MONITORING OF THE
I-39 KISHWAUKEE BRIDGE

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A report of the findings of
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I-39 Kishwaukee Bridge Monitoring

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This report details the continuous monitoring of the Kishwaukee Bridge. The data collected includes measurements such as bridge deck acceleration, temperature changes, and crack opening displacement data from local deformation gages. The monitored data also includes modal frequencies, shear strain at known crack locations, and daily truck traffic. The instrument response provides needed information for real-time inspection and planned maintenance and rehabilitation. The main objective of this research was to continue monitoring of the bridge through the retrofitting contract and beyond to validate that the design and retrofitting strategy performed on the bridge arrested the crack growth. The measurements collected from this study will be used to infer possible structural changes and to guide retrofit strategies for compromised components, ensuring the bridge’s integrity and stability into the future.
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EXECUTIVE SUMMARY

The Kishwaukee River Bridges, located about 4 miles south of Rockford, Illinois, are twin post-tensioned segmental concrete box girder bridges that were constructed using a balanced cantilever technique. During the bridge’s construction, the serious problem of cracking arose. Therefore, a health monitoring system (HMS) was implemented for this bridge.

This report details the continuous monitoring of the Kishwaukee Bridge using the HMS. The data collected included measurements such as bridge deck acceleration, temperature changes, and crack opening displacement data from local deformation gages. The monitored data also included modal frequencies, shear strain at known crack locations, and daily truck traffic. The instrument response provided needed information for real-time inspection and planned maintenance and rehabilitation.

The measurements collected from this project were used to infer possible structural changes and to guide retrofit strategies for compromised components, ensuring the bridge’s integrity and stability into the future.
# TABLE OF CONTENTS

ACKNOWLEDGEMENT ................................................................................................................ i  
EXECUTIVE SUMMARY ............................................................................................................. ii  
CHAPTER 1  KISHWAUKEE RIVER BRIDGE AND CRACKS ..................................................... 1  
  1.1  INTRODUCTION ............................................................................................................ 1  
  1.2  REASONS AND SCENARIO OF THE CRACKING ........................................................ 3  
CHAPTER 2  LONG-TERM HEALTH MONITORING SYSTEM ............................................... 6  
  2.1  INTRODUCTION ............................................................................................................ 6  
  2.2  DISTRIBUTED INTELLIGENT BRIDGE MONITORING SYSTEM ......................... 6  
  2.3  SENSOR ALLOCATION ............................................................................................... 21  
CHAPTER 3  LONG-TERM MONITORING RESULTS .............................................................. 27  
  3.1  TEMPERATURE ............................................................................................................. 27  
  3.2  CRACK OPENING DISPLACEMENT ............................................................................ 30  
  3.3  TRAFFIC LOAD ............................................................................................................. 34  
  3.4  FREQUENCY ................................................................................................................. 36  
CHAPTER 4  CURRENT STATE AND RETROFIT WITH EXTERNAL POST-TENSIONING .... 39  
  4.1  CURRENT STATE .......................................................................................................... 39  
  4.1.1  Loadings .................................................................................................................... 39  
  4.2  RETROFIT WITH EXTERNAL POST-TENSIONING .................................................. 52  
  4.3  SUMMARY .................................................................................................................. 57  
CHAPTER 5  CONCLUSION ....................................................................................................... 58  
REFERENCES ........................................................................................................................... 59
CHAPTER 1  KISHWAUKEE RIVER BRIDGE AND CRACKS

1.1 INTRODUCTION

The Kishwaukee River Bridges, located about 4 miles south of Rockford, Illinois, are twin post-tensioned segmental concrete box girder bridges that were constructed using a balanced cantilever technique. The 5-span 1,090 foot long bridges with 250-foot interior spans and 170-foot end spans were opened in 1980. Each box girder carries two 12-foot lanes and a shoulder of 10 and 6 feet. The out-to-out width of the box girder is 42 feet. The height of the box girder is 1,400 inches (Figure 1.1).

As a member of the first-generation of segmental structures, the Kishwaukee Bridge contains a single shear-key joint located near the centroid of the cross-section. These joints are vulnerable during polymerization of the epoxy because non-polymerized epoxy acts as a lubricant (Wang et al., 2001b). During this polymerization process, shear forces in the web are concentrated at the shear key which acts as a corbel. After the epoxy hardens, the whole joint will carry shear forces in the form of shear stresses that are uniformly distributed along the length of the joint.

Figure 1.1. Kishwaukee River Bridge
During the bridge’s construction, the serious problem of cracking arose. After completion of the bridge, it was discovered that the epoxy did not harden properly in most of the joints. The epoxy was not able to carry any shear stresses and instead was acting as a lubricant that caused reduction of friction coefficient. Therefore, a substantial part of the shear force was concentrated at the shear keys and caused severe cracking in the web (service state). As shown in Figure 1.2, the inclined cracks went through the joint because the horizontal length of the compression struts was limited to the length of one segment.

The visual inspection of the structure revealed the presence of a large number of cracks in the web of box girders, especially in the southbound bridge (Wang et al., 2001). Eight segments on either side of each pier of both bridges suffer from cracking. The pier segments that are provided with pre-stressed diaphragms have very few cracks. The four to five segments adjacent to the pier segments show extensive cracking, and the severity of the cracking reduces as one moves towards the center of the span (toward the closure segment).

The web of northbound bridge shows a few cracks, and the angle of the cracks is about 23°-25°. The cracks usually propagate from the bottom of the female keys and pass through two or three segments. The width of most cracks is about 0.016 in (0.4mm), and the widest cracks have a width of 0.018 in (0.45 mm) (Wang et al., 2001a).

Compared with the northbound bridge, the web of the southbound bridge has suffered from extensive cracking since its construction. The crack pattern is very different even within one segment (east and west web). The angle of the cracks varies from 10° to 42°. The widest cracks are very flat, sloping at 15 degrees and usually propagating from the bottom part of the female key towards the next segment. In many segments, the cracks propagate from the bottom
part of the male keys. These cracks are usually shorter and thinner than those that propagate from the female keys. In most segments, cracks end at the opposite joint and do not extend to adjoining segments. The widest cracks, which are located near the female key, have an average width of 0.03 in (0.75 mm), and the widest is 0.04 in (1.0 mm). Observation shows that the crack widths are smaller when they are farther away from the shear keys. But in the center of some segments, cracks are about 0.022 in (0.60 mm). The most frequently observed crack width is 0.016 in (0.40 mm).

1.2 REASONS AND SCENARIO OF THE CRACKING

The inclination of the cracks indicates that the webs in the southbound bridge were exposed to a combination of shear stress and vertical tensile stress that is uncommon in this type of structure. According to the construction report (Nair et al., 1982), the southbound bridge was constructed first. Cracking and other stress concentration were observed in the southbound bridge during construction. The bridge structure was essentially complete at that time except for the parapets and wearing surface. Obvious evidence of stress concentration was found near the joint between pier #1 and the first segment on the north side. In the east web of the box girder, the bottom of the male key at this joint had crushed and the inner surface of the key had spalled. The outer surface of the web of the pier segment had delaminated and spalled over a large area. These conditions are shown in Figure 1.3. There appears to have been a relative vertical movement or slip of about 0.5 inch (13 mm) in the east web at the distressed joint. The magnitude of this slippage could be inferred from the gap that appeared at the top of the shear key. There was no evidence of any relative vertical displacement at the joint in the west web. The upper surface of the top slab also did not show any sharp step at the joint, even above the west web. The relative movement at the web, combined with the absence of any noticeable movement at the top of the slab, indicated that there must be a horizontal delamination in the slab, probably in the plane of longitudinal post-tensioning bars. This was confirmed by testing with delamination detecting equipment.

Figure 1.3. Spalling and slipping in Kishwaukee River Bridge.
Examination of the distressed joint revealed immediately that the epoxy had not hardened properly and was soft and plastic. Subsequently, non-hardened epoxy was also found in a large number of other joints in the bridge. The epoxy was not able to carry shear stresses and instead was acting as a lubricant that caused a reduction in the friction coefficient. Therefore a substantial part of the shear forces were concentrated on the shear keys and became a cause of high vertical tensile stresses in the web, particularly close to the border of the female and male shear keys. The combination of shear and axial stresses caused the principal stresses to become higher than the tensile strength of concrete, see Figure 1.4a. When the concrete cracked, the stresses in the closest reinforcement raised considerably. Since the stiffness of cracked reinforced concrete was less than the uncracked sections, increasing loads due to the weight of further erected segments caused these cracks to propagate into the stiffer parts until they reached the opposite joint, see Figure 1.4b and Figure 1.4c. The first crack (Cracking pattern ‘A’) had appeared at the female shear key. Then, the cracking pattern ‘B’ appeared with increasing loads at the male key. Cracking pattern ‘C’ had developed due to the bonding between concrete and reinforcement.

Since shear flow $\tau_{Rj} = b_w \tau_{Rj}$ in the lower part of the joints has increased the stress concentration at the female key, cracking pattern ‘A’ which spread from the female key is wider and more extensive than cracking pattern ‘B’, as shown in Figure 1.4. $\tau_{Rj}$ is the residual shear resistance of the joints due to friction under high compressive axial stresses of concrete caused by pre-stressed strands. The stress analysis (Wang et al., 2001a) showed that the highest steel stresses are in the shear reinforcement and are located closest to the female shear key. The crack width of 0.03 in (0.75 mm) indicates that the reinforcement may have yielded.

A different crack pattern was observed in the segment #1 closest to the piers. These pier segments are directly supported by bearings. So part of the shear force can flow directly to the pier through the so-called compression struts as shown in Figure 1.5. Thus, despite the fact that the highest shear forces are present in this area, the webs of segment #1 usually have only a single major crack with a maximum width less than 0.016” (0.40 mm). However, the limited shear capacity of the joints had changed the compression stress field in the webs. A very rigid direct support caused concentration of compressive stresses at the shear keys. Shear keys were subjected to highly compressive stresses with various magnitudes depending on $\tau_{Rj}$ at the joints.
The best indication of above phenomenon is an event that happened 13 days after completion of the bridge, when the male shear key of segment SB1-N1 crushed and the inner surface of the key broke, as shown in Figure 1.3. This resulted in the slip of the web at the joint. Relative vertical movement between the pier segment and segment #1 was 5/8 in (16 mm). It has been conjectured that the bridge had been saved from structural collapse by the dowel effect of 94 pre-stressed strands crossing the joint (Wang et al., 2001a).

After bridge engineers had found the cracking problem, the joints were tested for the quality of epoxy, and defective joints were repaired by using steel pins. The pins were inserted and made the once smooth contact surfaces of the joints indented (toothed). Toothed surfaces have substantially improved shear resistance at repaired joints. Furthermore, steel pins have improved the transition of shear stresses across the joints. However, some flaws remained after the retrofit and prevented the bridge from completely returning to original status. Among the flaws were that redundant, steel stresses had not been removed from the shear reinforcement; highly compressive stresses in the concrete at the shear keys had not been reduced; and wide cracks had not been sealed.

Those steel pins seem to stop the propagation of shear cracks. However, it is unclear whether the shear cracks will remain stable in the future. Therefore, a long-term health monitoring system (HMS) is indispensable and was installed on the structure to provide continuous health information for the management of this bridge. Before the installation of this monitoring system, two static load tests and eight dynamic tests were conducted on the southbound bridge to determine its current health and to provide a baseline for the long-term HMS (Wang et al., 2000).
CHAPTER 2  LONG-TERM HEALTH MONITORING SYSTEM

2.1  INTRODUCTION

Eight dynamic tests, two static load tests, and a half-scale I-beam model test were performed from 1999 to 2000 in order to determine the existing health condition of the Kishwaukee Bridge. Based on these test results and subsequent FEM model analysis, a database containing the baseline health was established for long-term monitoring of the Southbound Kishwaukee Bridge. During the period of January through December 2001, IDOT contracted the Bridge Research Center of the University of Illinois to develop a smart monitoring system titled *Development and Installation of a Permanent Health Monitoring Station for the Southbound Kishwaukee Bridge*. The monitoring system has been in continuous operation along with maintenance activities since December 2001; the system has recorded temperature compensated strain, displacement, and mode frequencies data since then. The recent bridge diagnosis system with Fuzzy algorithms brought continuous health evaluation from global dynamic analysis to local static damage assessment online. This chapter will introduce the features of the HMS, including the types of sensors and their installed locations, and considerations to the analysis of the resulting data.

The major purpose of the bridge monitoring system developed for the Southbound Kishwaukee Bridge is as follows:

1) To enable users to monitor the health of bridges in real-time via internet.
2) To detect significant damage or degradation in structural components based on confidence limits driven statistically.
3) To automatically provide multi-level warnings to the bridge authority in time.

2.2  DISTRIBUTED INTELLIGENT BRIDGE MONITORING SYSTEM

Distributed Intelligent Bridge Monitoring System (DIBMS) is a new kind of bridge health monitoring system with a modular concept. It consists of three hardware systems and five software modules.

The features of DIBMS include:

1) *Ethernet-based data transmission*
   Sensors can be distributed in different locations and all data can be transmitted via internet from bridge to central server.

2) *Web-based application*
   Web interface is implemented between users and the system, making the client platform independent and easy to access.

3) *Multiple representations of data*
   Multiple methods to represent data: reports, graphs, data sheets, etc. Data can also be downloaded to local machine for further usage.

4) *Automatic processing and analyzing*
   Appropriate algorithms will be applied instantaneously on the collected data. Statistical data and analysis results will be generated and stored.

5) *Alarm/warning system*
   A multi-level alarm/warning system is embedded to locate sensor malfunctions, system errors, and structural damages in real-time.

6) * Powerful database system*
The database system allows for efficient operations with huge amount of data, and the database can be managed and maintained easily.

7) **User and enhanced security management**
   Multiple roles and privileges are designed to make the system secure and easily manageable.

8) **Flexible and expandable**
   It is convenient for users to add new sensor channels, bridge types, and additional functions of health evaluation in the monitoring system.

9) **Platform independent**
   The application software is developed with Java, which can be run on independent and different operation platform.

### 2.2.1 Structure of the DIBMS

As shown in Figure 2.1, the hardware system of DIBMS consists of two major parts: a sensor substation system and a main server system (including DB server system and application server system).

The hardware system of DIBMS has the following advantages:

- Distributed data pre-processing strategy
- Optimized modular structure strategy
- High capability of data archiving
- High efficiency of data transmission and processing
- High durability and reliability
- Convenient and easy maintenance

As shown in Figure 2.2, the software system of DIBMS consists of five modules: (1) input and pre-processing module; (2) database module; (3) analysis module; (4) web server/user interface module, and (5) expert system module. A detailed description about these subsystems and modules will be provided in the following chapters.
2.2.2 Sensor Substation System

In the traditional design of remote bridge monitoring systems, the data acquisition system in the bridge collects data from all the sensors then sends it back to the server in the administrator office. Only basic data manipulation such as filtering is performed at the bridge. All of the original data is transferred to the server. Hence for real-time health monitoring, it has been a challenge to handle the large amount of data that is generated automatically on a daily basis. The data places a heavy burden on the data transmission network and nearly overwhelms the hard disk of the server. Therefore, the concept of a distributed sensor substation, in which data can be pre-processed before transferring, was proposed (Wang et al., 2004). The purposes of data preprocessing can be described as follows:

- To release the burden of main servers.
- To reduce unnecessary data transmission.
- To save the space of database.
- To improve the efficiency of health assessment.

Software Modules:

1) Input & Pre-process Module: data collecting, data pre-processing, data transmission, and primary early warning.
2) Database Module: data archiving and data managing.
3) Analysis Module: real-time data post-processing for statistical analysis.
4) Web Server/User Interface Module: user management and information share.

Figure 2.2. Architecture of DIBMS software system.
The sensor substation has the following components (Figure 2.3):

- Microprocessor for data acquisition and preprocessing.
- Sensor network connection for network expandability.
- Ethernet controller for data transmission.
- Flash memory for data archiving.
- System backup for crash recovery and system protection.

The distributed sensor substation has a set of embedded processing programs. In the software system of DIBMS, it’s named Input and Pre-process Module. This module has the following major functions:

- Data acquisition: It collects real time data from the sensor network.
- Pre-Processing:
  - Data validation
  - Digital filtering
  - Temperature effect, traffic effect and FFT calculation
  - Data formatting & archiving
- Data transmission: Raw data and pre-processed data will be transferred and saved into the DB servers via internet.
- Early warning: Based on the pre-processed data compared with some preset thresholds, a sensor substation can send an early warning to the system administrator.
2.2.3 Database Server System

As shown in Figure 2.4, DB server hosts the four kinds of databases:

- Raw data database
- Processed data database
- Health data database
- Configuration database

![Figure 2.4. Scheme of database system.](image)

**Raw Data Database**

The raw data database stores and archives the data from the input and pre-process module. Both the analysis and expert system modules will use this data to analyze the health status of the bridge. The archiving mechanism for the raw data database is shown in Figure 2.5.

Due to the high acquisition frequency of sensors, data will accumulate quickly in the database (DB). This accumulation negatively affects the performance and maintainability of the table. To solve this problem, after a certain time interval (e.g., 1 day), the accumulated raw data will be moved to another archive table, and the current table will be cleared. This job is accomplished by a program that runs automatically on the database server. By this process, the raw data DB can be kept within a reasonable size. A user can also access the archive data via web when necessary. When the archive table has too much data (e.g., after 2 years), users can export history data to other media such as CD-R, DVD, or tape. If a user still wants to query this exported data, another dedicated DB or DB server can be established to handle the request.
Processed Data Database

The database for processed data stores and archives processed data generated by the analysis module. The data will be used by the expert system to analyze the health status of bridges. Figure 2.6 shows the flow chart about the generation of processed data database.

![Flow chart for processed data generation](image)

**Figure 2.6.** Generation of database for processed data.
**Health Database**
This database stores and archives the health data, health reports, and alerts generated by the expert system module.

**Configuration Database**
- Preset Database: To store the original static and dynamic characteristics of the bridge for health assessment, such as the bridge’s geometric and material parameters, original natural frequencies, design life, etc. These characteristics are determined after construction of the bridge is finished and cannot be modified.
- Adjustable Database: To store processing parameters of HMS, such as sampling rates, multi-level thresholds for warning/alert, etc. Privileged users can access and adjust these parameters via a web browser.
- System Database: To store all information pertaining to system management, such as a list of authorized users and their privileges and system configurations. The system administrator can manage this information via a web browser.

**2.2.4 Application Server System**
The application server system hosts the software modules (Figure 6.7).

**Analysis Module**
- Classical Statistical Method: Assume the Central Limit Theorem (CLT).
  - Averages taken from any experimental data will have a normal distribution.
  - The error for such a method will decrease slowly as the number of observations increase.
- Bootstrap Method:
  - The bootstrap method attempts to determine the probability distribution from the data itself, without recourse to CLT (Efron et al., 1993).
  - The bootstrap method is not a way to eliminate the error but to estimate it.
The most fundamental idea of the bootstrap method is to use some form of re-sampling with replacement from the actual data $x$, to generate $B$ bootstrap samples $x^*$ (Beran, 1988). Properties expected from replicated real samples are inferred from the bootstrap samples by analyzing each bootstrap sample exactly as the real data sample was analyzed. From the set of results of sample size, the inference uncertainties $B$ were measured from sample to (conceptual) population (Silverman, 1986). Suppose there are $n$ samples $x_i$. By randomly drawing samples $x$ with replacement, it will generate a large number of bootstrap sample sets $x^* 1, x^* 2, x^* 3, \ldots, x^* B$, where $B$ is the times of re-sampling. Then the bootstrap mean is,

$$
\varepsilon(x) = \frac{\sum_{b=1}^{B} \varepsilon(x^*_b)}{B} \tag{2-1}
$$

Bootstrap estimate of the standard error equals,

$$
se_{boot} = \sqrt{\frac{\sum_{b=1}^{B} [\varepsilon(x^*_b) - \varepsilon(x)]^2}{B - 1}} \tag{2-2}
$$

After obtaining $B$ bootstrap estimates of the parameter $m$ (mean), we can calculate the standard error of $m$ based on standard deviation of $B$ bootstrap estimates $SE_B$;

$$
[LCL, UCL] = [\hat{\mu} - t_{1-\alpha/2,n-1} \times SE_B, \hat{\mu} + t_{1-\alpha/2,n-1} \times SE_B] \tag{2-3}
$$

where $\hat{\mu} = \text{mean of the sample ( } \bar{x} \text{) ;}$

$SE_B = \text{standard deviation of the sample;}$

$t_{1-\alpha/2,n-1} = \text{t-statistic at } \alpha \text{ with } n-1 \text{ degrees of freedom.}$

The features of the Bootstrap Method are described as follows:

- Good agreement for Normal (Gaussian) distributions.
- For non-normal distributions, particularly for the tails, bootstrap method is better than classical estimators.
- Bootstrap method has no advantage over classical estimators when the sample size is large.
- With smaller sample sizes, the bootstrap method obtains better results than classical estimators.
- Bootstrap method is significantly more time-consuming than classical estimators.

**Expert System Module**

An expert system is a computer program that includes a representation of the experience, knowledge, and reasoning processes of human experts (DeWolf et al., 1989).
As shown in Figure 2.8, a rule-based expert system consists of (Daniel et al., 1992):

- **User interface**
  
  The user interface of expert system is a part of the web server/user interface module of DIBMS. It has the following features:
  
  - User-friendly
  - “Intelligent”:
    - Knowledge of how to present information
    - Knowledge of user preferences

- **Working Memory**
  
  The working memory of expert system is also called the finger-print bank (Database). It includes the following information:
  
  - The related codes of AASHTO, ACI and PCI for bridge evaluation
  - The accumulated essential information from the bridge monitoring system
  - Human users’ knowledge

- **Inference Engine**
  
  The inference engine controls overall execution of the rules. If a rule’s antecedent is satisfied, the rule is ready to fire and is placed in the agenda. When a rule is ready to fire it means that since the antecedent is satisfied, the consequent can be executed. It’s a type of reasoning mechanism, i.e., so-called chaining (Elkordy et al., 1993).

  There are two types of chaining: forward and backward. In forward chaining, the expert system is given data and chains forward to reach a conclusion. In backward chaining, the expert system is given a hypothesis and backtracks to check if it is valid. In DIBMS, backward chaining is used to derive the actual condition of a bridge based on rules which were determined from the bridge design codes and engineering experience.

- **Rule Base**
  
  Rule base is also called knowledge base. It’s the most important part in a rule-based expert system (Buchanan B., et al., 1984). It consists of:
  
  - Rules specified for a specific bridge. (All the geometric, material and structural parameters should be provided by bridge designers.)
  - Rules are of the form IF condition THEN action.
  - Rules can be specific, a priori rules - represent laws and codified rules.
- Rules can be heuristics. "Rules of Thumb" - represent conventional wisdom.
- Rules can be chained together.
- Certainty factors represent the confidence that a fact is true or a rule is valid.

Currently we use a Fuzzy logic algorithm in the expert system module.

**Web Server/User Interface Module**

This module is the interface between the system and the users. It implements a web application that allows users to access the Bridge Monitoring System via a user agent (e.g. browser). Through a web browser, a user can retrieve historical and real-time raw and processed data, statistical and advanced analysis results, bridge health diagnostic reports and maintenance suggestions related to the monitored bridge. The system administrators can perform the management tasks via this interface. A detailed example will be given in the next chapter.

This module offers the following functions to users:

- Data display (historical and real-time)
- Statistical data review
- Health report display
- Notification of warnings and alerts
- Maintenance suggestions
- Data downloading
- System configuration
- Data analysis configuration
- User and privilege management

### 2.2.5 Hardware Development of DIBMS

The first extended deployment of the completed hardware system with remote data transmission capabilities and remote operations occurred on December 10, 2001. At this time, all field hardware components were transported to the south-bound Kishwaukee Bridge and installed in the north side of pier 2. Approximately half of the gauges were connected to the monitoring system on December 10. The remaining sensors were installed prior to 2002 by graduate students at UIC. A more detailed description of the monitoring system components is presented in Section 2. In short, the monitoring system consists of a rack-mount UNIX-based PC (top chassis) which controls a multi-functional data acquisition card and modem. Signal conditioning modules and anti-aliasing filters are assembled in the lower chassis. To facilitate transportation and installation, these components are mounted in a ruggedized and vibration-isolated industrial rack enclosure. The enclosure contains an internal heater to keep the system warm in cold weather. Although equipped with a keyboard and mouse, the monitoring system is automatically powered-on and can be shutdown through a simple pushbutton sequence on the **Modem Status Display**. These conveniences are designed to ease setup and operation.

During the system calibration and debugging process, 56Kbps dialup modems were used to build a data transmission network with point-to-point TCP/IP protocol. In July 2003, a 128Kbs ISDN line was installed and the original dialup telephone line became the optional back-up communication tool. As of November 2004, an ADSL business line has been setup to provide the real-time data transmission at up to 1.5 Mbps/384Kbps.

The sensor substation system is designed based on a Microstar Laboratories DAP4000a 212 14bit data acquisition board that was installed in a Linux-based (SuSe 7.2) AT-industrial chassis, as shown in Figure 2.9. Based on the considerable number of reviews existing for
similar products, it was decided that this board would be ideal for the moderate level of analog signal monitoring required for this project. In addition to satisfying the necessary precision and aggregate bandwidth, the on-board processor of the DAP4000a greatly minimizes the programming efforts required for basic structure monitoring analyses. The board is operated with the Linux OS via supplied drivers and tested by the board manufacturer.

![DAP4000 DSP Board](image1)

(a) DAP4000 DSP Board

![Front Panel](image2)

(b) Front Panel

Figure 2.9. Sensor station in RPD MA-B AT rack-mount chassis.

The analog inputs to the board are brought to an expansion chassis with an interface backplane. Analog interface cards were selected to allow channel-by-channel sensor conditioning for the complement of strain gauges, accelerometers, displacement transducers, and thermocouples selected for bridge monitoring. The calibration for each channel is certified, the channels are reconfigurable if necessary, and they may be individually replaced in case of a malfunction. Equipment satisfying these specifications was selected in order to maintain the long-term viability of the system and to simplify any required maintenance.

### 2.2.6 Software Development of the DIBMS

The bridge monitoring software was installed on the Kishwaukee Bridge in 2001. Since this time, it has been collecting and processing data as well as generating evaluation and health reports. The expert system has been comparing the frequency distribution, shear stress/strain in the web reinforcement, traffic data, and curvature effect for the bridge. An automated warning/alarm system is in effect to provide notification of any local structural damage, system problems, sensor malfunction, or data errors. Data processing and results obtained from the Kishwaukee Bridge are shown in the next section.
Access to the system is password protected. Upon logging in with username and password, the first page provides basic information such as the status of the sensors and bridge health, temperature, and network connection, etc., as shown in Figure 2.10. A selection interface allows the user to choose their preferable monitoring segment by either clicking on the drawing or navigating a menu of text descriptions on the side bar. The user can also choose to view real-time monitoring, archived data, or the expert system analysis.

**Real-time Data**

By selecting the sensor channel and clicking display, the real-time data trend can be viewed. The preprocessing of data is also performed in real-time and can be displayed. As shown in Figure 2.11, frequency response from the acceleration data and response due to traffic effect can be viewed simultaneously in real-time.
Many channels can be viewed simultaneously. Details of a sensor channel are obtained by clicking on show details. The location on the bridge of each sensor and preprocessed data can be viewed. It is possible to zoom-in/zoom-out both x and y-axis to view the chart.

**Historical Data**

Old data is archived in the historical database, from which the user can select a range of days/dates to view. This module includes data charts and statistics reports related to all channels. Data from multiple channels can be retrieved in groups based on their relationships and can be compared upon user request. For strain and crack opening channels, the traffic effect data can also be retrieved. Figure 2.12 shows the historical information of strain (Channel: SB2-N16-E-S5) from April 1 to October 1, 2002.
For dynamic data, the frequencies of the first five modes each day are available to users. Analytical and statistical reports can also be obtained. The data can be viewed and downloaded in text format by clicking on view datasheets.

**Bridge Health Diagnosis**

This module utilizes statistical algorithms and rule-based strategies to analyze data for health reports. Various advanced analysis procedures can be performed by comparing the current data with the historical data. This module can be customized for more advanced analysis according to the user’s requirements. Frequency distribution is performed with two different methods in order to obtain the best confidence interval from the available data. Figure 2.13 shows the real distribution and bootstrap distribution of mode frequencies of the bridge.

![Image of historical database interface](image.png)

*Figure 2.12. Interface of historical database.*
Two types of assessment can be used:

- **Global Health Assessment:**
  - Frequency Analysis: Load Distribution and Integrity
  - Fatigue Analysis: Truck Traffic & Fatigue Life
- **Local Damage Assessment**
  - Strain/Curvature Analysis: Bending Stiffness and Capacity
  - Crack Opening Displacement Analysis: Shear Stiffness and Capacity
2.3 SENSOR ALLOCATION

2.3.1 Overview

Based on the results of the on-site dynamic tests, half-scale model test, and FEM simulation, it was determined that a long-term health monitoring system for the Kishwaukee bridge needed to consist of four types of sensors installed at critical positions along the bridge (Table 2.1):

1) Electrical Resistance Strain Gauges (SB2-N16 and SB2-S16)
2) LVDT sensors (Linear Variable Differential Transformer) (SB2-N4)
3) Thermocouples (SB2-N4)
4) Accelerometers (SB2-N13).

Figure 2.14 shows the location of the sensor substation and four kinds of sensors on the southbound bridge.

Figure 2.14. Sensor locations in Kishwaukee Southbound Bridge (Unit: m).
2.3.2 Strain Gauge Pairs

Routine strain monitoring conducted by strain gauges at critical positions can provide information on load and stiffness. Seven electrical resistance strain gauges (PL-60-11-3LT, Tokyo Sokki Kenkujo Co. Ltd.) were installed on the inside webs of segments SB2-S16 and SB2-N16. Another strain gauge (the eighth) was mounted on the surface of an undamaged concrete block as a reference gauge. Figure 2.15 shows strain gauge locations installed on one of the segments. The measured concrete strains were used to calculate flexural (bending) stiffness and to analyze traffic loads.

The strain gauges’ inspections occurred to determine if the connection between the strain gauge and concrete had deteriorated. Two years after the first installation, six of the eight strain gauges were found to be debonded. Hence in April 2004, eight old strain gauges were replaced with the same model of new gauges. Since then, no evidence of deterioration was found and no modification to strain data was made.

Table 2.1. Channel # and Corresponding Positions

<table>
<thead>
<tr>
<th>Channel#</th>
<th>Type</th>
<th>Location</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Thermocouple</td>
<td>SB2-N4-East</td>
<td>in concrete of the web</td>
</tr>
<tr>
<td>2</td>
<td>Thermocouple</td>
<td>SB2-N4-West</td>
<td>in concrete of the web</td>
</tr>
<tr>
<td>3</td>
<td>Thermocouple</td>
<td>SB2-N4-Top</td>
<td>in concrete of the slab</td>
</tr>
<tr>
<td>4</td>
<td>(reserved)</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>5</td>
<td>Thermocouple</td>
<td>outside SB2-N4</td>
<td>in the air</td>
</tr>
<tr>
<td>6</td>
<td>Thermocouple</td>
<td>SB2-N4-Bottom</td>
<td>in concrete of the slab</td>
</tr>
<tr>
<td>7</td>
<td>Thermocouple</td>
<td>SB2-N1</td>
<td>in the box on the wall</td>
</tr>
<tr>
<td>8</td>
<td>Thermocouple</td>
<td>SB2-N4</td>
<td>in the substation</td>
</tr>
<tr>
<td>9</td>
<td>LVDT Sensor</td>
<td>SB2-N4-West</td>
<td>on the external surface</td>
</tr>
<tr>
<td>10</td>
<td>LVDT Sensor</td>
<td>SB2-N4-West</td>
<td>on the external surface</td>
</tr>
<tr>
<td>11</td>
<td>LVDT Sensor</td>
<td>SB2-N4-West</td>
<td>on the internal surface</td>
</tr>
<tr>
<td>12</td>
<td>LVDT Sensor</td>
<td>SB2-N4-West</td>
<td>on the internal surface</td>
</tr>
<tr>
<td>13</td>
<td>LVDT Sensor</td>
<td>SB2-N4-East</td>
<td>on the internal surface</td>
</tr>
<tr>
<td>14</td>
<td>LVDT Sensor</td>
<td>SB2-N4-East</td>
<td>on the internal surface</td>
</tr>
<tr>
<td>15</td>
<td>LVDT Sensor</td>
<td>SB2-N2</td>
<td>as a reference sensor</td>
</tr>
<tr>
<td>16</td>
<td>(reserved)</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>17</td>
<td>Strain Gauge</td>
<td>SB2-S16-East</td>
<td>on the top of the web</td>
</tr>
<tr>
<td>18</td>
<td>Strain Gauge</td>
<td>SB2-S16-East</td>
<td>on the bottom of the web</td>
</tr>
<tr>
<td>19</td>
<td>Strain Gauge</td>
<td>SB2-S16-West</td>
<td>on the top of the web</td>
</tr>
<tr>
<td>20</td>
<td>Strain Gauge</td>
<td>SB2-S16-West</td>
<td>on the bottom of the web</td>
</tr>
<tr>
<td>21</td>
<td>Strain Gauge</td>
<td>SB2-N16-East</td>
<td>on the top of the web</td>
</tr>
<tr>
<td>22</td>
<td>Strain Gauge</td>
<td>SB2-N16-East</td>
<td>on the bottom of the web</td>
</tr>
<tr>
<td>23</td>
<td>Strain Gauge</td>
<td>SB2-N16-West</td>
<td>on the top of the web</td>
</tr>
<tr>
<td>24</td>
<td>Strain Gauge</td>
<td>SB2-N2</td>
<td>as a reference sensor</td>
</tr>
<tr>
<td>25</td>
<td>Accelerometer</td>
<td>SB2-N13</td>
<td>vertical</td>
</tr>
<tr>
<td>26</td>
<td>Accelerometer</td>
<td>SB2-N13</td>
<td>transversal</td>
</tr>
<tr>
<td>27</td>
<td>Accelerometer</td>
<td>SB2-N13</td>
<td>longitudinal</td>
</tr>
</tbody>
</table>
2.3.3 LVDT Sensor Pairs

Based on the test result and FEM analysis, we found Segment SB2-N4 to have the most severe shear damage on both its west and east webs. To measure the shear crack opening displacement (COD) on the box-girders of Kishwaukee Bridge, seven LVDT sensors were installed on the bridge (CDP-5 LVDT sensor, Tokyo Sokki Kenkujo Co. Ltd). Among those, two sets of LVDT sensor pairs (inclined and vertical) were mounted on both sides of the west web of SB2-N4. Another pair were mounted on the inside surface of the east web of the same segment. The seventh LVDT sensor was installed on the surface of an undamaged concrete block as a reference.

Figure 2.16 shows the locations of LVDT sensors installed in the bridge. Crack opening displacement (COD) recorded by the LVDTs can indirectly indicate the stress level of the shear reinforcement. This is useful for detecting the possible yielding of the shear reinforcement, and estimating the shear stiffness of bridge webs. COD values are also used in the traffic analysis.

Monthly inspections were performed to ensure that there was no debonding in the epoxy layer between the concrete surface and the LVDT. Results of the inspections indicated that debonding was not a problem and therefore there was no need to replace the LVDT sensors.
2.3.4 Thermocouples

Along with using transducers to measure cracking and strain, it has been proven that having temperature measured and compensated for is very important, as both the transducers and physical behavior of the structure are sensitive to its variations. Therefore, seven thermocouples were installed on the bridge to measure temperatures of the concrete and external and internal air. The thermocouple in the sensor substation provides temperature information for the automatic A/C control to ensure proper working conditions.

Figure 2.17 shows the location of thermocouples. All seven thermocouples are in good condition and are reliable for long-term monitoring. Results from both static and dynamic analyses are satisfactory after temperature compensation.
2.3.5 **Accelerometers**

The dynamic characteristics of a bridge are generally related to its stiffness, distribution of mass, and damping ratio. The dynamic responses of a bridge (such as frequency, damping ratio, and mode shape) are functions of the structural physical properties. It is therefore possible to monitor the dynamic characteristics of a structure for damage detection and health assessment. Accelerometers are used to measure these characteristics and their locations should be decided based on the following criteria:

1) To avoid the node of the mode shape.
2) To capture as many frequencies as possible.

In order to monitor the dynamic properties of the bridge, a tri-axial accelerometer was installed on the insider surface of the bottom deck of the Kishwaukee Southbound Bridge (Model 8392B2, K-Beam Accelerometer, Kistler Instrument Corp.). The accelerometer was fixed to a steel pad as shown in Figure 2.18.

According to the above placement criteria and the eigen-sensitivity calculation of the bridge, Segment SB2-N13 was chosen for the location of the accelerometer. Monthly inspections began shortly after installation and indicated that the accelerometer was working...
properly. In July 2003, a new high-accuracy single-axial accelerometer was installed at the same position to improve the resolution of vertical modes 2 and 4. The three accelerometer channels are AC coupled (gain of 2), with anti-aliasing pre-filtering with a cutoff frequency of 45 Hz.
CHAPTER 3 LONG-TERM MONITORING RESULTS

3.1 TEMPERATURE

Temperature measurements were recorded over a 5-year period from January 2002 to December 2006. Data from the seven thermometers were saved at 5-minute intervals into a server located in the UIC laboratory. Although some data were lost due to power supply and internet connection problems, the total loss over the five-year period was less than 30%. As shown in Figures 3.1 through 3.4, the approximately 370,000 data values saved at each node were sufficient to analyze and determine temperature patterns.

We can confirm the pattern of yearly changes from Figure 3.1. According to the amplitude of the temperature, the shaded outside temperature (T5) is highest and 20% higher than temperatures in the webs of the concrete box girder (T1 and T2).

![Figure 3.1. Five-year temperature recorded every 5 minutes. (To be continued)](image)
Figure 3.1. Five-year temperature recorded every five minutes.

Figure 3.2. Four-day temperature records of all thermometers.

The temperature lag of the concrete varies at different positions as shown in Figures 3.2 and 3.3. Based on temperature readings of the external shaded thermometer, the temperature
lag in the concrete at top and bottom positions is approximately 4 hours, while the lag in the web is 6 hours. In addition, temperature variations are also different. The temperature variation of the external shaded thermometer is about two times that of top concrete, three times that of bottom concrete, and six times that of web concrete. When the temperature variation of shade outside, for example, is 9°C, the temperatures of the top, bottom, and web concrete are 4.5°C, 3°C, and 1.5°C, respectively. These ratios and tendencies of the structural temperature are consistent in the summer and winter. We call these temperature gradients.

It is shown that temperature variations and lag at different positions of the same section are significantly different within a day. The reason is because the volume of concrete at each position is various, and the effective time to exposure to the outside temperature is not enough to equilibrate. If the exposure time is enough, the temperature variations at each position become equal as shown in Figure 3.4. The positional variation of the temperature lag can be reduced effectively by averaging the data on a daily basis. Use of the daily average is quite convenient and effective for discriminating the temperature effect on the long-term displacements or frequencies.

![Figure 3.3. Two-day temperature records during winter and summer 2004.](image)

![Figure 3.4. Daily average temperature records in July 2002 (summer) and Jan 2003 (winter).](image)

The maximum difference between positive and negative temperature was determined. The measured gradients are compared with the currently specified design gradients in the
AASHTO LRFD specifications (2002). Figure 3.5 shows the largest difference between data from the top (4 inch below the top of the deck), bottom (the surface of the bottom deck), and the coolest web thermocouples. The maximum positive gradient is produced in the summer (May 28, 2006), and the maximum negative gradient in the winter (Feb. 18, 2006). The positive gradient can appear even in winter when the bottom concrete temperature is higher than the top. However, the maximum always appears in summer because the temperature of top concrete in winter is much lower than that of bottom concrete.

![Figure 3.5. Temperature differences in 3 years.](image)

![Figure 3.6. Maximal negative and positive thermal gradients.](image)

A comparison was also made between the gradient magnitude in design specifications and the measured maximal daily variations. The magnitudes of the measured gradients are slightly larger than the design gradients, but the shape is similar to the specifications (Figure 3.6).

### 3.2 Crack Opening Displacement

Crack opening displacement was measured for a 5-year period, from January 2002 to December 2006. Data from the 4 LVDTs, two inclined and two vertical, were saved at 5-minute intervals into the UIC laboratory server. Although some data was lost due to power supply and
internet connection problems, the total loss over the 5-year period was less than 30%. As shown in Figure 3.7, the approximately 370,000 data values saved at each node were sufficient to analyze and determine crack opening patterns.

![Graphs showing monitoring results of COD within 5 years.](a) SB2-N4-W-C3, inclined  
(b) SB2-N4-W-C4, vertical  
(c) SB2-N4-E-C5, inclined  
(d) SB2-N4-E-C6, vertical

Figure 3.7. Monitoring results of COD within 5 years.

For the evaluation of long-term monitoring data in a statistical manner, crack displacements and temperature data are treated with daily average values. When the temperature increases, the crack is closed and LVDTs show negative values. Therefore the crack displacement is in inverse proportion to temperature as shown in Figure 3.8.
The correlation between crack displacement and temperature without a time axis is shown in Figure 3.9. By using least square methods, the linear function of crack opening due to temperature can be found as:

$$\Delta \ell(t) = \alpha \Delta T + \Delta \ell_0$$

(1)

where $\alpha$ is the temperature-displacement correlation slope, $T$ is temperature, and $\Delta \ell_0$ is a constant offset depending on the initial setting of the LVDTs. For instance, when the LVDT was installed on the west web, the temperature of the concrete was 6.7°C ($=70.39/10.47$), while the temperature of east web was 2.9°C ($=30.57/10.41$). This difference of 3.8°C, which is less than daily temperature variation, may be produced by the lag of installation. The two temperature correlation slopes of the west and east web are 10.47 and 10.41 respectively. This slope indicates the displacement, in units of micrometers that results from a unit temperature change at the measured area. Using equation (1), we can find the crack displacement due to temperature only, which is shown in Figure 3.10. After the temperature displacement is subtracted from the original data, the remains may show another pattern.
The remains, after the temperature effect is subtracted from the original data, have a tendency to gradually increase as shown in Figure 3.11. We may call the increment crack propagation due to fatigue. It is realized that COD from fatigue on the west web has increased from -94 μm to 38 μm and on the east web from -42 μm to 22 μm over the 5-year measuring period. The total crack propagations on the west and east web for 5 years are 132 μm and 64 μm respectively. This means average yearly increments on the west and east web are 26.4 μm and 12.8 μm respectively.

The difference in COD measurements between the two webs is due to the transversal non-uniformity of traffic loadings. Since most trucks are passing on the west lane of the southbound bridge, the west web certainly takes more internal shear stresses due to fatigue loading.

By using the same approach as subtracting temperature effect, the traffic pattern of COD can be evaluated. Figure 3.12 shows the result of COD after fatigue effect is subtracted from Figure 3.10.
At the west web, through which most heavy trucks are passing, the maximum and minimum distributions and variances calculated from probability density function are larger than in the east web as shown in Figure 12. These can support the conclusion we derived previously that much more fatigue cracks occurred on the west web.

![COD due to Daily Traffic load, West web](image1)

![Normal Distribution of COD, West web](image2)

(a) West web

![COD due to Daily Traffic load, East web](image3)

![Normal Distribution of COD, East web](image4)

(b) East web

Figure 3.12. Crack displacements due to traffic.

### 3.3 TRAFFIC LOAD

In 2004, the number of trucks were counted by visual measurement and the result was compared with the number of peak signals from the LVDTs. Both numbers were consistent. Based on this consistence, a threshold was established and imbedded in DIBMS to automatically count the number of trucks. The truck numbers collected from 2005 to 2007 are shown in Figure 3.13. Although some data are corrupt, we could find the pattern of daily truck numbers.

Based on the records, the daily average number of trucks was 2,500 and the maximum was around 4,200. With seasonal and weekly statistics considered, as shown in Figure 3.14 and 3.15, the number of trucks in the spring is 15% higher than in the winter, and the number of trucks on weekdays is twice the number on weekend days.
In view of long-term effects on a bridge, the traffic loads and temperature gradients, with respect to fatigue loads, may be the main source of crack propagation. In the case of the Kishwaukee Bridge, the effect of temperature gradients is two times as much as the designed live load of HS 20-44. However, fatigue load is especially controlled by loading cycles. Unlike the daily cycle of temperature gradient, the traffic load has more than 1,500 to 4,000 cycles per day. Therefore, we can conclude that the COD related to fatigue is mainly caused by traffic loads.
In order to measure the amount of the crack opening displacement due to traffic load, we measured the maximum crack opening displacement every 5 minutes for 4 months. The results are shown in Figure 3.16. The maximal COD on Sunday at midnight approaches zero.

The mean of the daily COD due to traffic load is 26.1 $\mu$m and the variance is 7.2 $\mu$m. The maximum of 50 $\mu$m corresponds to the result in previous section by using classification. The density of the data set measured for 4 months is evaluated by using the extreme value distribution. Extreme value distributions are often used to model the smallest or largest value among a large set of independent, identically distributed random values that represent measurements or observations.

![Figure 3.16](image)

**Figure 3.16** Maximal COD due to traffic load at west web

### 3.4 FREQUENCY

Data for the 1$^{\text{st}}$ and 3$^{\text{rd}}$ frequency of the superstructure was measured with the accelerometer and saved from 2004 to 2007. The frequency, which was saved hourly in an averaging manner, follows the tendency of the concrete temperature changes well. Figure 3.17 shows the tendency of both frequency and temperature.
Figure 3.18 shows correlation between the 3rd frequency and temperature using the same classification method that we applied for crack displacement and temperature correlation. We used the 2nd order polynomial function for the least square. The fitted curves are shown in Figure 3.19 and Figure 3.20. The average of 1st frequency was 1.644 Hz. And the yearly change of the 1st frequency is ±0.059 Hz, which is ±3.6% of the average.

Figure 3.17. 1st frequency and temperature.          Figure 3.18. 3rd frequency and temperature.

Figure 3.19. Correlation between 1st frequency and temperature.          Figure 3.20. Correlation between 3rd frequency and temperature.

With the correlation, we can evaluate the frequency history due to temperature as shown in Figure 3.21. The yearly variation of the frequency is ±0.050 Hz.

After the temperature effect is subtracted from original data, the remains are well distributed into a mean of zero without any other pattern as shown in Figure 3.22. The variation between the maximum and minimum of the remains was approximately ±0.025 Hz. It is a half of the yearly variation of temperature. Therefore, we can conclude that based on the long-term monitoring results there is no obvious change in the flexural stiffness of the superstructure even though the local damage on the web concrete progressed slowly.
Figure 3.21. 1st and 3rd frequencies due to temperature.

(a) 1st frequency
(b) 3rd frequency

Figure 3.22. 1st and 3rd frequency after temperature effect compensated (to be continued).

(a) 1st frequency history after temperature compensated
(b) Normal distribution
Figure 3.22. 1st and 3rd frequency after temperature effect compensated (continued).

(c) 3rd frequency history after temperature compensated
(d) Normal distribution
Figure 3.22. 1st and 3rd frequency after temperature effect compensated (continued).
CHAPTER 4 CURRENT STATE AND RETROFIT WITH EXTERNAL POST-TENSIONING

In this chapter, the current state of the superstructure is evaluated by finite element analysis. The self-weight, HS20 live load, temperature gradients, creep and shrinkage for 10000 days (27 years), and post-tension are considered in the analysis. In addition, we suggest the strengthening of the I-39 bridges over the Kishwaukee River with external post-tension. According to the strengthening concept, the final results show comparisons between current state and improved state.

4.1 CURRENT STATE

4.1.1 Loadings

4.1.1.1 Dead Load
Self weights are considered, including box girder, tendons, and Barriers.

4.1.1.2 Live Load
AASHTO Live load, HS20-44, is applied.

4.1.1.3 Thermal Gradients

Temperature gradient is applied according to Guide Specifications for Construction of Segmental Concrete Bridges, 1999.

<table>
<thead>
<tr>
<th>Zone 2</th>
<th>Positive Temp. (F)</th>
<th>Negative Temp. (F)</th>
<th>A (inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>46</td>
<td>-13.8</td>
<td></td>
</tr>
<tr>
<td>T2</td>
<td>12</td>
<td>-3.6</td>
<td></td>
</tr>
<tr>
<td>T3</td>
<td>5</td>
<td>-1.5</td>
<td></td>
</tr>
</tbody>
</table>

4.1.1.4 Post-tension
Because there is no record of longitudinal stressing for either the northbound or southbound bridges, an initial jacking force equal to 80% of the guaranteed ultimate tensile strength of the post-tensioning bars has been adopted for all erection stages. This assumption is representative of common construction practice at the time of design, and is in agreement
with transverse stressing records found in the *Preliminary Design Report, F.A. Rte 412 Over Kishwaukee River* (IDOT). Mr. Khaled Shawwaf, the Chief Engineer of Dywidag-Systems International who was involved in the redesign of the bridges, confirmed this assumption.

Based on historical data obtained from the National Climatic Data Center (NCDC) database, a relative humidity of 74% has been assigned to the environment. This value is also in agreement with Figure 5.4.2.3.3-1 of the current *AASHTO LFRD Bridge Design Specifications*. Relative humidity affects the calculated creep and shrinkage coefficients of the concrete and has a significant effect on the final distribution of force in the structure.

A classification of rapid-hardening, high-strength cement is assumed to calculate the superstructure concrete’s ultimate shrinkage coefficient. This assumption is based on the statement of “high-early-strength cement and steam heat were used” in *Design and Construction of the Kishwaukee River Bridges* (Nair et. al., 1982) with reference to the procedures used during casting of the superstructure segments.

An anchor set of 1/16” is used for all post-tensioning bars. This assumption is consistent with both the Guide Specs and the *AASHTO LRFD Bridge Design Specifications*.

### 4.1.1.5 Creep and Shrinkage

Strains are calculated in accordance with CEB-FIP 1978 Model Code for superstructures.

### 4.1.2 Materials

Material properties used in the analysis are listed in Table 4.2.

**Table 4.2. Material Properties**

<table>
<thead>
<tr>
<th>Materials</th>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Post-Tensioning Bars</td>
<td>Diameter</td>
<td>1.25</td>
<td>in</td>
</tr>
<tr>
<td></td>
<td>Area</td>
<td>1.25</td>
<td>in²</td>
</tr>
<tr>
<td></td>
<td>Wobble Factor</td>
<td>0.0002</td>
<td>/ ft</td>
</tr>
<tr>
<td></td>
<td>Friction Coefficient</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Modulus of Elasticity</td>
<td>30,000</td>
<td>ksi</td>
</tr>
<tr>
<td></td>
<td>G.U.T.S.*</td>
<td>150</td>
<td>ksi</td>
</tr>
<tr>
<td>Concrete</td>
<td>Unit Weight</td>
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<td>lb/ft³</td>
</tr>
<tr>
<td></td>
<td>f’c [Superstructure]</td>
<td>5.5</td>
<td>ksi</td>
</tr>
<tr>
<td></td>
<td>f’c [Substructure]</td>
<td>3.5</td>
<td>ksi</td>
</tr>
</tbody>
</table>

*Guaranteed Ultimate Tensile Strength*
4.1.3 Finite Element Model

Figure 4.1. Elevation layout of Kishwaukee River Bridge.

Figure 4.2. Division of cantilever elements.
4.1.3.1 Section and Properties

Table 4.3. Geometric Properties of The Cross-Section Of Concrete Components

<table>
<thead>
<tr>
<th>Segment</th>
<th>$h_s$</th>
<th>$A_c$</th>
<th>$I_c$</th>
<th>$I_z^*$</th>
<th>$S_{c,max}$</th>
<th>$z_{cg}^{**}$</th>
<th>$\kappa$</th>
</tr>
</thead>
<tbody>
<tr>
<td>#</td>
<td>[in/mm]</td>
<td>[m$^2$]</td>
<td>[m$^4$]</td>
<td>[m$^4$]</td>
<td>[m$^3$]</td>
<td>[mm]</td>
<td></td>
</tr>
<tr>
<td>6 - 17</td>
<td>8 / 203</td>
<td>7.480</td>
<td>13.321</td>
<td>29.200</td>
<td>4.531</td>
<td>2.248</td>
<td>2.678</td>
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<tr>
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<td>13 / 330</td>
<td>8.117</td>
<td>15.628</td>
<td>32.845</td>
<td>5.251</td>
<td>2.093</td>
<td>2.927</td>
</tr>
<tr>
<td>2</td>
<td>15 / 381</td>
<td>8.400</td>
<td>16.455</td>
<td>33.840</td>
<td>5.527</td>
<td>2.034</td>
<td>3.030</td>
</tr>
<tr>
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<td>17 / 432</td>
<td>8.688</td>
<td>17.192</td>
<td>34.530</td>
<td>5.783</td>
<td>1.980</td>
<td>3.131</td>
</tr>
<tr>
<td>Pier</td>
<td>18 / 457</td>
<td>8.833</td>
<td>17.527</td>
<td>34.740</td>
<td>5.903</td>
<td>1.955</td>
<td>3.181</td>
</tr>
</tbody>
</table>

$^*$) Torsion moment  
$^{**}$) Distance measured from bottom of the cross-section

- Cross-sectional area of composite member:
  \[
  A_i = \alpha_c A_c + \alpha_{tr} A_{tr} + \alpha_p (A_{p,lv} + A_{p,up}) - (A_{d,lv} + A_{d,up})  
  \]

- First moment of area of composite member:
  \[
  S_i = \alpha_c A_c z_{cg} + \alpha_{tr} A_{tr} z_{tr} + \alpha_p (A_{p,lv} z_{p,lv} + A_{p,up} z_{p,up}) - (A_{d,lv} z_{d,lv} + A_{d,up} z_{d,up})  
  \]

- Center of composite cross-section:
  \[
  z = S_i / A_i  
  \]

- Moment of inertia of composite member:
  \[
  I_i = \alpha_c \left[ A_c \left( z_{cg} - z_i \right)^2 \right] + \alpha_{tr} \left[ A_{tr} \left( z_{tr} - z_i \right)^2 \right] + \alpha_p I_{p,i} - I_d  
  \]
The cross-sectional area of concrete of the segment (Table 4.3); 
$I_c$ – moment of inertia of concrete cross-section (Table 4.3); 
$z_{cg}$ – center of gravity of concrete section, measured from bottom of segment; 
$A_{br}$ – sectional area of the barriers, $A_{br} = 2 \times 0.328 = 0.656 \, \text{m}^2$; 
$I_{br}$ – moment of inertia of the barriers, $I_{br} = 2 \times 0.00265 = 0.0053 \, \text{m}^4$; 
$z_{br}$ – distance of the barrier centroid from the bottom of segment, $z_{br} = 3.72 \, \text{m}$; 
$A_{p,lw}$ (up) – sectional area of prestressing bars located in the bottom (top) slab, 
$A_{p,lw(up)} = n_{p,lw(up)} \, A_{pl}$

Table 4.4. Geometric Properties of The Cross-Section of Composite Components

<table>
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<th>Segment</th>
<th>$h_s$</th>
<th>$n_{p,lw}$</th>
<th>$n_{p,up}$</th>
<th>$\alpha_c$</th>
<th>$z_{l}$</th>
<th>$A_i$</th>
<th>$I_i$</th>
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<td>[m$^4$]</td>
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<tr>
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<tr>
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<td>10.726</td>
<td>22.385</td>
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</table>
$A_{p1}$ – sectional area of the prestressing bar, $A_{p1} = 8.043 \times 10^{-4}$ m$^2$;
$z_{p,lw}$ (up) – distance of centroid of prestressing bars located in the bottom (top) slab;
$A_{d,lw}$ (up) – area of steel ducts where prestressing bars were embedded,
$A_{d,lw} = n_{p,lw} A_{p1}$.

$A_{d1}$ – area of duct with diameter 2 in (51 mm), $A_{d1} = 20.428 \times 10^{-4}$ m$^2$;

$\alpha_c$, $\alpha_{br}$, $\alpha_p$ - Concrete and reinforcement ratios: $\alpha_c = E_{cs} / E_c$ (variable);
$\alpha_{br} = E_{br} / E_c = 0.87$; $\alpha_p = E_p / E_c = 5.195$.

4.1.3.2 Tendon Layout

4.1.4 Results

4.1.4.1 Dead Load

Figure 4.5 and 4.6 show bending moment and shear force due to dead load.

Figure 4.5. Bending moment diagram due to dead Load w/ and w/o creep and shrinkage effect.
4.1.4.2 Post-tension

Figure 4.6 Shear Force Diagram due to Dead Load

Figure 4.7. Primary and Secondary Moments due to existing post-tension.

Figure 4.8. Total Moments due to existing post-tension.
Figure 4.9. BMD due to dead load and existing post-tension.

Figure 4.10. Total moments due to dead load and existing post-tensions.

Figure 4.11. Shear force diagram due to existing post-tension.
4.1.4.3 Creep and Shrinkage

Figure 4.12. Shear force diagram of dead load and existing post-tensions.

Figure 4.13. Moments due to creep at construction and 27 years later.

Figure 4.14. Moments due to shrinkage after construction and 27 years.
Figure 4.15. Moments due to creep and shrinkage at construction and 27 years later.

Figure 4.16. Moments due to dead load + post-tension, and plus live load + creep + shrinkage 27 years later.

Figure 4.17. Shear force due to creep and shrinkage after 27 years.
Figure 4.18. Total shear forces due to creep and shrinkage after 27 years.

Figure 4.19. Total shear forces due to stationary loads (dead load, post-tension, creep, and shrinkage) after 27 years.

4.1.4.4 Temperature Gradient

Figure 4.20. Moments due to positive and negative temperature gradients.
4.1.4.5 Live Load (HS20)

Figure 4.21. Shear forces due to positive and negative temperature gradients.

Figure 4.22. Envelope of moments due to live load, HS20-44.

Figure 4.23. Envelope of shear forces due to live load, HS20-44.
4.1.4.6 Current State

Figure 4.24. Envelope of moments due to live load and temperature gradients.

Figure 4.25. Envelope of shear forces due to live load and temperature gradients.

Figure 4.26. Envelope of current bending moments, after 27 years of construction.
The implementation of draped post-tensioning tendons external to the concrete and located inside of the box girder is a very effective way to reduce bending and shear stresses in the concrete box girder. Stresses caused by gravity are reduced by draped tendons. The ideally shaped parabolic tendon with high points at the piers produces a constant distributed load opposing the gravity-induced self weight of the box girder, thereby reducing net moment and shear forces. The axial force introduced by the tendons reduces tensile stresses in the concrete.

When using external tendons, the parabolic shape is approximated by a series of trapezoidal tendon runs, where the two top corners coincide with the piers and the bottom corners coincide with deviators mounted to the bottom slab.

**4.2 RETROFIT WITH EXTERNAL POST-TENSIONING**

The effectiveness of different tendon layouts has been studied by adding the tendons in the existing analytical model for the Kishwaukee River bridges.

The draped external post-tensioning chosen as the proposed layout is provided with two 12-strand, 0.6-inch tendons per web, encased in a smooth 3-inch diameter polyethylene (PE) duct. The two web tendons deviate in separate bottom slab deviators located in the fourth and seventh segments from the pier, while their top point is located along the centerline of each pier, see Figure 4.28.

In the end spans, only one of the two tendons in each web was deviated from the bottom slab deviator and anchored at the top of the box interior at the abutment diaphragm. The other tendon in each web continues horizontally and also anchors in the abutment diaphragm. This layout permits the anchoring of the tendons close to the web-bottom flange intersection in the box, which minimizes out-of-plane bending of the abutment diaphragm and minimizes the global end moment the post-tensioning tendons induced to the superstructure.
Figure 4.28. Layout of proposed external post-tensioning.

Provisions have been made for two additional tendons with the same size per web. This provides another 15% of the positive moment and 20% of the negative moment post-tensioning force, and exceeds the Guide Specs recommendation with 10% of the positive moment and negative moment by post-tensioning force. IDOT has two options regarding the installation of these tendons during the retrofitting: reserve them for future use or eliminate them if additional tendons are not deemed necessary. The layout of these provisional tendons is similar to that of the two 12-strand tendons to be installed at the time of the retrofit.

We optimized the layout by studying a series of anchor locations, and were able to maximize the effectiveness of the tendons while keeping them within the constraints of the existing concrete box girder and its post-tensioning bars.

Table 4.5. Tendon Properties

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<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
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</tr>
<tr>
<td>f_y</td>
<td>216</td>
<td>ksi</td>
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Table 4.6. Stressing Sequence

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</tr>
<tr>
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</tr>
<tr>
<td>3</td>
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<td>100%</td>
</tr>
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<td>4</td>
<td>7&amp;8</td>
<td>557.28</td>
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</tr>
<tr>
<td>5</td>
<td>5&amp;6</td>
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<tr>
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<td>1&amp;2</td>
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<td>50%</td>
</tr>
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<td>7</td>
<td>5&amp;6</td>
<td>456.9696</td>
<td>82%/2</td>
</tr>
<tr>
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<td>1&amp;2</td>
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<td>50%</td>
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</tr>
<tr>
<td>12</td>
<td>5&amp;6</td>
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<td>18%/2</td>
</tr>
</tbody>
</table>

4.2.1 Results

4.2.1.1 External Post-tension

Figure 4.29. Primary and secondary moments due to external post-tension.
Figure 4.30. Total moments due to external post-tension.

Figure 4.31. Primary and secondary shear forces due to external post-tension.

Figure 4.32. Total shear forces due to external post-tension.
4.2.1.2 Strengthened State

Figure 4.33. Comparison of bending moment between current state and external post-tension.

Figure 4.34. Total bending moments after strengthening.

Figure 4.35. Shear force comparison with stationary current state and external post-tension.
4.3 SUMMARY

The current state of the concrete box girder, 27 years after construction, has been evaluated in depth. Time dependent material properties from the CEB-FIP model code were used to consider the changes of compression strength of concrete. The strengthening concept with external post-tensioning was also calculated in bending moment and shear forces and compared with the current state. The strategy to retrofit is a very effective way to reduce bending and shear stresses in the concrete box girder.

The current maximum shear force due to stationary loads such as self-weight, creep and shrinkage, and post-tension is 7,450 kN at the end support (Figure 4.19), but the non-stationary maximum shear force due to temperature gradients and live load is 750 kN (Figure 4.25), only 10% of the stationary loads. Moreover, the two non-stationary shear forces due to live load and temperature gradient are 400 kN and 350 kN respectively. Although they are almost equal, the component leading to the crack propagation is estimated by truck load only. According to the long-term monitoring results, the count of cyclic loading by truck was 3,000 per day but the frequency by temperature gradient was only once a day. Moreover, the amount of crack propagation at the west web, above which trucks often passed, was double that at the east web. This proves that the estimation is appropriate.

The comparisons in Figure 4.33 and 4.35 show the effectiveness of the external post-tensioning. The external post-tension increases the current maximum change in negative moment at the support from 58.5 MN.m to 12.5 MN.m, and reduces the current change in shear force from 7,800 kN to 4,700 kN.
CHAPTER 5 CONCLUSION

To provide continuous health information about possible structural changes and to guide retrofit strategies for compromised components of the Kishwaukee Bridge, a distributed intelligent bridge monitoring system was developed. The distributed pre-processing algorithm was designed to improve the efficiency of data analysis, release the burden of main servers, reduce unnecessary data transmission, and save the space of the database. With the advantage of the data pre-processing module, the distributed sensor substation was proved to be reliable and efficient for data acquisition, processing, archiving, and transmission. The monitoring results indicate that the pre-processing algorithm is quite useful for real-time long-term health monitoring.

The long-term monitoring effort on Kishwaukee Bridge has yielded a variety of data which spans several years. These data include both global measurements from accelerometers and crack opening displacement (COD) data from local deformation gauges. The analysis of these data alerts us to possible structural changes and is useful to guide retrofit strategies for compromised components. The key findings included in this report are summarized below:

1) According to the monitoring record due to traffic effects, the crack opening displacements on the west web have progressed as much as 26 micrometers every year since January 2002. Although the flexural stiffness of the bridge is still unchanged, action should be taken to control the growth of shear cracks on the web within a reasonable limit. It is recommended to impose a restriction on the speed and weight of trucks going over the bridge.

2) Temperature change has a significant effect on the change of frequency and crack opening displacement. Based on the records of the health monitoring system, the maximum change of the first frequency due to temperature effect is from 1.57 Hz to 1.68 Hz. The average change of COD is about 10.4 micrometer with the temperature change of 1°C.

3) The time history and spectrum of acceleration are almost the same as the results measured in 2000. It seems that the modal frequencies of Kishwaukee Southbound bridge didn’t change a lot over these years.

4) For the two years of monitoring, the Average Daily Truck Traffic (ADTT) is roughly 2500. Compared with the record of ADTT from 2005 to 2006, the annual growth rate of truck traffic is constant.
REFERENCES
