Bridge Fatigue Service-Life Estimation Using Operational Strain Measurements

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Abstract: Fatigue service life of steel bridges subjected to cyclic live loads was studied using operational strain measurements of a full-scale bridge. Fatigue fracture can be brittle and sudden, potentially leading to catastrophic bridge collapse. A service-life index and function were proposed for determination of the ultimate service life and remaining service life of bridges. The proposed method was used to study fatigue behavior of a bolted connection of a full-scale steel bridge. A regression model was developed for calculation of the life index and the service life function. Instead of common practice of using nominal stress data, realizations of the maximum stresses were generated using the bootstrap method. This paper shows that the proposed model is applicable for service-life prediction of existing steel bridges using only a finite number of operational measured strains from heavy truck loads. **DOI: 10.1061/(ASCE)BE.1943-5592.0000860.**

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Introduction

Fatigue of steel bridges is an additive type of damage under cyclic live loads caused by daily traffic. The live loads are normally converted into nominal stresses. Fatigue life is then normally calculated by determination of both the variable-amplitude nominal stress and number of cycles (Lahti et al. 2000). Fatigue failure is assumed to occur at stress levels that are lower than the yield stress of steel under repeated alternating or cyclic stresses. Fatigue fracture is a brittle type of fracture that is either sudden and catastrophic or slow to propagate. Therefore, fatigue failure criteria should be considered during the design phase of steel bridges.

Several authors have evaluated the fatigue life of bridges using strain measurements (Alampalli and Lund 2006; Zhou 2006; Chan et al. 2001; Huckelbridge et al. 2002) as well as considering chloride environments (Cheung et al. 2009; Oh et al. 2007). Also, Ni et al. (2010), Wang et al. (2012), and Sahrapeyma et al. (2013) studied fatigue reliability assessment of steel bridges. For fatigue assessment and service-life prediction of existing steel bridges, Soliman et al. (2013) focused on service-life prediction for steel bridges by combining structural health monitoring (SHM) with a probabilistic bilinear stress–number of cycles (*S-N*) approach.

Yen et al. (2013) studied bilinear *S-N* curves using information regarding equivalent constant amplitude stress ranges and recommended a new slope for fatigue life estimation. The recommended slope below constant amplitude fatigue limit was based on an analytical derivation regarding cumulative fatigue damage. Moreover, Kwon et al. (2012) focused on fatigue life estimation below the constant amplitude fatigue threshold (CAFT) for steel bridges on the basis of a probabilistic approach. The research improved fatigue life estimation by integration of a bilinear *S-N* approach into a probabilistic framework. Uncertainties of the fatigue deterioration process were modeled using the approach. Application of the proposed approach was illustrated using an existing bridge that was expected to experience finite fatigue life.

In extending the field of fatigue life, Roeder et al. (2005) considered fatigue cracking of coped stringers riveted to floor beam connections for better estimation of service life and life extension. The study developed tools to evaluate damage limitation methods for the coped stringer connection, and to better understand cracking and crack growth. As a result, a design procedure was proposed for improving performance and extending fatigue life. Razmi et al. (2013) presented guidelines to evaluate elastic and plastic deformations for determination of the amount of deformation in piles, and establish the fatigue life of the piles in integral-abutment bridges. Their research showed that seasonal fatigue life was much smaller than daily fatigue life. For evaluation of measured response effects as disturbance in the interpretation of fatigue life prediction results, Leander and Karoumi (2012) researched quality assurance of measured response intended for fatigue life prediction. Also, statistical distributions for monitored stress ranges were presented to assess fatigue life. The objective of their research was to find reliable approaches and methods for a quality assurance of the monitored result for fatigue life prediction. Saranik et al. (2013) studied experimental and numerical models for fatigue life prediction of bolted connections. They reported that a probabilistic life prediction model of a beam could be identified by a Monte Carlo simulation

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methodology. They showed that the information presented by the *S-N* curve can provide data on the fatigue damage characteristics of these connections. Maximov et al. (2012) focused on enhancement of fatigue life of the net section in fitted bolted connections. They developed an approach to introducing nearly uniform residual hoop stresses. They showed that the residual hoop stresses enhance the fatigue life of the net section in these connections.

This literature review reveals that it is possible to assess fatigue life of steel bridges if measured responses are available for their entire lifetimes. Although this can be accomplished for laboratory component testing, such data are typically not available for full-scale bridges. Because measured responses of steel bridges during their lifetimes are generally unavailable, a tool is needed to predict future responses from actual measured responses, which is the primary goal of this paper. The authors used the bootstrap method to predict future measured responses. It provides a simple approach for generating plausible sets of stress data that reproduce the empirical probability distribution of measured stress data, which can in turn be used for service-life prediction. Bootstrapping is implemented by randomly sampling from an independent measured data set with replacement to create additional data sets that can be used for further statistical analysis. Efron (1979), Carey (2004), Sahrapeyma et al. (2013), and many others describe the bootstrap approach, and Politis (2003) and Vogel and Shallcross (1996) provide guidance on the use of bootstrap methods for data series that exhibit temporal dependence. Bigerelle and Iost (1999) and Bigerelle et al. (2006) used bootstrap analysis for fatigue lifetime prediction. Their research showed that the bootstrap is a powerful tool for modeling probability density function of fatigue life time prediction. Similarly, Follen et al. (2014) applied the bootstrap to a bridge engineering problem concerned with damage detection.

Most researchers predict the service life, or fatigue life, of steel bridges under average daily traffic. However, they do not normally consider changes in the average daily traffic loads (Zhou 2006; Chiewanichakorn et al. 2007). At best, they define average daily traffic using a constant annual rate (Kwon and Frangopol 2010; Alampalli and Lund 2006). Using a constant annual rate is unrealistic because it ignores the natural and important variations in average daily traffic. This paper introduces a service-life function for estimation of remaining service life that considers actual changes in average daily traffic. In addition, previous researches limited their analysis to only stress ranges and number of cycles for fatigue life only using quantitative values (Li et al. 2002, 2003; Chan et al. 2001). This paper takes a different approach by proposing a quantitative and qualitative life index for ultimate service-life prediction of existing steel bridges.

A regression analysis was performed based on stresses and associated number of cycles for examining changes in the life index. The service-life function was defined as the number of cycles experienced by a steel bridge under actual changes of daily traffic using only measured strains. This study predicted future stress ranges and considered the relevant number of cycles that defines the service life of a bridge on the basis of previous measurements. Each truck type and weight has a different truck influence line at the strain gauge location used with its own stress range during truck travel. The stress ranges at a strain gauge location for different truck paths are different. After obtaining the stress range from measured strain time history during each truck travel, the highest value of stress range was used to determine the nominal stress. Then, the nominal stresses were used to calculate fatigue life. For this well-designed bridge, the calculated nominal stress of the Powder Mill Bridge (PMB) in the target bolted connection was lower than the CAFT of stress. This is the expected design practice for new bridges to avoid fatigue issues. To demonstrate the fatigue life prediction methodology using the existing measured strain data at the PMB, maximum stresses were used to calculate fatigue life instead of using nominal stresses. For this purpose, a stress concentration factor was used to convert nominal stresses to maximum stresses. The maximum stress is the largest or highest algebraic value of a stress range during truck travel compared to the smallest algebraic value. This is the algebraic difference between the maximum and minimum values. Each truck type and weight has a different truck influence line with its own stress range. In this paper, maximum stress per truck is the algebraic difference between the maximum and minimum. Again, to demonstrate the proposed method on an actual bridge, maximum stresses were obtained from measured strains and were used instead of nominal stresses at a critical location. The maximum-stress approach yields more conservative results than use of nominal stresses for service-life prediction with minimal computational efforts. This method can estimate repair, retrofit, and replacement costs over the service life of existing steel bridges, which can play an important role in transportation systems everywhere. Here, service life and fatigue life are assumed to have the same meaning and behavior.

Powder Mill Bridge and Instrumentation

The PMB over the Ware River in Barre, Massachusetts, was opened to traffic in September 2009 (see Fig. 1). This bridge was selected because it is a typical structural system that is frequently used in most existing continuous steel girder bridges within the U.S. highway transportation network. It is a three-span continuous steel girder bridge that has a 200-mm-thick composite concrete deck slab and carries two lanes running north-south for a total of 47 m (154.2 ft) across the river. The main span is 23.5 m (77.1 ft) in length, whereas the two end spans are 11.75 m (38.5 ft). A field splice is located close to the north pier in the center span, where the steel girders are bolted to plate girders. Although the PMB is in a rural location, it supports heavy truck traffic from the Barre-Martone regional landfill and recycling facility near the bridge. The steel girders are spaced at 2.25 m (7.4 ft) in the center. Steel diaphragms that laterally connect the steel girders are spaced at 5.875 m (19.3 ft), located at seven stations along the length of the steel beams. There is a water main under the bridge between Girders 4 and 5 that is supported by smaller steel diaphragms and smaller spacing.

The PMB was instrumented with a long-term SHM system containing a variety of sensors. The data acquisition system (DAQ) and all analog-to-digital convertor boxes were installed under the bridge

Fig. 1. PMB and data acquisition system (images by Masoud Sanayei)

and are invisible from the outside. Six different types of sensors were installed: strain gauges, steel temperature sensors, embedded concrete temperature sensors, uniaxial accelerometers, biaxial tilt meters, and pressure plates. These six different types of sensors comprise 200 sensors that were permanently installed on the bridge during the construction process. The large number of sensors makes the PMB one of the most instrumented bridges of its size, allowing the bridge to function as a full-scale outdoor laboratory. Sanayei et al. (2012) provided a complete description of the PMB and its instrumentation. As part of this monitoring system, 100 strain gauges were installed to measure strains at steel girders, as shown in Fig. 2. Most of the strain gauges are attached to the fatigue prone positions close to supports and midspans. All sensors are connected to data acquisition boxes, which are located near the south abutment. Collected static strain measurements were used for the heavy truck event-based service-life prediction. Subsets of these sensors have been used for different studies. In this research, only one set of strain measurements from one strain gauge was used for fatigue service-life studies.

The strain gauges are type KFG-5-350-C1-11L3M3R (Omega Engineering, Inc., Stamford, CT), roughly 9.4 mm (0.37 in.) with resistance of 350 Ω. Strain gauges are installed at critical locations where pairs of strain gauges are placed at the top of the bottom flange and the bottom of the top flange at each location, for a total of four at each station. This instrumentation plan is designed to record the response of the bridge under daily traffic loads and to establish the strain histograms at each location. These histograms illustrate the bimodal nature of the probability distribution of the truck travel stress output or probability density function (PDF) of maximum stresses. It would be difficult to capture the behavior of such bimodal PDFs without the use of the bootstrap method, as advocated in this study. The instrumentation for service-life prediction was selected for measurements at strategic locations with high stress concentrations that might experience cracking.

The annualized average 24-h traffic volume at a specified section of highway is the average daily traffic (ADT) volume, which is calculated by dividing traffic volume during a period by the number of days in that period. All existing steel bridges exhibit responses through loading interactions under daily traffic volume, which can lead to cumulative fatigue. For these load cases, the steel girders' operational responses to trucks of unknown weight and size are recorded. The collected strain output data are referred to as measured strain data under heavy truck loadings. In this paper, the maximum measured strain data from heavy-truck events were used to estimate statistical distribution functions that predict the service life of the PMB under normal operating conditions. In other words, using the measured data on operational strain as a result of daily traffic, the PMB service life was predicted on the basis of a service-life function using a nonparametric statistical technique, the bootstrap method.

In measured signal processing, there is always some level of ambient vibrations, measurement error, and measurement electronic noise. Measurement errors are filtered to create relatively clean strain measurements for moving trucks. One of the most common filters is the moving average in time domain. The moving average filter removes random noise from operational strain data. Because measurement noise generally has higher frequency content than structural responses, a moving average filter can remove some of the noise while retaining the response. A strain-measurement postprocessing program selectively collects strains from only the heaviest truck events by excluding data that do not exceed a threshold strain measurement. The moving average filter is then applied to the truck event in an effort to reduce measurement error, and the maximum measured strain in each truck event is used for bridge service-life prediction.

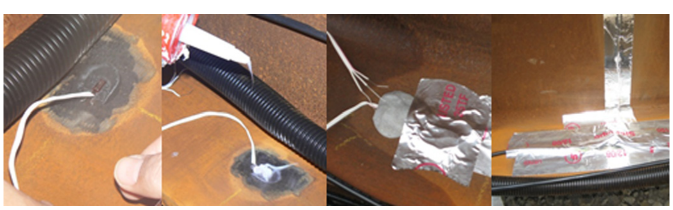

Fig. 2. Strain gauge installation process

Finite-Element Modeling for Critical-Point Fatigue Analysis of the PMB

Bolted connections with splice plates are widely present in existing steel bridges because they offer a large load-carrying capacity and easy assembly. These connections are typically the most critical type for fatigue damage and failures. They are sensitive to fatigue cracking because of high-stress fields around bolt holes in steel plates and segments of steel girder webs and flanges. In the PMB's splice section, shown in Fig. 3, the top and bottom flanges of the spliced girders are bolted to eight steel plates bridging the two girders. Therefore, uniaxial stress ranges at steel plates and flanges of

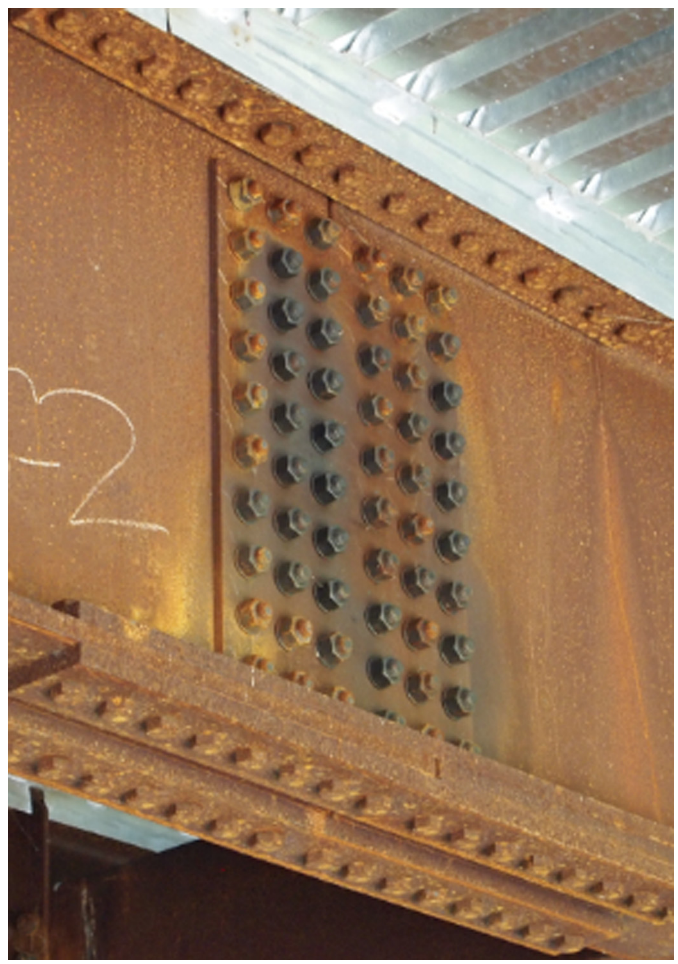

Fig. 3. PMB bolted connections as fatigue-critical locations (image by Masoud Sanayei)

the connection are obtained to investigate fatigue life by *S-N* curve. In other words, the longitudinal tensile stresses (σ_t) present near the bolt holes are used to evaluate fatigue life, because fatigue failure in bolted connections is provoked by initiation and growth of fatigue cracks around the holes. The longitudinal tensile stresses can gradually open these cracks and eventually result in crack growth. A fatigue failure will occur after a specified set of repeated loading cycles. Because a failure of one of these bolted connections is likely to result in serviceability or life safety issues, the fatigue failure of the PMB is calculated on the basis of fatigue life of bolted connections of different girders under heavy truck events.

To determine the most critical girder for evaluating fatigue, it is necessary to determine which girder experiences the highest loading under truck events. To accomplish this, a three-dimensional (3D) finite-element (FE) model for the PMB was developed. The FE model was created using four-noded shell elements for the steel girders and eight-noded solid elements for the concrete deck. Because the girders are supported by elastomeric neoprene bearing pads at all piers and abutments, translational and rotational springs were used to model the boundary conditions. The pads are very soft in the horizontal direction and have a minimal impact on bridge deck stresses and strains caused by gravity loads. The full description of this calibrated FE model is given by Sanayei et al. (2012). The model is called a coarse solid/shell element model that has been shown to be sufficiently accurate and computationally efficient. The geometry of the PMB is modeled as accurately as possible to ensure the FE model well represents the response of most critical steel girders under moving heavy truck loads.

After creating the FE model of the bridge, it was analyzed under different truck loading patterns running along the length of the bridge. Assumed truck loading paths were used to excite each girder for daily traffic–based service-life prediction to determine the most critical girder for fatigue service-life prediction. The HS20-44 design truck, with an axle longitudinal distance of 4.3 m (14 ft) and a concentrated tire loading lateral distance of 1.8 m (6 ft) for a truck weighing 358.7 kN (40 kips), was used for all load paths (AASHTO 2013a).

Fig. 4 shows a 3D view of the PMB model deck with the north in the upper-right corner and the load path X1 highlighted. The six longitudinal steel girders supporting the deck are numbered G1–G6, from left to right. The analysis showed that Girder G2 exhibited the highest stresses under Load Path X1, as shown in Fig. 4. In Path X1, the right tire is placed over Girder G2, and the left tire is 0.61 m off of the northwest curb to induce higher stresses. The crawl speed of the truck is approximately 1.34 m/s. The exterior Girders G1 and G6 did not exhibit a high enough response as a result of geometric limitations for loading imposed by the curb and sidewalk.

The aforementioned FE model analysis was performed with a truck of known size and weight for determination of the most critical girder. However, because daily traffic is not controlled by size, weight, or path, an automated system of SHM and service-life prediction was required that was independent of truck characteristics. For this study, the measured response of strain gauge No. 26 (SG-26) was used. SG-26 is located on the second steel girder (G2) near the north pier connection. SG-26 is on the interior side of the bottom flange of the steel girder. This specific gauge was chosen because it is located roughly at the location that receives the maximum stresses.

Strain and Stresses for Fatigue Analysis

A long-term monitoring program was configured on site to automatically and continuously collect measured strain data from heavy truck events. Data were collected for 175 days spaced intermittently between December 2013 and May 2014 (approximately 6 months). In total, 44,463 measured strain data, which exceed the specified threshold, were collected from the daily traffic volume at a sampling rate of 50 Hz.

Temperature-related effects are the most common causes of changes in strain measurements. Moreover, variations in measured strain data are affected significant by seasonal changes. The strain data from the vehicle events are used to estimate a nonparametric cumulative probability distribution of maximum strain measurements. To remove the temperature effects for use in fatigue calculations, each strain gauge record is zeroed out based on the initial reading before the truck drives onto the bridge. The ambient temperature does not vary significantly during the short time that a truck passes the bridge.

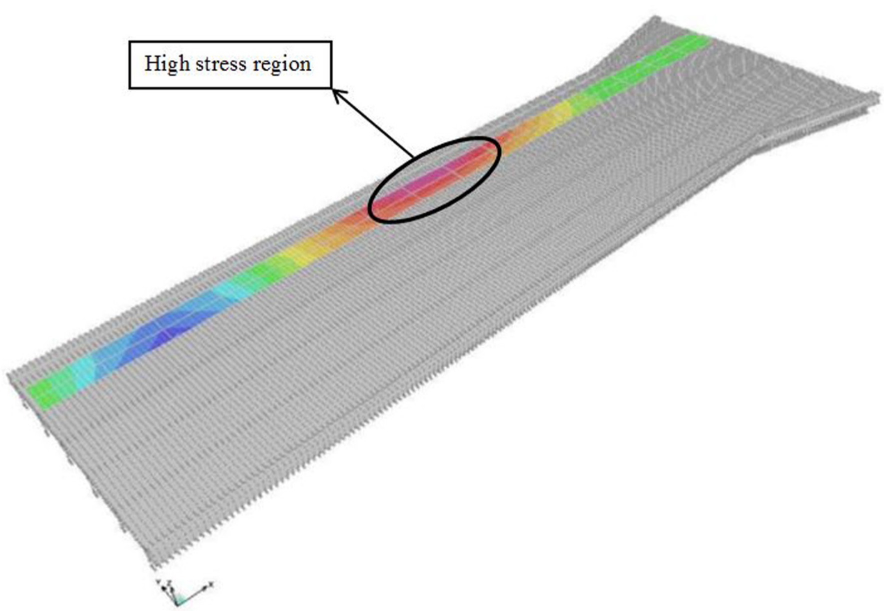

Fig. 4. FE model of the PMB (critical load path on Girder 2)

There is a CAFT of stress for each material's *S-N* curve, which can also be called the endurance limit, the fatigue limit, or the fatigue strength. It is a threshold stress level below which the specimen will not develop fatigue cracks during the expected life of the bridge. In contrast, below this threshold, the material may endure an infinite number of cycles, giving it an infinite fatigue life. For the PMB, the σ_{CAFT} is equal to 69 MPa (10 ksi), determined according to the experimental *S-N* curve of A572 steel (Woo et al. 2002). The equation of the experimental *S-N* curve for this steel alloy is expressed as

$$\log N = 15.088 - 4.097 \log S \qquad (1)$$

Here, the constant amplitude fatigue threshold of strain is evaluated on the basis of CAFT of stress using linear elastic behavior ($\varepsilon_{\text{CAFT}} = \sigma_{\text{CAFT}}/E$). A heavy truck event is defined as an event from the maximum strain output from SG-26 that is greater than $\varepsilon_{\text{CAFT}} = 345$ ($\mu\varepsilon$) based on its histogram. The measured strain data, which are more than the CAFT, experienced by the connection resulted in a total of 1,225 measured heavy truck events.

According to Hooke's law, maximum stresses are linearly proportional to measured strain data for linear-elastic behavior of steel materials. The modulus of elasticity of steel used is 200,000 MPa (29,000 ksi) for all steel girders of the PMB. Because the bending stresses in the steel girder flanges are mostly uniaxial, the measured axial strains are multiplied by the modulus of elasticity for calculation of axial stress. Thus, the calculated maximum stresses are uniaxial stresses in the longitudinal direction, defined as the *x*-axis in the model shown in Fig. 4. By assuming that only the elastic stress region is considered for fatigue behavior in existing steel bridges, the maximum stresses are calculated using 1,225 measured heavy truck events. The tensile maximum stresses above the endurance limit tend to open cracks and can cause fatigue failures in bolted connections.

Nominal stress is defined by dividing the axial load by the net cross-sectional area at a notch or hole. This kind of stress is calculated by simple elastic theory, ignoring any plastic behavior. The nominal stresses are used to evaluate the service life of existing steel bridges. Also, there are stress concentrations near holes, notches, and other changes in sections. This distribution contains calculated stresses that have larger magnitudes than the nominal stress over the section area. A stress concentration factor is defined as a ratio of maximum stress to nominal stress. In other words, the maximum stress level is equal to the concentration factor multiplied by the nominal stress. The calculated nominal stress of the PMB in the target bolted connection is lower than the constant amplitude fatigue threshold of stress. To demonstrate the service-life prediction method, the maximum stresses are used for the PMB based on fatigue stress concentration factor (k_f). The fatigue stress concentration factor (k_f) for this type of bolted connection with the given geometry is calculated to be equal to 3.2 (Pilkey and Pilkey 2008). Use of the maximum stress instead of the nominal stress leads to a more conservative service-life prediction for the PMB.

Fatigue Analysis Using the Bootstrap Method

A PDF of measured strains is needed to characterize the probability of occurrence of the maximum stress data. The nonexceedance probability associated with a maximum stress (σ) is determined by its cumulative distribution function (CDF). The complement of the CDF yields the exceedance probability of a maximum stress value and is known as the survival distribution function (SDF). For the 1,225 heavy truck events for SG-26 collected from the PMB, and i = 1–1,225, the absolute maximum stress values per event (σ_i) are ranked such that σ_1 is the largest maximum stress value related to ε_1 and σ_{1225} is the smallest related to $\varepsilon_{\text{CAFT}}$. The largest maximum stress value (σ_1) is 130 MPa (18.85 ksi), and the smallest value (σ_{1225}) is 69 MPa (10 ksi). The SDF is a plot of the ordered values of the absolute maximum stress per heavy truck event against their probability of exceedance (p_i). The probability of exceedance is computed using a Weibull plotting position, $p_i = i/(m + 1)$, where m is the sample size (m = 1,125) and i is the rank of the observation, as shown in Fig. 5. The Weibull plotting position is appropriate because it yields an unbiased estimate of the exceedance probabilities of the stress values, regardless of the form of their underlying PDF (David and Nagaraja 2003).

A key concern is a determination of the minimum number of measured strains needed for the bootstrap resampling strategy to ensure that the resampled distribution mimics the original empirical SDF during each bootstrap experiment. Fig. 6 illustrates the behavior of estimates of three quantiles from an SDF derived using varying numbers of events analogous to the analysis (Follen et al. 2014). Fig. 6 illustrates that a stable SDF results when the maximum stress value at a certain probability of exceedance does not change significantly with the addition of more data from the same distribution. Inspection of Fig. 6 indicates that at least 1,000–1,200 heavy truck events are required as the minimum sample size for implementing a bootstrap approach for determination of the SDF. Therefore, selection of the sample size comprising 1,225 heavy truck events should yield a realistic and stable SDF for the PMB. This uniquely collected sample is considered adequate to generate stable and independent sets of SDFs from the bootstrap experiments that follow.

For each structural girder, there are many points on the experimental *S-N* curve. Each point on the *S-N* curve determines a specific stress level on the S-axis and a corresponding number of cycles on the N-axis. For a typical girder, the corresponding number of cycles is normally on the order of several millions, which defines the fatigue life. In fact, this value is the total number of cycles experienced during the lifetime of the girder. Therefore, it is possible to predict the service life of the steel girder if the total number of cycles is known. Because only a finite number for both stress points and corresponding number of cycles could be obtained from the 1,225 measured heavy truck events, the bootstrap method was used to generate the several million cycles of data needed for this analysis by resampling the measured events. There is an equivalent of 1.5 cycles for each heavy truck event crossing the PMB on the basis of the AASHTO LRFD bridge design specification (AASHTO 2013a).

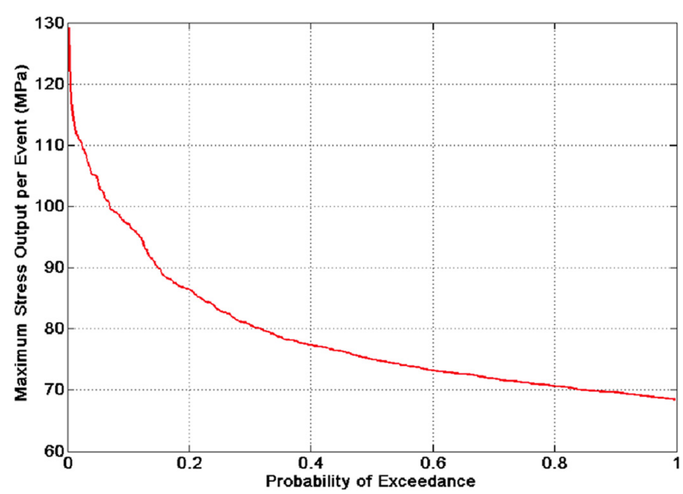

Fig. 5. SDF of measured stress data

Because there is a continuous time history for maximum stress data, a rainflow method is used for counting more accurately the number of fatigue cycles. The rainflow method is beneficial for using the actual number of induced cycles, which is 1.5 for each heavy truck event for this case study. Therefore, the rainflow method is used to validate the code value of 1.5.

The fatigue of steel bridges is an additive type of damage under cyclic live loads caused by daily traffic. If the number of cycles of traffic volume is specific during the lifetime of a steel bridge, then fatigue life is normally calculated using this number of cycles. However, it is not possible to measure this number of cycles during the lifetime of a steel bridge, thus only a small subset of this number of cycles is measured. Then, the number of cycles during the lifetime of a steel bridge is calculated based on these measured numbers of cycles using the bootstrap method. The bootstrap is implemented using a matrix of the measured number of cycles and the measured maximum stresses. Then, from this matrix, a bootstrap sample of maximum stress data and a related number of cycles is drawn randomly; with replacement, this is repeated until Miner's rule is satisfied. The SDF of maximum stress outputs is developed using the bootstrap method to reach this goal, as shown in Fig. 7.

Here, the bootstrap is implemented by sampling randomly, with replacement, from the measured maximum stress data set to create additional data sets that can be used in further statistical analysis. The theory of the bootstrap has shown that, for independent and identically distributed random variables, such resampling with

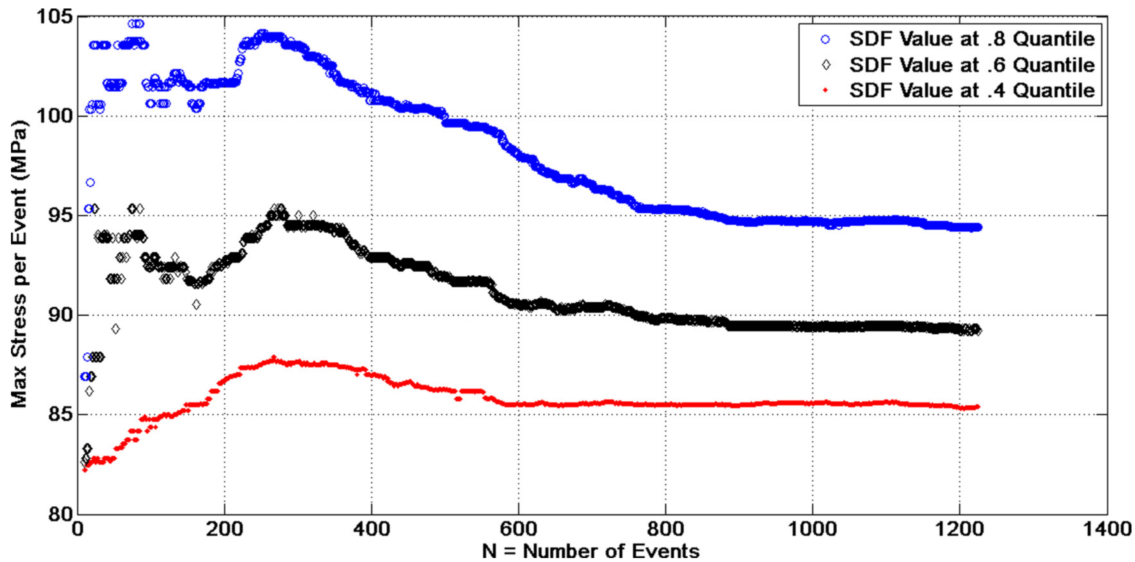

Fig. 6. SDF stresses versus number of events

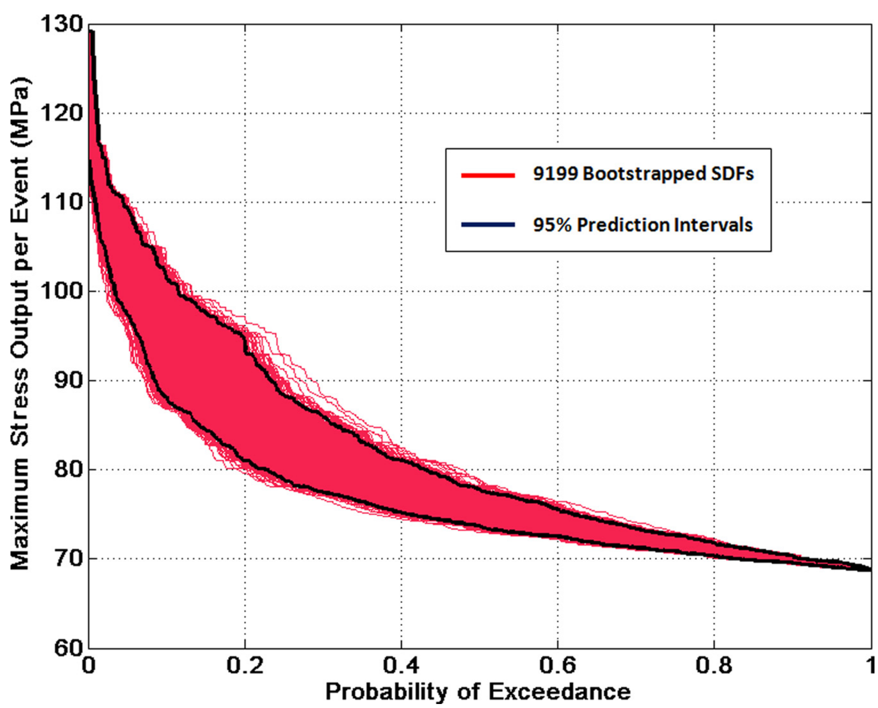

Fig. 7. Bootstrapped SDF of measured heavy truck events

replacement preserves the PDF, CDF, and SDF of the data. Again, the bootstrap method is used to generate new measured data until Miner's rule is satisfied. The bootstrap method is used to develop 9,199 new sets of resampled data from the 1,225 measured maximum stress data. Each new resampled data set is then used to develop 9,199 new resampled SDF curves. Fig. 7 shows the SDF resulting from each of the resampled data sets obtained via bootstrapping, resulting in m = 1,225 measured stress data per SDF. Also, each SDF is generated using measured stress data, sorted in descending order. Two prediction intervals are plotted on the SDFs in Fig. 7, as originally proposed by Vogel and Fennessey (1994) for hydrologic applications and more recently used by Follen et al. (2014) for a bridge application. Each of 9,199 curves or 9,199 sets of maximum stress is made up of the ordered values $[\sigma_{(i,j)}]$ for I = 1–1,225, as previously defined, and j = 1–9,199. For 95% prediction intervals with 9,199 curves, 5% of the maximum stress data at each i value should be outside the intervals; that is, the 95% prediction intervals at each i value are $PI_{\text{HIGHER BOUND}(i)} = \sigma_{(i,230)}$ and $PI_{\text{LOWER BOUND}(i)} = \sigma_{(i,8969)}$. The interpretation of the prediction intervals in the figure is critical to determine the number of maximum stresses that provide sufficient basis for a reliable and reproducible definition of the life index.

Miner's Rule

Miner's rule is one of the most widely used cumulative damage models for fatigue life prediction of steel bridges because of repetitive heavy truck loads. According to this rule, a percentage of fatigue damage at a specified stress level is defined by a number of loading cycles on the basis of the stress level. Also, the total damage is defined as the fatigue failure of the steel girder occurring when the number of cycles reaches 100%. All stress points on each SDF curve in Fig. 7 are converted into their largest maximum stress point with the corresponding number of cycles accounted for failure using Miner's rule. Then, the largest maximum stress points are derived as $(\sigma_1)_j$, where σ_1 is equal to the largest maximum stress level for curve j. For example, the largest maximum stress level is 130 MPa (18.85 ksi) for the first curve. Also, all numbers of cycles associated with bootstrapped SDF curves are converted into their equivalent number of cycles (N_j^*), which is given as

$$N_j^* = (n_1)_j + \sum_i \frac{(N_1)_j}{(N_i)_j}(n_i)_j \tag{2}$$

where i = number of measured stress cycles (1–1,225 in this study); j = number of SDF curves in descending order (1–9,199); $(n_1)_j$ = number of stress cycles applied at largest maximum stress point (σ_1) per curve (1.5); $(n_i)_j$ = number of stress cycles applied at other maximum stress points (σ_i) per curve (1.5); $(N_1)_j$ = number of cycles to fatigue failure applied at maximum stress level (σ_1) per curve, which is determined using the experimental S-N curve or Eq. (1); and $(N_i)_j$ = number of cycles to fatigue failure applied at each stress level per curve, which is determined using the experimental S-N curve or Eq. (1).

The bootstrap is continued until $\sum[(N^*)_j/(N_1)_j] = 1$ for j = 1–9,199 based on Miner's rule, resulting in a total of 9,199 generated SDF curves. According to the obtained points, a plot of the largest maximum stress of bootstrapped curves versus cumulative number of cycles was drawn, as shown in Fig. 8. Therefore, for all points, the largest maximum stress values of each curve are ranked from the largest to the smallest, and the cumulative dependent number of cycles $(N_{\text{cumulative}})$ is given as

$$N_{\text{cumulative}} = N_1^* + \sum_j \frac{(N_1)_1}{(N_1)_j} N_j^* \tag{3}$$

where j = number of SDFs (1–9,199 for the PMB); N_1^* = equivalent number of cycles relate to the first curve; N_j^* = equivalent number of cycles related to the jth curve; $(N_1)_1$ = number of cycles to fatigue failure applied at the largest maximum stress of the first curve, which is determined using the experimental S-N curve or Eq. (1); and $(N_1)_j$ = number of cycles to fatigue failure applied at the largest maximum stress of the jth curve, which is determined using the experimental S-N curve or Eq. (1).

The total number of all stress cycles is used because fatigue damage is cumulative and related to the number of loading cycles experienced by each specimen of the bridge load-carrying system. The plot in Fig. 8 contains 9,199 points that are representative of the largest maximum stress point of each bootstrapped curve. Also, the cumulative dependent number of cycles of the PMB is determined using Eq. (3), which is 4,156,261 cycles. This calculation is based on a bootstrap with a 5% prediction interval, and shows the ultimate number of stress cycles above the endurance limit that the target steel connection of the PMB can endure before occurrence of a fatigue failure resulting from heavy truck loads.

Service-Life Prediction of the PMB Based on Service-Life Function

The main goal of this paper was to propose a service-life function to predict the service life of existing steel bridges based on life index. The life index (μ_j) is defined using a mapping of all stresses to a range of [0, 1] as

$$\mu_j = \frac{(\sigma_1)_j - (\sigma_1)_{\text{maximum } j}}{(\sigma_1)_{\text{minimum } j} - (\sigma_1)_{\text{maximum } j}} \tag{4}$$

In this paper, the minimum and maximum j values are equal to 1 and 9,199, respectively, where $(\sigma_1)_1$ =130 MPa (18.85 ksi) and $(\sigma_1)_{9,199}$ =69 MPa (10 ksi). This index shows variation of the age of steel bridges under daily traffic loads. The life index is calculated for inferring the service life of the PMB using Eq. (4). Aging of the PMB over a period of time is shown by the gradual decrease of life index based on a linear interpolation, as shown in Fig. 9. The life index points are ranked from the largest to the smallest in the figure, meaning the life index decreases gradually as the PMB ages. The life index points simulate different degrees of fatigue deterioration in the G2 bolted connection. This index starts from unity, indicating that the PMB is undamaged and no repair deemed necessary. In contrast, when the life index is less than 1, the PMB is subjected to fatigue cycles and it is moving toward fatigue failure until a collapse occurs as $\mu_{j=9,199}$ approaches $N_{\text{cumulative}}$. The $\mu_{j=9,199}$ is equal to zero and $N_{\text{cumulative}}$ is 4,156,261 cycles in the collapse scenario. The life index is used as a reliable fatigue signature to envision the existing condition of the PMB under load cycles. Also, this paper confirms that the index is sufficiently sensitive to fatigue damage and results in a good indicator for prediction of fatigue life for the PMB subjected to heavy truck events. Fast decrease of the life index of an existing bridge warns of rapid fatigue deterioration of the existing steel bridges, which necessitates close attention for required repair of the bridge. There is one response variable and one predictor variable, the life index points and the cumulative number of cycles, respectively, which are shown in Fig. 9. The life index points have provided sufficient information to define and develop a service-life function for the PMB.

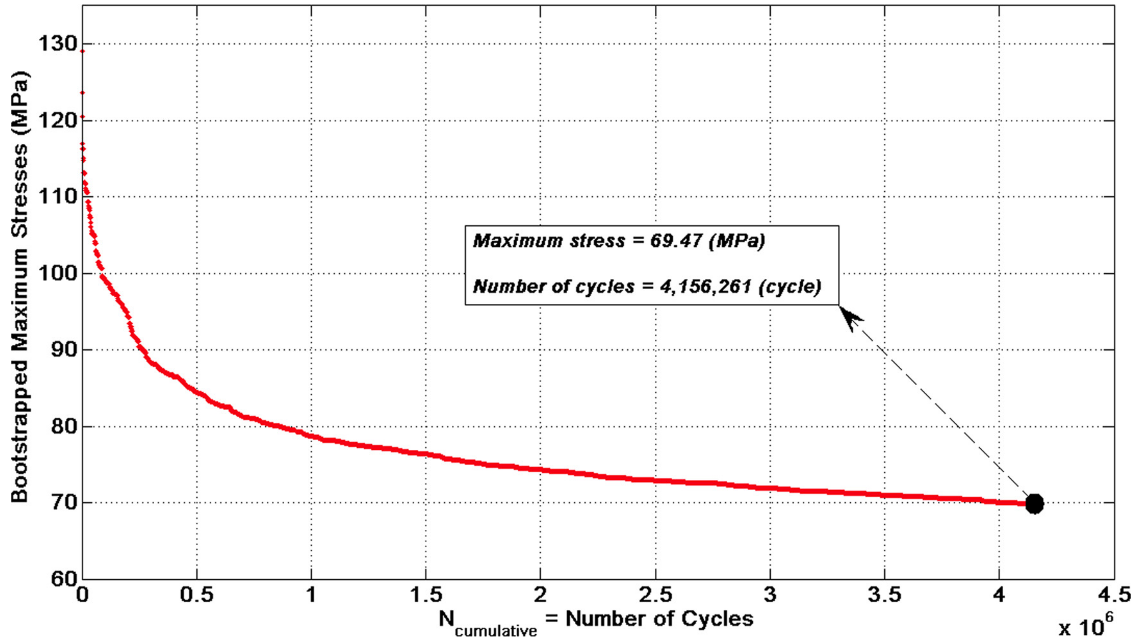

Fig. 8. Largest maximum stress of bootstrapped curves versus cumulative number of cycles

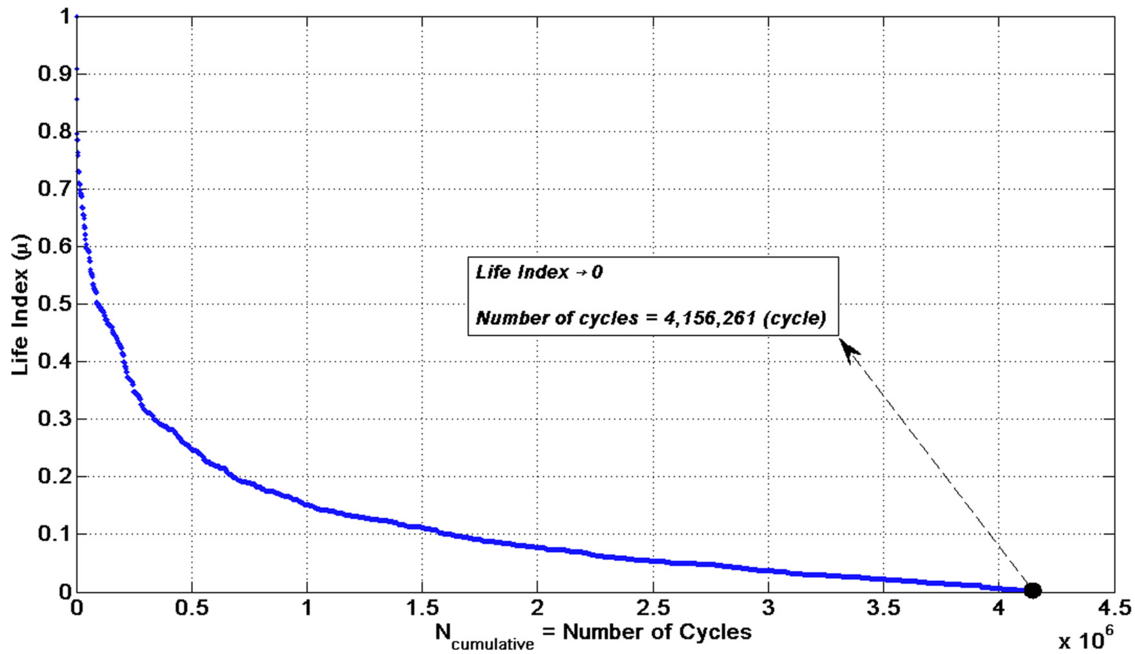

Fig. 9. Life index points versus cumulative number of cycles

There are several mathematical functions for curve fitting to scattered data in several applied literature sources that are used for conventional linear computations. This paper presents an estimation function that is general, simple to compute, and invariant using a single shape of function comprising both life indices and cumulative number of cycles. The main goal of this function is to consider actual changes in average daily traffic for remaining service-life prediction. Therefore, a regression model is proposed for modeling service-life function [$f(\mu)$] using a curve that is fitted through the life index points (μ_j versus $N_{\text{cumulative}}$). The regression analysis is used for investigation of relationships between life index and number of cycles to fatigue failure. The service-life function is a decreasing function of the life index. Thus, an inverse exponential function is used to predict fatigue life based on actual changes in traffic volume. This function takes the following general form:

$$f(\mu) = \left(\frac{a - \ln(1-\mu)}{b}\right)^{c} \tag{5}$$

where constants a, b, and c for the PMB = 0.421, 20.25, and –3.929, respectively. The function $f(\mu)$ is used here to approximate the complex nonlinear relationship of the life index with 95%

prediction interval corresponding to future life index predictions, as shown in Fig. 10.

Remaining Service Life

The service life of new and in-service bridges can be estimated on the basis of the proposed function duration of service and ADT. ADT is equal to a total heavy truck traffic volume during a given time period, ranging from 1 day to 1 year, divided by the number of days in that time period. As stated, whereas the life index is used as an index for prediction of maximum service life of the PMB, the service-life function is used to assess remaining service life of the bridge at every arbitrary time interval. The predicted remaining service life derived from the model can contribute to effective management of steel bridges. The remaining service life of existing steel bridges is estimated by the difference of $N_{\mathrm{cumulative}}$ and the service-life function $[f(\mu)]$ cycles divided by $(N_{\mathrm{ADT}})_{\mathrm{Heavy\ trucks}}$ as

$$T = \frac{(N_{\mathrm{cumulative}}) - f(\mu)}{(N_{\mathrm{ADT}})_{\mathrm{Heavy\ trucks}}} \tag{6}$$

where (N_{ADT}) = number of cycles for ADT in 1 year for heavy trucks. The remaining service life of the PMB after 5 years is determined using Eq. (6) and is given as

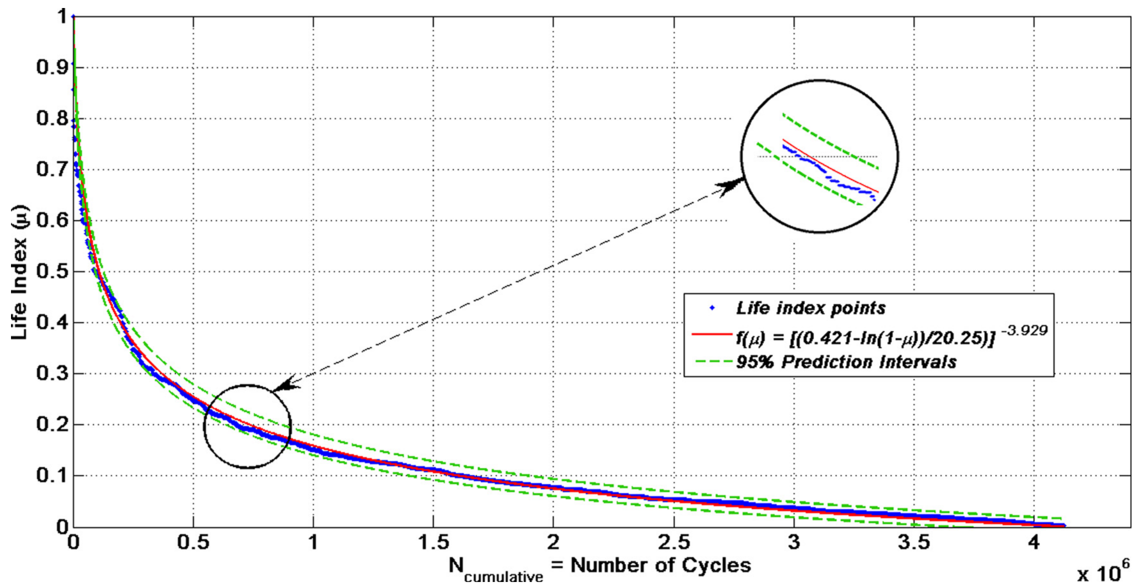

Fig. 10. Inverse exponential service-life function of the PMB

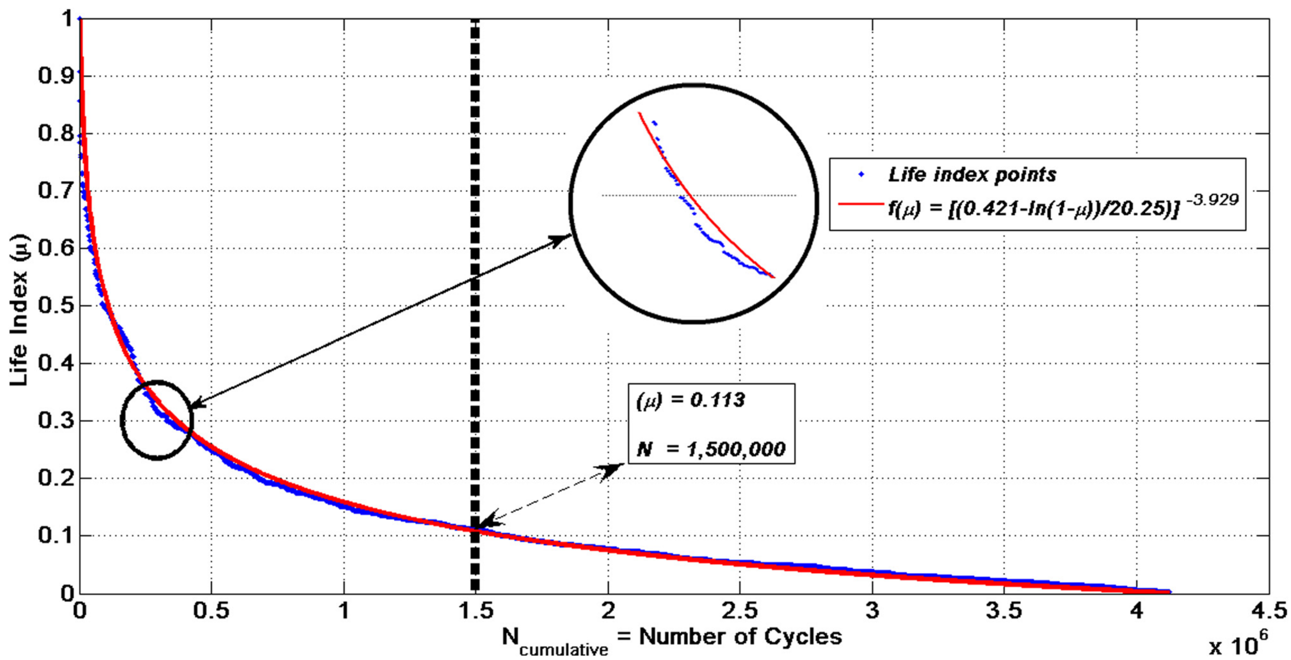

Fig. 11. Remaining service life based on service-life function

$$T = \frac{4,156,261 - 12,250}{\left(\frac{1,225}{175}\right)} = 592,001(\text{days}) = 1,621 \ (\text{years}) \tag{7}$$

Therefore, the remaining fatigue life of the PMB is approximately 1,600 years with a 95% prediction interval according to Eq. (7). This is possible and acceptable because the new existing steel bridges are designed to have an infinite service life (Lund and Alampalli 2004), and the PMB is one of them. The average annual daily traffic (AADT) contains data for 24 h, 365 days per year; however, the value will vary year to year. In fact, because of increasing demand, the AADT for a given bridge generally increases. Hence, the number of cycles can change throughout the PMB's service life. For example, it is assumed that the measured largest maximum stress level of the PMB extracted is 70.84 MPa (10.15 ksi) as a result of heavy truck events after 50 years. The life index related to this stress level is calculated using Eq. (4), that is 0.113 ($\mu = 0.113$). Then, the induced number of cycles at this stress level is determined by Eq. (5), which is 1,500,000 cycles [$f(\mu) = 1,500,000$ cycles], as shown in Fig. 11. As result, the PMB's remaining service life using Eq. (6) is given as

$$T = \frac{4,156,261 - 1,500,000}{2,450} = 1,084 \ (\text{years}) \tag{8}$$

The aforementioned equation proved that Eq. (6) has a better predictive capability compared with other proposed methods for service-life prediction because it is capable of considering actual changes in the ADT for fatigue life calculations. According to a traditional method, the remaining service life after 50 years will be calculated as

$$T = 1,621 - 50 = 1,571 \ (\text{years}) \tag{9}$$

The remaining service-life value in Eq. (9) is not based on consideration of actual changes in ADT, whereas the actual remaining service life is 1,084 years as calculated in Eq. (8). As a result, the presented method is particularly suitable for prediction of remaining service life of existing steel bridges and realizing traffic volume changes.

Conclusions

A SHM system was developed and installed at the PMB in Barre, MA. Operational strain measurements were collected for almost 6 months, leading to a sample of 1,225 truck excitations and response measurements. The series of independent strain measurements were converted into measured stress data, which were below the constant amplitude fatigue threshold. A stress concentration factor was used as a multiplier for converting nominal stresses to the maximum stresses. A fatigue analysis based on the bootstrap method using Miner's rule was performed for assessment of the remaining number of cycles. The following results were obtained from this research study:

1. A statistical method was introduced for estimating the number of fatigue cycles of existing steel bridges under heavy truck events using a nonparametric bootstrap method combined with a regression analysis. The proposed method was able to effectively predict service life using only operational strain measurements.
2. The bootstrap method as a nonparametric statistical tool was successfully used to predict possible future fatigue cycles. This method is simple and uses brute force computer computations to replace the complex theoretical assumptions and relationships required for most statistical approaches used to summarize data.
3. The proposed service-life index can quantify variation of the age of steel bridges under daily traffic loads. This index provides both a quantitative and qualitative index for ultimate service-life prediction of existing steel bridges. This index could be further used in bridge management. Here, the life index of a bridge was investigated to determine the capacity of the bridge under repetitive loads. Changes in the life index were sufficiently sensitive to the fatigue damage of bridges caused by heavy truck events.
4. The proposed service-life function can predict steel bridges' remaining service lives for any arbitrary future time period within the range of values considered in these experiments. The fitted service-life function was capable of considering actual changes in the ADT for prediction of remaining service life. Generally, the predicted remaining service life derived from the model could contribute to effective management of steel bridges, whereas the timely repair and retrofit increase the safety levels in bridges and decrease costs.
5. This paper documents that regression analysis can provide a powerful tool for determining accuracy of statistical functions for service-life prediction. Consequently, the proposed method assists bridge engineers, bridge owners, and state officials in the objective assessment of deteriorated bridges for retrofit or replacement of in-service bridges. The proposed methodology has successfully predicted the remaining service life for the PMB.

Future Work

The proposed method can only predict the remaining life of a steel bridge based only on its fatigue behavior. Future work in the field of service-life prediction of existing steel bridges should consider inclusion of multiple damage effects and environmental conditions. Consideration of these effects and conditions may improve both the quality and accuracy of available models and the proposed function. By doing so, researchers could investigate temperature effects on the service-life function. Other multivariate factors that can be included in the service-life function are corrosion, scour, earthquake, typhoon, and fire. Multivariate regression methods are ideally suited to the development of such relationships. It is notable that this method is general and applicable to all steel structures that might be subjected to fatigue-inducing cyclic loads. In addition to bridges, examples include mechanical systems, aerospace systems, power transmission, and civil constructed facilities.

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