crossing Road accessed at mile point 26.3 on the Haines Highway. The bridge structure is made of a two-span riveted steel parker truss (see Figure 1). The total length of this bridge is 74 meters (243 feet). The superstructure consists of various box sections with inverted channel sections riveted to two steel plates. The bridge driving surface is a timber deck and this deck is supported by a series of timber girders connected to transverse I-beams. Both spans rest on a central concrete abutment and the side banks.

Figure 1. Klehini River Bridge

Recent AKDOT&PF inspections reported damage in a variety of structural members; the included torn gusset plates, cracking at rivet holes, damaged or missing lateral bracing, damaged sway bracing, and etc. Weld repairs were also identified at several locations of the structural elements. Gouges, flame cut holes, bullet holes, and tack welds for cracks on the truss members were also great concerns to potential degradation.

DEVELOPMENT OF THE STRUCTURAL HEALTH MONITORING SYSTEM

This study addresses specific issues associated with the bridge in question, i.e. torn gusset plates, cracks at rivet holes, damaged or missing lateral bracing, damaged sway bracing, and the soundness of identified weld repairs on structural elements at several locations. The proposed monitoring plan includes extracting modal characteristic using accelerometers, and local diagnostic monitoring through the use of strain and crack gauges.
Since damages and deteriorations exist at many locations on the bridge, it is impractical to install sensors at all locations that are damaged. Therefore, optimization of the sensor layout for the selected bridge was based on the results of a moving load analysis, modal analysis and the latest inspection reports.

Moving Load Analysis

Three-dimensional linear elastic finite element global models of the Klehini River Bridge were prepared using SAP2000 (Computers and Structures, a finite element analysis computer program), (Figure 2). The model represented the current as-built structural configuration. Truss members, girders, stringers and floor beams were modeled using frame elements that have three translational degrees of freedom (DOFs) and three rotational DOFs at each node. The deck was modeled with shell elements.

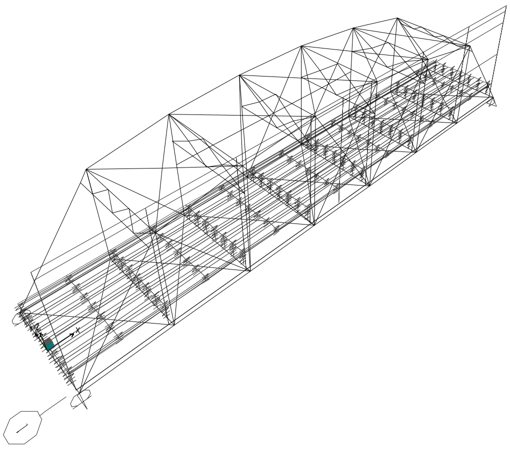


Figure 2. Global finite element model

Most of the finite element global models for the truss structure were based on the assumption that the members (elements) were connected together with hinges (the traditional axial 2-degree of freedom element). This structure is extremely rusty and bent in places. Thus, the actual member connection conditions are nearly semi-rigid. In order to estimate the influence of the member connection type on bridge response, three finite element models (Model-1, Model-2, and Model-3) were developed. In finite element Model-1, the truss element connections were assumed to be hinged. In finite element Model-2, the truss element connections were assumed to be rigid. In finite element Model-3, the truss element connections were assumed to be hinged, but the poor support conditions at the abutment caused by connections oxidation and soil build up around the support were considered in the model which is assumed the roller supports as pin point supports. It was the purpose of this exercise to find the upper and lower bound solution possibilities. Consider because of the deterioration, the expansion bearings are likely not free to rotate. Thus, we studied the influence of bearing fixity on structural response. This condition occurs only if the expansion bearings were unable to move.

Bridge bearings were modeled using pin point to connect the superstructure and pier to simulate the actual behavior and it only resist movements without resist moments. The fixed bearing behavior at a pier was modeled by simply releasing the rotational DOFs in the vertical bending plane of the bridge. For Model-1 and Model-2, the expansion bearing behavior at the abutment was modeled by assigning roller restraints in the longitudinal direction and hinge restraints in the transverse direction at the bearings. In other words, the DOFs allowed are the longitudinal translation and the vertical bending rotation. For Model-3, the expansion bearing behavior at the abutment was modeled using a fixed bearing to approximate the abutment’s poor support conditions

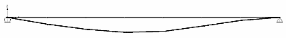
The values used from the living Load analysis were based on results from the three models discussed above; these results were used to find the critical sections and determine the predicted states of stress. There is only one traffic lane on the Klehini River Bridge and the vehicle class was defined to by three types of vehicles. The bridge was evaluated for the following vehicles: HL-93K; HL-93M; and HL-93S. The vehicles were allowed to move in both directions along one lane of the bridge. The program was used to evaluate the maximum and minimum response throughout the structure due to the placement of these different vehicles.

It was proposed that strain gauges were to be used to provide a stress history in the members. The stress history is used to determine if a member is over stressed and or if there any are bending stresses in the member. The strain diagrams for the different models provide a priority arrangement for the strain gauges. It is essential that peak compression and peak tensile strain be monitored in the member. Members that may experience fatigue are likely to have both large tension and large compression strains and these should be monitored as their failure state will be lower than those not experience stress reversals. The largest peak compression strain will likely occur at the end of the bridge in the top chord. The largest peak tensile strain will likely occur in the middle of the bridge in the bottom or lower chord. Peak compression-tensile appears at the outside of diagonals and these members are likely to experience fatigue issue.

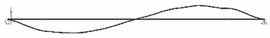
Modal Analysis

Modal analysis can be used to determine the actual stiffness of this bridge. Stiffness matrices are dominated by higher modes and flexibility matrices are dominated by lower modes. So the actual stiffness of the bridge can be identified by adjusting the stiffness matrices until the finite element model’s higher modes are equal to the measured modes. In order to measure higher modes and determine natural frequencies, an accelerometer sensor plan should be chosen based on the initial finite element modal analysis results. The positions of the accelerometers depend on the lower mode shapes in the longitudinal, transverse, vertical and rotational directions.

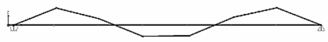
In a finite element modal analysis, natural frequencies, mode vectors and mass participation factors were determined by the Ritz–vector method. The mass participation factor for a mode provides a measure of how important the mode is for computing the response to the acceleration loads in each of the three global directions. In building design, there is a rule of thumb that the accumulated modal mass participation factor in every direction is over 90%. An analysis of the bridge specified a need for a total of 120 modes to achieve this percentage



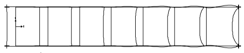
(a) 1st bending: 0.1240 s



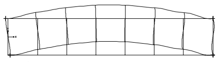
(b) 2nd bending: 0.0716s



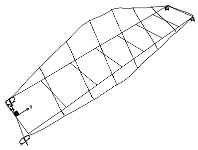
(c) 3rd bending: 0. 0534 s

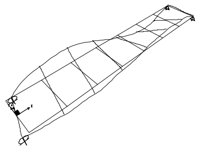


(d) 1st longitudinal: 0.1962 s



(e) 1st transverse: 0.1852 s





(f) 1st torsion: 0.0874s (g) 2nd torsion: 0.0473 s

Figure 3. Mode shapes and natural periods

The accelerometer sensor plan follows standard procedures for acquisition of dynamic properties (or signature) of the structure. Lower modes and corresponding frequencies were planned to be measured by accelerometers. From the modal analysis, lower natural periods and mode shapes for four directions have been successfully identified (Figure 3). Because of the limited number of accelerometers, the accelerometers should be placed at fixed positions to measure the first three modes and corresponding frequencies. The finite element modal analysis was used to predict the mode shapes. That gives a guideline for the placement of accelerometers.

According to the modal analysis results, accelerometers were placed at the bridge deck level (bottom chords of the trusses) along the length of the bridge to measure the natural frequencies and mode shapes of the bridge structure. This information can also be used for monitoring the global condition of the bridge and identify the accuracy of finite element model.

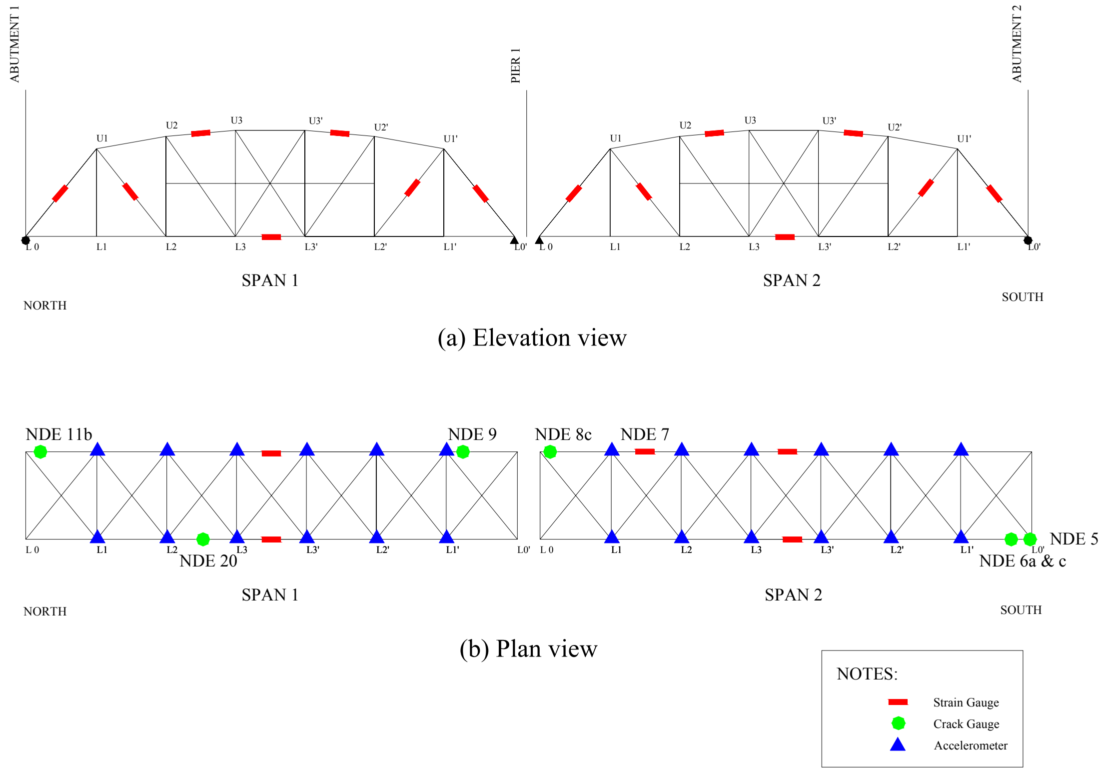
Crack Gages

The crack gauges will show movement and progression of cracking at the sensor locations. Crack gauges are also able to track the number of loading cycles for establishing remaining service life. According to the 2011 QA Services report. There were 22 cracks that were identified and seven of them were selected for crack propagation monitoring.

RESULTS AND DISCUSSIONS

Preliminary Sensor Layout

A preliminary sensor layout (including accelerometers, strain and temperature sensors, crack gauges, etc.) is shown in Figure 4. It was proposed that a total of 56 sensors should be installed for monitoring this bridge. However, since this study is aimed to monitor gradual degradation of the bridge, the sensor arrangement does not cover all the cracks but provides information about changes in the load path when cracks gradually increase in length. The design of the bridge structure allows for the use of a minimal number of temperature compensation sensors. In this case a total of four temperature sensors were placed in each truss. A preliminary structural analysis showed that the diagonal members of the trusses are fracture critical members. For this reason a strain sensor should be placed to monitor these members. This resulted in a total of eight sensors. As the main load path the lower chord members should also be monitored, especially those in the lower chords with weld repairs. Strain sensors were located near the middle points of each truss for an additional four sensors and one sensor was used to monitor the weld repair in the lower chord truss. Sixteen strain sensors were allocated for the monitoring the top chords of each truss.



(a) Elevation view

(b) Plan view

Figure 4. Preliminary sensor layout on trusses

Because of the poor conditions around the bridge supports at the abutments (the conditions included steel oxidation and a buildup of soil around the bearing supports), the expansion bearings should be monitored for rotation with tilt meters. If the supports are not free to rotate as they should, the bridge may experience twists. Finally, an additional seven crack sensors were proposed to be located near specific defects in gusset plates and channel flanges; these are to monitor for crack activity. The following table is a brief summary of number and locations of these sensors.

Table 1. Summary Number of Sensors

Sensor and Locations	**Number of Sensors**
Strain Sensors on the Top Chord Members	16
Strain Sensors on the Diagonal Members	8
Strain Sensors on the Lower Chord Members	5
Crack Sensors	7
Portable Accelerometers	12
Tilt Meter (at expansion supports)	4
Temperature Sensors	4
Total	56

Equipment

In this study, we selected fiber optic sensors because this technology is stable over long periods of time and it is ideal for use in a Structural Health Monitoring system.
The structural health monitoring system is composited by five parts: sensors, sensor multiplexer, sensor interrogator, controller & storage (Figure 5) and remote computer.

Sensor interrogator sends four optical signals (laser) to the sensor multiplexer. The multiplexer have four switchers which switch each laser to four channels and the total channel number is increased to sixteen. The laser comes to each sensor after the multiplexer. There is an optic Bragg grating in each optical sensor which can reflect certain type of wavelength back to the interrogator. The interrogator can indicate the change of optical signal and transfer the optical signal to the digital signal then send it to controller & data Storage. Controller & Storage calculates the data and send the data to remote computer by DSL internet.
Fiber optic sensors can be connected in series. Fusion splices are preferred in order to minimize loss. Armored cable, cable in conduit, or other similar type of protection helps the sensors with weather exposure, protection against animals and damage by people. The local computer, sensor interrogator and sensor multiplexer are supplied by Micron Optics, Inc.

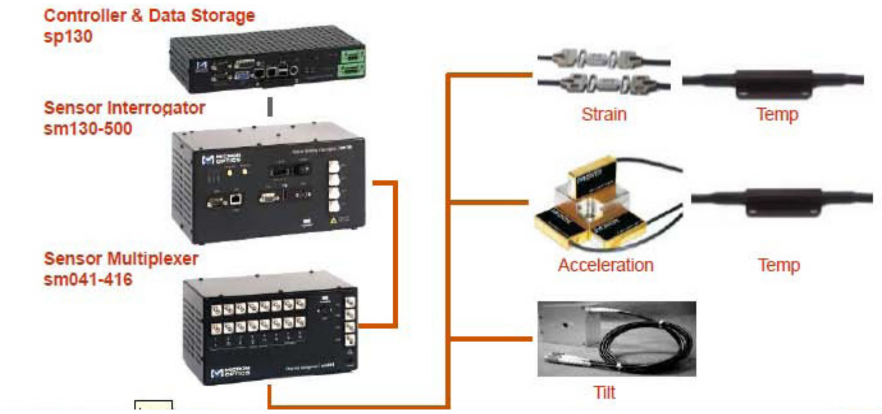


Figure 5. System configuration

CONCLUSIONS AND FUTURE WORK

This paper has presented the design method for developing a structural health monitoring system. The Fiber Optic Sensor system was selected for the harsh cold conditions. A global finite element model was built based on as-built conditions and the findings resulted from a SAP2000 3-dimensional finite element model simulating the structural condition. A Moving load analysis following the guidelines of the American Association of State Highway and Transportation Officials (AASHTO) 2007 was used to identify the critical members in this structure. Strain gages were proposed to be placed on those critical members to ensure the live load won't exceed the design limits caused by actual traffic conditions. From the modal analysis, the lowest mode shapes and natural periods in vertical, transverse, longitudinal and rotational direction were found based on the mass participation factor. The mode shapes indicated the best position to place the accelerometers. After field measurement of the mode shapes and natural periods, it is proposed that the field dynamic results will be calibrated with the finite element results which can identify the accuracy of the finite element model. The preliminary layout of crack gages was based on recent inspection reports and field inspections. Cracks were classified into three kinds: cracks at the end of lower chord lower flanges, cracks at the mid-span outside gusset plants, and weld repair at the end-span gusset plants. Cracks were selected for monitoring based on the possibility if the structure could experience expansion or contraction movements.
Using the calibrated finite element model, the load rating can be conducted based on the guidance of the Manual for Bridge Evaluation and the load rating results can show the condition of each bridge member and provide information so the bridge owner may determine if the structure should is satisfactory, needs repair or needs to be replaced.

ACKNOWLEDGMENTS

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