Experience made with the International Benchmark on Dynamic Bridge Monitoring and Assessment at the Wayne Bridge in New Jersey

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^^ABSTRACT: A representative composite bridge has been selected by the Federal Highway Administration (FHWA) for the international collaboration between the Long Term Bridge Performance Project (LTBP) initiated in the U.S. and the FP7 Project IRIS in Europe. Various teams from the U.S., Japan, Korea, UK and EU have performed separate assessment routines including dynamic monitoring and system identification. The various approaches were compared and harmonised in order to achieve a standard model for this type of bridge, representing more than 40% of the bridge stock in the FHWA network. This comprises a total of about 240.000 bridges which could subsequently be tested and assessed following a standard procedure.
The present paper intends to present a strongly life-cycle oriented approach, developed by the authors. This is based on a dynamic monitoring campaign and an accompanying visual inspection at the Wayne Bridge, New Jersey.

KEY WORDS: Structural Health Monitoring; Dynamic Measurement; Life-Cycle Assessment.

1 INDRODUCTION

In the course of the IRIS Project the Wayne Bridge on Highway 202 in Wayne County - New Jersey has been selected for the 1st international test case within the US DOT – Long Term Bridge Performance Program. The main objectives were:

- To demonstrate the current practices in the U.S., Japan and Europe on bridge inspection and assessment
- To study the socio-cultural differences between the approaches and compare the results
- To work towards the harmonized assessment procedure

For reasons of a consistent documentation of the field of investigations of each working group and the corresponding results, the present paper covers only those parts of expertise provided by the authors.

2 BRIDGE DESCRIPTION

The Wayne Bridge (Figure 1) is located on the highway US 202 & NJ 23, leading across US 202 (Mountain View Boulevard), the Ramps M & N and the Norfolk Southern Railroad in Wayne, New Jersey. The structure consists of two separate load bearing structures – one for each driving direction. The bridge comprises four spans of simply supported composite welded steel plate girders and has a total length of 130.64 m and a total width of 37.64 m. The Southbound Structure was opened to traffic in 1983, the Northbound Structure in 1984.

The bridge shows numerous fatigue cracks, bearing problems, joint performance problems, flexibility/vibration problems, extensive deck cracking and movements [4]. The overall condition rating (D.O.T. Bridge inspection) was fair – due to the superstructure [1][2].

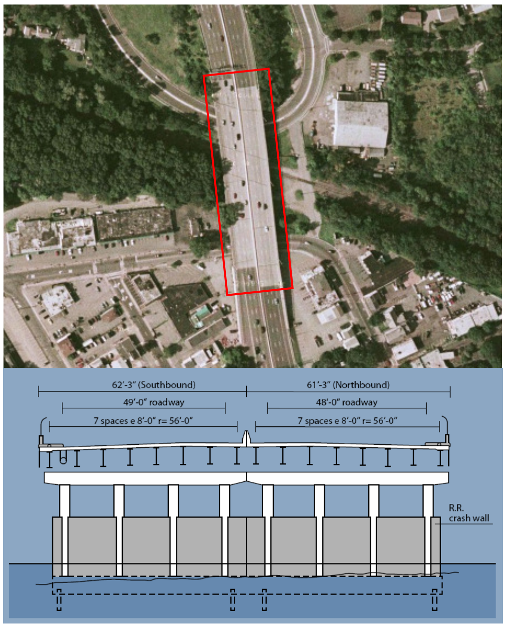


Figure 1. Overview of the Wayne Bridge in New Jersey

As detailed documentations of the bridge (drawings) and inspection reports have been provided in advance a targeted planning of the European campaign was possible.

3 OBJECTIVES OF THE CAMPAIGN

3.1 Critical Questions

The investigation aimed to answer the following questions [3]:

- Does this bridge have any strength/capacity issues? If so, what are they?

- How can technology be used to identify, quantify and understand these issues? Structural Health Monitoring?
- What is the root cause of any deficiencies in the bridge?
- What, if any, retrofits or interventions would you recommend?
- Maintenance and repair investigations for a 75-year life cycle?

3.2 Applied methodology

In order to clearly address the stated problems the approach described below has been developed and standardized by the authors.

The multi-level procedure is based on the entire lifecycle of the structure and considers certain characteristics of the structure gained from the following investigations:

- Visual inspection of the critical elements with detailed assessment following the Austrian RVS 13.03.11 regulations
- Design loading and specifications
- Capturing the dynamic characteristic of the structure using the BRIMOS® wireless methodology
- Identification of the bridge behaviour on expert level to create the basis for an understanding of the observed damages and physical properties

All relevant results of the listed components are incorporated into a probabilistic lifecycle model. Furthermore additional parameter studies enable to provide explicit answers to the stated critical questions.

4 VISUAL INSPECTION

4.1 Concept

The Austrian Guideline on bridge inspection RVS 13.03.11 [6] specifies that every bridge has to be surveyed at least every 6 years following a given procedure. The detailed results are combined to a bridge rating which is used for decision making.

In the presented case a selective inspection has been carried out based on the information provided by the documentation material received in advance. Even if it was decided to concentrate on the critical issues only, the inspection has been complete enough to allow integral assessment and rating of all necessary elements of the structure. A comparison to other assessment approaches is feasible.

4.2 Results of the visual inspection

The inspection did not show any extraordinary findings in addition to what has been documented before or reported by the other teams. Basically the structure appeared to be in fair condition with existing local problems (Figure 2).

The damages point out the typical issues of corrosion and cracking. Corrosion can be mainly attributed to water leaking from the expansion joints and the activities of the pigeons. Most severe damages are to be seen at the abutments and comprise mainly the end cross girders. The corrosion protection of the outer steel beams is intact which gives the bridge a reasonable appearance. Generally the corrosion condition of the main girders is good. The concrete parts of the structure also show a very reasonable appearance. There is heavy traffic on the structures in both directions with a considerable portion of trucks.

The cracks in the steel structure are mainly attributable to improper detailing of the structural steel components. Bracings - directly connected to the webs - without the required respective stiffeners on the opposite side invite damages. This creates an overstress in case of differential movement. Nevertheless this problem is most probably not progressive after the initial overstress has been released by cracking. Furthermore this damage concentrates on parts of the steel structure which are not the main bearing elements. Therefore any collapse without warning can be excluded. No immediate intervention is necessary.

The concrete structures are generally in good condition. The deck consists of a solid concrete slab without waterproofing and pavement. From below one can see only the steel sheets that have been used as a form for concreting. No traces of water seepage have been found. The slab is cracked at the cantilever beams where the bottom surface can be seen. These cracks correlate with the joints in the barriers which is an expected behavior. There is very little lime traces visible at these cracks which suggests that only small amounts of water penetrate here.

The abutments are cracked at the wing walls as well (both abutments, both sides). Looking at the abutment situation from the top it shows that there are water traps where water is invited to seep into the embankment behind the abutment wall. Freeze and thaw circles might have created the problem of cracking.

Concrete spalling with the appearance of local corrosion is to be seen on one of the piers. The corbel supporting the outer beam of the southbound structure (pier dividing span 1 and 2) is cracked. As this structural detail appears several times and the loading in this particular place is assumed to be far from being excessive this damage might also be caused by construction activities. Eventually a temporary overloading caused the cracks which subsequently allowed corrosion of the reinforcement. As there is sufficient redundancy at this particular place of the structure no immediate action is required.

Figure 2. Typical findings at the Wayne Bridge

As the bridge behaves rather lively there is extraordinary load on bearings and expansion joints. Traces that the dynamic action has taken a toll on the equipment can be found everywhere. Some bearings, especially at the border of the structure, have been found to be heavily corroded. Additionally some single units are slightly twisted. Nevertheless they still provide the required functions. Some reasonable retrofit work will be necessary here. A surprise failure of a bearing is not really to be expected and, if happening, would not lead to a progressive collapse without warning.

The expansion joints are a major problem. In this type of structure with its dynamic behaviour a perfect expansion joint does not exist. The main challenge of the assessed joints is the water tightness which is not given currently. Especially at the sidewalk of the structure the rubber seal shows brittle cracking. Assessing the bottom view of the structure, all steel parts and bearings are suffering of heavy corrosion in the mentioned area.

Furthermore proper detailing is missing particular with the transition from the embankment to the bridge where considerable impact is created by traffic. At the abutments, the drag plates have to be included in all expansion joint retrofit proposals as they have to be considered as one system.

According to the RVS 13.03.11 regulation the Wayne Bridge is classified in category 3 out of 6 implying that the structure is in good condition with moderate problems.

4.3 Comparison Austrian Rating vs. U.S. Rating

The following table (Table 1) shows and compares the available range of possible maintenance condition states. Their corresponding rating reflects the extent of damage according to US and Austrian standards. A conversion of the available DOT assessment of 2008 into the Austria rating system has been made. The bridge ratings on the one hand and the subsequent remaining capacity with regard to the early warning level on the other hand were derived. In fact the result of the inspection 2010 has confirmed the rating of 2008. In other words both procedures arrive at about the same total score. From the correspondence we learned that the Japanese score might be less favourable.

Table 1. Comparison of the bridge rating according to RVS 13.03.11 (2010) with DOT (2008) [1] assessment.

	Rating	Capacity
USA	5	0.50
AUSTRIA	3	0.62

The calculated capacity in Table 1 is to be understood as remaining safety with regard to the early warning level.

5 DESIGN LOADING AND SPECIFICATIONS

The design of the Wayne Bridge followed the regulations stated below:

Design specification:

1977 AASHTO Standard specifications for highway bridges including interim 1979 with N.J.D.O.T., modifications dated June 1, 1978; revised Feb. 29, 1980

Design loading:

AASHTO HS 20 - 44 + 10% or an alternate military loading of two axles, four feet apart each axle weighing 24,000 LBS., whichever produces greater stress.

6 STRUCTURAL HEALTH MONITORING

6.1 Monitoring Concept

The objective of the monitoring campaign was to identify the key performance indicators of the structure with regard to their relevance for civil engineering issues [8]:

- The bridge structure's relevant eigenfrequencies and corresponding mode-shapes:
  Load bearing capacity and operability
  Evaluation of the bearings
  Distribution of the global and local dynamic structural stiffness in the bridge's lengthwise and transversal direction
- Sensitivity analysis to investigate the progression, the character, the stability and probable changes in the energy content of the relevant eigenfrequencies:
  Load bearing capacity and operability,
- Energy dissipation path in the structure's lengthwise direction:
  Dissipation of the induced vibration energy, localization of problematic sections
- Vibration intensity at the entire bridge deck:
  Localization of weak points with regard to fatigue threat
- Comparison of measured values with the results of the finite element model:
  Reference to the undamaged initial condition

6.2 Dynamic Measurement

The dynamic measurement with BRIMOS® 11.02 was carried out on the 21st and the 22nd of June in 2010. The temperature during the measurements varied between 22°C (75°F) and 36°C (96.8°F). The measurements were done under ambient conditions on the one hand (environmentally excited vibrations) and under mostly unrestricted traffic - the traffic was limited to two lanes - including passages of heavy vehicles on the other hand.

Figure 3. Documentation of the BRIMOS® Measurement

Due to the time constraints and limited available equipment – only 2 sets of BRIMOS® Wireless equipment have been brought in the U.S. – a sensor layout with 40 different positions has been chosen. This procedure allowed limiting the campaign to one day for each driving direction (Day 1: Southbound Structure; Day 2: Northbound Structure). A total recording time of 2x9 hours of monitoring data are available for analysis and assessment.

At the Northbound Structure so-called In-Depth Monitoring was carried out at all four spans. At the Southbound Structure In-Depth Monitoring was conducted at Span#1 and Span#2, while at Span#3 and Span#4 so-called Hot-Spot Monitoring was used (Figure 4).

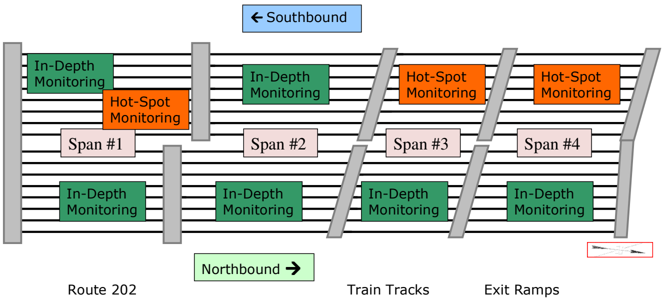


Figure 4. Overview – BRIMOS® measurements.

6.3 *Dynamic System Identification*

In addition to the dynamic measurement a finite element model of the bridge was developed with the software RFEM. To obtain a reasonable estimation in terms of model parameters only a model of Span#1 Southbound Structure was made (Figure 5).

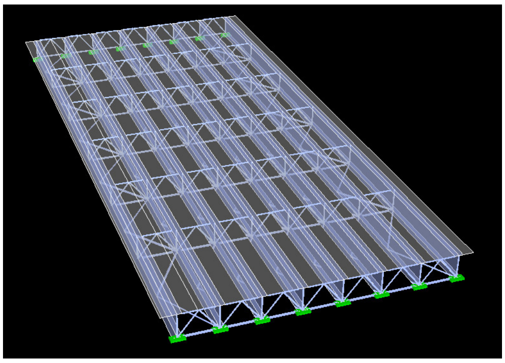


Figure 5. Numerical model Span#1 Southbound Structure using beam and plane elements.

In total the model comprises 720 beam elements, 9100 plane elements and 7200 nodes.

The calculated parameters serve as expected values based on the undamaged condition. The comparison of the results from the model with those of the measurements supports the assessment of the structural condition and enabled the identification of torsional mode shapes even if the measurement was done in one straight line only (utilization of 3D-accelerometers).

The following table (Table 2) includes all analytically computed eigenfrequencies for Span#1 Southbound Structure, which have been considered to be relevant for further evaluation.

Table 2. Expected values of eigenfrequencies based on the numerical calculation.

Mode Shape	Expected Frequency Values [Hz]
1st bending mode	3.29
1st torsional mode	3.39
2nd bending mode	6.37
3rd bending mode	7.00
2nd torsional mode	8.48
4th bending mode	11.28
5th bending mode	11.50

Examples for corresponding mode shapes are represented in Figure 6 - Figure 9.

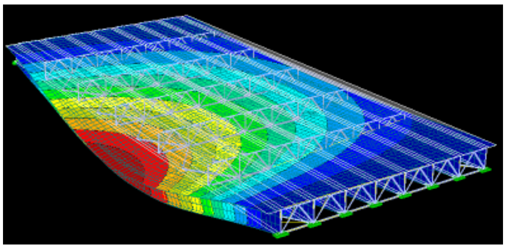


Figure 6. 1st mode shape 3.29 Hz – 1st bending mode

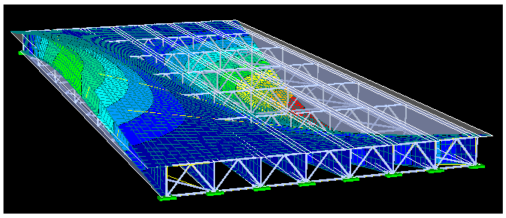


Figure 7. 2nd mode shape 3.39 Hz – 1st torsional mode

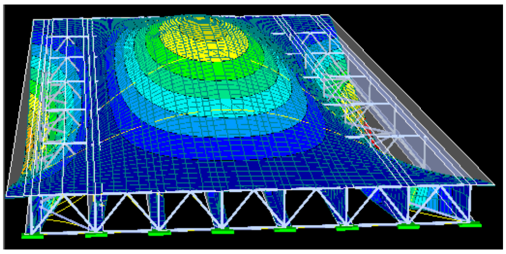


Figure 8. 3rd mode shape 6.37 Hz– 2nd bending mode (plane, cross)

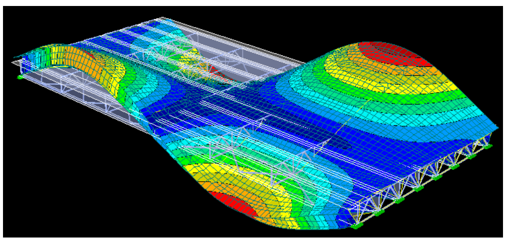


Figure 9. 5th mode shape 8.48 Hz - 2nd torsional mode

6.4 *Eigenfrequencies*

Since three-dimensional accelerometers are used for BRIMOS® measurements, it is possible to distinctively determine the dynamical characteristics of the structure in longitudinal, vertical and transverse direction. A three-dimensional accelerometer was used as reference sensor during the whole measurement - positioned at 40% of each span's length. Therefore the influence of the varying traffic intensity on the measured data as well as on the results of the analysis can be considered. The following Figure 10 shows a typical measurement signal recorded at the Wayne Bridge.

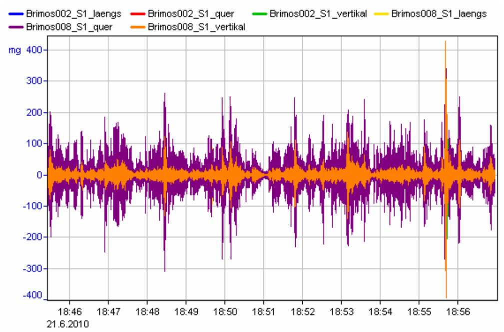


Figure 10. Characteristic acceleration signals, all measured signals

The measurements were made under ambient conditions on the one hand (environmentally excited vibrations) and under normal – partly restricted - traffic including passages of heavy vehicles on the other hand. For each sensor array one measurement file with a length of 11 minutes and a sample rate of 1000 Hz (= 1 millisecond) was recorded.

The eigenfrequencies which are extracted from the measurements can be understood as main indicators of the effective dynamic stiffness of the structure.

The frequency spectra (e.g. Figure 11) of all spans show a clear dynamic character in all three analysed dimensions. The relevant eigenfrequencies are primarily located in the range of 3.0 to 21.0 Hz. All of those eigenfrequencies are global eigenfrequencies (bending and torsion) linked with the dynamic stressing of whole spans.

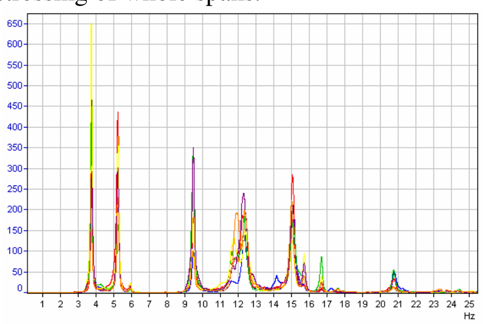


Figure 11. Typical frequency spectra (ANPSD), 0.2-25 Hz, vertical direction, Span 2, Northbound Structure

Deviations between the eigenfrequencies of the different spans primarily arise from the different geometrical properties (geometry: skew progression - ground view, span length and section height). The measured deviations correspond with the expected ranges. No extraordinary characteristics indicating limited operability or damage of the primary load bearing structure have been identified.

6.5 *Mode Shapes*

The curvature of the mode shapes is an important parameter for the assessment of the structural integrity and its operational characteristics (e.g. exhibiting highly stressed areas). Furthermore mode shapes enable the identification of changing boundary conditions, e.g. the settlement of a bridge support. The following figures (Figure 12 - Figure 14) exemplarily show characteristic mode shapes derived from the measurements at the Northbound Structure, Span 1.

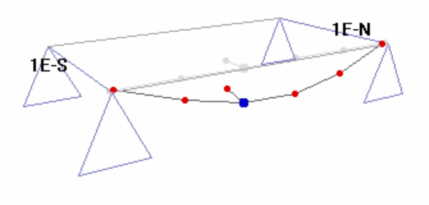


Figure 12. 1st bending mode - 3.73 Hz

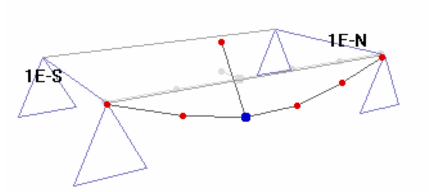


Figure 13. 2nd bending mode - 10.11 Hz, plane and cross

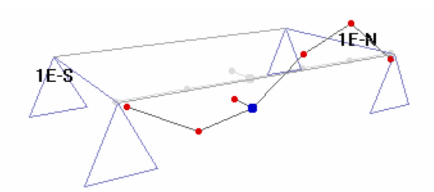


Figure 14. 3rd bending mode - 12.04 Hz

The determined mode shapes are characteristic for this kind of structure and correspond to the expected modes of vibration on the one hand and to the calculated modes shown before on the other hand (see Chapter 6.3. Dynamic System Identification). Distinctively occurring mode shapes indicate a satisfying structural maintenance condition.

6.6 *Vibration Intensity*

Intensive dynamic loading causes fatigue-failure of structures. The vibration intensity gives an impression of the energy-impact into the structure. The intensity for the investigated bridge object is determined at all sensor positions and included into a diagram, which represents the corresponding risk level.

The analysis of the vibration intensity shows the relation between the structure's eigenfrequencies and its corresponding oscillation amplitude. As a basic principle lower amplitudes are permissible for higher eigenfrequencies. If the oscillation amplitudes exceed a limit, damage by vibration based overstraining (fatigue) to the structure or to structural elements has to be expected due to vibration stress.

The vibration intensity is subdivided into 4 zones, ranging from low probability of damage due to dynamic stress (Zone I) up to very high probability of damage (Zone IV) (Classification according to C. F. Beards 1996 [7]).

The vibration intensity determined for all the existing outer beams is shown in the following Figure 15. The bridge object responds sensitive to traffic impact loading and shows values of vibration intensity in the range I and II but also already increased values in the range III. Range II indicates a zone, where damage might be caused by continuous dynamic stress. Range III indicates a zone, where permanent dynamic stressing causes damage to substantial parts of the structure.

The frequently occurring fatigue cracks are considered to be obviously consequences of the high vibration intensity. The damaged and probably too small dimensioned expansion joints intensify the force transmission into the structure and contribute to the high vibration intensity.

The frequency which is excited the most from traffic is located around 10 Hz and represents either the 4th or 5th vertical eigenmode. This frequency occurs in intensity level IV once (span 1 southbound), in intensity level III 3 times (span 1, 2 and 4 northbound) and in intensity level II for the rest of the spans.

Intensity IV is definitely a damaging level and corresponds with the cracks found in the deck slab in span 1 southbound. A loading impact that regularly occurs in intensity zone III means that in the medium-term damage from vibration shell be expected. Consequently this will most probably cause further crack patterns in the deck slab within the next five years.

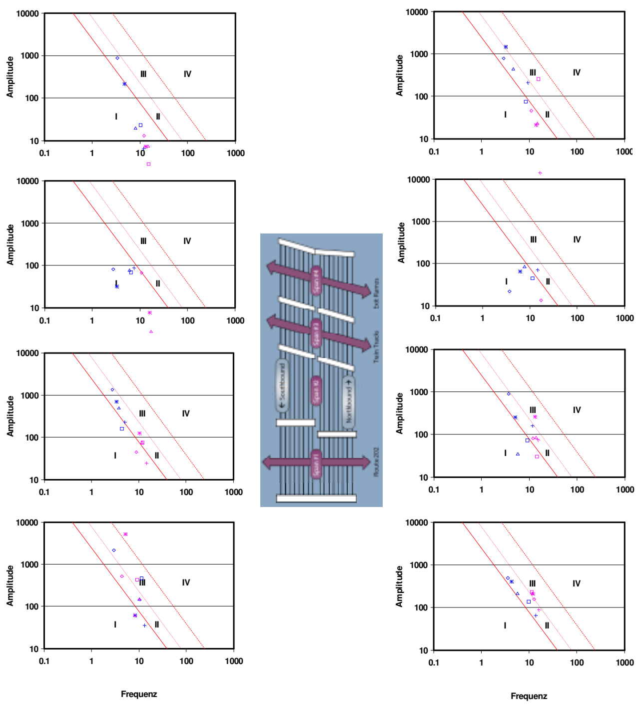


Figure 15. Distribution of Vibration Intensities over the Bridge

6.7 *Dissipation of induced Vibration*

The energy dissipation path is a suitable indicator for the condition of a structure and the main girders respectively. "Problematic zones" mostly dissipate energy caused by friction which is reflected in an increase of the local damping values. Higher damping values in the range of abutments or piers are system-based and thus have no direct influence on the assessment of the structure's condition.

As this analysis requires a sufficient number of positions recorded on each span the energy dissipation path has been determined only for 5 of the 8 spans. In these cases energy dissipation should be visible at the moveable bearings.

The given results show 3 regular cases and 2 cases of extraordinary behaviour.

Span 2 northbound does not show any energy dissipation indicating that the moveable bearings are not excited to any movement from dynamic loads. This indicates a malfunction of these bearings which might lead into overstress in the beams.

At span 4 northbound energy dissipates at the "wrong" bearings, which might either mean that there is a mistake in the drawings or the fixed bearings are malfunctioning (these bearings have not been inspected).

The pattern of damping values at the other analysed spans is typical for such a structural type. The damping analysis primarily reflects the dominantly occurring system damping due to the mechanical behaviour of the bridge. Increased values from local spots according to material damping which would point out additional certain damage of mechanically grave extend were not determined.

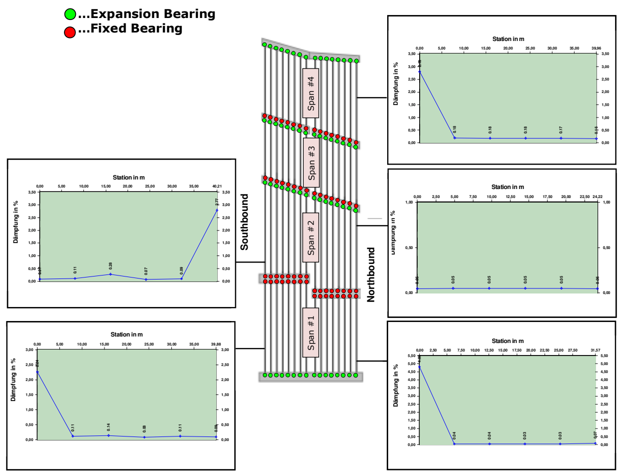


Figure 16. Recorded Energy Dissipation Paths.

6.8 *Evaluation of Structural Integrity*

So-called trend cards are an essential evaluation instrument in the context of full-scale measurements on bridge structures. Trend cards are obtained by evaluating frequency spectra taken from several measurements, telescoping them together and viewing them from above. For reasons of a better descriptiveness a two-dimensional visualisation is chosen. Showing the behaviour of the structure during certain timeframes of monitoring observation trend cards enable the identification and assessment of extraordinary behaviour.

For every span of the bridge object related trend-cards were derived. Figure 17 shows the spans' relevant stiffness-patterns in the vertical direction over the measurement's entire time period in the range from 0.2 to 25 Hz represented by the reference sensor defined for each span. The different quality of the trend cards depend on the total length of records available. The eight trend cards given here do not show any unusual behaviour and therefore confirm that the global structural behaviour is satisfactory. Considering the high vibration intensity given before this is a sign that there is no damage affecting the global structural behaviour yet. Extraordinary changes indicating arising deficiencies can not be detected in any of the trend-cards.

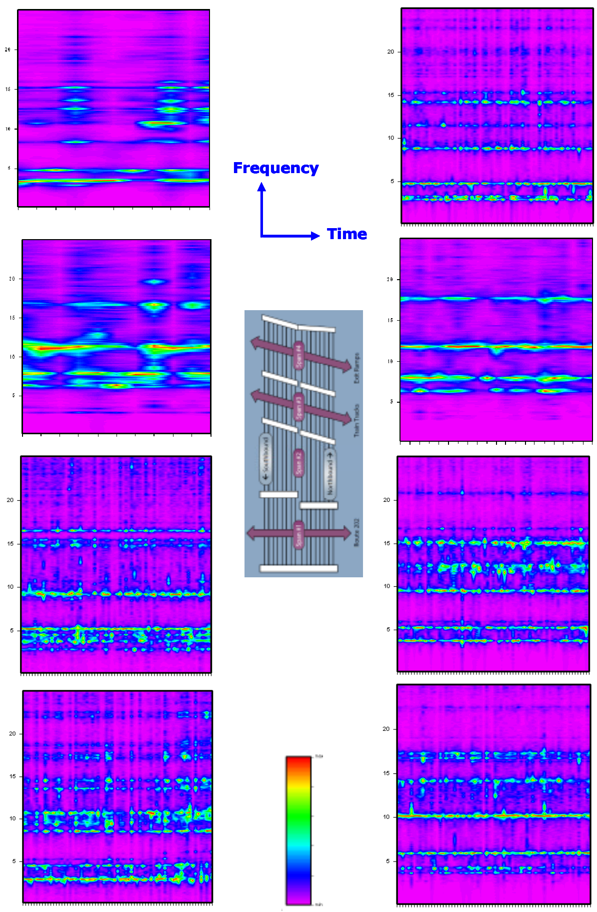


Figure 17. Trend Cards over the entire recording Period for all Spans

6.9 Summarizing judgment from dynamic monitoring

The present assessment indicates that the structure's load-bearing capacity and its operability are available to a satisfactory extent at the time of investigation.

An imminent threat of collapse is currently not recognizable. According to the performed measurements no immediate action concerning traffic loading restrictions or rehabilitation measures is required.

The structure's dynamic response reveals the bridge to be in fair condition, even though a high level of vibration intensity induced into the structure is clearly evident. The vibration intensity partly lies in the range of category II and III indicating that continuously occurring dynamic loading on this level causes damage at significant parts of the structure and supports an accelerated ageing process. The frequently occurring fatigue cracks are considered to be obviously consequences of the high vibration intensity.

The damaged and eventual too small dimensioned expansion joints intensify the force transmission into the structure and contribute to the high vibration intensity.

Since high vibration intensities in general come along with an accelerated consumption of lifetime the observation of the structure's lifecycle curve will be of more significance in the future.

The analysis of the eigenfrequencies over time and along the lengthwise direction of the bridge shows a regular global maintenance condition indicating vital condition in terms of bending and torsional resistance and structural integrity.

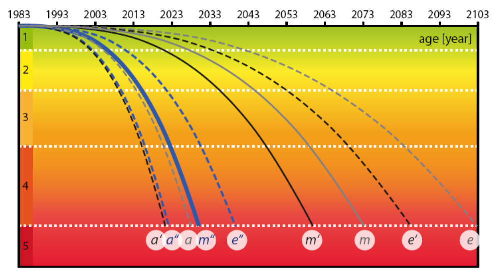
The energy dissipation path shows three normal cases and two cases of extraordinary behaviour with occurring deviations concerning the documented bearing concept.

A strictly visual driven rating would classify the bridge object in fair condition with local problems. The broadening of expertise based on the second evaluation stage (assessment based on dynamic measurements and numerical simulation) leads to a clear increase of the structural overall condition index – indicating the structure to be in good condition with local damage.

7 LIFE CYCLE ANALYSIS

To answer the main question in terms of the remaining structural life expectation and demanding maintenance and repair options for a 75-year lifecycle the newly developed Life Cyle Methodology has been applied. The results of the visual inspection, the design code safety level and the results from dynamic monitoring are incorporated. For further reading see [9].

It is shown that under the current circumstances the theoretical remaining lifetime is estimated to be 21 years only (Figure 18). The result meets the expectation that without any retrofit the lifetime of the structure is limited.



Residual life expectancy [years]	Initial range		Adaptation		Enhanced prediction	
Lower bound	a	18	a'	11	a''	12
Mean	m	**63**	m'	**50**	m''	**21**
Upper bound	e	93	e'	76	e''	30

Figure 18. Derived Life Cycle Curve (Condition Index) enhanced Model => Range of Life Expectancy.

7.1 Recommended Retrofit Interventions

To extend the remaining lifetime of the Wayne Bridge – based on the results of the dynamic monitoring and the visual inspection - the following retrofit interventions are recommended:

Superstructure:
Renewal of corrosion protection, repair of the concrete surface and the corroded stringers, replacement of the wind bracings to handle the problems with the occurring fatigue cracks, and proper replacement of the bearings

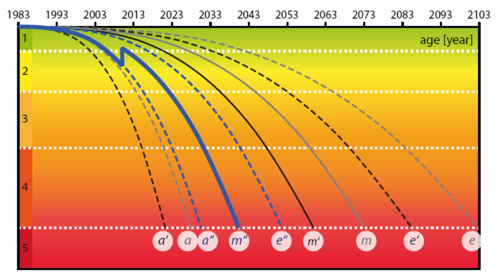
Substructure:
Renewal of corrosion protection, repair of spallings, holes and concrete pockets as well as removal of contamination of the concrete surface, deep injection of cracks

Dewatering:
Establishing of an effective drainage concept

Expansion Joints:
Proper detailing & design

- Full replacement (abutment area) or
- Partial replacement (at least seals above the piers)

In case that the bridge receives the proposed retrofit durability issues and above all a reduction of the intensive vibration to reasonable levels are most likely ensured. The remaining lifetime of the structure can be expected to increase to at least 31 years (Figure 19).



Residual life expectancy [years]		Initial range		Adaptation		Enhanced prediction		Prediction due to maintenance
Lower bound	a	18	a′	11	a″	12	a″	21
Mean	m	**63**	m′	**50**	m″	**21**	m″	**31**
Upper bound	e	93	e′	76	e″	30	e″	41

Figure 19. Derived Life Cycle Curve (Condition Index) enhanced Model incl. Maintenance Interventions => New Range of Life Expectancy.

8 CONCLUSIONS

A strictly visual driven rating would classify the bridge object to be in fair condition with local problems.
Deficiencies are mainly situated in dewatering and in the actual vibration intensities.
The broadening of expertise from visual inspection based knowledge to the second evaluation stage (assessment based on dynamic measurements and numerical simulation) leads to a clear increase of the structural overall condition index – indicating the structure to be in good condition (in terms of structural resistance) with local damage.
The structure's load-bearing capacity and its operability are available to a satisfactory extent.
The current dynamic analysis of the Wayne Bridge with BRIMOS® revealed an accelerated consumption of the global lifetime.
The remaining lifetime is estimated to be 21 years only.
The remaining lifetime can be considerably enlarged by investing into a reasonable retrofit.

ACKNOWLEDGMENTS

VCE as the coordinator of the IRIS group is very grateful to NJDOT, PB and FHWA & Drexel University for the perfect support & the kind arrangement in the course of preparing and conducting the bridge inspection and measurement campaign. We thank all of you for the support and help provided.

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