IDENTIFICATION OF GAPS IN STRUCTURAL HEALTH MONITORING TECHNOLOGIES FOR BRIDGES

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Abstract

Previously researchers in the area of non-destructive damage evaluation (NDE) envisioned a bridge management system where sensors fed measured responses such as strain and acceleration into a damage detection algorithm. Structural health monitoring (SHM) has recently emerged as another technology that would enable an engineer to evaluate the safety of a bridge. The objective of this paper is to identify gaps in structural health monitoring technologies that must be closed to enable an owner to implement this technology. First, the needs of the owner are identified. Next, several SHM applications using the Damage Index Method are summarized. Based on these applications, weaknesses in the technology are identified.

Introduction

Previously researchers in the area of non-destructive damage evaluation (NDE) envisioned a bridge management system where sensors fed measured responses such as strain and acceleration into a damage detection algorithm. In turn, this algorithm would determine if the bridge had deteriorated to the point where safety to the traveling public had been compromised, and the system would then notify the appropriate public officials [Farrar and Jauregui, 1996]. While significant advances have been made toward achieving this goal, NDE technologies are hardly ready for implementation. Capitalizing on this perceived failure, the area of structural health monitoring (SHM) has emerged as an alternative technology that would enable an engineer to evaluate the safety of a bridge.

Given the perceived poor interaction between the areas of non-destructive damage detection / evaluation and structural health monitoring, the objective of this paper is to identify gaps in structural health monitoring technologies that must be closed to enable an owner to implement this technology. First, the needs of the owner are identified. Next, several applications using the Damage Index Method, as part of a structural health monitoring system, are briefly summarized. Based on these applications, weaknesses in the technology are identified.

Need

Prior to discussing the current status of structural health monitoring, let us first reexamine the need for structural health monitoring of bridges. Specifically, this need is

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based on: (a) the continued deterioration of the infrastructure; (b) restoring serviceability of the bridge after an extreme event; and (c) the introduction of new materials in bridge construction. Of all the reasons for implementing a structural health monitoring system, the continued deterioration and increasing functional deficiency of the civil infrastructure is perhaps the most significant. In the United States alone, 27.5% of bridges were structurally deficient or functionally obsolete in 2000 [ASCE, 2003]. It should be noted that this same argument was made to support development of non-destructive damage evaluation methods several decades ago. Secondly, events over the past decade have forced State Bridge Engineers to consider the effects of extreme events other than earthquakes on the response of a bridge. The most visible of these events has been the series of tragic terrorist attacks against the U.S., both nationally and abroad. While not as sensational as the terrorist attacks, recent barge mishaps on the nation's waterways have identified other bridge vulnerabilities. Lastly, fibre-reinforced polymer (FRP) composite materials were introduced as an effective material for strengthening structures subjected to seismic events. Since then these materials have also been investigated as a means to rehabilitate or strengthen a bridge. While there are benefits to these materials (such as high strength-to-weight and stiffness-to-weight ratios), there are limitations as well. One such limitation is the lack of data to evaluate the long-term performance of bridges rehabilitated with these materials [Karbhari et al., 2000].

A renewed interest among bridge engineers to mitigate the effects of deterioration and extreme events on bridges, as well as reopen these structures immediately after such an event, represent a significant challenge that structural health monitoring could realistically meet. Owners and managers, such as State Bridge Engineers, need answers to specific performance issues, such as serviceability, reliability and durability, to mention a few. That is: (a) has the load capacity or resistance of the structure changed (serviceability); (b) what is the probability of failure of the structure due to a predefined load (reliability); and (c) how long will the structure continue to function as designed (durability)?

Structural health monitoring has been defined by some as *the use of in-situ, nondestructive sensing and analysis of structural characteristics, including the structural response, for detecting changes that may indicate damage or degradation* [Housner et al., 1997]. A health monitoring system that detects changes that may indicate damage or degradation in the civil structure does not go far enough to satisfy the needs of the owner. What is needed is an efficient method to collect data from a structure in-service and process the data to evaluate key performance measures, needed by the owner, such as serviceability, reliability and durability. For this work, the definition by Housner et al. [1997] is modified and structural health monitoring is defined as the use of in-situ, nondestructive sensing and analysis of structural characteristics, including the structural response, for the purpose of identifying if damage has occurred, determining the location of damage, estimating the severity of damage and evaluating the consequences of damage on the structure. Structural deterioration or damage is defined as a change in stiffness of the structural element.

Application of Structural Health Monitoring

Although methods of structural damage assessment have been proposed [Park et al., 1997; Stubbs et al., 2000] development of Level IV NDE methods are still pending. All the NDE methods developed to-date can be classified into one of four levels according to their performance [Rytter, 1993]. These performance levels include: (1) Level I – methods that only identify if damage has occurred; (2) Level II – methods that identify if damage has occurred and simultaneously determine the location of damage; (3) Level III – methods that identify if damage has occurred, determine the location of damage as well as estimate the severity of the damage; and (4) Level IV – methods that identify if damage has occurred, determine the location of damage, estimate the severity of damage, and evaluate the impact of damage on the structure. It should also be noted that although the relative performance of several prominent Level II Methods has been evaluated by Farrar and Jauregui [1996] using experimental data obtained from a field structure, no such work has been done for a Level III or IV Method.

The technical literature is replete with papers related to non-destructive damage detection and evaluation, and structural health monitoring schemes [e.g., Rytter, 1993; Doebling et al., 1996 and Chang, 2005]. A significant effort has focused on developing data collection procedures, damage detection schemes and health monitoring methods to monitor the integrity of civil structures. An underlying assumption beneath this paper is that the development of structural health monitoring technologies that are able to answer the bridge owner's questions requires a Level III or IV non-destructive damage evaluation (NDE) algorithm. Therefore, the intent here is not to discuss the development of structural health monitoring technologies, but rather present results from several applications and discuss gaps in the transfer of technology between structural health monitoring and non-destructive damage detection. In the following section, the application of SHM to four different bridge types is presented. These include a suspension bridge, a reinforced concrete box girder bridge, a reinforced concrete T-girder bridge and a bridge constructed using frp composite materials. We shall begin with the suspension bridge.

Application #1: Suspension Bridge

Completed in 1963, the Vincent Thomas Bridge (VTB) is located in San Pedro, California and was among the first suspension bridges to be supported on a pile foundation. The four-lane cable-suspension bridge, as shown in Figure 1, crosses the Palos Verdes fault in Long Beach Harbor. The structure is approximately 1850 m long and consists of a main span of 457 m, two suspended side spans of 154 m each, and two approaches of approximately 545 m length at the east and west ends. The two main towers are 112 meters high. VTB is an important commercial link between the Port of

Los Angeles in San Pedro, and the Terminal Island and Long Beach Freeway.

FIGURE 1: VIEW OF THE VINCENT THOMAS BRIDGE

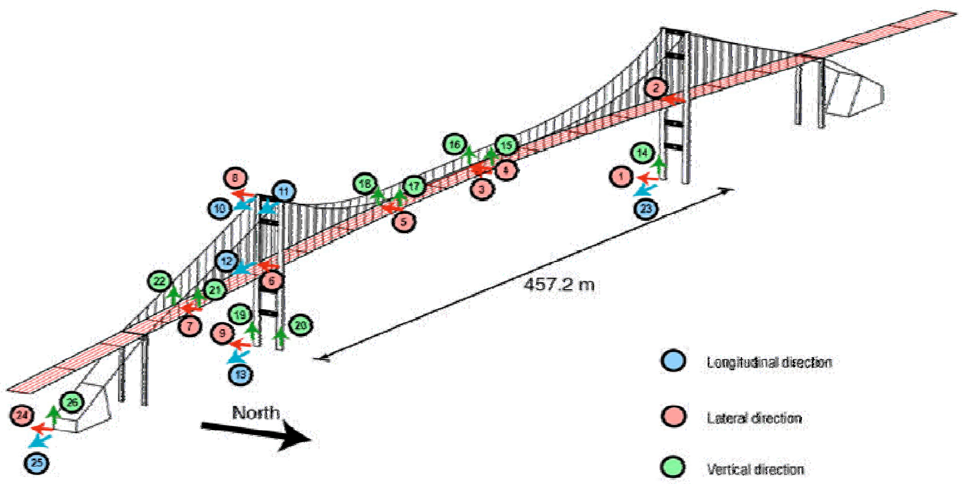

FIGURE 2: SENSOR LAYOUT ON THE VINCENT THOMAS BRIDGE

VTB is a good example of an attempt to implement a structural health monitoring system that is unable to produce the information needed by an owner. As noted by Masri et al. [2004], the system installed is only able to extract natural frequencies and damping ratios from the data collected. Figure 2 shows the layout of the 26 sensors mounted on the bridge in 1980. Note that the eastern half of the bridge is more densely instrumented than the western half of the structure. Thus, extracting reliable mode shapes from this data is virtually impossible. Given that frequencies are not a useful indicator of damage, such a system eliminates any damage detection algorithm that utilizes mode shapes. However, work is on-going to evaluate the structural safety of the bridge using the damage index method.

Application #2: Concrete Box-Girder Bridge

The Lavic Road Overcrossing is a two-span bridge in-service in Southern California and represents a typical bridge constructed by Caltrans. Constructed in 1968, this two-span reinforced concrete box girder bridge is supported by a single column bent, and simply supported abutments. The bridge is located in San Bernardino County, California and carries local traffic over Interstate Route 40. The structure is oriented in a North-South direction and spans the four-lane interstate highway. The south span is 37.5 m long while the north span is 36 m long. The superstructure is a 2.1 m deep reinforced concrete box girder, which includes a 10.4 m wide deck (including overhangs) and four 203 mm wide webs spaced at 2.7 m. The bridge is supported approximately at mid-span by Bent #2 which consists of a column 1.5m in diameter. The column is supported on a spread footing resting on sand. The abutments consist essentially of an end diaphragm supported by a spread footing (i.e., an end diaphragm abutment).

Using the NDE method described earlier, the bridge was first evaluated in 1997. Visual damage to the bridge following the Hector Mine Earthquake (October 1999) was minimal and the bridge was evaluated immediately following the earthquake [Choi et al., 2004]. As seen in Figures 3a & 3b, the bridge did sustain nominal damage due to the earthquake. This was later verified by a visual inspection.

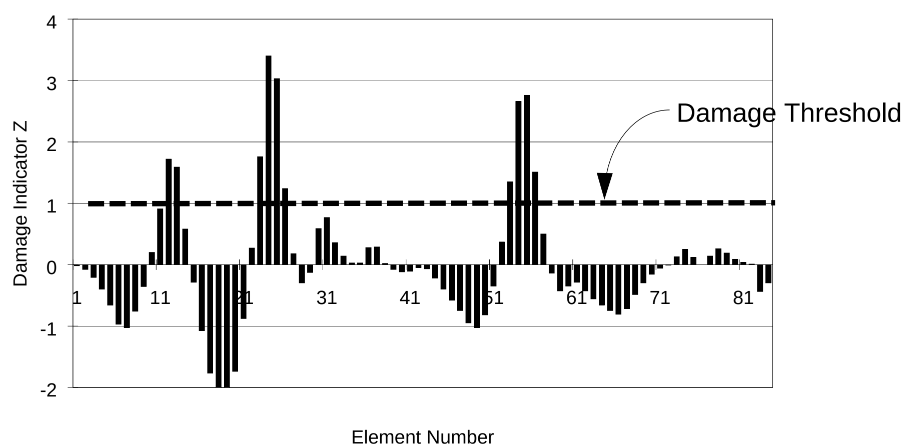

FIGURE 3a: DAMAGE LOCALIZATION RESULTS USING THE FIRST BENDING MODE ALONG THE EAST GIRDER (September 1999)

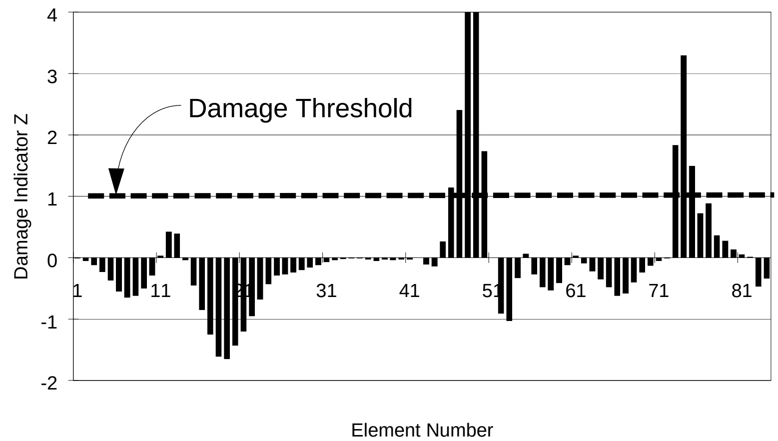

FIGURE 3b: DAMAGE LOCALIZATION RESULTS USING THE FIRST BENDING MODE ALONG THE EAST GIRDER (October 1999).

Application #3: Reinforced Concrete T-Girder Bridge

This bridge was built in 1964 and is a 340 foot (103.6 m) long, 5 span, two-lane highway bridge spanning an aqueduct canal [Sikorsky et al., 2001]. The superstructure consists of a cast in-place, continuous, reinforced concrete T-girder, monolithically connected to the bents. The 6-1/4 inch (158.75 mm) thick reinforced concrete deck spans transversely between the 7.25 feet (2209.8 mm) center-to-center spaced girders.

In the spring of 1998, this bridge experienced punching shear failures at two locations on the bridge superstructure. After analysis of the structure, a strengthening measure was proposed that included carbon fibre reinforcement bonded directly to the bottom soffit with an epoxy adhesive. Table 1 presents the results of the damage assessment showing a maximum increase in stiffness after strengthening of over 20%. The resultant flexural demand – capacity ratios were calculated for each span. It should be noted that the flexural capacity was determined based on a theoretical formulation and has not been confirmed with laboratory experimental evidence.

TABLE **1**: RESULTS OF DAMAGE ASSESSMENT

Span	Average Increase in Stiffness			Flexure
	Baseline	December 1999	December 2000	D/C
1	0	-0.04	-0.01	0.81
2	0	-0.24	-0.02	0.92
3	0	-0.12	-0.01	0.85
4	0	-0.16	-0.02	0.87
5	0	-0.03	-0.03	0.81

Application #4: FRP Composite Deck Bridge

The bridges carrying State Route 86 across the Kings Stormwater Channel near the Salton Sea are two span structures that carry two lanes of traffic and are 13.0m (42.5 feet) wide. A reinforced concrete bridge carries southbound traffic. The bridge carrying northbound traffic is constructed using fiber-reinforced polymer (FRP) composite materials and is 20.1m (66.0 feet) in length. The bridge system consists of a two-span beam-and-slab superstructure with a multicolumn intermediate pier. The superstructure is composed of 6 longitudinal carbon shell girders with 10 mm (3/8 inch) wall thickness and 343 mm (13.5 inch) inside diameter, which are connected across their tops with a modular FRP deck system. The damage localization results in Figures 4a & 4b indicate the development of the separation of the deck panels between 26 Sep 01 and 05 May 02. For these damage locations, the severity estimation approached 1.0 indicating a complete loss of capacity at those elements. This separation was subsequently verified by field investigation [Sikorsky et al., 2003].

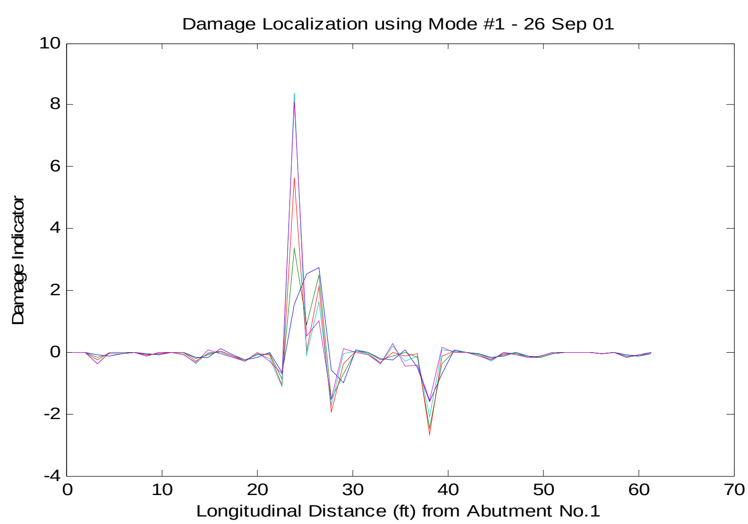

FIGURE 4a: DAMAGE LOCALIZATION USING MODE #1 – 26 SEP 01

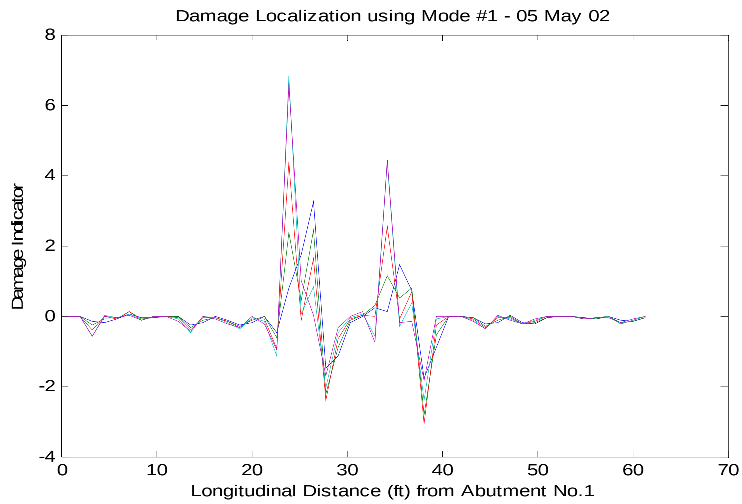

FIGURE 4b: DAMAGE LOCALIZATION USING MODE #1 – 05 MAY 02

Discussion

The above applications demonstrate that a Level III non-destructive damage detection method, such as the Damage Index Method, can be incorporated into a structural health monitoring system. These applications also provide results useful to a bridge owner. It should be noted however, that this technology still requires additional work to allow bridge owners to implement structural health monitoring on a wide-scale basis. Based on these applications, the following areas requiring additional work are identified. First, the VTB application demonstrates the need to install a sensor system that can support a damage detection algorithm. In other words, selection of the damage detection algorithm should occur before a sensor system is installed. Second, while a success at the time, the Lavic Road application points out the need to estimate the severity of damage. Damage localization by itself is not sufficient. Third, the Byron Road application demonstrates the ability of SHM to evaluate long-term performance (i.e., durability) of a strengthening scheme using a new material. However, additional work is needed to relate the damage indicator to a failure mechanism. Fourth, the Kings Stormwater application demonstrates the Damage Index Method can be incorporated in a permanent monitoring system that allows the Engineer to remotely monitor the bridge. As in the other applications, additional work is needed to relate damage localization and severity estimates to failure mechanisms and structural capacity. It should be noted that only the VTB and Kings Stormwater applications have permanent sensor installations.

Summary

As a minimum, an owner needs a SHM System that can evaluate the ability of the structural system to function as designed and estimate the remaining service life of that structure. Previous researchers in the area of non-destructive damage evaluation (NDE) envisioned a bridge management system where sensors fed measured responses such as strain and acceleration into a damage detection algorithm. In turn, this algorithm would determine if the bridge had deteriorated to the point where safety to the traveling public had been compromised, and the system would then notify the appropriate public officials. While significant advances have been made toward achieving this goal, NDE technologies are hardly ready for implementation. Capitalizing on this perceived failure, the area of structural health monitoring (SHM) has emerged as a relatively simple technology to implement that would enable an engineer to evaluate the safety of a bridge.

Given the perceived poor interaction between the areas of non-destructive damage detection / evaluation and structural health monitoring, the objective of this paper was to identify gaps in structural health monitoring technologies that must be closed to enable an owner to implement this technology on more widespread basis. First, the needs of the owner were identified. Next, several SHM applications using the Damage Index Method were briefly summarized. To conclude then, the following weaknesses in the technology were identified based on these applications.

- A damage detection algorithm is needed to successfully implement SHM.
- The damage detection algorithm should define the sensor system.
- An estimate of damage severity is needed; in addition to damage localization.
- Extensive work is needed to link damage detection results with failure mechanisms to estimate remaining service life and load capacity

Acknowledgments

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