ACOUSTIC EMISSION TESTING OF IN-SERVICE CONVENTIONALLY REINFORCED CONCRETE DECK GIRDER SUPERSTRUCTURES ON HIGHWAY BRIDGES

FINAL REPORT

SPR 633

ACOUSTIC EMISSION TESTING OF IN-SERVICE CONVENTIONALLY REINFORCED CONCRETE DECK GIRDER SUPERSTRUCTURES ON HIGHWAY BRIDGES

Final Report

SPR 633

by

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16. Abstract

Three reports were produced from research sponsored by the Oregon Department of Transportation on acoustic emission (AE). The first describes the evaluation of AE techniques applied to two reinforced concrete (RC) bridge girders, which were loaded to mimic in-service conditions. The main goal was to get a thorough understanding of how AE methods can be used with RC and in what way these methods can assist in maintaining the state's aging RC deck girder bridges. Recommended settings for data acquisition and processing were evaluated. In addition to the complex full-scale beam components, studies were performed on smaller test specimens that improved understanding of stress wave propagation through reinforced concrete and the response of acoustic emission sensors in detecting these waves. Some qualitative and quantitative assessment methodologies were described, and examples and limitations of the methods were presented. Source locations in three dimensions were performed, and strategies on how to best deploy sensors were evaluated using Monte Carlo Simulations.

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APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	millimeters squared	mm^2
ft^2	square feet	0.093	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometers squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3
	NOTE: Volumes greater than 1000 L shall be shown in m^3.			
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit	(F-32)/1.8	Celsius	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
m^2	meters squared	1.196	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	kilometers squared	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celsius	1.8C+32	Fahrenheit	°F

*SI is the symbol for the International System of Measurement

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1.0 PURPOSE OF AE TESTING

Most commonly used forms of non-destructive testing (NDT) applied to structural load testing such as visual testing (VT), Magnetic Particle (MT), Dye Penetrant (PT), Ultrasonic (UT), Radiography (RT) and strain gage or crack width/length measurements require a minimum level of accumulated damage to occur prior to reliable detection. All of these methods respond to the accumulation of damage and not the actual damage process. AE has the ability to respond to the damage process itself irregardless of the level of accumulated damage. Additionally modern AE testing equipment and procedures can be very sensitive to the formation of structural damage. These two characteristics can make AE a useful testing method when assessing the in-service performance of structures subjected to changing loads.

The primary benefit of including AE testing into a structural performance test is to develop AE testing criteria that can be used to identify the occurrence of structural damage in real time with very high sensitivity. Correlating the AE criteria with load demand from test and ambient loads can provide meaningful insight into the level and nature of structural damage occurring and accumulating in the members being tested. This can be particularly useful when the structure must be kept in-service when structural load ratings and / or observed structural conditions indicate the structure is likely operating at a lower level of reliability (gray area) then the owners normally prefer. Leaving a highway bridge that fits this situation in-service while a new structure is being designed and constructed is a good example of an appropriate application of including AE in a structural health monitoring program. Any significant structural damage that occurs will very likely be detected by AE which can be used to trigger on-site visual assessment and more in-depth evaluation of the other structural performance parameters such as strains and crack motions.

1.1 APPLICABLE STRUCTURES

These testing procedures were developed specifically for conventionally steel reinforced concrete highway bridge superstructures with emphasis on the deck girder configuration. In addition the examples shown were developed using vintage (1950’s) materials and construction practices. In all likelihood they are equally applicable to any commonly encountered conventionally steel reinforced concrete structural member subjected to variable loads.

2.0 RECOMMENDATIONS FOR TESTING AND MONITORING OF RCDG BRIDGES USING AE

Based on the laboratory and in-service bridge test results determined from this research a set of recommended guidelines for testing vintage RCGD highway bridges that are subject to diagonal tension cracking was developed and is described below.

2.1 VISUAL INSPECTION OF BRIDGE

A visual inspection of the bridge, in accordance with National Bridge Inspection Standards, should first be performed to identify the general physical condition, identify, locate, map and measure all visually detectable cracks or other forms of damage. Even if a recent bridge inspection report is available, it should be verified with an inspection prior to planning the testing procedures. Review of maintenance activities over the life span of the bridge is also desirable.

2.2 STRUCTURAL LOAD RATING

The most current load rating for the bridge should be obtained and the sections with the lowest rating factors should be compared to the physical damage observed in the inspection and determine if there is reasonable correlation. Preferably the rating was performed using one of the more modern codes such as LRFR (Load Resistance Factor Rating) or better yet using the procedures specified in SPR 350 (*ODOT 2005*). The loads considered in the rating should be compared to both the real ambient loads and the loads that can be practically applied with test trucks. A load rating performed using the test trucks weights, axle configurations and load placement is highly desirable for making the best use of the test data.

2.3 SELECTION OF CRITICAL REGIONS OR SECTIONS ON STRUCTURAL MEMBERS

An analytic load rating such as AASHTO LRFR should be performed on the structure of interest prior to developing the structural load testing procedures. The demands from both test trucks and in-service loads should be compared to the capacities along the entire length of the structural member to be investigated. Critical sections and failure modes at these sections should be identified.

2.4 IDENTIFICATION OF POTENTIAL DAMAGE MECHANISMS IN THESE REGIONS

Based on the axial, shear and moment capacities and demands at the critical sections of interest identify potential damage or failure mechanisms such as flexural cracking, diagonal tension cracking, compression failure, shear-compression failure, rebar debonding, rebar pullout, etc. This information can be used to predict the origin and progression of visually observable damage and identify common structural performance measurement approaches such as rebar strain, concrete crack width, girder deflection, etc. to be used to correlate with the AE data. In addition it can also provide guidance on AE sensor positions relative to the girder section being tested.

Based on the visual inspection and load rating data the appropriate test section on the bridge girder lines should be selected. The ideal section will contain significant diagonal tension cracks that are all spaced approximately the same distance for the nearest vertical support structure, i.e. bent or pier on each girder line. The cracks should cross nearly the full depth of the stem at an angle, preferably close to 45 degrees, with one or more shear stirrups crossing the crack. If several of these sections are present then choose one considering maximum crack width and / or ease of access. Ideally strain gages and / or crack mouth opening displacement (CMOD) transducers can be installed on each girder line at the chosen section with the strain gage attached to a shear stirrup that crosses a diagonal tension crack and measuring CMOD near the strain gage location.

2.4.1 Select AE Sensor Type and Array Deployment

If low-frequency, e.g. 60 kHz resonant type sensors are available; their use is preferred due to greater sensitivity and potential coverage than higher frequency units. Commercially available high-fidelity sensors are generally not sensitive enough to be of practical use for field testing. The array deployment should be decided considering the sensor type, number of sensors and desired area of coverage. For general application when more then one crack is present, a widely spaced linear array deployment centered at mid depth of the stem will provide the greatest coverage. Sensor spacing up to approximately 6 feet can be used with the 60 kHz sensors and 1 ½ to 2 feet for 150 kHz sensors. Depending on the number of sensors available and the locality of cracks, using more than one linear array, e.g. one array on each girder line, may be desirable.

If there is one particular crack or other damage feature that is of concern then using planar arrays that cover the defect area are preferable. Ideally enough sensors can be employed to deploy two or more planar arrays of 3 channels or more around the defect area. When determining the exact location of each sensor, it should be kept in mind that non-symmetric sensor placement around the defect will provide the greatest accuracy for source location algorithms if they are to be used. Mounting sensors on both sides of the stem and the bottom face is desirable for optimal coverage of the damaged region. It is also desirable to locate and mark on the girder stem the location of all stirrups in the test area using a rebar locator.

Another factor to consider is access to the test surfaces. Bridges will typically require access equipment ranging from ladders to man lifts. In many instances use of these tools will involve traffic control which may be the deciding factor sensor placement. Safe access is a must because installation of the parametric and AE sensors and related lead wires is time consuming work that requires comfortable and steady access.

2.4.2 Selection of structural performance criteria and ability to track load level on test section (analog correlation)

AE testing requires a variable load cycle to be applied to the structural member being tested. For highway bridges this will often be accomplished with the application of live load in the forms of test trucks of known weight and axle configuration and ambient or in-service truck loads. Thermal stress, wind or girder loads induced by substructure or foundation motion could also be used for specific applications.

The conventional structural performance measurements applied provide two main functions: 1) quantifying load position during the application for use in correlating the AE data to load application and 2) quantifying the real demand on the test section from the applied loads.

Two practical and common methods are often used for concrete highway bridge superstructures being the crack mouth opening displacement (CMOD) of existing structural cracks in the concrete and axial strain in reinforcing steel.

Crack width measurements have the advantage of being easily measured using a simple potentiometer or LVDT and are one of the most common measurements taken during routine bridge inspections of such structures. The width of such cracks can be used as a qualitative indication of accumulated damage; the wider the crack, the more potential accumulated damage at the section. Typical crack widths for flexural and diagonal tension cracks in the types of structures investigated under SPR 644 range from 0.01 to 0.08 inches. Crack width displacement can be used to estimate the position of the test or ambient truck on the span relative the section of interest; in both flexure and shear cracks the displacement typically reaches a maximum value when the section is at maximum demand. Typical crack width displacements measured during the study have amplitudes ranging from 0.0002 to 0.008 inches.

The disadvantages of crack width measurements is that they cannot be directly related to actual demand on the test section because they can in some cases close up permanently if new cracks form in the girder. It is purely a qualitative measurement of accumulated damage and instantaneous demand. Because crack width measurements are so easily obtained they should almost always be included in the test or monitoring program.

Strain measurements in the reinforcing steel can provide the best quantification of both actual structural demand on the test section and the position of the test truck relative to the test section. The strain gages should be mounted to the reinforcing steel at the location where the concrete is cracked for maximum sensitivity and ease of correlation with damage calculations. For diagonal tension cracks the shear stirrups are instrumented and for flexural cracks the tension bars are instrumented. Strain ranges typically encountered will range from 30 to 300 microstrains (900 to 9000 psi in steel). Sensitivities of 1 microstrain (30 psi in steel) are practically achieved in practice. This high level of sensitivity can easily detect the crossing of a 3000 lbf GVW passenger car and in some cases a 200 lbf human walking across the span.

Strain gages with very short gage lengths (1/8 to ¼") are typically required for deformed reinforcing steel. It is most desirable to install the gage between deformations; if necessary one or two deformations can be ground smooth in the region of gage installation.

The disadvantage of reinforcing steel strain measurements is the potential difficulty of properly installing the gages on bars that are often buried 1 to 4 inches inside of the concrete girder. The greater the concrete cover the more difficult the installation. Only ¼ Wheatstone bridge configurations are practical in this application. Proper installation is paramount to the successful use of strain gages.

2.4.3 Data Acquisition Equipment Location

A location to setup the data acquisition equipment should be selected that is safe to work from for both the operators and equipment. Lead wire runs must also be considered when choosing the location. The AE system can tolerate lead wire runs exceeding 200 feet if necessary, but typically CMOD and particularly strain transducers will not unless signal conditioning can be applied at the transducer. If a medium to long term health monitoring system is to be employed, then vandalism and theft of the expensive test equipment must also be considered.

2.4.4 Mount and Check the structural performance and AE sensors

The structural performance sensors should be installed prior to mounting the AE sensors, especially if strain gages are to be used. This will reduce the chances of damaging the AE sensors. The AE sensor fixtures should be mounted in the sensor locations determined in Section 3.4.2.

3.0 STRUCTURAL PERFORMANCE SENSOR INSTALLATION

Sensors that measure or respond to structural performance are very important to include in an AE test program. The primary function is to correlate the AE data to the loading demand on the structure. The most common sensors used for concrete bridge testing are strain gages, linear displacement transducers, load cells, tilt meters and accelerometers. Each of these sensor types have different methods of testing installation quality which should be followed.

3.1 AE SENSOR MOUNTING

AE sensors must have the receiving aperture acoustically coupled to the surface of the structural component being tested. Proper acoustic coupling of the sensor is paramount to running a successful AE test. The surface of the concrete must be ground smooth and relative free of voids. A quality acoustic couplet, such as DOW vacuum grease or silicon grease, proves to be the most effective short term couplant. This fills the small gap and voids between the concrete surface and the AE sensor aperture. The sensor must be held firmly in place with a normal force of 5 to 20 lbf for proper acoustic coupling. Small clamping devices can be attached to the concrete surface using CA glue and springs plunder or screw head used to press the AE sensor onto the surface. Acoustic isolation between the clamp and the sensor is desirable and easily accomplished with a 1/8 to ¼ inch this soft rubber gasket between the top of the AE sensor and the mating surface of the plunger or screw head. For applications such as SHM where the AE sensors are going to be left in place for long period attaching the sensor to the structure with appropriate glue is recommended. The silicon grease can dry out over time and greatly decrease the quality of the acoustic coupling. Epoxies, CA and hot glue have been found to work well. No clamping device is required when this approach is used.

Strain relief for the lead wires connecting the AE sensor to the AE data collection system is required to prevent any loading on the AE sensor other then the clamping force.

3.2 AE SENSOR INSTALLATION TESTING (BEFORE, DURING AND AFTER)

Mechanical pencils using 0.5mm diameter lead of 2H hardness are a standardized AE source to apply to the surface of the test member as detailed in ASTM E976. Using this form of AE source for calibration of AE sensors provides repeatable high amplitude, broad spectrum excitation that closely resembles the shot rise time and duration AE source that simulates brittle facture failure mechanisms. Once the AE sensors have been installed and acoustically coupled to the concrete test section performing pencil lead break calibrations is the most efficient and effective method of quantifying the quality of the installation. Such calibration methods are somewhat statistical in nature. At least five pencil lead breaks should be applied within 2 inches of each sensor. The peak amplitudes recorded from each break should typically be between 93 and 99 dB with an extreme range of less then 3 dB for most sensors. If this is not the case repeat the test for two addition sets of five breaks. If either data set does not meet this criteria reinstall the AE sensor until the criteria is met. If this is not possible then either the AE sensor or its installation (the lead wires or the AE data channel) likely have a fault and must be corrected.

Once the AE sensor installation passes this test continue breaking pencil leads in the same spot in groups of five over increasing distances from the sensor in 3 inch increments. The results of this testing procedure will establish the maximum distance a strong AE source can be expected to be detected from this particular sensor in this particular installation. This effective distance or range is typically 12 to 48 inches using the recommended AE sensors on 4000 psi concrete structures free of significant cracks.

Many AE sensors are capable of imparting an AE source into the test medium with a calibration pulse. A decaying sinusoidal electrical pulse is sent to the AE sensor causing it to respond as a driver. On resonant type AE sensors the aperture will respond with a large amplitude decaying sinusoidal motion that has many more cycles then the input pulse. The magnitude of this AE source is comparable to the pencil lead break. The frequency content tends to be monochromatic and at or near the peak sensitivity frequency of the sensor.

Some AE testing equipment offers such a calibration pulse option. An impulse voltage signal with peak amplitude ranging from 10 to 400 V is sent to a particular AE sensor. Some AE sensors require the use of a preamplifier between the transducer and the data collection system. The preamplifier can be damaged by this calibration pulse if it is not designed to accommodate passing of this voltage.

A major advantage of using this option for testing sensor installation is convenience. Once the sensors acoustic coupling and effective range have been determined with the pencil break method they can be rechecked with the calibration pulse method. Provided the AE sensor spacing is sufficiently small (12 to 48 inches) at least one sensor in the array should be able to detect and measure the response of from another AE sensor acting as an AE source with the calibration pulse.

Sensor installation testing should be performed at a minimum before and after the loading of the structure. It is very desirable to test more frequently for example between various loading sequences. The calibration pulse method makes this very practical to perform.

3.2.1 Set AE Thresholds

With the data acquisition system up and running the triggering thresholds for each AE channel should be set. Ideally five minutes of data with no alternating loads on the structure should be measured. The RMS levels from each sensor can be used to determine the lowest threshold levels which for a quality AE system and sensor should be around 3 times the RMS value. Dynamic threshold levels can be implemented with good success on AE systems so equipped (*Schumacher 2008*). Once the thresholds are set ambient traffic should be allowed to cross the structure. The recording threshold, which is typically higher than the detection threshold, can be set by observing the response of the system to insignificant loads such as small passenger cars.

3.2.2 Run Controlled Load Cases

Once the data acquisition system is fully operational and threshold set, the controlled loading can be conducted. It is almost mandatory that no other alternating and preferable no additional static load are on the test span or the attached approach spans during the application of the controlled loads. With highway bridges this can be challenging and will require proper planning of traffic control. On high volume highways the rolling blockage performed in low volume hours of operation is recommended. Typically under these conditions static load cases cannot be applied. If permitted, static load cases are desirable for unambiguous calculations of the calm ratio (*Lovejoy 2006*). Quality field notes should be taken during the test runs so there is no ambiguity regarding which loading case corresponds too which data set. Photographs of each load case are very useful for post processing of the data.

Typical load protocols can be as follows:

3.2.2.1 Load – hold – unload

With no other live loads on the bridge the test trucks are slowly positioned on the span to produce maximum demand on the test section. Once in place the truck remain static for 1 to 10 minutes and then are removed one at a time. If traffic control allows it is desirable to perform this sequence with first one, then two and then three trucks combined, incrementally increasing the maximum load. If practical this sequence should be repeated in each travel lane.

3.2.2.2 Slow continuous load – unload

Have a single truck cross the span(s) in each lane at a constant speed of 5 to 10 mph. Repeat this sequence with two trucks nose to tail or side by side depending on what configuration produces the highest demand on the test section. If the structure carries a major highway that cannot tolerate bridge closures this approach can be very effective when a rolling blockade is employed to keep ambient loads off of the structure during the controlled test run.

3.2.2.3 Fast continuous load – unload

Repeat the slow continuous load – unload sequence at normal highway speeds. Impact can have a significant effect on peak demand levels and load rate can also effect damage progression in concrete and hence affect the AE results.

3.2.3 Ambient Load Cases

Typically the controlled loads will not be as severe as the upper end of the ambient loads. For this reason it is desirable to collect at least 100 ambient load cases. Often times the most severe loads will occur when a combination of trucks are on the span at the same time. This event is probabilistic and thus requires a significant amount of loading to capture. A full time structural health monitoring system will provide the best chances of capturing such events.

3.2.4 Calculate Damage Parameters

For each controlled load case the calm ratio, severity and historic index should be calculated. The calm ratio is a single parameter that characterizes the current state of accumulated damage in the test section. This parameter represents the entire load case and generally will not change unless more damage is imparted to the test section. If insufficient AE activity is recorded, i.e. less then 200 hits, then the value can vary greatly, though typically on the orders of 0.1 to 10. A stable calculation of the calm ratio should be repeatable in the range of 0.1 to 1.0.

The severity and historic index for each AE channel should be calculated over each load case data set. Using J=50 and N-K=200 is a good starting point for the factors needed to calculate the severity and historic index respectively. Expect the lighter load cases, i.e. test truck crossing being mostly supported by non-instrumented girders, to not yield reliable or for that manner any results with these two parameters. Again more then 200 hits are required to begin calculating the historic index and 50 for the severity. The data are calculated per channel and typically presented that way. Peak values from all channels over the entire load case are also a useful way to present the overall results.

4.0 ASSIGNMENT OF DAMAGE LEVEL

The structural damage level from diagonal tension cracks has been defined to fit into three categories or levels ranging from the least amount of damage at level 1 to the most severe damage at level 3. These damage levels can be determined from visual observation of the girders, structural demand measurements such as strain gage data and lastly AE data. Visually the ODOT crack comparator tool separates the three levels based on maximum crack widths in the girders high shear zone. Extensive laboratory testing performed under the project found a correlation between the shear crack widths and maximum loading level relative to capacity as well as the AE damage parameters.

4.1 ODOT CRACK COMPARATOR RATING

Crack widths are divided into three ranges as defined on the ODOT crack comparator tool. Level 1 corresponds to crack widths that are visible to the naked eye but are less than 13 mils. These are considered to be hairline cracks, and no particular action on the bridge inspector's part is required for such cracks. Level 2 corresponds to crack widths between 13 and 25 mils and is required to have their extents traced on the beam by the inspector. Level 3 corresponds to cracks with widths greater then 25 mils. These cracks are to have their extents traced and maximum width measured and recorded with the date of the inspection on the beam and on a crack map which is to be included in the bridge inspection report.

4.2 STRUCTURAL DEMAND RATING

The diagonal tension crack widths were found to correlate with the previous maximum structural demand on the test girders. Using the ODOT crack width levels to define structural demand levels that correlate the following ranges are recommended:

Level 1 – Maximum previous loading / Ultimate capacity < 0.5

Qualitative interpretation – Light loading with no serviceability concerns

Level 2 – 0.5< Maximum previous loading / Ultimate capacity < 0.7

Qualitative interpretation – Moderate loading that may justify continued surveillance or refined load capacity calculations when operating at the higher end of the range.

Level 3 – Maximum previous loading / Ultimate capacity > 0.7

Qualitative interpretation – Heavy loading is implied and will require refined load capacity calculations and possibly a full time structural health monitoring system, if the loading is not reduced or the capacity increased.

AE response to demand/damage levels

The AE damage parameters, Felicity ratio, calm ratio, severity and historic index were correlated with the three levels of structural demand and damage. The recommended interpretation of each is discussed below.

4.2.1 Equation 1

Felicity ratio = Load or demand at start of AE activity in current load cycle / Maximum previous load or demand

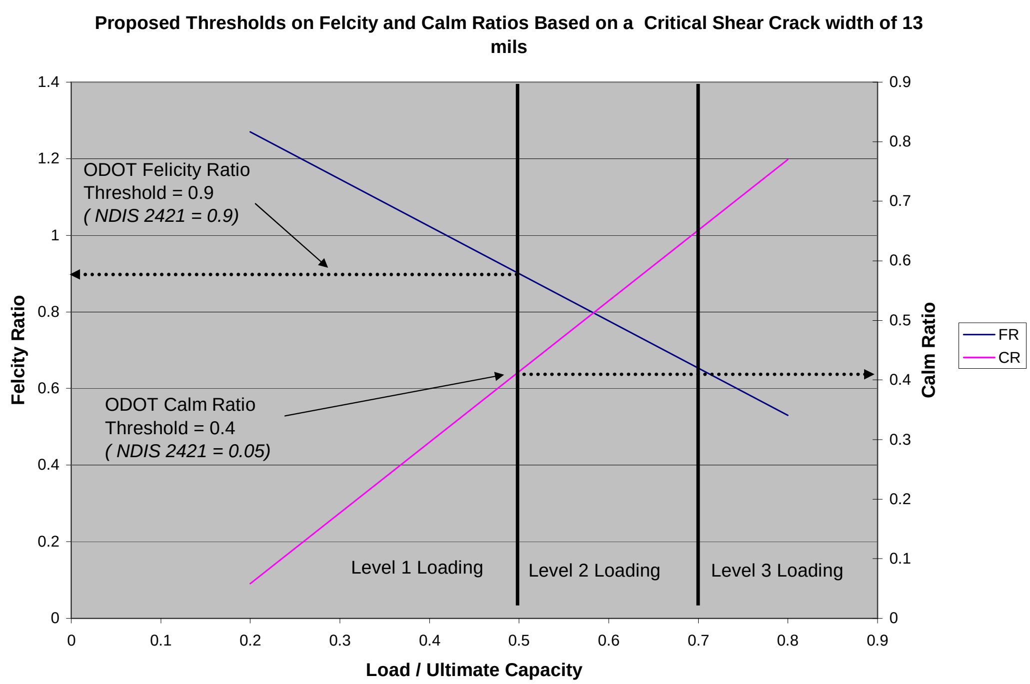

Figure 1: Thresholds for Felicity and calm ratios based on a critical shear crack width of 0.013 inches.

Based on the 42 full sized laboratory beam test data the Felicity ratio was found to have a reasonable fit to a linear response with respect to the loading level and is shown in Figure 1. Felicity ratios greater then 0.9 indicate accumulated damage corresponding to level 1. Felicity ratios between 0.9 and 0.65 indicate accumulated damage corresponding to level 2. Felicity ratios less then 0.65 indicate accumulated damage corresponding to level 3.

As discussed by Lovejoy (*2006*) and Schumacher (*2008*) this interpretation of the Felicity ratio is only applicable to previously unloaded test beams that are subject to a series of monotonically increasing load – unload cycles. If any other significant load cycles are repeated between increasing loads – unload cycles, i.e., fatigue loading, the correlation between the Felicity ratio and damage state shown no longer exists. In addition to this limitation is the fact that nearly all in-service highway bridges have not only accumulated fatigue damage but have an unknown loading history. These observations render the Felicity ratio of little practical use when testing in-service structures.

4.2.2 Equation 2

calm ratio = total unloading AE hits / total loading AE hits

Based on the 42 full sized laboratory beam test data the calm ratio was found to have a reasonable fit to a linear response with respect to the loading level and is shown in Figure 1. Calm ratios less then 0.4 indicate accumulated damage corresponding to level 1. Calm ratios between 0.4 and 0.65 indicate accumulated damage corresponding to level 2. Calm ratios greater then 0.65 indicate accumulated damage corresponding to level 3.

The calm ratio has been found to be a robust AE damage parameter for the subject application which provides a pseudo-quantitative characterization of damage state. Laboratory data has shown the relationship depicted in Figure 1 to hold true even with millions of cycles of fatigue damage imparted to the test beam (ref. #3). From a field testing perspective it is also very practical to specify loading protocols which produce AE data from which the calm ratio is well defined and easy to calculate.

Intensity Analysis - Another applicable method for identifying and classifying structural damage using AE parameter data is Intensity analysis as presented by Fowler et al. (*Fowler 1989*). The origins of Intensity Analysis come from the Fiber Reinforced Plastic (FRP) pressure vessel industry and are more developed for this class of structure. These methods were first applied to pre-stressed concrete highway girders by Fowler et al. in 2001 (*TXDOT 2001*). These methods were again applied to a new bridge constructed with pre-stressed concrete girders in Poland in order to establish a baseline response for future monitoring in (*Golaski 1989*).

Intensity is a measure of the structural significance of an AE source. Two parameters are needed to determine the Intensity, the historic index and the severity. The historic index, H (t), weighs the average signal strength of the last 20% or 200 hits, which ever is the smallest number, of hits to the average signal strength of all hits for the load protocol. Analytically, it is a method for determining changes in slope in the cumulative amplitude versus number of hits curve which can identify the arrival of the “knee” in the curve. Providing early detection of the “knee” in the cumulative amplitude versus hits curve is useful for identifying new damage as it occurs in the loading curve.

Equation 3 defines the historic index. It can be seen to be a function of time and is calculated over all peak amplitudes (S_{0i}) from i to N, where N is the total number of hits measured up to and including time *t.* Limits on N-K are imposed to meet the above definition as shown.

4.2.3 Equation 3

$$H(t) = \frac{N}{N-K} \sum_{t=K+1}^{N} S_{0i} / \sum_{i=1}^{N} S_{0i}$$

N= total number of AE hits up to and including time *t*
S_{0i} = Signal Strength of the i^{th} hit
K= empirical parameter

Historic index does not apply for N<200 hits
K=0.8N for 200 < N < 1000
K=N-200 for N> 1000

The severity Index, S_r , is defined as the average of the 50 largest peak amplitude hits striking a particular sensor. A significant increase in Severity can indicate the onset of more serious structural damage as the loading progresses. It has been found that severity increases sharply at the "knee" in the cumulative amplitude versus hits curve [ref. #3, 3b]. The severity is numerically defined in Equation 4. The parameter J, the number of peak amplitudes to average over, is an empirically derived constant much like N-K in the historic index.

4.2.4 Equation 4

$$S_r = \tfrac{1}{J} \sum_{i=1}^{i=J} S_{0i}$$

S_{0i} = Signal strength of i^{th} hit
J = empirical parameter ranging from 10 to 50 with 50 being the value used in Lovejoy (*2006*). Severity does not apply until N > 50

An example of the response of the H and Sr to the structural demand on a typical test beam is shown in Figure 2. Data points represent maximum values of H and Sr over individual load – unload cycles. In all test data analyzed the responses of these parameters demonstrated the following traits: 1) Both H and Sr show relatively low values at states of low damage, 2) Both H and Sr increase in magnitude when the diagonal tension cracks form and begin extension which is just prior to or at the transition from loading level 1 to level 2, 3) Both H and Sr values drop in magnitude and then stabilize during shear crack extension and coalescing which corresponds to loading level 2 and 3) Both H and Sr increase rapidly when the girder is loaded to near ultimate capacity.

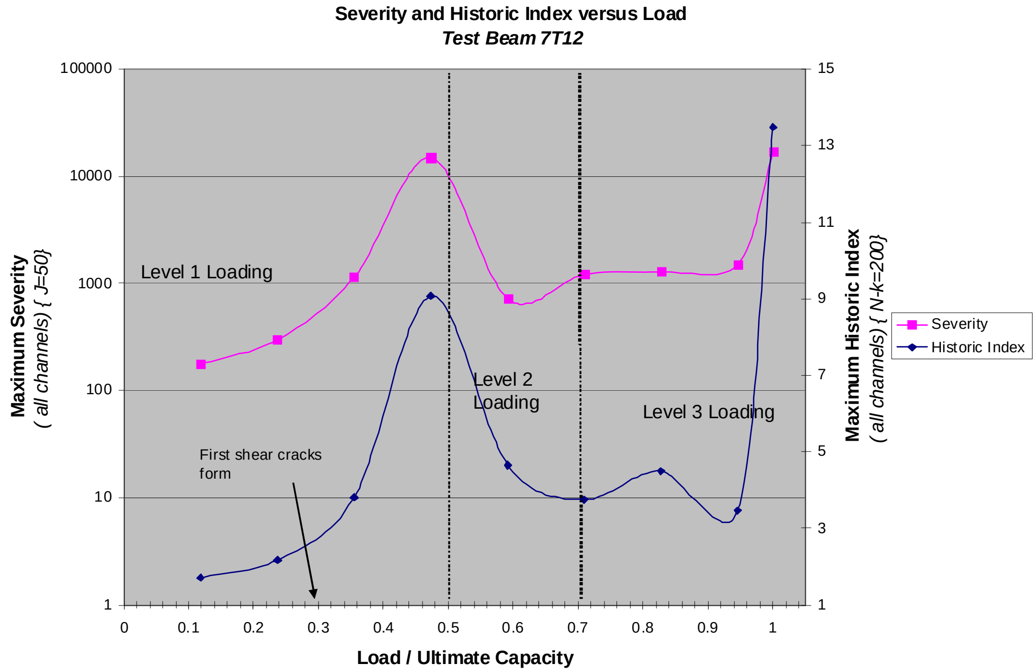

Figure 2: Summary of maximum severity and historic index from a typical full scale RCDG test beam loaded with monotonically increasing magnitude load – unload cycles.

The primary advantage of using both H and Sr in this application is the high sensitivity to the formation of diagonal tension cracks. The historic index has been found to increase by a factor of 3 to 7 and the severity to increase by 1 to 3 orders of magnitude during such events.

Once H and S_r are calculated for each channel over the load protocol they can be correlated by plotting the historic index as the abscissa and the severity as the ordinate. Intensity grading curves can be imposed on these plots that are developed from experimental data pertinent to the structure being tested. A schematic example of such a grading chart is shown in Figure 3. In general the chart is divided into zones which define the structural significance of the AE depending on where they occur on the plot. Sensors which plot towards the upper right of the chart indicate greatest significance and sensors which plot either at the lower left or below a minimum severity are considered to be of less or no significance respectively. Recommend actions can be applied to each zone ranging from no action required, through various levels of follow up NDE or analysis, up to taking the structure out of service. Both the grading zones and recommended actions are application specific.

Figure 3 shows the intensity chart from the test results shown in Figure 2 with hypothetical grading criteria defined. Using the interpretation described above it is clear to see that the two load cycles corresponding to 47% and 100% of ultimate capacity show the most intensity of all load cycles. The former level corresponded to the formation of the diagonal tension cracks and the later to the shear-compression failure mode.

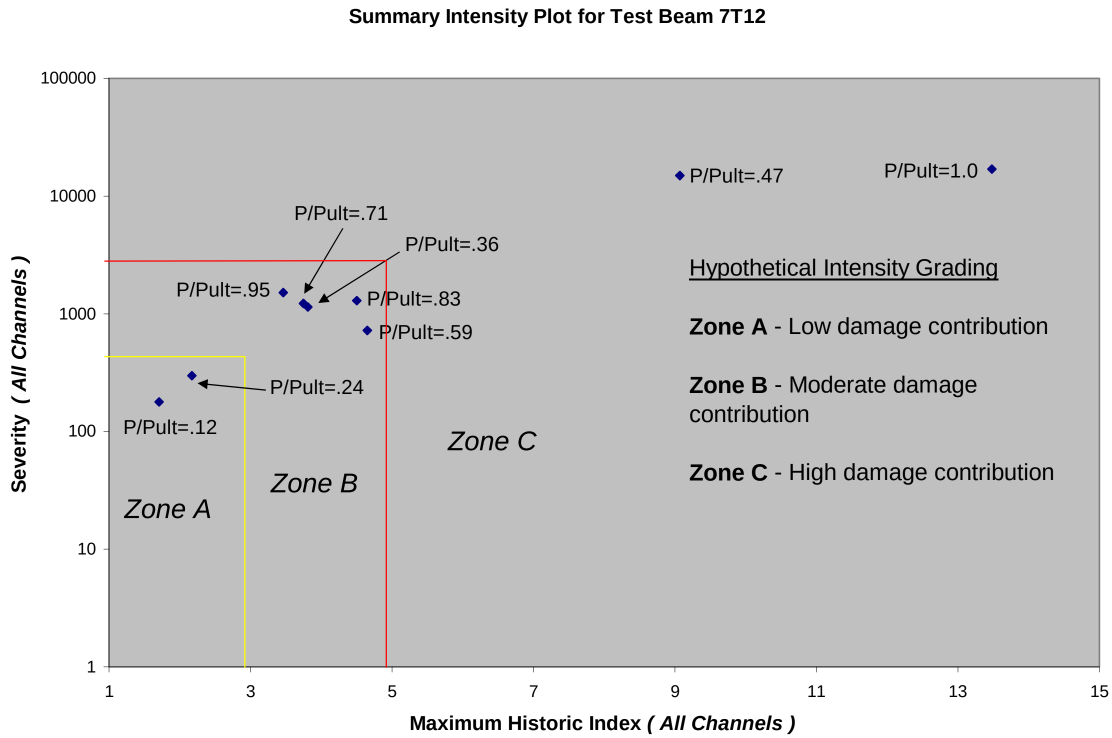

Figure 3: Example Intensity chart with hypothetical grading criteria.

4.2.5 Compare Parametric Data to Load Rating

Preferable rebar strain was recorded so that the actual loads imposed on the test girder(s) can be compared with calculated results from the load rating. The more sophisticated the load rating the better they will agree. It is also acceptable to apply the strain data to the load rating for fine tuning if it is done in the manner prescribed in LRFR. Load rating factors can now be confidently assigned to each test load case.

4.2.6 Developing Intensity Grading Criteria

If the measured loads can be brought within reasonable agreement with load rating results then acceptable operational limits or threshold can be assigned to the severity and historic index responses. Base lines can be established from the test data by assigning Intensity grading criteria for each of the controlled load cases based on the load rating factors.

These grading criteria will only be directly applicable to the maximum level of loading applied during the controlled loads. As previously discussed ambient loads will likely exceed these levels. Based on the ambient data collected, a representative range of maximum severity and historic index can be determined and applied as a threshold level for a structural health monitoring system. Because many of these bridges are very similar in design and construction, test results from one bridge to another can at least be compared to help refine or extend the loading ranges that the intensity grading criteria cover and thus mature over time much like it has in the pressure vessel industry.

4.2.7 *b*-Value Analysis

Another statistical way to look at AE data is the so-called *b*-value analysis. The relationship was established by Gutenberg and Richter in 1949 and has been used to characterize earthquake amplitude distributions as well as to analyze slope-stability in geotechnical and material science applications. The magnitude-frequency distribution relationship is defined as:

$$\log_{10}(N) = a - b \cdot M_L \tag{4.1}$$

Where M_L is the magnitude of an event on the Richter scale, N is the number of events that lie within $M_L \pm \Delta M_L$. a and b are empirical constants, where b describes the slope of the magnitude-frequency diagram. The basic concept is that this *b*-value (the slope) drops significantly when damage becomes more localized. In the field of reinforced concrete, *b*-value analysis has been used by several researchers to monitor structural deterioration (*Colombo 2003 and Shiotani 2000*). Commonly in AE applications, the maximum hit amplitude in dB is multiplied by a factor of 1/20 and replaces the earthquake magnitude M_L. This yields *b*-values in the same range as seen in seismic applications. The *b*-value for each set of AE amplitude-frequency distributions can be estimated with Matlab employing a linear curve-fit over the mean ± one standard deviation as suggested by Rao et al. (*2005*). Standard errors are given as $S_E = b/\sqrt{n}$ where n is the number of samples (consecutive AE hit amplitudes) used. A suggested value for n that was used in (*Schumacher 2008*) is 100. Figure 4 visualizes what *b*-values represent: the slope of the cumulative frequency distribution of a set AE hit amplitudes.

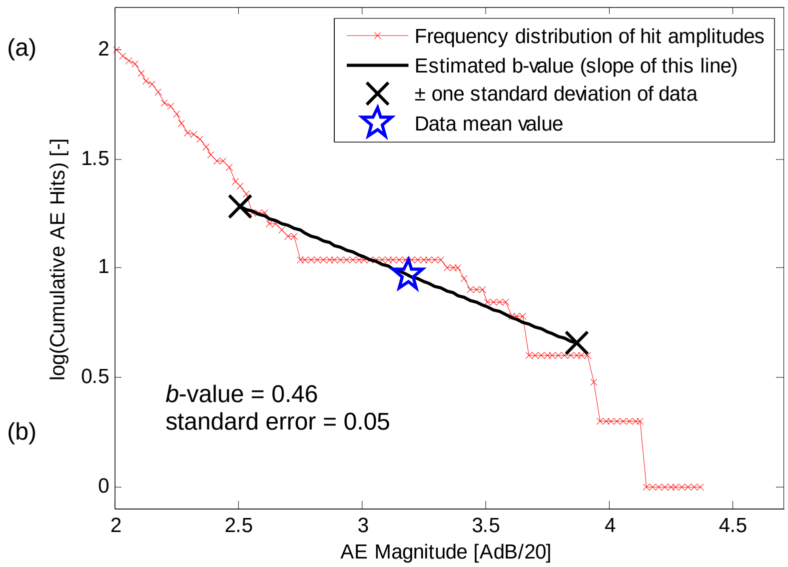

Figure 4: Example of one *b*-value over a set of 100 AE hit amplitudes

Typically, when low amplitude service level loads are applied, the minimum *b*-value from one loading cycle is well above 1.0 suggesting that there is no damage (cracking) being imposed. If a very high load is applied to the structure, possibly one the structure has never experienced before, the *b*-value will drop significantly below 1.0, in some cases even below 0.5.

b-Value analysis appears especially well suited for implementation in a structural health monitoring system since it is computationally inexpensive and, theoretically, only one sensor is needed. Shown in Figure 5 is an example of continuous *b*-value evaluation. In (a) estimated *b*-values computed over a set of 100 AE hit amplitudes are shown and (b) illustrates the total applied force. In this case, a new overload was applied to a full-scale girder specimen as described in (*Shiotani 2000*), i.e. a force that the specimen had not experienced before. As anticipated, the *b*-value dropped well below 1 (full line), even below 0.5 (dashed line) when this new load level was reached, suggesting that localized damage is occurring, e.g. cracks are forming or growing. The AE sensor used was located on top of the web of an inverted T-specimen about 1.10 m (43 in.) away from the edge of the left bearing plate (which corresponds to the column face in case of a real bridge).

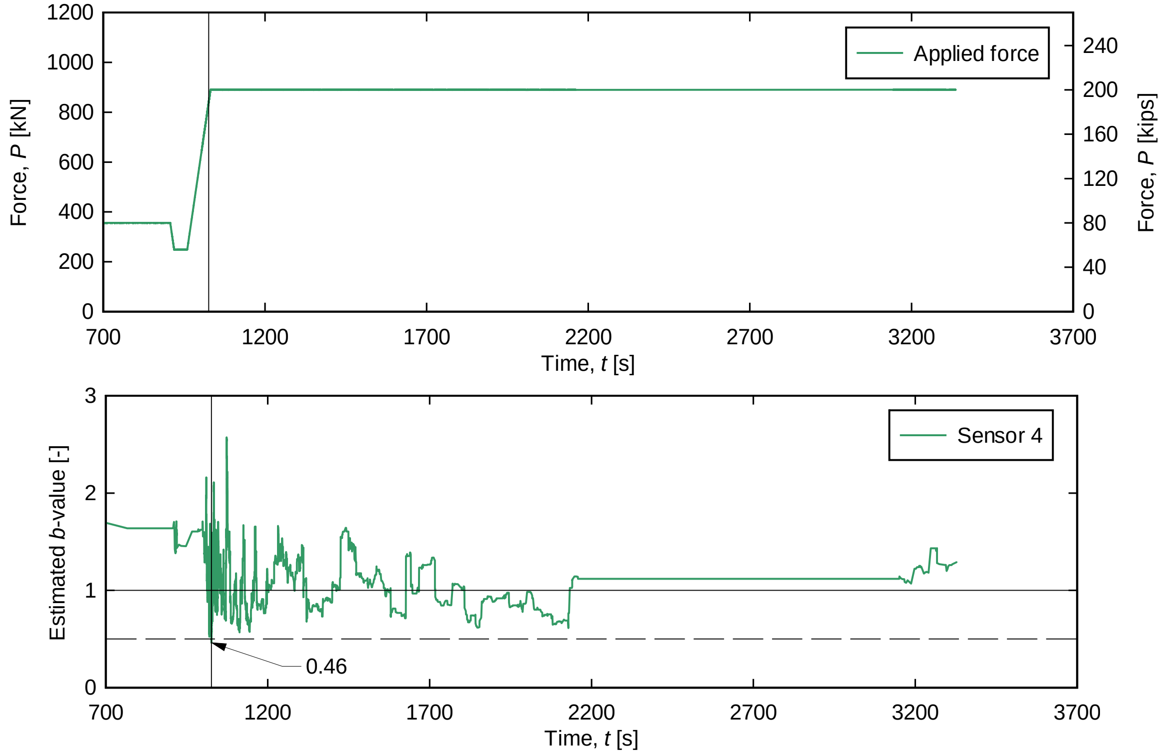

Figure 5: Example of continuous *b*-value analysis for one AE sensor

It was observed that *b*-value time histories differ from sensor to sensor. This is most likely caused by the different sensor locations which affect the AE hit amplitude data and therefore the *b*-values as well, i.e. *b*-values are also a function of the sensor location with respect to the damage source.

It is recommended to apply continuous *b*-value analysis to AE data recorded during structural in-service testing to all sensors and then determine an averaged minimum *b*-value for each loading case as described in section 1.1.9.

One issue that should be kept in mind is that *b*-values are sensitive to high amplitude noise, e.g. found from passing vehicles with studded tires. It is recommended that in addition maximum AE hit amplitudes are averaged over 5 to 10 hit samples and the standard deviation calculated simultaneously. Vehicles with studded tires tend to produce a group of only high amplitude AE events with very few low AE hits which yields a high average AE hit amplitude value with a small standard deviation. This makes them distinguishable from real sources.

4.3 IMPLEMENTATION OF AE TESTING INTO A STRUCTURAL HEALTH MONITORING SYSTEM

There are at least two reasons to proceed with the design and installation of a structural health monitoring system after the above described testing has been completed. The first and primary reason is that the calculated load rating predicts an under capacity structure for carrying the expected loads. This would be especially true if the load rating in question was refined with strain gage data and still predicted a under capacity situation. When this occurs, it is often not feasible to limit loads on the bridge due to political reasons, and repairs or replacement will take time to implement. Assuming the owners do not fear the bridge will pose safety issues, (i.e. potential collapse or excessive deflections, which will likely be the case for the subject bridges), a structural health monitoring system can provide reliable assurance that the structure is performing adequately under the service loads until the repairs or replacement can be implemented. Past practices, under these conditions, have been addressed by sending out a bridge inspector to the structure on an increased frequency. NBIS standards require a maximum of 2 year periods between inspections. In some cases this has been reduced by owners to a period of 7 days. Though sending a real person to a troubled structure who knows what to look for on a very repetitive cycle does provide a level of comfort for the owners, it is often very impractical and of questionable value.

For bridge structures that have moderate to high use and value, a more effective and efficient approach is to install a structural health monitoring system that can continuously measure important structural responses such as stirrup strain, CMOD and AE. Such systems, as are currently being implemented on several ODOT bridges, can provide real time continuous monitoring that can be easily accessed by maintenance engineers from the office. Not only is this far more convenient then constant physical inspection, it is more effective because the monitoring is continuous. Historic trends are also more easily identified form the data which can warn of increasing damage accumulation, rebar stress ranges and CMOD. Thresholds on both the structural and AE data can be set, and the system can notify the owner when they have been exceeded, thus causing a real person to investigate the structure physically, lending the onsite inspection much more meaningful.

By establishing intensity grading criteria, AE could be readily implemented for this purpose. No other means of non-destructive testing currently available is better suited to detect the occurrence of structural damage over a large area in real time then AE when properly applied. Results from this research have shown that the formation and significant extension of diagonal tension cracks in RCDG's can be readily detected and approximately located. Using these AE damage parameters along with rebar strain and / or CMOD to confirm a service load produced the AE as opposed to other sources such as electrical interference or environmental conditions can provide the grounds for a very sensitive and reliable structural health monitoring system on the subject structures.

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