Performance based bridge design by infusion

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ABSTRACT: The durability condition of concrete highway bridges in Michigan, USA is discussed. A methodology for durability condition assessment is proposed. The life-cycle performance based design of bridges is a process that is being developed within the last decade. In this process, the bridge durability performance expectation (optimal performance) is understood as a goal. A feedback control process is advised for bridge design where the feedbacks are developed utilizing the differences between optimal and measured performance. In this context, health monitoring and its infusion to the design process needs to be an essential ingredient in performance based design. This paper will discuss the SHM infusion process for unremarkable concrete highway bridges.

1 INTRODUCTION

Health monitoring by condition assessment of highway bridges is in principle to assure the safety of in-service bridges and for developing rehabilitation remedies and repair procedures that can be directly incorporated into the Capital Preventive Maintenance (CPM) and Capital Schedule Maintenance (CSM) activities. CPM and CSM are the strategies adopted by Highway agencies to extend the bridge service life that reduces the need rehabilitation and reconstruction activities.

The methodology developed for the durability assessment of highway bridges and their components is based on the principles of statistical structural health condition monitoring and assessment as presented by Lee & Aktan (1997). The four basic steps of the methodology are: selection of samples, condition assessment, diagnosis of distress, and development of solutions and remedies. Condition assessment step includes detailed field investigation of a representative group of bridges. Field investigations are often performed by visual inspection incorporated with destructive and nondestructive testing (NDT) techniques. The diagnosis phase includes the identification of distress mechanisms and their progression by analyzing the field inspection data. Further, numerical simulation models are developed, analyzed, and verified against the observed distress. The analytical models are used for in-depth analysis of distress causes and their impact on bridge safety. The cost effective causes of action to minimize, eliminate or manage the distress are developed during the solution phase. Finally, recommendations and revisions are suggested for the design and construction specifications. This article describes the implementations of the methodology to an unremarkable group of prestressed concrete (PC) I-beams and PC side-by-side box-beam superstructures.

The long term goal of the research is to introduce the performance based design procedures to the highway transportation infrastructure as discussed by Aktan & Aktan (1999). These implementations are studied for the Michigan highway system, which is comprised of more than 192,000 km length with over 12,000 bridges, of which more than 3300 are prestressed concrete. Since the last decade, in Michigan, new roads or bridges have rarely been constructed. Bridges are replaced primarily due to structural deficiencies or functional obsolescence. There is inspection, maintenance and condition monitoring programs established for these bridges based on asset management procedures. Though these programs are useful for maintaining and budgeting for maintenance, they are not designed for generating feedbacks for improving the design and construction of the next generation of bridges.

Unremarkable highway bridges are designed and constructed based on functional and operational needs that are defined in terms of strength and serviceability requirements. Bridge durability is often incorporated by stipulations defining allowable crack width, expansion joint and bearing provisions for the prevention of water intrusion to the bridge underside, stipulations for surface water drainage also for prevention of water intrusion. The bridge design is governed by the design manuals of localities. These manuals are primarily based on AASHTO Bridge Design Specifications AASHTO (1998). However, often there are significant differences between the AASHTO and state provisions as in the case in Michigan, MDOT (2003a). Bridge construction is governed by standard construction specifications described in MDOT (2004).

As of 2001, the national highway system (NHS) in Michigan is comprised of more than 15,000 km. under the Michigan Department of Transportation (MDOT) jurisdiction. Currently, there are more than 5800 bridges on the NHS that include over 1000 prestressed concrete (PC) bridges. Bridges are replaced due to structural deficiencies or functional obsolescence. During the period from 1995 – 2000, on the NHS 482 PC bridges were replaced. Out of 482 bridges, 281 were reconstructed using PC side-by-side box-beam and 139 using PC I-beam systems. Moreover, because of construction advantages one type of PC bridge, specifically the side-by-side box-beam became the choice for spans under 35 meters, suggested in MDOT (2003).

Knowing that the majority of unremarkable bridges are prestressed concrete (PC), one of the research tasks is to evaluate and assess the evolution of PC bridge design. The assessment process will evaluate the relationship between the evolving design procedures and bridge durability performance. The performance study completed on the inventory of side-by-side box-beam bridges in Michigan NHS is utilized for evaluating the impact of changing design provisions on measured performance. If weak relation appears between the design evolution and bridge durability performance then the input process to the design needs to be investigated. In that case, the second research task is to develop a proper information and feedback process that will control the design evolution process for better performing bridges. This article will present the proposed framework that is adopted from the adaptive control process and field verified in a study to evaluate the performance of PC I-beam bridges. This framework is called "Design by Infusion"

2 EVOLVING SIDE-BY-SIDE BOX-BEAM DESIGN PROCEDURES

In the US, the first major prestressed concrete (PC) highway bridge was Walnut Lane Bridge in Philadelphia in 1951. In 1954, prestressed concrete was introduced to Michigan. This bridge type became popular because the beams can economically be cast in plants at span length competitive to steel beams. There are 1037 PC bridges with design types of side-by-side box-beams, spread box-beams and I-beams on the NHS under the jurisdiction of the Michigan Department of Transportation. Of these PC bridges, 236 are side-by-side box-beam bridges. The histogram shown in Figure 1 demonstrates the construction distribution over the last 50 years.

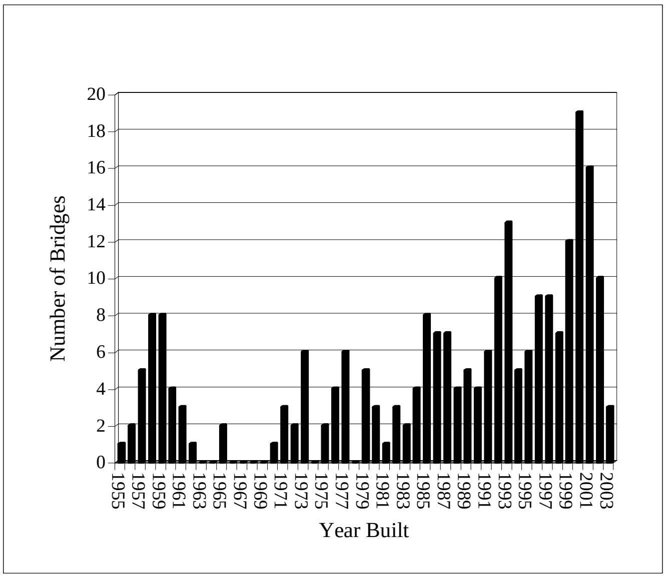

Figure 1. Histogram of side-by-side prestressed concrete box-beam bridges in Michigan on the NHS.

Current construction practice of side-by side box-beam bridges is as follows: the beams are firmly set at their place adjacent to each other, and the longitudinal joints between beams are grouted to the full-depth to form a tight, solid joint. After the grout is allowed to cure for 48 hours, beams are tied together by transverse posttensioning at the ends and at specified intermediate locations along the span. For beam heights greater than 33 inches, two tendons are used at each posttensioning location. A seal washer is placed between the box-beams before grouting in order to prevent any intrusion of shear-key grout into the posttensioning tendon ducts. The force applied to each tendon during posttensioning is 82.5 kips and 104.5 kips for the design loads of HS20 and HS25, respectively. Finally, a six-inch reinforced concrete deck with a single layer of mesh reinforcement is placed. Required time delay between the deck placements upon transverse posttensioning is not specified.

In Michigan, the earliest documented precast prestressed side-by-side box-beam bridge was designed in 1954 and constructed in 1956. The box-beam configurations given in the plans are: single cell of 27 × 36, 33 × 36, and 42 × 36 inches and double cells of 17 × 36 and 21 × 36 inches. At that time, the concrete strength requirement was 5000 psi. The box-beam height varies between 17 to 42 inches while the beam width is constant at 36 inches. The stirrups used for shear are of open shape and did not extend to the bottom flange of the box-beam. The spacing specified between adjacent beams is ¼-inch.

For beams that span up to 40 feet a transverse tie-rod is placed at the center of the span to tie the beams together. For the spans greater than 40 feet, tie-rods are placed at each 1/3 span location. Three-inch thick concrete or two-inch thick bituminous wearing ride surface is placed over the beams. Documentation indicates that deeper shear-key configuration at posttensioning locations was introduced in 1958. The box-beam design and dimensions were modified in 1974. Post 1974, all the beams are formed by single cells (17 × 36, 21 × 36, 27 × 36, 33 × 36, 39 × 36, and 42 × 36 inches) with the exception of a 12 × 36 inches solid box section. Open stirrups were replaced with closed stirrups with the 1974 design changes. The spacing between adjacent beams was increased from ¼-inch to ½-inch. Between 1973 and 1990, concrete strength between 5000 and 6000 psi was specified. Since than, concrete strength can be specified up to 7000 psi with the cast in place deck concrete of 4000 psi.

Transverse posttensioning force and tie-rod locations were changed in 1983 and in 1985 prestressing tendons were specified replacing tie-rods. Posttensioning forces and the tendon locations along the beam length were again changed in the same year, as shown in Table 1 and Table 2. Also in 1985, full-depth grouted shear-keys were specified and the spacing between adjacent beams was increased to 1½ inches. In 1988, a 6-inch thick reinforced concrete deck was introduced and the beams were cast with slab ties to achieve composite action. Transverse posttensioning positions along the beam height were specified in 1990 as given in Table 3.

Longitudinal deck cracks over the shear keys between box-beams have been reported since the construction of the very first side-by-side box-beam bridge. Water collecting inside the hollow beams and subsequent longitudinal girder cracking has also been reported as one of the commonly observed distress forms. The primary goal of preventing water intrusion into the keys between the beams and inside the beams has not been satisfactorily resolved. In an effort to eliminate the longitudinal cracks, and improving the shear design the open stirrups were replaced with closed stirrups, tie-rods with posttension tendons, wearing surface with six-inch thick reinforced concrete deck, and posttension locations and shear-key configuration were also changed. The spacing between adjacent box-beams was increased up to 1½ inches expecting better grouting and joint performance.

Current box-beam dimensions include a 48-inch wide single-cell box, in addition to the 36-inch wide box, shown in MDOT (2003). Bottom flange thickness of the 36-inch wide box-beam varies from 4.5 inches for a single layer to 6 inches for double layers of prestressing strands while top flange thickness remains at 5 inches. Top and bottom flange thickness of a 48-inch wide box-beam is 6 inches. The spacing between adjacent beams is allowed to vary between 1½- to 3 inches.

Table 1. Transverse posttension tie-rod locations along the beam length (1983)

Span Length (ft)	Tie-Rod Locations
Up to 50	1 at center of span; 1 at each end of beam
Over 50 to 62	2 at center of span (11 ft apart); 1 at each end of beam
Over 62 to 100	1 at center of span; 1 at each quarter point; 1 at each end of beam
Over 100	2 at center of span (11 ft apart); 1 at each quarter point; 1 at each end of beam

Table 2. Transverse posttension tendon locations along the beam length (1985-Present)

Span Length (ft)	Tendon Locations
Up to 50	2 at center of span (11 ft apart); 1 at each end of beam.
Over 50 to 62	1 at center of span; 1 at each quarter point; 1 at each end of beam
Over 62 to 100	2 at center of span (11 ft apart); 1 at each quarter point; 1 at each end of beam
Over 100	1 at each end of beam with 5 equally spaced between

Current construction practice is as follows: the beams are firmly set at their place, and the longitudinal joints between beams are grouted to the full-depth to form a tight, solid joint. After the grout is allowed to cure for 48 hours, beams are tied together by transverse posttensioning at the ends and at intermediate locations along the span as specified in Table 1. A seal washer is placed between the box-beams before grouting in order to prevent any intrusion of shear-key grout into the posttensioning tendon ducts. Two tendons are used at each posttensioning location for beam heights greater than 33 inches (Table 3). The force applied to each tendon during posttensioning is 82.5 kips and 104.5 kips for the design loads of HS20 and HS25, respectively (For spans under 36’ transverse posttensioning is limited to 83 kips for HS20 and 82.5 kips for HS25 Loading). Finally, a six-inch reinforced concrete deck with a single layer of reinforcement is placed. Delay time between the deck placement and transverse posttensioning is not specified.

Table 3. Transverse posttension tendon locations along the beam height (1990-Present)

Beam Size	Description
12" x 36"	At each location place 1 tendon 5.5 inches below top of the beam
17" x 36" 21" x 36" 27" x 36"	At each location place 1 tendon at mid depth of the beam
33" x 36" 39" x 36" 42" x 36"	At each location place 2 tendons, 1 at each third point of the beam depth

3 PERFORMANCE DATA COLLECTION

3.1 *Selection of Inspection Samples*

In order to document the evolving design procedures on side-by-side box-beam bridge superstructure performance, 15 in-service bridges were identified and inspected (Attanayake et al. 2005). These 15 bridges were selected from a group of 236 side-by-side box-beam bridges that are on the NHS. Bridges with large skew often require special design considerations. In order to include only the bridges with standard design, the inspection samples were selected with a maximum skew angle of 30 degrees. Eight bridges were selected from the pre-1960 design pool. The remaining 7 bridges represented the post –1988 design, utilizing 6-inch composite concrete deck and transverse posttensioning tendons.

3.2 *Documented Deck and Beam Distress*

The original wearing surface of inspected pre-1960 bridges was either concrete or bituminous. In most cases, the wearing surfaces had been replaced with an overlay of PC concrete or latex modified concrete or asphalt. When asphalt is used as the wearing surface, a water proofing membrane is often placed over the beams. In all cases, longitudinal deck cracking and distressed joints over the abutments and piers were common to all bridges. On most of the pre-1960 bridge decks, crack sealants were applied over the longitudinal cracks. Crack sealants appeared to be effective over wide cracks (Figure 2). Nevertheless, some of the large width longitudinal cracks were sealed; however, there were significant length of deck cracks that were not watertight. The concrete wearing surface on some of the bridge decks was patched with polymer concrete, but the cracking along the patch border allowed water penetration. Concrete wearing surface was placed with a wire mesh reinforcement thus, susceptible to delamination and spalling observed as potholes. Expansion joints over the abutments and the piers exhibited extensive distress. The continuous concrete bridge deck (in order to eliminate the joints over the piers) showed substantial cracking above the piers, especially a continuous transverse crack directly over the pier. The drainage systems of the older bridges were ineffective.

Figure 2. Bridge deck distresses and repair

Longitudinal cracking was also common in post-1988 bridge decks (Figure 3). Most deck joints exhibited some form of distress or breakdown. In an effort to eliminate the joints over the piers and abutments, the recent practice is to place the deck slab continuous and to design the deck as a continuous member for live loads. After placing the deck slab, construction joints are sawed and sealed at specified locations as given in the bridge plans. However, transverse cracking was still observed directly over the piers as seen in Figure 4. In some cases, construction joints were inadequately sawed and cracks formed propagating within the vicinity of the saw cut joints.

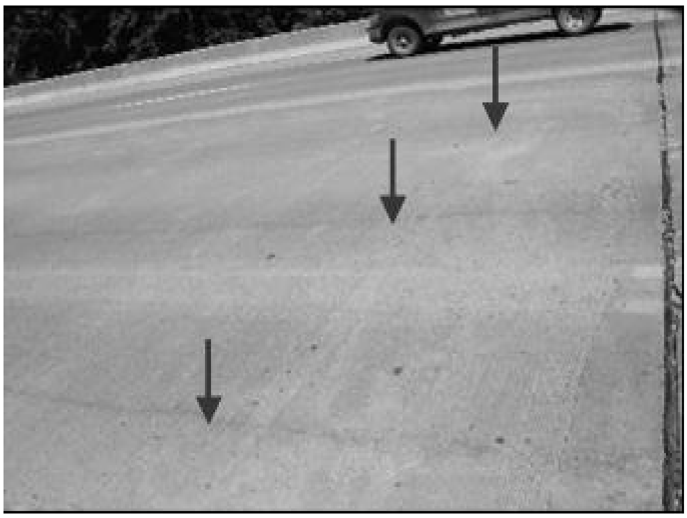

Figure 3. Bridge deck distresses (longitudinal cracks)

All the pre-1960’s bridges showed signs of prolonged exposure to moisture along beam edges and beam soffits. Small shear-keys used in old bridges could not be visually inspected. Overnight rains sometimes helped to identify the leaky joints (Figure 5). Moisture on stub abutment was a clear indication of the volume of water that flowed through the shear-keys. Heavy calcium carbonate deposits (efflorescence) signify the amount and length of time of leakage through the joint (Figure 6). The leakage from the joints over the piers and abutments must have been the cause of bearing corrosion. A maintenance program was initiated to drill drain holes shown in Figure 7 along the bean soffits to drain the water, if any, collected inside the box-beams. The rust stain around the drain holes is an indication of the active corrosion damage.

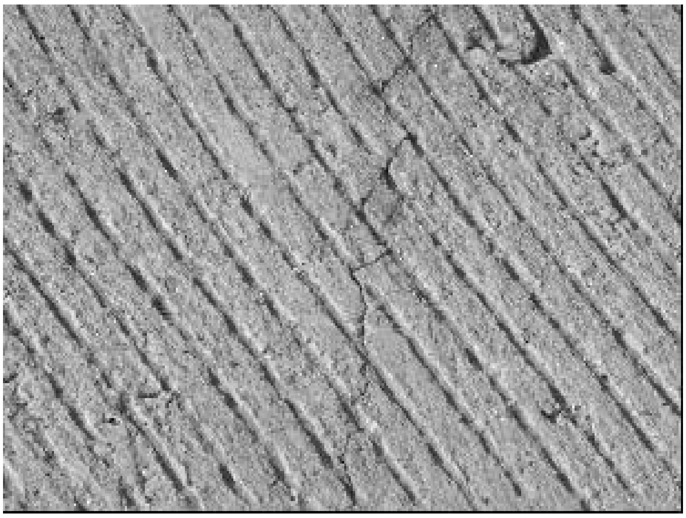

Figure 4. Close-up of transverse cracking of continuous deck over pier

Figure 5. Moist beams, leaky joints, and moisture on stub abutment

The repeated exposure to moisture and subsequently chlorides resulted in initiating and intensifying tendon corrosion leading to loss of tendon cross-section, concrete cracking, delamination, spall, and finally the breaking of tendons (Figure 8). Delamination, spall and breakage of tendons were concentrated along the beam edges (areas of heavy moisture exposure). Longitudinal cracking at the box-beam soffits appears either due to corroding tendons or due to pressure developed by ice formed by freezing of water collected in the beam hollow core. The amount of moisture on the beams with cracked soffits was considerably heavier than that on the other beams (Figure 9). Corroded tie-rods, rust stain, and efflorescence were visible at the stress pockets. As shown in Figure 10 exposed and broken tendons were also observed on fascia beams due to high-load-hits or collision damage.

Figure 6. Calcium carbonate deposits along the shear key

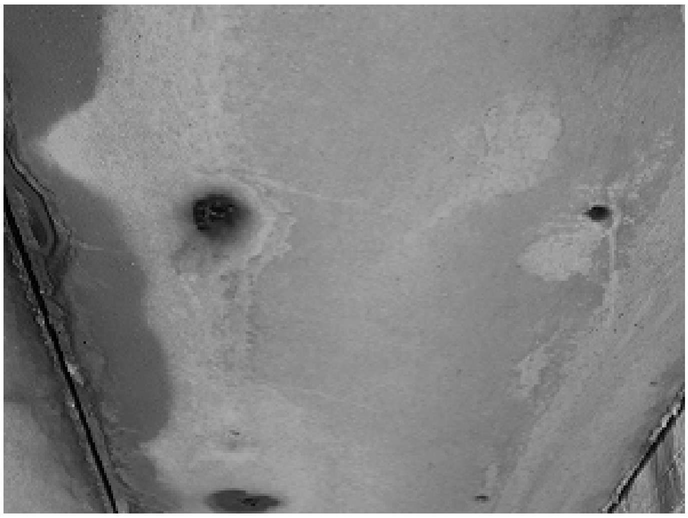

Figure 7. Drain hole with rust staining

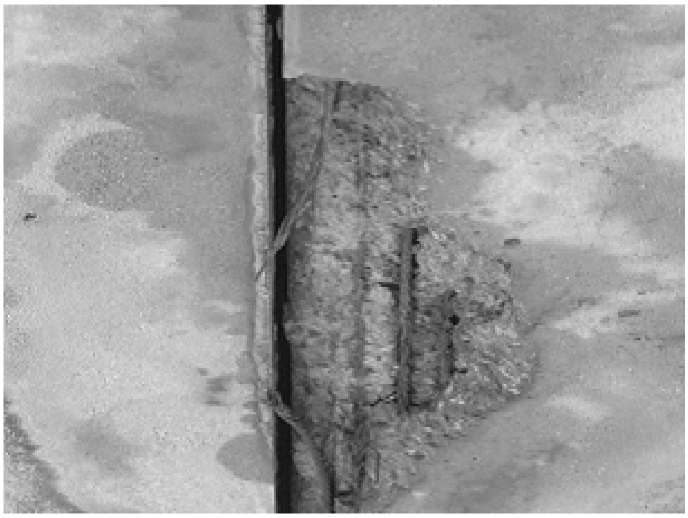

Figure 8. Corroded and broken tendons

Figure 9. Longitudinal cracks on box-beam soffits

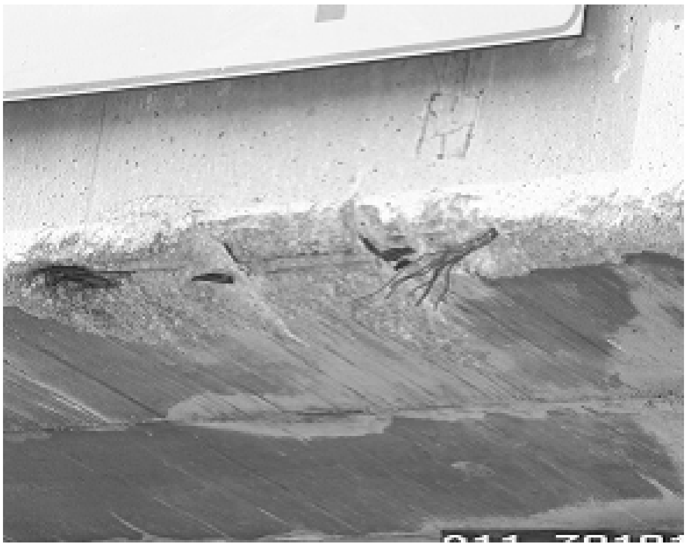

Figure 10. Close-up of high load hits

Figure 11. Efflorescence on the cracked post 1985 design shear key

Figure 12. Moisture on beam soffits and post 1985 design shear keys

4 PERFORMANCE ASSESSMENT

The field inspection goal was to document the type and progression of distress in order to evaluate the effects of evolving design procedures on side-by-side box-beam bridge superstructure performance. The beam and shear-key condition data illustrated that all the beams showed repeated and prolonged exposed to moisture except only in one bridge. The inspection data indicated that leaky expansion and construction joints and surface water penetrating and leaking from the longitudinal cracking were common to all the bridges, regardless of the age. The precursor to beam distress was often the longitudinal deck cracking. Surface water penetrates and leaks through the longitudinal cracks sometimes getting trapped within the shear keys. The beam sides concealed by the shear-key were often saturated. Corrosion initiated when the moisture with chlorides penetrated to the tendons and stirrups. Prior to the 1970’s, the box-beams were fabricated using stiffened cardboard to form the internal cavities. With leakage, water collected inside the boxes and initiated tendon corrosion which emanated itself as longitudinal cracks along the beam soffit since approximately the mid-seventies the beams are cast with solid Styrofoam blocks forming the cavity. The water no longer collects in the cells and longitudinal cracking problems appear to have been resolved. The majority of concrete spalls, tendon corrosion and tendon breaks are concentrated along the beam edges. Tendon breakage was observed very infrequently.

The primary performance of issue of water intrusion to the bridge underside appears to persist. The changes to the shear key geometry, introducing a cast in place deck and improving transverse connectivity by introducing posttensioning, later increasing the posttensioning forces did not improve these performance issues. The impact of the use of Styrofoam to form the beam cavities is uncertain. Moisture absorption of Styrofoam is negligible. However, moisture ingress persists and moisture collecting and condensing along the boundary with the Styrofoam is a possibility. At this time, invasive tests were not performed to positively establish the box interior moisture state. Assessment in that case is based on conjecture.

5 PROCESS FOR DESIGN BY INFUSION

Basis for design by infusion can be most simply described as a linear regulator control process. In this process, the goal is to establish a control input so as to drive the plant from an initial state to a constant final state. In this process, the reference state (bridge service life) is assumed constant. The state is described as the specifications that control design and construction (uniform specifications). External actions such as the exposure, live loads are acquired as part of the state. The optimal control is determined based on acquired performance data and user selected gains in order to keep the plant response (deterioration rate) constant also as presented by Aktan & Aktan (1999). A simplified block diagram of the control process is shown in Figure 13. In this process, all functions are known or well defined except the acquisition of performance data. The primary work performed in this study deals with defining the performance assessment process by utilizing the health monitoring data. In presenting this process, a previous study on causes and cures for PC I-beam end deterioration (Aktan et al. 2002).

Using the Michigan Department of Transportation bridge inventory database, the full stock of PC I-girder bridges were identified and classified according to their attributes. Next, 20 bridges were selected as a statistical representative group, and a detailed field inspection was conducted on the PC I-girders of each bridge. Also, a national survey was sent to all the State Highway Agencies to obtain their observations and practices on PC I-beam bridges. Once the field and survey data was gathered, it was analyzed to identify and document the common distress states of PC I-beam ends. In conjunction, a statistical analysis was performed on the PC I-beam bridge inventory and inspection data to identify causes and progression of distress. To verify the cause of distress, analytical investigations were conducted using numerical models. The last step dealt with the development of comprehensive recommendation to be incorporated in the uniform specifications.

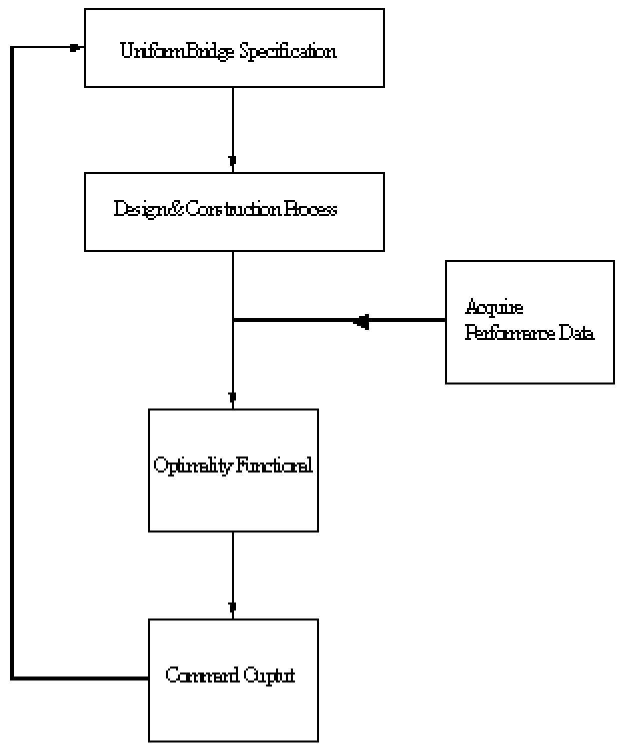

Figure 13. Design infusion by optimal feedback control

The field investigation, which was a component of data collection, revealed that beam-end distress progresses rapidly if cracking is present. Beam-end cracking was observed even in new girders in the plant and on girders of recently built bridges. The survey of state highway officials also corroborated the prevalence of girder end cracking. A 3-D finite element (FE) model of AASHTO PC I-girder was utilized to identify the causes of the cracks appearing in the mid-web and on the transition zone between web and bottom flange.

Numerical modeling analysis of a full bridge brought out several additional mechanisms that contribute to end cracking. These mechanisms were related to rigid concrete diaphragms and non-functional elastomeric bearings. The recommendations included sealing of beam-ends at the precast plant prior to shipment. New diaphragm details were proposed especially utilization of steel diaphragms that allow ventilation of bridge underside.

6 CONCLUSIONS

The primary goal of the paper is to express the need for an established process for improving the performance of unremarkable bridges by performance based updates to the uniform specifications. The essence of performance based design is the ability to understand that the performance expectation (optimal performance) is always greater than measured performance. Perfection is pursued by designing a feedback process where the difference between measured performance and expected or optimal performance is minimized and a set of new provisions or updates to existing provisions are developed for incorporating in the uniform specifications.

Two case studies are presented. The first case study dealt with the performance or side-by-side box-beam bridges where the changes in the uniform specifications since 1956 are presented. The field performance is evaluated by a statistical representative sample of these bridges. The performance study revealed that the changes to uniform specifications were not able to improve the performance of side-by-side box-beam bridges toward the expected (optimal) performance.

The second case study presented the health monitoring of PC I-girder bridges in Michigan. The health monitoring protocol was developed suitable for optimal feedback control process. The process incorporated field data collection as well as the use of comprehensive analytical models. A through understanding of the performance was established prior to comparisons to optimal performance. Recommendations were developed for the uniform specifications with the goal of guiding the bridge performance toward optimal.

7 ACKNOWLEDGEMENTS

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