Automated Bridge Load Rating Determination utilizing Strain Response due to Ambient Traffic Trucks

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Abstract: Approximately 11% of bridges in the United States are categorized as structurally deficient and there is a marked need of more accurately evaluating true structural capacity. Structural Health Monitoring (SHM) systems can provide a timely indication of the need for maintenance, repair, rehabilitation and replacement of bridges and can greatly improve the apportionment and management of limited resources. This paper presents an Automated Ambient Traffic (AAT) approach for determining load rating of bridges monitored by the BECAS SHM system under ambient traffic. The AAT approach was developed through a process integration of truck detection, bridge model calibration, and bridge load rating: (1) The quasi-static bridge strain response and the characteristics of associated trucks are collected; (2) Multiple trucks are randomly sampled from a historic Weigh-In-Motion (WIM) database; and (3) For each combination of strain response and truck selection, an Finite Element (FE) model is calibrated and used to calculate a load rating. Sampling strategies were discussed for appropriately quantifying the influence of uncertainties of truck characteristics on the calibration and load rating results. To demonstrate this approach, a sample three-span, five-girder, and two-lane steel girder/concrete deck (I-80) bridge was utilized. A load rating of the I-80 Bridge using the Traditional Known Truck (TKT) approach was performed to provide benchmark results. The results of the calibration and load rating using the AAT approach were derived using three different sampling strategies and compared to those using the TKT approach. The sampling strategy, selecting strain response with a spectrum of higher peak girder strains, associated trucks with a spectrum of higher gross vehicle weights, and two truck events on south and north

lanes respectively for a calibration, resulted in the best calibration and load rating results. It was concluded that the AAT approach using the BECAS SHM System is a reliable method for continuously estimating the load carrying capacity of bridges.

Key Words: Ambient Traffic Trucks; Finite Element Model Calibration; Bridge Load Rating; Structural Health Monitoring.

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1 **Introduction**

2 According to the American Society of Civil Engineering's 2013 Report Card for America's
3 infrastructure [1], the nation's 607,380 bridges, with an average age of 42 years, are in a state of
4 needed improvement and approximately 11% of these bridges are classified as "structurally
5 deficient". And, unfortunately, there appears to be a growing number of bridge collapses
6 reported by mainstream media. There is a marked need of realistically evaluating the true
7 structural capacity of bridges to prevent such catastrophic events. Further, the FHWA requires
8 that bridges carrying a public road should be regularly inspected and evaluated for safety with an
9 interval no more than two years. However, visual inspections are difficult to perform, can put the
10 traveling public in dangerous roadway situations, and require inspectors to work in somewhat
11 precarious situations [2]. At the same time, an estimate by the Federal Highway Administration
12 (FHWA) indicates that twenty billion dollars per year is needed from federal, state, and local
13 governments to meet the goal of eliminating deficient bridges by 2028 [1]. However, the funds
14 currently utilized for this purpose are far less than that needed.

15 Bridge load rating is an important aspect of operating and maintaining safe bridges. The
16 Traditional Known Truck (TKT) approach is typically utilized to provide the most accurate
17 understanding of how a bridge resists external loads and consists of the following features: (1)
18 on-site, controlled tests using known trucks crossing the bridge are conducted to collect actual
19 bridge response; (2) truck characteristics such as gross vehicle weight, axle weights, axle
20 spacings, and transverse and travel positions are measured during those tests; (3) bridge load
21 carrying capacity can be assessed by analyzing the test data and updating/calibrating established
22 FE models. The TKT approach has been well demonstrated in various literature [3,4,5,6] Chajes
23 et al. [3] conducted an experimental test of a slab-on-girder bridge designed with non-composite

1 steel girders and calibrated the parameters of the established bridge model through appropriate
2 analysis of measured data and assumptive simplification. It was found that unintended composite
3 action and support restraint increased the bridge load rating over that predicted using traditional
4 codified approaches. Wipf et al. [4] and Davids et al. [5] found that bridge ratings determined
5 using a calibrated bridge model were, in general, greater than traditional code-based ratings.
6 Sanayei et al. [6] calibrated a baseline bridge model through comparison with nondestructive test
7 data and manual model updating. They also found that the calibrated FE model generally
8 resulted in higher overall load rating factors than those using conventional approaches. Other
9 research studies on improving load rating results through conducting nondestructive bridge
10 testing and establishing and updating FE models were reported by Chajes and Shenton [7], Yost
11 et al. [8], and Schlune et al. [9].

12 It is widely accepted that bridge load carrying capacity can be most accurately estimated
13 using FE models that are optimized to match field measured response data. However, field tests
14 commonly utilized for bridge load rating are sometimes not implemented due to spatial, time and
15 cost restrictions as well as difficulties associated with traffic interruption which may cause
16 significant economic losses and inconvenience to users. Continuous load rating of bridges with a
17 Structural Health Monitoring (SHM) system that relies on ambient traffic would provide an
18 effective solution that results in more accurate assessments while minimizing mobility impacts.
19 SHM systems have been taken as an effective solution to provide a timely indication of the need
20 for maintenance, repair, rehabilitation and replacement of aging bridges and greatly improve the
21 apportionment and management of limited resources. In the recent past a SHM system known as
22 BECAS has been fully developed for which continuous load rating is an important addition.

The objective of this paper is to present an Automated Ambient Traffic (AAT) approach for continuously determining load ratings of bridges monitored by a SHM system under ambient traffic. The first section describes the AAT Approach in terms of the approach framework, SHM system, truck event detection, bridge model calibration and load rating, and selection of sampling strategies. The second section introduces the configuration, instrumentation, and Finite Element (FE) modeling of a sample three-span, five-girder, two-lane steel girder/concrete deck bridge on Interstate-80 (I-80). The third section presents the load rating of the I-80 Bridge using the Traditional Known Truck (TKT) approach to provide a basis for comparison with the AAT approach. The fourth section presents the load rating results of I-80 Bridge using the AAT approach compared to those obtained using the TKT Approach. The last section gives the summary and conclusions of this study.

Automated Ambient Traffic Approach

AAT Approach Framework

The AAT approach for continuously assessing the bridge load carrying capacity was developed based on a strain-based SHM system (introduced later and known as BECAS) which remotely monitors the response of a bridge under ambient traffic. The AAT approach was realized through a process integration of truck detection, bridge model calibration, and bridge load rating. The approach framework includes an automated step-by-step procedure illustrated in the flowchart shown in Figure 1: (1) The quasi-static bridge response and the characteristics of the associated trucks from single truck events are collected by the BECAS SHM system using a developed truck detection methodology; (2) Based on the available truck information for each single truck event, multiple trucks are sampled from a historic Weigh-In-Motion (WIM) database by using

appropriate selection criteria.; (3) For each combination of strain response and associated truck loads, an FE model is calibrated; and (4) The updated bridge model is utilized to perform bridge load rating. Note that multiple load cases with different truck travel positions from each truck event are utilized to calibrate the bridge model. Strain response from either one or more truck events can be utilized for bridge model calibration since additional truck events simply increase the amount of loading cases for model calibration. The operations shown in the flowchart (see Figure 1) are achieved using a custom-developed software applications known as BECAS Processing Engine and BECAS Load Rating that automates the entire process of truck detection, bridge model calibration, and bridge load rating.

SHM System

The strain-based SHM system for this work has basic hardware components including sensor network, data logger, desktop computer, network switch, router, and cellular modem, and office server, schematically illustrated in Figure 2. The sensor network consists of electrical resistance strain gages deployed on a bridge to collect its strain response. The instrumentation details of the demonstration bridge will be described later. The data logger consists of a CR9000x module with programming developed using the CRBasic language from Campbell Scientific. The data collected from the data logger are transferred to the desktop computer through the network switch. The data are stored temporarily on the desktop computer before being sent to the Linksys router via the network switch. The data are then dispatched to the cellular modem and then transmitted to the office server via 4G cellular communication. The software controlling the data transfer involves a standard File Transfer Protocol (FTP). The data files are collected every minute with an appropriate sampling rate.

The entire SHM system is known as BECAS and consists of multiple, automated software applications (BECAS Merge, BECAS Processing Engine, BECAS Damage Detection, BECAS Load Rating, etc.). The BECAS system was developed by the Iowa State University Bridge Engineering Center and is offered commercially by Advanced Structural, LLC. The system generates in excess of 10 GB of data daily. Although the raw data sets are permanently stored, intermediate analysis results are not stored with only final results being retained using a custom developed file accounting system.

Single Truck Event Detection

Utilizing the SHM system, single truck events (i.e., instances where only one truck travels across a bridge) and its associated travel lane can be accurately detected using strategically placed strain gages. Concurrent events, where more than one truck simultaneously on the bridge see Figures 3(a) and 3(b), are identified and subsequently abandoned. Single truck events consisting of a single five-axle truck on the bridge are extracted from the SHM system records and are used as the base data for bridge model calibration as shown in the flowchart in Figure 1. The procedure of detecting single truck events is not introduced herein due to length limitations but is described in detail by Phares et al. [10], Greimann et al. [11], and Lu [12].

Truck parameters used for bridge model calibration consist of axle spacings, travel position, gross vehicle weight, axle weights, and transverse position. Axle spacings and longitudinal travel position can be detected using the strains recorded by gages strategically placed on the deck bottom as conceptually illustrated in Figure 3(c). For two lane bridges, each sensor line consists of four deck gages (designated DL11, DL12, DL13, DL14 at sensor line 1 and DL21, DL22, DL23, and DL24 at sensor line 2) as illustrated in Figure 3(d). Axle spacings

and travel position can be detected using the strain rate response recorded by these strain gages as introduced as follows.

Two longitudinally aligned sensors, DL11 and DL21 are utilized for the illustration. Using proprietary algorithms peaks in the data can represent the five axles of a detected truck, as shown in Figure 4(a). The truck speed (V) can be determined by:

$$V = d_{12} / t_{12} \quad (1)$$

where, d_{12} = the distance between the two deck bottom sensor lines, as shown in Figure 3(c); t_{12} = the time duration that it takes for the truck to travel from sensor line 1 to sensor line 2. With the calculated speed, the four axle spacings of the truck (i.e., A-SPC #1, A-SPC #2, A-SPC #3, and A-SPC #4 as shown in Figure 4(b)) can then be determined as the product of the speed and timestamp differences. Converting the strains from the time-domain to the more user-friendly truck position-domain is accomplished using the truck speed, timestamps of deck peak strains, and locations of sensor lines. The quasi-static strain response for each detected five-axle truck event can then be extracted through a zeroing and filtering process which eliminates strain components due to temperature, dynamic effects, and high frequency noise, as discussed by Phares et al. [10], Doornink [13] and Lu [12].

Bridge Model Calibration and Load Rating

The Iowa State University Bridge Engineering Center (BEC) routinely performs bridge load rating using a set of commercially available software applications [14], including WinGen, which is used for bridge model generation and load test simulation, and WinSac, which is used for structural analysis, model calibration, and load rating computation. For calibration, WinSac includes algorithms for making direct numeric comparisons between measured and computed strains. The bridge parameters are calibrated through a process of minimizing the difference

between the measured and computed strains using a least squares approach. Four different statistical values, absolute error (AE), percent error (PE), scale error (SE) and correlation coefficient (CC), are used to describe the model's ability to represent the actual structure, and can be determined by:

$$AE = \sum |\varepsilon_R - \varepsilon_C| \quad (2)$$

$$PE = \frac{\sum (\varepsilon_R - \varepsilon_C)^2}{\sum \varepsilon_R^2} \quad (3)$$

$$SE = \frac{\sum \max |\varepsilon_R - \varepsilon_C|_{gage}}{\sum \max |\varepsilon_R|_{gage}} \quad (4)$$

$$CC = \frac{\sum (\varepsilon_R - \mu_{\varepsilon_R})(\varepsilon_C - \mu_{\varepsilon_C})}{\sum \sqrt{(\varepsilon_R - \mu_{\varepsilon_R})^2 (\varepsilon_C - \mu_{\varepsilon_C})^2}} \quad (5)$$

where, ε_R = measured strain; ε_C = strain calculated using the FE model; $\max |\varepsilon_R - \varepsilon_C|_{gage}$ = maximum absolute strain differences between measured and calculated strains in each gage; $\max |\varepsilon_R|_{gage}$ = maximum absolute strain in each gage; μ_{ε_R} = average recorded strain in each gage; μ_{ε_C} = average calculated strain in each gage.

The calibrated bridge FE model (calibrated bridge parameters) is then used to perform a load rating using WinSac. The load rating factor (RF) is calculated using the Load Factor Rating (LFR) Method per AASHTO Standard Specifications [15]:

$$RF = \frac{C - A_1 D}{A_2 L(1 + I)} \quad (6)$$

where, C = capacity of the member; D = dead load effect on the member; L = live load effect on the member; A_1 = factor for dead loads, equals 1.3; A_2 = factor for live loads, equals 2.17 for Inventory level and 1.3 for Operating level; I = impact factor for live load effect.

The live loads applied to the load rating bridge model are the AASHTO HS-20 trucks. Various loading cases with different transverse positions should be taken into account to account for one or more trucks traveling across the bridge in critical locations. These transverse positions must extend for one side of the bridge with one outer wheel line of the outer truck located at 0.6 m (2 ft) away from the bridge parapet [15]. The dead loads consist of the self-weights of the superstructure components which can include girders, stringers, floor beams or diaphragms, deck, and parapets. The shear and moment capacity of all bridge components are calculated based on appropriate portions of the AASHTO Standard Specifications [15]. Load envelopes are calculated such that the rating factors are computed for all the FE elements. The lowest rating factor for all of the elements are taken as the rating factor of the bridge. Note that for demonstration and simplification purposes in this study, the girders of the demonstration bridge, as the major bridge members of resisting dead and live loads, are the only components utilized to calculate bridge load rating at the Inventory level.

Sampling Strategy Determination

For each calibration and load rating utilizing a single truck event, the loads due to a truck and associated strain response are used in the calibration process. However, from the SHM system, the accurately measured truck characteristics include the truck speed, axle spacings, travel lane, and longitudinal travel position, while the gross vehicle weight, axle weights, and transverse position are not measured since no specific measurements have been implemented. As a result, the truck details for each truck event are only partially known and cannot be utilized for bridge

1 model calibration without making estimates. The AAT approach was developed to address and
2 mitigate the influence of the uncertainties associated with these unknown truck parameters on the
3 calibration and load rating results. Using the AAT approach, trucks are sampled from a Weigh-
4 In-Motion (WIM) database utilizing certain sampling criteria established based on the known
5 truck characteristics. For each single truck event, each calibration and load rating is performed
6 utilizing a truck sampled from the WIM database. By repeating this process many calibrations
7 and load ratings can be completed which result in statistical distributions of calibrated bridge
8 parameters and load ratings which reflect the influence of the uncertainties on the results of
9 calibration and load rating. Since the manner in which the trucks are selected from the WIM
10 database has an impact on the results of calibration and load rating, a reasonable sampling
11 strategy should be determined so as to properly select trucks from the WIM Database. As an
12 illustration, several possible sampling strategies are presented in the following section.

13 A WIM database was created using the data collected from the Dallas and Jasper County
14 weigh stations located on I80 in Iowa. Figure 5 shows frequency histograms (based upon the
15 WIM database) for the four axle spacings that define a five axle truck's basic geometry. Figure 5
16 indicates that the axle spacings can vary significantly for different five axle trucks; the variability
17 is most pronounced for spacings #1 and #3. Five-axle trucks with certain configurations are
18 further categorized as shown in Table 1: (1) subgroup #1 - axle spacings #1, #2, #3, and #4 are 3-
19 6.7 m, 1.2-1.5 m, 7.6-12.2 m, and 1.2-1.5 m (10–22 ft, 4–5 ft, 25–40 ft, and 4–5 ft), respectively;
20 (2) subgroup #2 - axle spacings #1, #2, #3, and #4 are 4.9-5.8 m, 1.2-1.5 m, 9.1-10.7, and 1.2-1.5
21 m (16–19 ft, 4–5 ft, 30–35 ft, and 4–5 ft), respectively. The frequency histograms of gross
22 vehicle weight for the five axle trucks in subgroups #1 and #2 are shown in Figures 6(a) and 6(b),
23 respectively. As indicated in Figure 6, the five axle trucks in subgroups #1 and #2 have a similar

1 pattern of truck weight distribution although the trucks are sorted utilizing different axle spacing
2 ranges for the four axles. Accordingly, five axle trucks in subgroup #2 with smaller ranges of the
3 axle spacings are desirable because subgroup #2 minimizes the uncertainties associated with
4 some truck parameters.

5 To correlate truck weight with the bridge response, frequency histograms of the peak
6 girder strains from gage D2_BF (see Figure 9(c)) and gage I2_BF (see Figure 9(c)) for 2,310
7 south-lane five-axle truck events detected by the SHM system were compared and are shown in
8 Figures 7(a) and 7(b), respectively. Similarly, the same was completed for the peak girder strains
9 from gage D4_BF (see Figure 9(c)) and gage I4_BF (see Figure 9(c)) for 2,247 north-lane five-
10 axle truck events detected by the SHM system which are shown in Figures 7(c) and (d),
11 respectively. Note that the selected truck events have axle spacings as defined by the subgroup
12 #2 criteria. Based on comparisons between Figure 6(b) and Figure 7, it was found that the
13 distributions of detected peak girder strains have a similar pattern to that for the truck weight for
14 the five axle trucks in subgroup #2, indicating a strong correlation between the peak girder
15 strains and truck weight.

16 With the similarities of the distributions of the peak girder strains and truck weight in
17 mind, three sampling strategies were developed to address uncertainties of the gross vehicle
18 weight and individual axle weights as tabulated in Table 2. For sampling strategy A, sampling
19 criteria were established which included the truck weight spectrum of subgroup #2 (89-356 kN
20 [20-80 kips]) and the girder strain spectrums (South lane: 30-80 and 25-75 microstrains for gages
21 D2_BF and I2_BF respectively; North lane: 30-80 and 27-77 microstrains for gages D4_BF and
22 I4_BF respectively) as listed in Table 2 and illustrated in Figure 6(b) and Figure 7. Either a south
23 or north lane event can be utilized for a bridge model calibration.

The concept of sampling strategy B is that heavier trucks are correlated with higher bridge responses which can then be utilized to further reduce uncertainties by taking advantage of the truck weight-response relationship and similarities in weight distribution between axles. As shown in Table 3, the heavier trucks of subgroup #2 (truck weight ranging from 334 to 356 kN [75 to 80 kips]) have less variations in gross vehicle weight, axle weights, and axle weight ratios between the five axles. In particular, the mean value of axle weight #1 equals 0.67, the mean values of axle weights #2 to #5 approximate to 1.0, and the standard deviations of the axle weight ratios between different axles are less than 0.06, as shown in Table 3. Accordingly, a spectrum of larger truck weight (75 to 80 kips) and a spectrum of larger girder strains (South lane: 75-80 and 70-75 microstrains for gages D2_BF and I2_BF, respectively; North lane: 75-80 and 72-77 microstrains for gages D4_BF and I4_BF, respectively) are utilized to set the sampling criteria for sampling strategy B as listed in Table 2 and illustrated in Figure 6(b) and Figure 7. Either a south or north lane event can be utilized for a bridge model calibration.

The concept of sampling strategy C is that a bridge model calibration utilizing truck events in both south and north lanes involves higher strains in most of the girders, which further reduces uncertainties. Accordingly, sampling strategy C is an improved version of sampling strategy B through simultaneously (i.e., superposition) utilizing both south and north lane events during model calibration. To illustrate the steps followed for sampling both strain response and trucks, the sampling criteria has been incorporated in the flowchart for bridge model calibration and load rating and is shown in Figure 8. Note that, for sampling strategy C, double the amount of loading cases-due to two single truck events-are utilized for each model calibration.

It should be reiterated that no matter which sampling strategy is used, for each calibration the strain response of a truck event is selected based on the sampling criteria which includes the

spectrums of peak girder strains from the SHM system database, and a truck is selected from the WIM database based on the sampling criteria which includes the spectrum of gross vehicle weight and the measured axle spacings related to the selected truck event. Further, due to the precision of the truck detection process, the ranges of the detected axle spacings minus/plus 0.1 m (0.4 ft) are used to slightly relax the selection criteria and to ensure a sufficient amount of trucks can be found in the WIM database. To take into account uncertainties of truck transverse position during model calibration, the transverse position of each selected truck is randomly sampled from a uniform distribution with a range extending the lane center minus 0.5 m (1.5 ft) to the lane center plus 0.5 m (1.5 ft).

Details and FE Modeling of a Demonstration Bridge

I-80 Bridge

The previously mentioned I-80 Bridge shown in Figure (9a), crossing Sugar Creek and carrying eastbound traffic in central Iowa, was used to demonstrate the AAT bridge load rating determination process. The I-80 Bridge has three spans with a 15-degree skew, a total length of 62 m (204 ft) and a roadway width of 12 m (40 ft). The bridge supports two eastbound traffic lanes with a posted speed limit of 113 km/h (70 mph). The nominal 191-mm (7.5-in) thick cast-in-place reinforced concrete deck is supported by five steel girders, two abutment diaphragms, two pier diaphragms, and seven intermediate diaphragms. The girders are continuous over the three spans with 18.6-m (61-ft) end spans and a 78-ft center span. Figures 9(b), 9(c), and 9(d) illustrate the bridge plan, typical cross section and gages installed on the bridge. The dimensions of the girders and diaphragms are shown in Table 4. Within the negative moment region, the exterior and interior girder flanges have cover plates with dimensions of 356

mm×14 mm×5.6 m (14 in.×9/16 in.×18 ft- 6 in.) and 356 mm×16 mm×5.6 m (14 in.×5/8 in.×18 ft- 6 in., respectively. And the girders are spliced at locations 17.6 ft away from both piers. The spacing between the girders is 2.9 m (9 ft-6 in.). An idealized roller support is at both abutments and at the east pier and an idealized pinned support is at the west pier. Both abutments are stub concrete and the two piers are open two-column, concrete cantilevers.

The instrumentation of the I-80 Bridge consists of 69 strain gages installed on the steel girders and 8 strain gages installed on the concrete deck. As shown in Figure 9(b), the red dots represent 33 strain gages installed on the girder bottom flange and the yellow dots represent 18 strain gages mounted on the top and bottom flanges of the girders. The bridge cross-sections with instrumentation installed are labeled from A to O and the girders are labeled from 1 to 5 as shown in Figure 9(c). The gages are designated using a unique combination of cross-section, girder, and location. For example, D2_BF represents the gage at Section D, Girder 2, on girder bottom flange; D2_TF represents the gage at section D, girder 2, on the girder top flange. The girder strain gages were installed on the bottom of the top flange and top of the bottom flange as shown in Figure 9(d).

FE Modeling

The FE model of the previously described I-80 Bridge was established as shown in Figure 10(a). The girders and diaphragms are modeled using a two-node beam element, which has three translational and three rotational degrees of freedom at each node. The deck is modeled using a shell element which has three translational and three rotational degrees of freedom at each node and incorporates both bending behavior (kirchhoff plate element) and tension membrane behavior. Restraint of the girders at the abutment is modeled using rotational spring elements. As shown in Figure 10(b), the beam elements for the girders share common nodes with

the shell elements for the deck at the composite centroid location. The section properties of each beam element are determined based on the composite section consisting of the transformed deck and steel beam. The diaphragms only share common nodes with the girder elements at the connection location (not with the deck). Linear elastic material models are used for the concrete and steel, respectively. This is due to the fact that the bridges can commonly sustain more than twenty AASHTO HS20 trucks at failure as indicated by Gheitasi and Harris [16] and Deng et al. [17], due to the structural redundancy and system-level effects. For the research in this study, the bridge is only exposed to approximately two HS20 truck loads and the behavior of the bridge materials are in the elastic range.

To calibrate the established FE model, a set of bridge parameters significantly correlated to the bridge response are selected for the model optimization process. Typical bridge parameters consist of the moments of inertia of the girders and diaphragms, elastic modulus of the deck, and spring constants at the supports. For the I-80 bridge, the seven bridge parameters to be calibrated, as illustrated in Figure 10(c), are the moment inertia of the exterior girder cross-sections in the positive moment region (I_{GE-P}), moment inertia of the exterior girder cross-sections in the negative moment region near piers (I_{GE-N}), moment inertia of the interior girder cross-sections in the positive moment region (I_{GI-P}), moment inertia of the interior girder cross-sections in the negative moment region near the piers (I_{GI-N}), modulus of elasticity of the deck (E_D), spring constant for exterior girders (R_E), spring constant for interior girders (R_I). Theoretical non-composite and composite values were utilized to establish lower and upper bounds for each of these parameters and are tabulated in Table 5. The initial values of the elastic modulus of the deck were set to be the plan-specified values, and the upper and lower limits were set as 25% higher and 25% lower than the plan values. The initial values for the girders were set as plan

values considering full composite action with the deck and railings. The upper and lower limits of the moments of inertia of the girders were set as 25% higher than plan values considering full composite action and 25% lower than plan values considering non-composite action, respectively. The spring constants for both interior and exterior girders at the abutments were set to have an initial value of 113 kN-m/rad (1000 kip-in/rad), a lower limit of 0, and an upper limit of 1017 kN-m/rad (9000 kip-in/rad). It should be noted that the end restraint needs to be critically evaluated because the bridge load rating will be over-estimated if too much unintentional support restraint is provided by the abutments [3]. As described previously, the parameters are calibrated through minimization of the difference between the calculated and measured strain values.

It should be noted that the calibrated moments of inertia are significantly correlated with the centroid positions of the girder cross-sections. To realistically evaluate the centroid positions of different types of girder cross-sections, the strain responses in the top and bottom gages were utilized to derive the neutral axis location. Take, for example, the cross-section of the interior girder in the positive moment region. As shown in Figure 11, the effective width of the deck is equal to the girder spacing, 2.9 m (114 in). The transformed deck section was derived using the stiffness ratio between the girder steel and deck concrete. Based on the composite girder section properties, the centroid position of the girder cross-section was calculated to be 759 mm (29.89 in.) away from the bottom gage location. The gages, D2_BF and D2_TF, mounted on the bottom and top flanges of the steel girder, are shown in Figure 12(a), and the distance between D2_BF and D2_TF is 862 mm (33.96 in.). The strain responses in D2_BF and D2_TF are shown in Figure 12(a) (positive in tension). Based on mechanics of materials, the strain profile is illustrated in Figure 11 and the neutral axis location can be derived by:

$$\bar{y} = \frac{\varepsilon_b}{\varepsilon_b + \varepsilon_t} d \quad (1)$$

where, $\bar{y}$ = neutral axis location relative to the bottom gage location; ε_b = strain in the bottom gage; ε_t = strain in the top gage; d = distance between the two gages.

Since small strain readings are not reliable, a minimum required strain difference between the top and bottom gages (i.e., the minimum delta strain) was used to pick strain responses utilized for calculation of the neutral axis location. For the gage pair D2_BF and D2_TF, the minimum delta strain was set to 15 micro-strains. The calculated neutral axis location for this gage pair is plotted in Figure 12(b) and has the mean of 717 mm (28.22 in.) and a standard deviation of 8 mm (0.311 in.). Likewise, the neutral axis locations of the exterior girders in the positive moment region, exterior girders in the negative moment region, and interior girders in the negative moment region were also calculated as tabulated in Table 6. The calculated neutral axis locations are slightly different from these determined using the traditional hand calculations based on transformed section properties. Accordingly, the calculated means of neutral axis locations were imported into the FE model as the centroids of the girder cross-sections for the beam elements.

Load Rating using Traditional Known Truck Approach

Bridge model calibration and load rating using the TKT approach was performed to provide information for validating the adequacy of the AAT approach. For the TKT approach, the strain response collected from field tests using trucks with known parameters were utilized for bridge model calibration. On-site controlled tests using trucks at crawl speed crossing the bridge were

1 conducted with a three-axle dump truck employed as the control truck. The axle and wheel
2 configurations of the three-axle dump truck are illustrated in Figure 13 and the axle spacings,
3 axle weights and total weight of the truck are summarized in Table 7. During the tests, the south
4 lane was closed to other traffic and the controlled trucks crossed the bridge in different
5 transverse positions at crawl speed. Only test data which were not affected by the presence of
6 ambient traffic were utilized for model calibration. The truck speeds and transverse positions for
7 the five tests utilized for calibration are summarized in Table 8. It should be noted that the strain
8 data from the crawl speed tests were filtered to fully eliminate any dynamic and noise effects.

9 Five bridge model calibrations and load ratings were performed utilizing the data from
10 the five tests. The final optimized parameters for each bridge model calibration are shown in
11 Table 9. Among the five calibrations, the variations within the seven optimized parameters are
12 small and indicate the robustness of the calibration process. The statistical values illustrating the
13 accuracy of the calibrated models are also shown in Table 9. Small errors (including percent
14 error and scale error) and a correlation coefficient larger than 0.97, were generally found. Note
15 that only the load carrying capacities of girders were evaluated because the strain responses in
16 these components were utilized for bridge model calibration. The means and standard deviations
17 of the bridge parameters, statistical values, and minimum rating factors are also shown in Table 9.
18 Due to the small standard deviations shown in Table 9, it is safe to conclude that the bridge
19 parameters and rating factors can be well determined using the TKT approach.

21 **Load Rating using Automated Ambient Traffic Approach**

22 The details of sampling of strain responses and WIM trucks for the bridge model calibration are
23 shown in Table 10. For each sampling strategy, 100 runs of calibration and load rating were

performed using the automated software BECAS Load Rating. For comparison purposes, one demonstration using a south lane event is presented for sampling strategy A, two demonstrations using a south and north lane event respectively are presented for sampling strategy B, and one demonstration using both the south and north lane events is presented for sampling strategy C as shown in Table 10. The final selected truck events and associated truck characteristics are summarized in Table 11. Calibration and load rating results obtained using the AAT approach through different sampling strategies are summarized and compared with those determined using the TKT approach shown in Table 12. As before, only the load carrying capacities of girders were considered. The means and standard deviations of converged parameter values, statistical values, and minimum rating factors were calculated for 100 runs of calibrations and load ratings using each sampling strategy (see Table 12). As indicated in Table 12, sampling strategy C is the best of three strategies while sampling strategy B is better than sampling strategy A. When using sampling strategy C, the means and standard deviations of errors are relatively smaller and the correlations are greater than 0.95. The means and standard deviations of the statistical values and minimum rating factors are comparable to those obtained using the TKT approach. The mean values of bridge parameters using sampling strategy C are in good agreement with the results using the TKT approach, although some differences in optimized bridge parameter values are found especially for the exterior girder moments of inertia as shown in Table 12. Smaller differences in bridge parameter values, statistical values, and rating factors were obtained using sampling strategies B and C compared to those using sampling strategy A.

Figure 14 shows that a wide spread of the minimum rating factor (ranging from 1.2 to 1.9) was found using strategy A, a smaller spread of the minimum rating factor ranging from 1.3 to 1.7 was obtained using sampling strategy B, and a smallest spread of the minimum rating factor

1 ranging from 1.5 to 1.7 was obtained using sampling strategy C which is most close to the result
2 obtained using the TKT approach. For example, the mean, standard deviation, and range of the
3 minimum rating factor using sampling strategy C are 1.60, 0.06, and 1.5-1.7, respectively. The
4 Iowa Department of Transportation currently performs bridge load rating using LARS Bridge
5 [18], which uses a single line girder method and is not capable of modeling lateral load
6 distribution effects. A minimum rating factor of 1.28 was obtained using LARS Bridge [18]
7 which is much lower than the mean of 1.60 obtained using the AAT approach with sampling
8 strategy C and the mean of 1.65 obtained using the TKT approach.

9 One of the 100 calibrations using sampling strategy C that had a percent error of 8.4%,
10 scale error of 5.3% and correlation coefficient of 0.9596 was taken as an example to illustrate the
11 calibrated results. The trucks randomly selected for this calibration from the WIM database are
12 shown in Table 13. The strain time histories calculated using the calibrated FE model are in good
13 agreement with test data as shown in Figure 15 and Figure 16 for sections D for the south and
14 north lane events respectively.

16 **Summary and Conclusions**

17 An Automated Ambient Traffic (AAT) approach was introduced for continuously determining
18 bridge load rating utilizing ambient traffic trucks. The AAT Approach was developed by process
19 integration of various algorithms that independently complete the truck detection, bridge model
20 calibration, and bridge load rating. These automated algorithms are contained within the BECAS
21 software suite being commercialized by Advanced Structural, LLC Accounting for uncertainties
22 in estimating gross vehicle weight, axle weights, and transverse position, different sampling

1 strategies were utilized to select bridge response and truck characteristics for bridge model
2 calibration.

3 To demonstrate this approach, a typical three-span, five-girder, and two-lane steel
4 girder/concrete deck bridge was utilized. Load rating of the I-80 Bridge using the Traditional
5 Known Truck (TKT) approach was performed to provide information for validating the
6 adequacy of the AAT approach. The following conclusions were made from the load rating using
7 the TKT approach:

- 8 • Small errors including percent error and scale error and good correlations were obtained.
- 9 • The bridge parameter values and rating factors were accurately and consistently
10 determined from multiple different measured datasets.

11 Calibration and load rating results using the AAT approach with different sampling
12 strategies were compared with those obtained using the TKT Approach. The means and standard
13 deviations of the converged parameter values, statistical values, and minimum rating factors
14 were calculated for multiple calibrations and load ratings using each sampling strategy. The
15 following conclusions were made:

- 16 • When using sampling strategy C, the means and standard deviations of the statistical
17 values and minimum rating factors were comparable to those obtained using the TKT
18 approach.
- 19 • The mean values of bridge parameters using sampling strategy C were in good agreement
20 with the results using the TKT approach. The smallest spread of the minimum rating
21 factor were obtained using sampling strategy C which is most close to the result obtained
22 using the TKT approach.

- A smaller rating factor obtained by the Iowa Department of Transportation using the single line girder method, was much lower than that obtained using the AAT approach with sampling strategy C and that obtained using the TKT approach.
- The AAT approach is a reliable method for continuously estimating the load carrying capacity of bridges and strategy C was recommended for the AAT approach.

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- 17

Figure Caption List

- 1
- 2 Figure 1 Flowchart of Automated Step-by-Step Procedure of Bridge Load Rating using AAT
- 3 Approach
- 4 Figure 2 Schematic of SHM System Components
- 5 Figure 3 Trucks Traveling on a Bridge: (a) Side-by-Side Event; (b) One-after-Another Event; (c)
- 6 A Truck Traveling on the Bridge
- 7 Figure 4 Peak Strains Induced by a Five-Axle Truck: (a) Peak Strain Rates in Sensors DL11 and
- 8 DL21; (b) Five-axle Truck corresponding to Peak Strain Rates
- 9 Figure 5 Frequency Histograms of Axle Spacings (1 ft = 0.305 m): (a) Axle Spacing #1
- 10 (b) Axle Spacing #2; (c) Axle Spacing #3; (d) Axle Spacing #4
- 11 Figure 6 Frequency Histograms of Gross Vehicle Weight (1 kip = 4.448 kN): (a) Five-Axle
- 12 Trucks of the Subgroup No. 1; Five-Axle Trucks of the Subgroup No. 2
- 13 Figure 7 Frequency Histograms of Maximum Girder Strains in Different Gages for Trucks of
- 14 Subgroup No. 2: (a) Maximum Strain in Gage D2_BF – South Lane Events; (b) Maximum Strain
- 15 in Gage I2_BF – South Lane Events; (c) Maximum Strain in Gage D4_BF – North Lane Events;
- 16 (d) Maximum Strain in Gage I4_BF – North Lane Events
- 17 Figure 8 Flowchart of Automated Ambient Traffic Approach using Sampling Strategy C
- 18 Figure 9 View, Plan, Cross-section and Strain Gages of I80-Bridge: (a) View; (b) Bridge Plan;
- 19 (c) Labels of Cross-Sections, Girders, and Gages; (d) Bridge cross-section
- 20 Figure 10 Details of FE Model of I-80 Bridge: (a) Bridge Model and Gages; (b) Modeling of
- 21 Girder and Deck; (c) Optimized Bridge Parameters
- 22 Figure 11 Gages on Interior Girder Cross-section – Positive Moment Region
- 23 Figure 12 Neutral Axis Determination based on Gage Pair D4_BF and D4_TF: (a) Strain
- 24 response of Gage Pair D4_BF and D4_TF; (b) Neutral Axis Location
- 25 Figure 13 Axle and Wheel Configurations of Dump Trucks (ft, 1 ft = 0.305 m)
- 26 Figure 14 Frequency Histograms of Minimum Rating Factors using Different Sampling
- 27 Strategies
- 28 Figure 15 Comparisons of Strain Time Histories between Collected Data and FE Results using
- 29 AAT Approach – South Lane Event & Section D (1 ft = 0.305 m)
- 30 Figure 16 Comparisons of Strain Time Histories between Collected Data and FE Results using
- 31 AAT Approach – North Lane Event & Section D (1 ft = 0.305 m): (a) Gages D1_BF and
- 32 D2_BF; (b) Gages D3_BF, D4_BF, and D5_BF

Table 1 Subgroups of Five-Axle Trucks with Different Axle Spacing Ranges

Subgroup	Axle Spacing #1 (ft)	Axle Spacing #2 (ft)	Axle Spacing #3 (ft)	Axle Spacing #4 (ft)
#1	10–22	4–5	25–40	4–5
#2	16–19	4–5	30–35	4–5

Note: 1 ft = 0.3 m

Table 2 Sampling Strategies with Different Strain and Truck Weight Spectrums

Sampling Strategy	South Lane Events			North Lane Events		
	WIM Truck Weight (kip)	D2_BF (10 ⁻⁶)	I2_BF (10 ⁻⁶)	WIM Truck Weight (kip)	D4_BF (10 ⁻⁶)	I4_BF (10 ⁻⁶)
A	20–80	30–80	25–75	N/A	N/A	N/A
	N/A	N/A	N/A	20–80	30–80	27–77
B	75–80	75–80	70–75	N/A	N/A	N/A
	N/A	N/A	N/A	75–80	75–80	72–77
C	75–80	75–80	70–75	75–80	75–80	72–77

Note: 1 kip = 4.448 kN

Table 3 Axle Weight Distribution of Heavy Trucks of Subgroup No. 2

Truck Selection		Gross (kips)	A-WT#1 (kips)	A-WT#2 (kips)	A-WT#3 (kips)	A-WT#4 (kips)	A-WT#5 (kips)	R ₁₂	R ₂₃	R ₃₄	R ₄₅
Subgroup #2	Average	57.02	10.39	12.12	11.97	11.19	11.34	0.94	1.01	1.15	0.99
	Std dev	14.80	0.91	3.42	3.35	4.15	4.17	0.32	0.05	0.31	0.08
Heavy Trucks of Subgroup #2	Average	76.64	11.10	16.49	16.25	16.33	16.47	0.67	1.02	1.00	0.99
	Std dev	1.19	0.83	0.66	0.66	0.80	0.80	0.06	0.04	0.07	0.06

Note: R₁₂ — Ratio of A-WT#1 to AWT#2; R₂₃ — Ratio of A-WT#2 to AWT#3; R₃₄ — Ratio of A-WT#3 to AWT#4; R₄₅ — Ratio of A-WT#4 to AWT#5; 1 kip = 4.448 kN

Table 4 Dimensions of Girders and Diaphragms

Bridge Components	Type	Flange		Web	
		Width (in.)	Thickness (in.)	Depth (in.)	Thickness (in.)
Interior Girders	36WF150	11.972	0.94	33.96	0.625
Exterior Girders	36WF135	11.94	0.794	33.962	0.598
Abutment Diaphragms	15E33.9	3.4	0.65	13.7	0.4
Pier Diaphragms	24WF76	8.985	0.682	22.546	0.44
Intermediate Diaphragms	16WF36	6.992	0.428	14.994	0.299

Note: 1 in. = 25.4 mm.

Table 5 Parameter Values and Ranges of I-80 Bridge

Parameter	Non-composite Plan Value	Composite Plan Value	Lower Limit	Upper Limit
I_{GE-P} , in ⁴	7680	28677	5760	35846
I_{GE-N} , in ⁴	12791	33899	9594	42374
I_{GI-P} , in ⁴	8895	24926	6671	31158
I_{GI-N} , in ⁴	14761	35737	11071	44672
R_E , kips-in./rad	1000	1000	0	9000
R_I , kips-in./rad	1000	1000	0	9000
E_D , ksi	3834	3834	2876	4793

Note: I_{GE-P} — moments inertia of exterior girders in the positive moment region; I_{GE-N} — moment inertia of exterior girders in the negative moment region near piers; I_{GI-P} — moment inertia of interior girders in the positive moment region; I_{GI-N} — moment inertia of interior girder cross-sections in the negative moment region near piers; E_D — modulus of elasticity of deck; R_E — spring constant for exterior girders; R_I — spring constant for interior girders; 1 in⁴ = 416,231 mm⁴; 1 ksi = 6.895 MPa; 1 kip-in/rad = 0.113 kN-m/rad.

Table 6 Neutral Axis Determination at Different Girder Cross-sections

Girder Cross-sections	N.A. Based on Section Properties (in.)	N.A. Based on Strain Response			
		Base Gages	Mean (in.)	Standard Deviation (in.)	Minimum Delta Strain (10^{-6})
Exterior Girders in Positive Moment Region	35.04	D5_BF & D5_TF	30.53	0.382	3.5
Exterior Girders in Negative Moment Region	33.90	G5_BF & G5_TF	31.33	0.183	3.5
Interior Girders in Positive Moment Region	29.89	D4_BF & D4_TF	28.22	0.311	15
Interior Girders in Negative Moment Region	26.75	G4_BF & G4_TF	27.41	0.496	13

Note: Neutral Axis Location — Relative to the bottom gage location; Minimum Delta Strain — Minimum strain difference between the top and bottom gages; 1 in. = 25.4 mm.

Table 7 Parameters of Three-Axle Dump Truck

Truck Type	A-SPC #1, ft	A-SPC #2, ft	A-WT #1, kips	A-WT #2, kips	A-WT #3, kips	GVW, kips
Dump Truck	15.25	4.50	15.5	16.2	16.2	47.9

Note: A-SPC — Axle spacing; A-WT — Axle weight; GVW — Gross vehicle weight; 1 ft = 0.305 m; 1 kip = 4.448 kN.

Table 8 Crawl Speed Tests

Test ID	Truck Type	Speed, ft/s	Travel Lane	Transverse position, ft
CT1	Dump Truck	6.5	South	18.5
CT2	Dump Truck	6.6	South	18.5
CT3	Dump Truck	6.8	South	16.2
CT4	Dump Truck	5.8	South	16.2
CT5	Dump Truck	6.5	South	18.6

Note 1 ft = 0.3 m; 1 ft/s = 0.3/s.

Table 9 Calibration and Load Rating Results using Crawl Speed Dump Trucks

Test ID	I_{GE-P} , in ⁴	I_{GE-N} , in ⁴	I_{GL-P} , in ⁴	I_{GL-N} , in ⁴	R_E , kips- in./rad	R_I , kips- in./rad	E_D , ksi	AE , 10 ⁻⁶	PE	SE	CC	Min. Rating Factor
CT1	32770	36450	23550	33430	437.5	7000	5400	3868	5.9%	6.4%	0.9778	1.65
CT2	32770	37260	23450	34230	437.5	7000	5400	4506	6.4%	6.8%	0.9737	1.65
CT3	30870	37250	24930	34050	5736	7000	5400	4629	4.8%	9.1%	0.9776	1.66
CT4	30110	37870	24200	33280	2122	7000	5400	4461	4.8%	8.6%	0.9779	1.64
CT5	32770	36620	23250	33900	437.5	7000	5400	5869	5.8%	8.0%	0.9735	1.65
Mean	31858	37090	23876	33778	1834	7000	5400	4667	5.5%	7.8%	0.9761	1.65
Standard Deviation	1277	569	688	407	2300	0	0	734	0.7%	1.2%	0.0023	0.01

Note: AE – Absolute error; PE – Percent error; SE – Scale error; CC – Correlation coefficient; 1 in⁴ = 416,231 mm⁴; 1 ksi = 6.895 MPa.

Table 10 Sampling of Truck Events and WIM Trucks for Bridge Model Calibration

Sampling Strategy	Event No.	Travel Lane	Strain Bin (10 ⁻⁶)		Amount of WIM Trucks	WIM Truck Weight Bin (kips)
Strategy A	1	South Lane	30-80 (D2_BF)	25-75 (I2_BF)	100	20-80
Strategy B	2	South Lane	75-80 (D2_BF)	70-75 (I2_BF)	100	75-80
	3	North lane	75-80 (D4_BF)	72-77 (I4_BF)	100	75-80
Strategy C	2 & 3	South & North Lanes	75-80 (D2_BF) & 75-80 (D4_BF)	70-75 (I2_BF) & 72-77 (I4_BF)	100 & 100	75-80 & 75-80

Table 11 Events and Ambient Traffic Five-Axle Trucks

Event No.	Peak Strain 1	Peak Strain 2	Truck No.	Speed, ft/s	A-SPC #1, ft	A-SPC #2, ft	A-SPC #3, ft	A-SPC #4, ft
1	69.8 (D2_BF)	70.3 (I2_BF)	Five-axle Truck 1	104.2	18.83	4.58	34.58	4.5
2	77.6 (D2_BF)	71.2 (I2_BF)	Five-axle Truck 2	104.2	16.15	4.58	34.17	4.17
3	77.43 (D4_BF)	72.1 (I4_BF)	Five-axle Truck 3	102.4	16.79	4.5	31.15	4.09

Table 12 Calibration and Load Rating Results using the AAT Approach with Different Sampling Strategies

Calibration and Load rating Approach				I_{GE-P} , in ⁴	I_{GE-N} , in ⁴	I_{GI-P} , in ⁴	I_{GI-N} , in ⁴	R_E , kips-in./rad	R_I , kips-in./rad	E_D , ksi	AE , 10 ⁻⁶	PE	SE	CC	Min. Rating Factor
AAT approach	Strategy A	Event 1	Mean	11411	13326	21616	26013	1321	8776	4624	11727	13.2%	12.2%	0.9311	1.49
			STVD	3788	3619	4564	6835	1762	226	938	2311	5.9%	1.1%	0.0323	0.17
	Strategy B	Event 1	Mean	15655	18528	25465	34943	5694	8500	5524	9393	9.0%	11.7%	0.9546	1.45
			STVD	3569	3591	919	734	1114	0	177	582	0.9%	0.8%	0.0049	0.10
		Event 2	Mean	31299	38440	21653	33329	6859	8500	5850	7415	7.2%	8.9%	0.9694	1.57
			STVD	3892	4842	1848	2085	0	0	0	397	1.0%	0.3%	0.0016	0.06
	Strategy C	Event 1 & 2	Mean	30222	36782	22334	33852	6859	8500	5850	17207	9.0%	5.2%	0.9575	1.60
			STVD	4160	5595	1161	940	0	0	0	853	1.0%	0.4%	0.0042	0.06
TKT approach			Mean	31858	37090	23876	33778	1834	7000	5400	4667	5.5%	7.8%	0.9761	1.65
			STVD	1277	569	688	407	2300	0	0	734	0.7%	1.2%	0.0023	0.01

Note: STVD - Standard Deviation; 1 in⁴ = 416,231 mm⁴; 1 ksi = 6.895 MPa; 1 kip-in/rad = 0.113 kN-m/rad

Table 13 Parameters of Selected Five-Axle Trucks

Trucks	A-SPC #1, ft	A-SPC #2, ft	A-SPC #3, ft	A-SPC #4, ft	A-WT #1, kip	A-WT #2, kip	A-WT #3, kip	A-WT #4, kip	A-WT #5, kip	GVW, kip
Five-axle Truck 1	16.2	4.3	33.5	4.1	10.31	17.4	16.9	16.45	16.45	77.51
Five-axle Truck 2	17	4.5	31.9	4.1	10.97	15.83	17.07	16.5	16.59	76.96

Figure 1 Flowchart of Automated Step-by-Step Procedure of Bridge Load Rating using AAT Approach

Figure 2 Schematic of SHM System Components

(a) Side-by-Side Event

(b) One-after-Another Event

(c) A Truck Traveling on the Bridge

(d) Deck Gages in Sensor Lines 1 and 2

Figure 3 Trucks Traveling on a Bridge

(a) Peak Strain Rates in Sensors DL11 and DL21

(b) Five-axle Truck Corresponding to Peak Strain Rates

Figure 4 Peak strains Induced by a Five-Axle Truck

(a) Axle Spacing #1

(b) Axle Spacing #2

(c) Axle Spacing #3

(d) Axle Spacing #4

Figure 5 Frequency Histograms of Axle Spacings (1 ft = 0.305 m)

(a) Five-Axle Trucks of the Subgroup No. 1

(b) Five-Axle Trucks of the Subgroup No. 2

Figure 6 Frequency Histograms of Gross Vehicle Weight (1 kip = 4.448 kN)

(a) Maximum Strain in Gage D2_BF – South Lane Events

(b) Maximum Strain in Gage I2_BF – South Lane Events

Figure 7 Frequency Histograms of Maximum Girder Strains in Different Gages for Trucks of Subgroup No. 2 (Continued)

(c) Maximum Strain in Gage D4_BF – North Lane Events

(d) Maximum Strain in Gage I4_BF – North Lane Events

Figure 7 Frequency Histograms of Maximum Girder Strains in Different Gages for Trucks of Subgroup No. 2

Figure 8 Flowchart of Automated Ambient Traffic Approach using Sampling Strategy C

(a) View

(b) Bridge Plan (the red dot — a strain gage on the girder bottom flange; the yellow dot — two strain gages on the girder top and bottom flanges; and the red cross — a deck gage)

(c) Labels of Cross-Sections, Girders, and Gages

(d) Bridge cross-section

Figure 9 View, Plan, Cross-section and Strain Gages of I80-Bridge (1 ft = 0.3 m; 1 in. =25.4 in.)

(a) Bridge Model

(b) Modeling of Girder and Deck

(c) Optimized Bridge Parameters

Figure 10 Details of FE Model of I-80 Bridge

Figure 11 Gages on Interior Girder Cross-section – Positive Moment Region

(a) Strain response of Gage Pair D4_BF and D4_TF

(b) Neutral Axis Location

Figure 12 Neutral Axis Determination based on Gage Pair D4_BF and D4_TF

Figure 13 Axle and Wheel Configurations of Dump Trucks (ft, 1 ft = 0.305 m)

Figure 14 Frequency Histograms of Minimum Rating Factors using Different Sampling Strategies

(a) Gages D1_BF and D2_BF

(b) Gages D3_BF, D4_BF, and D5_BF

Figure 15 Comparisons of Strain Time Histories between Collected Data and FE Results using AAT Approach — South Lane Event & Section D (1 ft = 0.305 m)

(a) Gages D1_BF and D2_BF

(a) Gages D3_BF, D4_BF, and D5_BF

Figure 16 Comparisons of Strain Time Histories between Collected Data and FE Results

using AAT Approach — North Lane Event & Section D (1 ft = 0.305 m)

Abstract

Approximately 11% of bridges in the United States are categorized as structurally deficient and there is a marked need of more accurately evaluating true structural capacity. Structural Health Monitoring (SHM) systems can provide a timely indication of the need for maintenance, repair, rehabilitation and replacement of bridges and can greatly improve the apportionment and management of limited resources. This paper presents an Automated Ambient Traffic (AAT) approach for determining load rating of bridges monitored by the BECAS SHM system under ambient traffic. The AAT approach was developed through a process integration of truck detection, bridge model calibration, and bridge load rating: (1) The quasi-static bridge strain response and the characteristics of associated trucks are collected; (2) Multiple trucks are randomly sampled from a historic Weigh-In-Motion (WIM) database; and (3) For each combination of strain response and truck selection, an Finite Element (FE) model is calibrated and used to calculate a load rating. Sampling strategies were discussed for appropriately quantifying the influence of uncertainties of truck characteristics on the calibration and load rating results. To demonstrate this approach, a sample three-span, five-girder, and two-lane steel girder/concrete deck (I-80) bridge was utilized. A load rating of the I-80 Bridge using the Traditional Known Truck (TKT) approach was performed to provide benchmark results. The results of the calibration and load rating using the AAT approach were derived using three different sampling strategies and compared to those using the TKT approach. The sampling strategy, selecting strain response with a spectrum of higher peak girder strains, associated trucks with a spectrum of higher gross vehicle weights, and two truck events on south and north lanes respectively for a calibration, resulted in the best calibration and load rating results. It was concluded that the AAT approach using the BECAS SHM System is a reliable method for continuously estimating the load carrying capacity of bridges.