REMOTE GLOBAL MONITORING OF THE MICHIGAN STREET BRIDGE STURGEON BAY, WISCONSIN

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Remote Global Monitoring of the Michigan Street Bridge
Sturgeon Bay, Wisconsin

by

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Abstract

A remote global monitoring system is being operated on a 65 year old lift bridge in Sturgeon Bay Wisconsin. The system uses both strain gages and clinometers to continuously monitor the "health" of fracture critical components of the bridge. Ruggedized miniaturized data loggers are locally mounted on the structure at the individual test sites. Spread spectrum RF links network the individual data logger computers to a central host computer. The host computer is connected to a dedicated telephone line via modem to allow remote access to the recorded data as well as real time monitoring capabilities. This system uses commercially available equipment. It provides a practical cost effective solution to a problem that is not unique to this structure, the need for intensive continuous monitoring to assure safe operation of aging structures during life extension.

Background

The Michigan Street Lift Bridge was built in 1930. It crosses Sturgeon Bay, in the city of Sturgeon Bay, Wisconsin, a major ship building area. It is one of two bridges that provide the only routes to the Door County peninsula. Sturgeon Bay also provides a major shipping route between Lake Michigan and Green Bay. The structure consists of thirteen spans for a total length of 1,413 feet. There are seven spans east of the lift span, five of which are combination steel-concrete girder and two are overhead steel truss. West of the lift span there are five overhead steel truss spans. Figure 1 shows an overall view of the bridge from the west.

The overhead truss, rolling bascule lift span is 161'-6" in length. Each leaf of the span is 80'-9" in length. Roadway width is 24 feet with one sidewalk 8 feet wide. The two bascule girders are spaced 26'-6" apart. The bridge opens approximately 3,600 times a year for a total of 7,200 cycles.

Maintenance and repair history on the rolling bascule span is lengthy and important in determining the causes of conditions discussed. A summary of the history is as follows:

- 1930 New Structure
- 1959 New Steel Grid Floor
- 1960 Extensive Damage Repairs (Ship Hit Bridge)
- 1965 Estimated Date for Field Welds Placed on Bascule Girder Rivets and Rolling Plate
- 1970 Bridge Painted
- 1978 Fix Superstructure Deterioration
- 1984 Bridge Painted

- 1988 New Electrical Components on Drive System
- 1992 Rebuild Motors

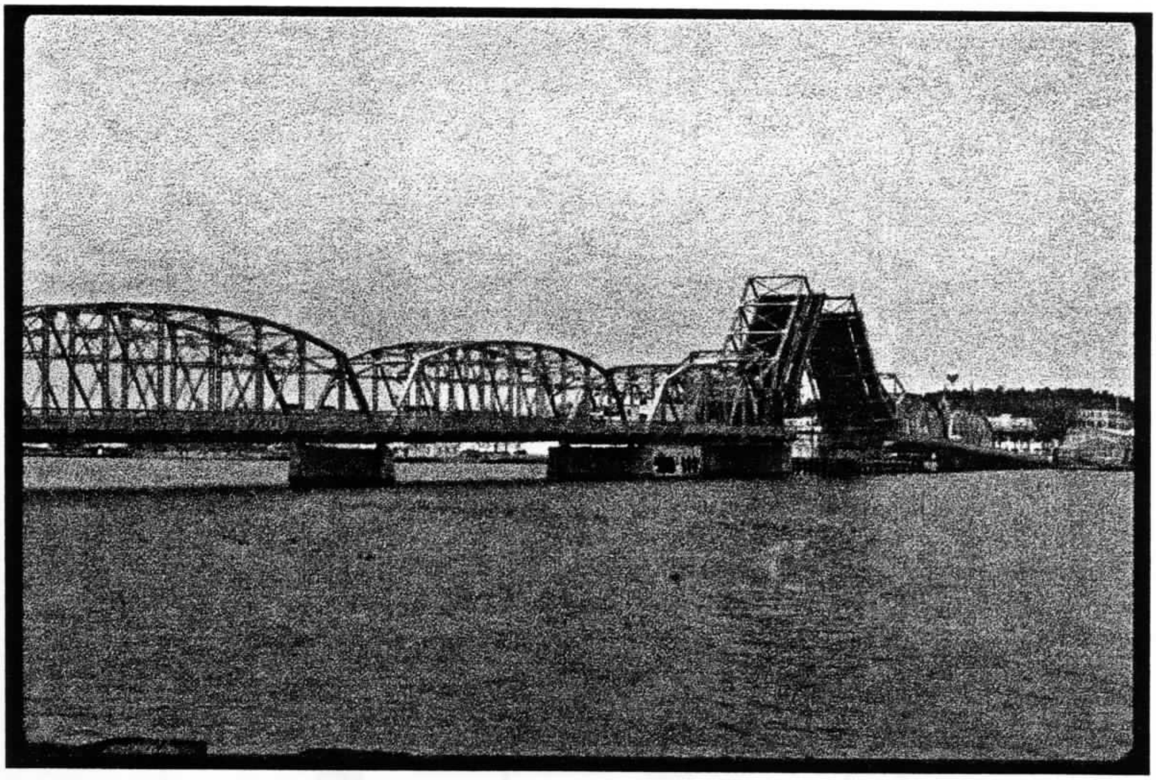

Figure 1 Overall View of Michigan Street Bridge.

Two components, a track girder and a segmental girder support the bascule girder. The track girder remains in a fixed position at all times and the segmental girder moves as the bridge opens and closes.

As shown in Figure 2, both the track girder and the segmental girder have two web plates spaced at 1'-2". A cast steel (A24 Material) track plate with teeth is attached to the top of the box girder with two single lines of rivets. This assembly acts as the track girder. The segmental girder has a rolled steel plate (A7 Material) with holes that match the teeth in the track plate. This assembly is also attached with two single lines of rivets. Structural material in the bascule girders is (A7 Material). All mechanical equipment to operate the bridge is located above the roadway along with the counterweights for each leaf.

FRACTURE CRITICAL INSPECTION

June 1994 Initial fracture critical inspection was performed by Wisconsin DOT at the request of persons doing routine inspections on the bridge. They noted movement in the rolling plate at the point of attachment to the segmental girder during opening and closing of the bridge

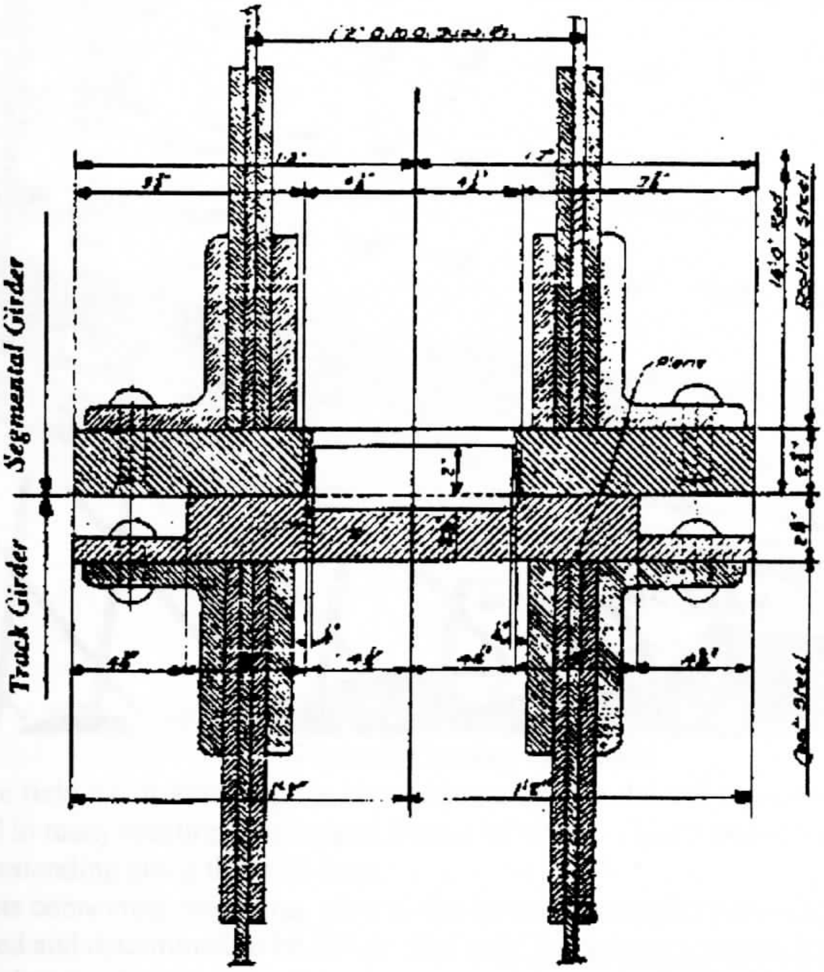

Figure 2 Track girder - segmental girder detail.

This inspection found serious cracking throughout the segmental girder. A majority of the cracks initiated from field repair welds that were placed in approximately 1965 although the date could never be clearly defined. Field welds were placed along the edge of the rolling plate and connection angles. Figure 3 below shows a view of this cracked field weld. Also, welds were placed around the rivet heads on the angles. Apparently this was done as an attempt to stop movement where the rolling plate was attached to the segmental girder.

The cracking was continuous along the rivet lines in the connection angles for several rivets. Cracks also migrated at random patterns from rivet heads. The connection angles were also cracked at the 90-degree corner of the angles from bending in several locations. Vertical bearing stiffeners were worn into the connection angle as much as 1/8".

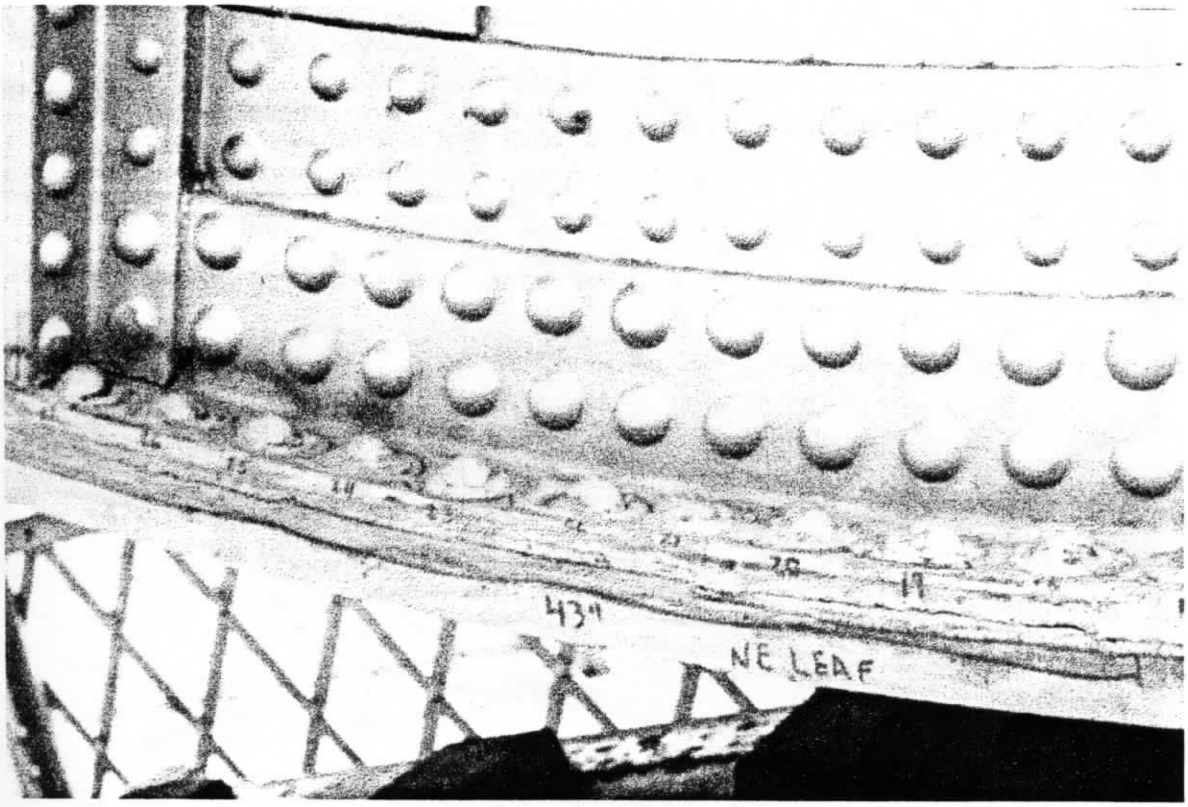

Figure 3 Cracked field weld.

The field welds attaching the connection angles to the rolling plates were also cracked in many locations for several inches in length. Figure 4 shows one of the large cracks extending along the rivet line.

All rivets connecting the rolling plate to the connection angles were ultrasonically inspected and determined to be failed. The only holding mechanism for keeping the rolling plate in place at several locations was corrosion in the rivet holes.

Emergency repairs were performed by placing bolts in failed rivet locations. The bridge was load posted and placed on monthly inspection cycles the number of openings was reduced as much as possible.

August 1994 Inspection found increased cracking from field welds on rivet heads and field welds attaching the connection angle to rolling plate. New cracks were found in the rolling plate. These were located at the corners of the holes where the track tooth engages. The cracks were diagonal and varied in length.

Cracks were found in the 90-degree corner of the connection angles attaching the cast steel track plate to the box track girder. This was due to flex as the bascule span operated. Movement was visual.

Design started on rehabilitation plans and work was scheduled for February 1995, the minimal usage time for both vehicle traffic and ship traffic.

September 1994 Cracking continued, but the growth rate had decreased. A new area of concern was located. The bascule span had settled with the many years of use and the pinion gear (drive gear) teeth were bearing on the rack teeth, thus assuming load that was not considered in the original design. The original design gap was 1/2".

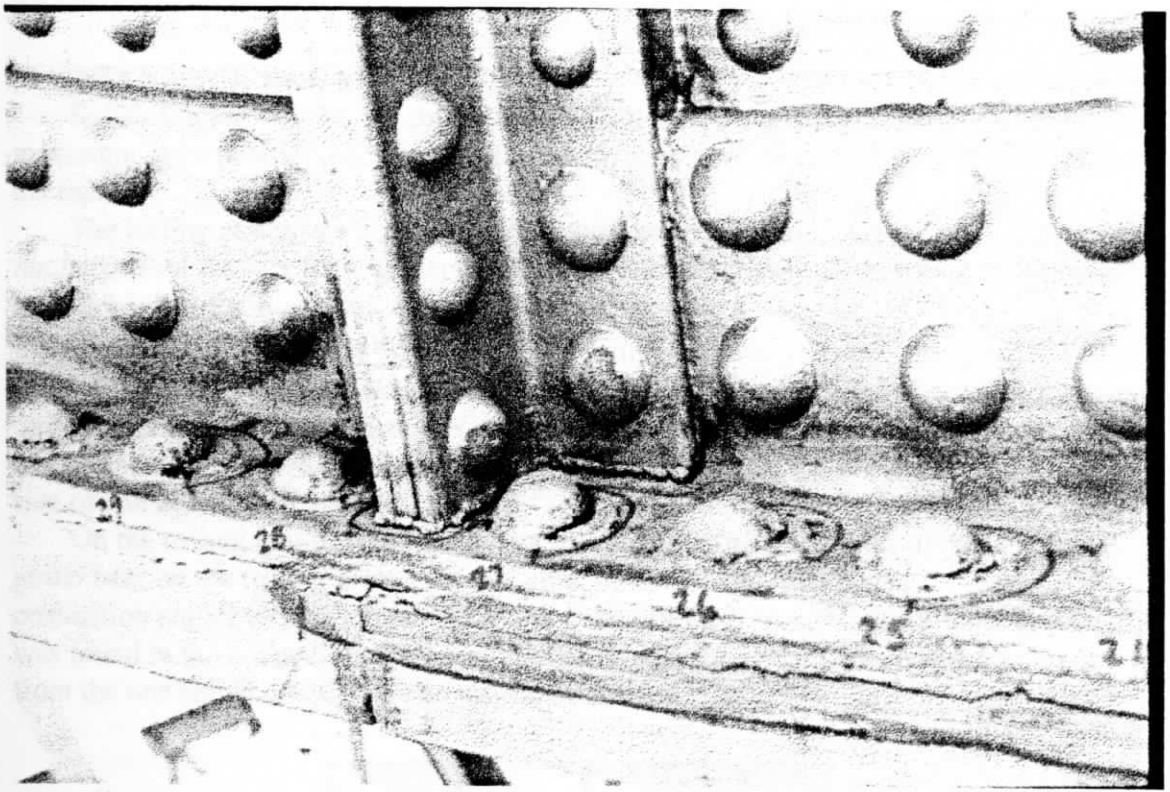

Figure 4 - Large crack running along rivet line in segmental plate attachment.

October, November, December 1994, January 1995 Conditions continued to change, but at a much slower rate due to considerable decrease in operating cycles in the winter months.

Design plans were completed with several meetings to work out details for field conditions. A contractor was selected and material was ordered. All bolts were custom fabricated to match details of existing rivets.

MAINTENANCE REHABILITATION DESIGN

The design for rehabilitation took into consideration that the bridge was to remain in service a limited number of years until a new bridge can be constructed. The bridge would be permanently closed during the rehabilitation contract. It was decided to leave us much of the existing material in place as possible to achieve alignment when putting the segmental and track girders back in place after repairs.

To accomplish this, the severely cracked connection angles connecting the rolling plates to the segmental girders and the connection plates connecting the cast steel track plate to the box girder were reinforced by adding an additional plate on the exterior side. This reinforcement provided a clamping force on the angles. Also, all cracks were to have holes drilled at the ends. The cracks in the 90-degree corners of the angles were to have holes drilled at the ends, with no other repair.

All vertical bearing stiffeners were to be attached similarly by clamping new material on the exterior surfaces.

A325 high-strength bolts were specified. These had to be custom fabricated to meet the conditions of existing rivet details.

MAINTENANCE REHABILITATION

Rehabilitation started in February 1995. The bascule girders were opened to their maximum open position and blocked to remain permanently open during the first part of the repairs.

The rolling plates were removed from the segmental girder and the track plates were removed from the box track girder. Both were marked clearly so they could be placed exactly back at their original location after repairs were accomplished.

Unexpected conditions found Deterioration was more severe than expected. The cast steel track plates had considerable cracking on the bottom side. Where the cast steel track plates connected to the angles, there was heavy wear up to 1/8", and severe corrosion with deep pitting. Figure 5 shows the severe cracking observed on the back side of one of the track plates.

On the rolling plates, wear had taken place where the web plates of the segmental girder bear on the rolling plates. In this area, up to 1/8" had worn away. At the connection angles there was also wear and deep pitting corrosion. Additional cracking was found in the connection angles along the rivet lines. The cracks had migrated out from the one single crack that was visible on the top side.

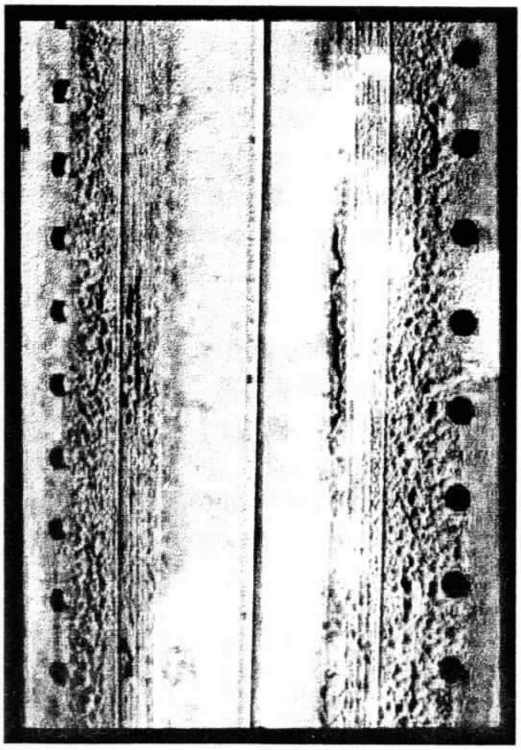

Figure 5 Showing severe cracking wear and corrosion on back side of track casting.

Additional rehabilitation repairs After several contacts, we determined that it would take several weeks to fabricate new cast steel track plates. This would excessively delay re-opening of the bridge. To accommodate contract time and the traveling public, we decided to repair existing plates.

Cast steel track plates were taken to a fabrication shop and weld repaired under close supervision. Weld material and welding processes were selected that would be most compatible with the existing material. The process included restraints to minimize distortion, preheat, and post-heat to minimize cracking and internal stress. The cast steel track plates were cleaned to remove deep pitting corrosion and primed with paint.

Deterioration from wear and original casting flaws made it impossible to completely weld repair two cast steel track castings. Because of the fracture critical nature of these castings, we contacted Northwestern University's industrial research laboratory, BIRL and requested that they develop a strain gauging system to allow continuous monitoring of these plates from a remote location. Strain gauges were installed on the two plates while they were in the fabrication shop. The gauges were located between the two web plates of the box girder that is inaccessible when the cast steel track plates are in position on the bridge.

Rolled steel plates on segmental girders were also taken to a fabrication shop for cleaning to remove deep pitting corrosion. Holes were drilled at ends of diagonal cracks in corners of tooth sockets. A prime coat of paint was applied.

To provide complete bearing surfaces when re-installing the cast steel track plate and rolled steel plates, titanium putty with a compressive strength of approximately 18 Ksi was selected. It would be applied immediately before final bolting to all bearing surfaces to assure that it would remain in a liquid form until bolts were tightened.

After adjustments for unexpected deterioration, the rehabilitation work progressed on schedule. The cast steel track plates and rolling plates on segmental girders were re-assembled with new clamping plates, titanium putty and bolts. New bearing stiffeners were placed by clamping with new materials and bolts. Teeth on the pinion gears and rack castings were machined to provide a clearance of 1/8".

Strain Gage Installation

A total of 16 weldable Measurements Corporation type CEA-06-W250A-350 strain gages were mounted on the two most badly deteriorated track plates. The plates were the first segments on the south east and south west quadrants of the bridge. The gages were mounted in quad groups with two gages in the longitudinal (rolling) direction and two gages mounted transverse to the plates long axis. This pattern was reproduced in two areas on each of the two castings. A close up of a typical gage mounting pattern is shown in Figure 6.

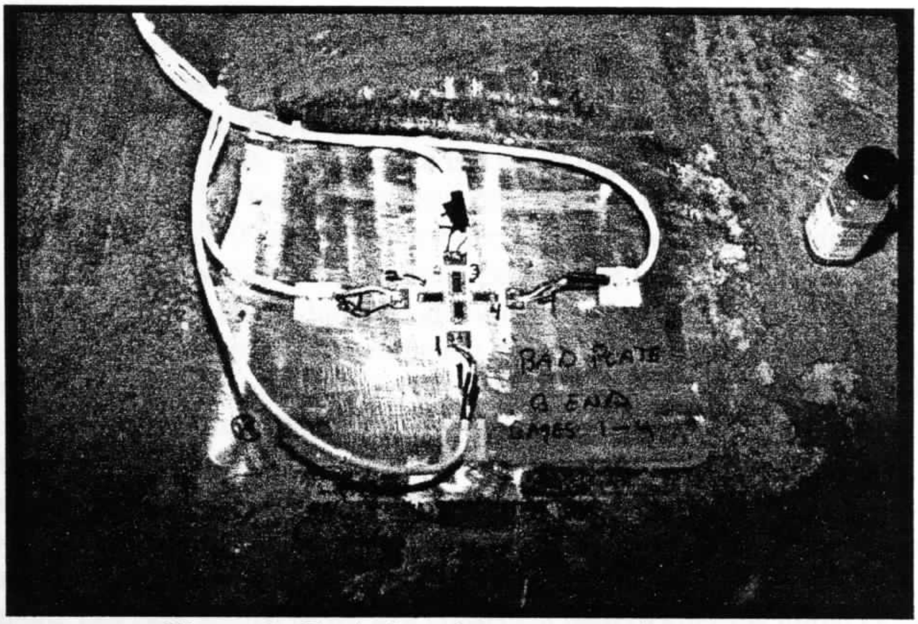

Figure 6 - Typical strain gage mounting pattern

The placement of the gages was governed by two requirements. The first was to monitor strains in the most severely damaged areas during lift cycles and secondly to monitor strains in the bearing areas resulting from live traffic loading. One transverse and one longitudinal gage was needed in each area to monitor both components of strain. We decided to add additional redundant pairs in each of the four monitored areas to allow for potential gage failure over the projected 6 to 7 years of life for the rehabilitated structure. As we shall see in later discussions, this was a fortuitous decision. The gages were protected with Measurements "M" coat "F" system which is their most durable system. This system uses multiple layers of protective tape and neoprene rubber with specially formulated adhesives. A photograph of the protected gages is shown in Figure 7.

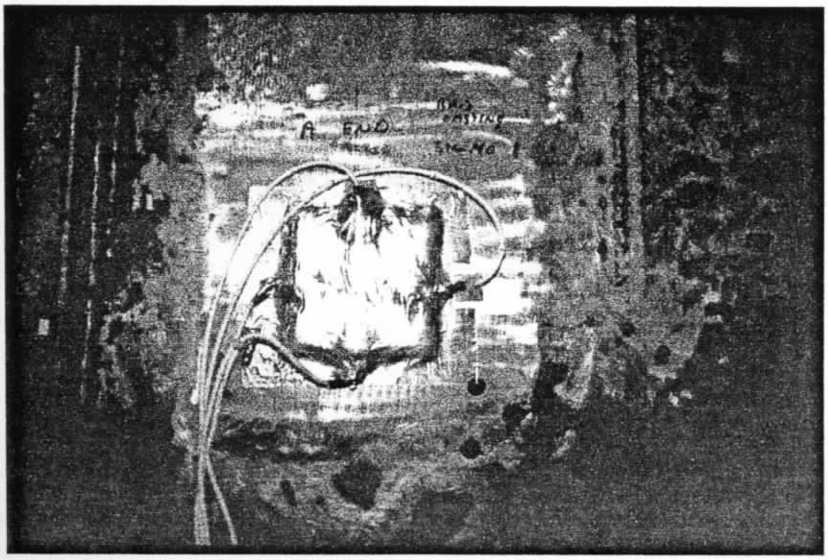

Figure 7 Gages with protective coating in place.

Initial Testing

The initial closing cycle of each bascule leaf was monitored closely for any areas of interference. During these tests, longitudinal and transverse strain gages were also monitored. A Somat S-2100 field computer was used to monitor and record the strain gage readings for these tests. This device is a small rugged data logger approximately the size of a 3" by 5" card file that can be connected to a lap-top computer. The lap-top is used to initialize the S-2100 and set up the desired test as well as providing real-time display capabilities. The S-2100, if desired, can then be left in place on the test site to record test data for later up-loading and analysis. For the initial tests, the S-2100 was used in both real-time display and permanent data recording modes. Very high strains were observed in the severely damaged area of the west #1 casting. A plot of the first two cycles for the longitudinal gage is shown in Figure 8. The first peak was produced when the bridge rolled over the gage site for the first lowering cycle. The peak is over 1000 µstrain and shows a permanent offset of 200 µstrain following the first cycle. This response is indicative of possible damage and permanent distortion of the casting. The other gages did not show either the high initial reading or the permanent offset. Furthermore, this gage settled down to a consistent pattern after the initial closing. A plot of a subsequent lift cycle is shown in Figure 9. Prior to recording this plot, the gages were re-zeroed. The offset is no longer observed and the peak strain has reduced to 760 µstrain. Additionally, during these tests some movement of the main piers was felt when the bridge was raised to its maximum height. As a result of this observation, we decided to install clinometers on each of the two main piers. The clinometers were installed in

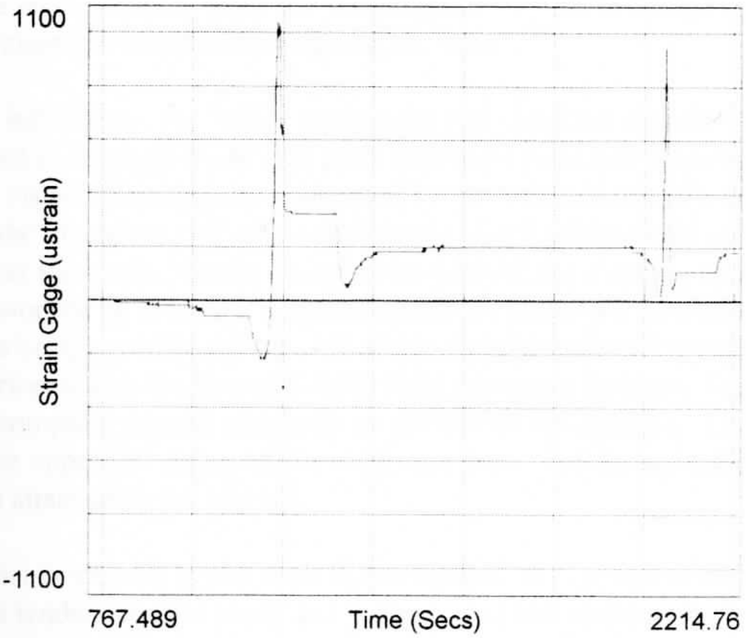

Figure 8 Strain readings from longitudinal gage at 77-inches, south west track casting #1

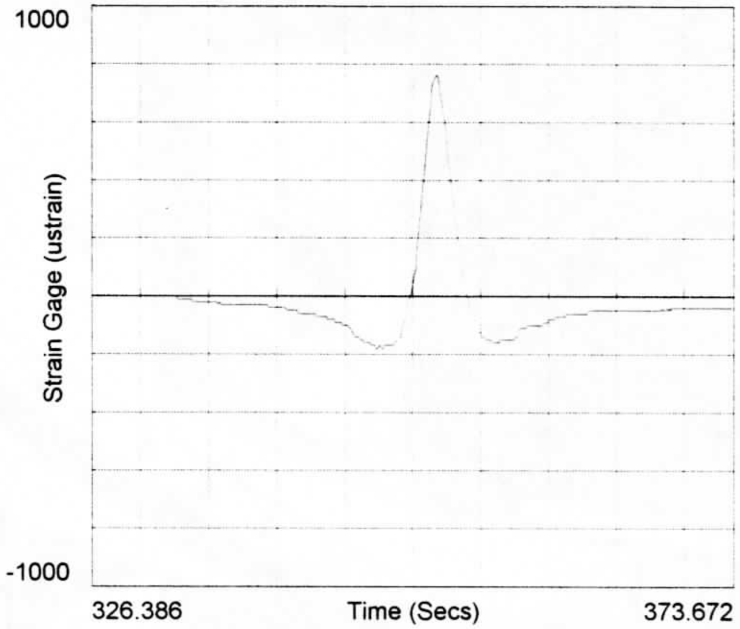

Figure 9 Subsequent plot for longitudinal gage at 77-inches, south west track casting #1

orthogonal pairs to allow measurement of both longitudinal and transverse components of tilt. The clinometers are Lucas AccuStar devices capable of 0.001 degree resolution. The measured tilt at the pier cap is 0.1 degree longitudinal with no measurable transverse component. The piers tilt away from the shipping channel as the bridge opens. These readings have remained consistent throughout the tests.

Following these initial tests, the Somat equipment was installed in permanent protective enclosures located in areas on the bridge piers that were protected from view by casual passers by. The installed sensors are connected by short cables routed through water-tight conduit to the fiberglass protective enclosures. Each enclosure is mounted on the main pier cap near the track castings on opposite sides of the shipping channel. The two test sites are networked by means of a spread spectrum radio link to a host computer that is installed in the bridge tenders office. All of the signal conditioning and data logging functions are performed by the Somat S-2100 field computer systems. Figure 10 shows one of the field computer remote sites with its protective box opened. The clinometers can be seen in the upper left corner of the enclosure box. The spread spectrum radio link's antenna is attached to the box lid.

The host computer is a 66 MHz 486 desk top computer. It is mounted in a utility room below the bridge tenders control room and is connected to a dedicated telephone line to allow remote access.

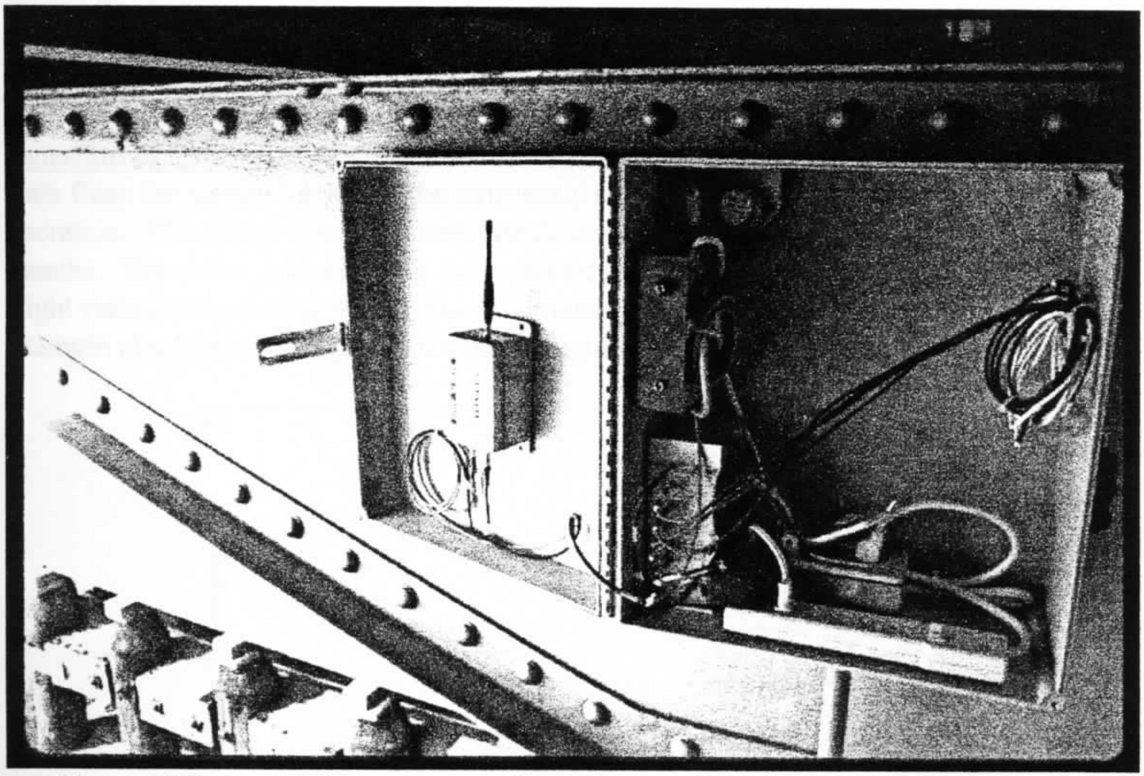

Figure 10 Remote field computer, clinometers and RF link mounted in protective enclosure.

A total of six sensors (4 strain gages and two clinometers) are being utilized at each of the two test sites. The data are continuously recorded and analyzed in several different analysis modes and the results are stored in battery backed memory in the field computers where it can be uploaded to the host on command. The data is also archived on the host computer and made available for later plotting and display. Additionally, real-time data monitoring is provided to allow assessment of individual lift cycles. The real-time mode displays data from any 4 sensors at a particular test site. It can be observed using a computer equipped with a fast modem (14.4 KB or higher) from any remote site equipped with a telephone connection.

Results

This monitoring system has been in continuous remote operation since early July of 1995. Prior to July, the system was operated in a manual mode that required weekly trips to the bridge to upload the data. To-date over 3000 lift cycles have been monitored. Ambient temperatures have ranged from 105 degrees F. to minus 20 degrees F. During early operation occasional computer lock-ups were experienced. These lock-ups required the bridge tender to power down the locked computer. Upon power restoration, the system recovered. These occurrences were very infrequent (once or twice a month). we decided to increase system reliability by adding stabilized power supplies to the remote field computer installations and an un-interruptable power supply to the host set-up. The primary source of computer malfunctions was the poor power line regulation caused by operation of the bridge lift motors. Our instrumentation uses the same power buss as the bridge. The stabilized power sources eliminated most of the computer problems.

Detection of Deterioration

Data from the sensors proved to be extremely consistent for the first few weeks of operation. The bridge averaged between 20 and 25 lift cycles per day during the summer months. The strain gage plots for successive cycles were nearly congruent except for slight variations in bridge speed which is manually controlled by the operator. An example of a typical data plot is shown in Figure 11.

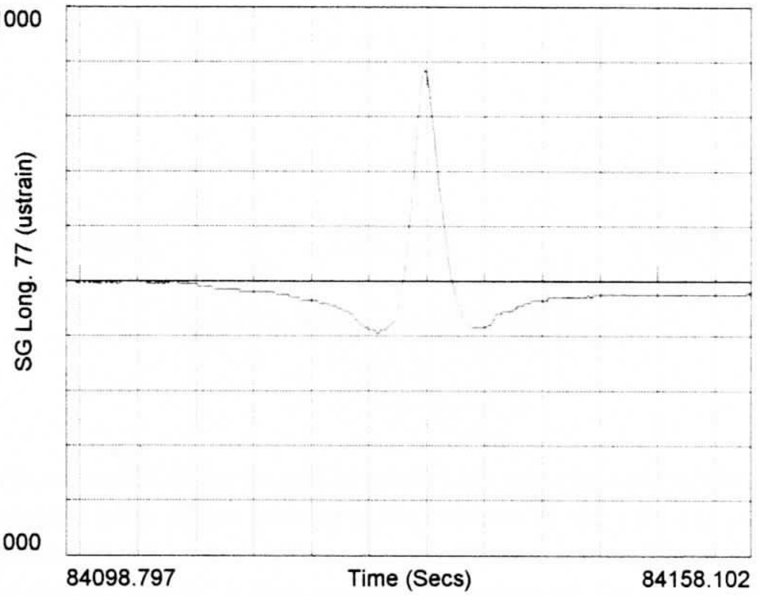

Figure 11 Typical data plot

During the second week in August we began to observe radical changes in the data from the strain gage mounted at the most severely damaged site on the south west track casting. Initially, the departure from normal occurred in an isolated lift cycle and then returned to normal. However, after one additional day of normal operation, the abnormal readings increased in frequency. The abnormalities consisted of large amounts of drift in the strain gage zero and large discontinuous excursions of the strain readings during a cycle. Figure 12 below shows these phenomena..

We alerted the appropriate Wisconsin DOT personnel of these observations and expressed our opinion that they were probably related to progressive damage in the region of the strain gage in question. The large thermal drifts can be caused by crack propagation under the gage reducing the heat sink supplied by intimate contact between gage and casting surface. Similarly, the large discontinuous strain excursions could be caused by weakening of the casting in the damaged area resulting from crack propagation.

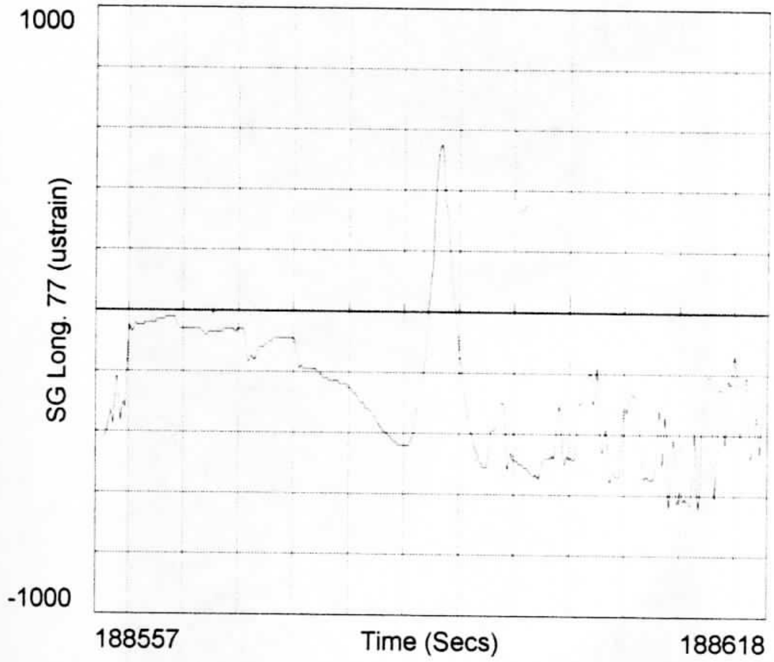

Figure 12 Abnormal strain reading

We met with Wisconsin DOT personnel at the bridge site and used a fiber optic boroscope to view the back of the casting in the vicinity of the strain gage that was producing the abnormal data. Access was gained through a check hole that had been drilled in a large surface crack. A photograph taken through the fiber scope is shown in Figure 13. Cracks are visible running into the strain gage cluster. These cracks were not present when the casting was repaired and are the probable cause of the abnormal strain gage readings.

Since these observations in mid August the damage continued to progress and successive gages were severed as the cracks propagated. The redundant gages that were originally installed in this area greatly extended our effective monitoring time for this damaged area. The final gage which was a longitudinal gage mounted farthest from the crack area severed in January of 1996 almost 5 months later. This incident demonstrated the effectiveness of this approach for remote global bridge monitoring.

Conclusions

The instrumentation used for these tests is all off-the-shelf commercially available equipment which up till now has been primarily used in the automotive industry. This application clearly shows the applicability of this technology to a vital area of concern mainly, remote monitoring of critical aging highway bridges during unavoidable life extension.

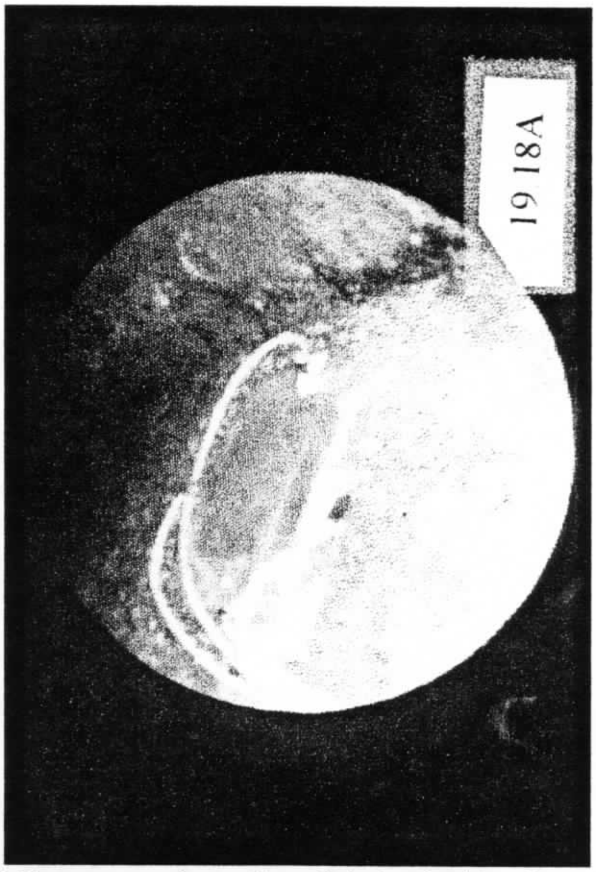

Figure 13 Boroscope view of crack in back side of track casting

Future work in this area will concentrate in two areas. These are, scale up to cover more test sites on larger structures, and automated data interpretation (the "smart" structure.) The current system allows the bridge engineer to call the bridge from a remote location and check for abnormal operation. The "smart" bridge will call the bridge operator and inform him of abnormal or dangerous operating conditions.