17th ASCE Engineering Mechanics Conference
June 13–16, 2004, University of Delaware, Newark, DE

EM2004

IN-SERVICE LOAD & RESISTANCE FACTOR RATING METHOD FOR BRIDGES

Degang Li[1], Baidurya Bhattacharya[2] (Member, ASCE) and Michael J. Chajes[3] (Member, ASCE)

ABSTRACT

Historically, bridges are evaluated using allowable stress and load factor rating methods. Load rating made in these traditional methods does not correspond to any standard and quantifiable measure of safety and the resulting ratings are often quite conservative. The newly emerging AASHTO load & resistance factor rating (LRFR) method can lead to consistent and uniform safety. But its factors are derived from conservative traffic and multiple presence assumptions, and not based on site-specific information (although the LRFR manual does discuss the derivation of live load factors based on WIM data). This paper presents a live load probabilistic model based on site-specific data, which allows the elimination of a substantial portion of live load effect modeling uncertainty, as well as a substantial portion of structural analysis modeling error. Random occurrence rate of peak loads and Bayesian updating of measurement uncertainties are considered. Gumbel distribution is found to fit the projected maximum live load very well. Sensitivity studies show the projected maximum live load is not sensitive to the threshold strain above which events are recorded, as long as the threshold is sufficiently high. Based on the new live load

---

[1] Corresponding author: Lawrie & Associates, LLC, 6202 Old Franconia Road, Alexandria, VA 22310, email: lidegang@ce.udel.edu, fax: 703-719-6142

[2] Center for Innovative Bridge Engineering (CIBrE), Department of Civil and Environmental Engineering, University of Delaware, Newark, DE 19716, USA

[3] Center for Innovative Bridge Engineering (CIBrE), Department of Civil and Environmental Engineering, University of Delaware, Newark, DE 19716, USA

model, the instrumented bridge is evaluated for specified service period and target reliability. This proposed In-Service Load & Resistance Factor Rating (ISLRFR) method is illustrated using a slab-on-steel girder bridge on I-95. Site-specific bridge response data (peak live load strain) are collected using an In-Service Bridge Monitoring System (ISBMS) developed at the Center for Innovative Bridge Engineering (CIBrE) at the University of Delaware.

Keywords: Bridge Rating, LRFR, Reliability, Conditional Probability, In-Service Data

INTRODUCTION

Highways play a significant role in the nation's economy. Bridges are a very important part of the highway system. As transportation needs increase and the bridges continue to age and deteriorate, while maintenance and repair operations are deferred due to limited budgets, more and more bridges are classified as structural deficient. To optimize the allocation of the limited fund, methods for accurately assessing a bridge's true load carrying capacity are needed.

There are three existing rating methods: Allowable Stress Rating method (noted ASR in the following), Load Factor Rating method (noted LFR in the following), and Load & Resistance Factor Rating method (LRFR). ASR and LFR methods do not correspond to standard and quantifiable measure of safety. Reliability analysis shows that the steel girders, reinforced concrete T beams, and prestressed concrete girders designed by AASHTO (1992) show considerable variation in the reliability indexes (Nowak et al., 2000). The safety criteria in LRFR are consistent and uniform. But the load and resistance factors are derived from conservative truck traffic and multiple presence assumptions.

Also, bridge rating is different from bridge design. When bridges are designed, the behavior of the as-built bridge, as well as the nature of the site-specific traffic, can only be estimated. Bridge design is by necessity conservative, and many secondary sources of stiffness and strength are either neglected in design, or are too difficult to compute. When load rating a bridge, however, the best model is the bridge itself. Existing rating methods use simple analytical models and deterministic parameters. The model parameters come from original design specifications, and in some cases input from visual inspections. Not surprisingly, these parameters are often conservative. The actual performance of many bridges is better than theory predicts due to better load distribution, unintended composite action, unintended continuity, participation of secondary members, etc. When a structure's computed theoretical safe service live load capacity is less than desirable, it may be beneficial to take advantage of some of the bridge's inherent extra capacity if it is available.

In theory, to accurately rate a bridge, we need to take into account two things: one is a good bridge analysis model, the other is the site specific truck traffic. It is time-consuming and expensive to get the necessary information. The existing methods (ASR, LFR, LRFR) can hardly take both into account. A load test can result in a good bridge analysis model, but it has nothing to do with the truck traffic. WIM data can result in a good truck traffic model, but it is irrelevant to the bridge model. Therefore a new method – In-Service Load & Resistance Factor Rating (ISLRFR) will be proposed in this paper. It can lead to more accurate bridge rating.

PROPOSED LIVE LOAD MODEL

In-Service Bridge Monitoring System (ISBMS) is used to collect the in-service bridge response data. This system is developed at the Center for Innovative Bridge Engineering (CIBrE) at the University of Delaware. Detail information is shown in the work by Holloway (1999). Bridge 1-791 on I-95 is used to illustrate the proposed method. This bridge is a

3-span continuous, slab-and-steel-girder that consists of 2 traffic lanes and one breakdown lane and carries a large amount of traffic between Philadelphia, PA and Wilmington, DE. It was designed with the approach spans being non-composite and the center span being composite. Figure 1 shows the bridge in plan view. It is oriented at a slight 8-degree skew. The span lengths of the approach spans are identical at 35' and the span length of the center is 58' (Reid, 1996).

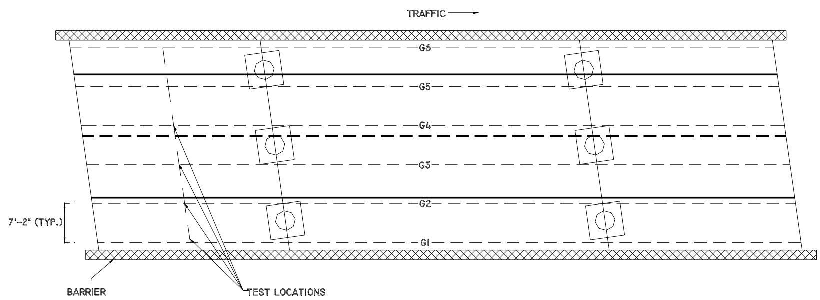

FIG. 1. Plan view of bridge 1-791 (Courtesy of Holloway)

From the collected in-service data, it is found that girder G3 controls. Its collected in-service data is shown in Figure 2. Only the strains above the threshold 85με are recorded. In 11 days, 533 peak strains are collected.

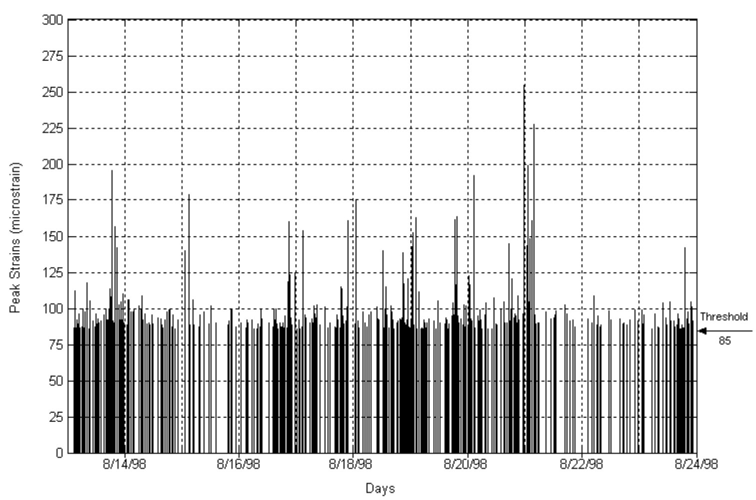

FIG. 2. Peak live load strain from G3 in bridge 1-791

Next, Peaks Over Threshold (POT) method is used to check the extreme value distribution corresponding to the collected peak strains. The expression for the Generalized Pareto Distribution (GPD) is:

$$G(y) = \text{Prob}\,[Y \le y] = 1 - \{[1+(cy/a)]^{-1/c}\},\ a > 0,\ [1+(cy/a)] > 0 \qquad (1)$$

Equation 1 can be used to represent the conditional cumulative distribution of the excess Y = X - u of the variate X over the threshold u, given X > u for u sufficiently large. The cases c > 0, c = 0, and c < 0 correspond (respectively) to Frechet (type II extreme value), Gumbel (type I extreme value), and Weibull (type III extreme largest values) distributions (Pickands, 1975, Castillo, 1988).

The de Haan estimation method (de Haan, 1994) for the estimate of c and a is shown in Equations (2) - (4). Let the number of data above the threshold be denoted by k, so that the threshold u represents the (k+1)th highest data points. The highest, second, …, kth, (k+1)th highest variates are denoted by $X_{n,n}$, $X_{n-1,n}$, $X_{n-(k+1),n}$, $X_{n-k,n} \equiv$ u, respectively. Compute the quantities

$$M_n^{(r)} = \frac{1}{k}\sum_{i=0}^{k-1}[\ln(X_{n-1,n}) - \ln(X_{n-k,n})]^r, r = 1,2 \tag{2}$$

The estimators of c and a are,

$$\hat{c} = M_n^{(1)} + 1 - \frac{1}{2\{1-[M_n^{(1)}]^2/[M_n^{(2)}]\}_1} \tag{3}$$

$$\hat{a} = uM_n^{(1)}/\rho_1 \tag{4}$$

where, $\rho_1 = 1, \hat{c} \geq 0; \rho_1 = 1-\hat{c}, \hat{c} \leq 0$. The calculated $\hat{c}$ is shown in Figure 3.

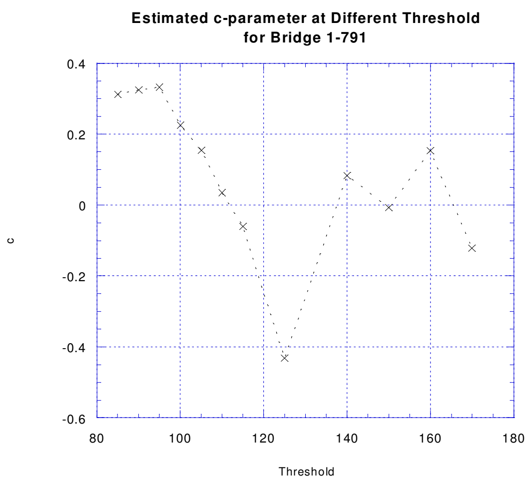

FIG. 3. Estimated $\hat{c}$ at different thresholds

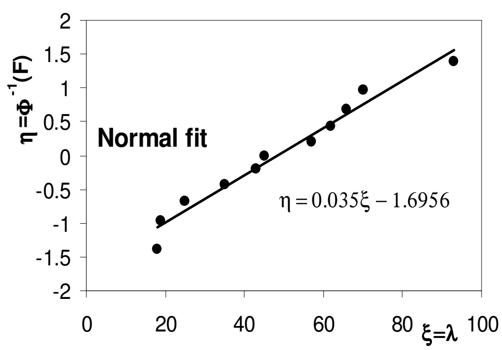

FIG. 4. Normal fit for random occurrence rate

It can be seen that $\hat{c}$ varies around 0, showing the peak live load is most likely Gumbel distributed. Next a different method will be used to verify the results of POT analysis and to estimate the distribution parameters.

To continue our analysis, we have the following assumptions:

- The occurrence rate Λ of the collected peak strains is a random variable.
- For a fixed value of Λ, the point process N(t) is a Poisson process.
- The peak strains are identically distributed and statistically independent of each other with

CDF $F_L(l)$.

The Λ and F_L are not precisely known and can be estimated from the in-service data. The CDF of the maximum peak strain during an interval of length t can be obtained from Equation (5) – (6).

$$F_{L\max,t}(l) = \iint_{all\ \hat{\Lambda}\ and\ \hat{F}_L} F_{L\max,t}(l \mid \hat{\Lambda}, \hat{F}_L(l)) f_{\hat{\Lambda}} f_{\hat{F}_L(l)}\, d\hat{\Lambda}\, d\hat{F}_L(l) \tag{5}$$

$$F_{L\max,t}(l \mid \widehat{F}_L(l) = p(l), \widehat{\Lambda} = \lambda) = \exp(-\lambda t(1 - p(l))) \tag{6}$$

It is found from probability fitting (Fig. 4) that for 1 day $\widehat{\Lambda}$ is normal distributed with mean 48.5 and COV 59%. And $\widehat{F}_L(l)$ is Beta (q,r) distributed with $q = n\widehat{p} + 1, r = n(1 - \widehat{p}) + 1$ (Bhattacharya, 2004). Monte-Carlo simulation is used to solve the Equation (5).

Gumbel and Frechet probability fittings are shown in Figure 5. Weibull distribution is limited at the right side. So it is not a good candidate. Obviously Gumbel distribution fits the data points very well. From Figure 5(a), we can also get the distribution parameters u=3.284/0.0201=163.4, α=0.0201 for 1 day. The projected live load for 2-year is shown in Table 2.

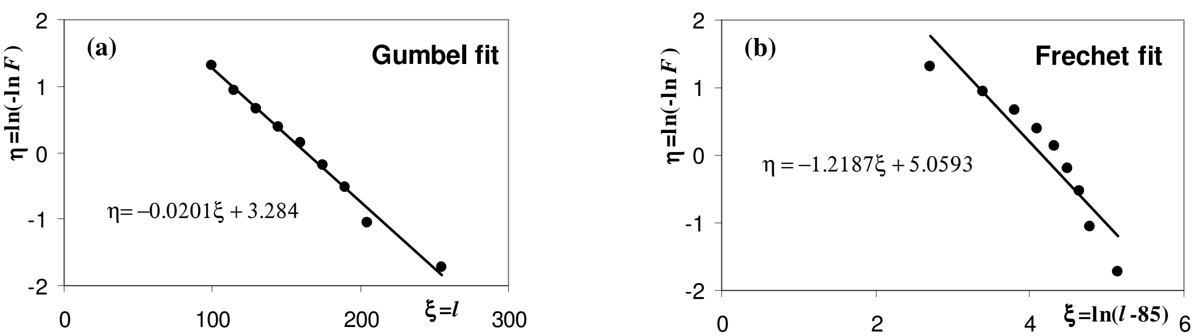

FIG. 5. (a) Gumbel probability fitting (b) Frechet probability fitting

The choice of the nominal live load is arbitrary. However, we need to choose the nominal live load in a manner that is acceptable to the bridge community. The choice of the predicted two-year return period load effect, L_{2yr} seems most reasonable in this regard. By definition L_{2yr} is exceeded on an average once every two years (the usual inspection interval) and is equal to the median annual maximum. It is also very close to 1-year mean live load. The 1-year median value is 475.7 με.

There is a key assumption in the above calculations. The collected peak strains are assumed to be an iid (independent and identically distributed) sample. Three tests (Turning Point Test, Difference-Sign Test and Rank Correlation Test) are performed to investigate this assumption. These three tests are briefly described in Appendix. The results are shown in Table 2, which clearly shows this iid assumption cannot be rejected at 95% confidence levels.

Table 2 Non-Parametric Tests for Independence

Sample n=533(511)	X	μ	σ	$\lvert X-\mu \rvert/\sigma$
Turning Point Test	353	339.3	9.5	1.44
Difference Sign Test	252	255	6.5	0.46
Rank correlation Test	63307	65152	5784	0.32
95% Confidence Limit $\Phi_{1-0.05/2}$=1.96				

RATING FACTOR FOR THE INSTRUMENTED BRIDGE

If we neglect the deterioration during inspection period, the limit state equation will be:

$$g(t) = R - D - L(t) \tag{7}$$

where, R = the bridge resistance, D = the dead load and L(t) = the maximum live load during inspection period t. The bridge rating equation is:

$$RF = \frac{\phi R_n - \gamma_D D_n}{\gamma_L L_n} \tag{8}$$

where, ϕ = resistance factor, γ_D= dead load factor, γ_L= live load factor, R_n=nominal resistance, D_n= nominal dead load and L_n=nominal live load with impact.

Since we do not have test data on the resistance and dead load for this instrumented bridge, the statistical parameter assumed in the LRFD codes calibration (NCHRP 454, 2001) will be used in the following calculation. Strain instead of truck weights or moment is collected. So strain will be used to evaluate this bridge.

The steel grade of bridge 1-791 is A36. Its yield strain is 1241 $\mu\varepsilon$. To take into account the extra capacity after the yield, the ultimate nominal resistance (ε_u) is taken as the yield strain (ε_y) multiplied by the ratio of inelastic moment ($M_{inelastic}$) and yield moment (M_{yield}), shown in Equation 9. $M_{inelastic}$ and M_{yield} can be calculated based on the codes.

$$\varepsilon_u = M_{inelastic}/M_{yield}\varepsilon_y \tag{9}$$

The statistical parameters of bridge 1-791 are shown in Table 3.

TABLE 3. Statistical Parameters Bridge 1-791

Items	Bias	COV	Nominal Value ($\mu\varepsilon$)	Distribution
Resistance R	1.12	10%	1.16×1241=1440	Lognormal
Dead Load D	1.03	8%	96	Normal
Live Load (2-yr) L	1.09	12.3%	475.7	Gumbel
M_U/M_y=936/807=1.16 ε_y=36/29000=1241 $\mu\varepsilon$				

Next, we will set the target reliability β. Ideally, the selection of target β should be an economic issue that reflects both the cost of increasing the safety margins and the costs associated with component failures. But the target β in current LRFD bridge design codes and LRFR manual are based on past performance experience and engineering judgment. The implied target β 3.5 and 2.5 in LRFD design codes and LRFR manual are calculated based on the assumption that live load is normal distributed. When the live load distribution is changed, the target β should be recalculated. This is a tremendous work. Unfortunately, we do not have the time and resource to do that. Some analysis (Li, 2004) shows when the live load distribution is changed from Normal to Gumbel while maintaining the bias and COV, the calculated target β will go down. To avoid confusion and to be simple, the target β is conservatively set as 2.5 in the following calculation.

Using Rackwitz-Fiessler procedure (1978) for FORM, it can be found that to reach reliability index 2.5 for 2-year flexural limit state of bridge 1-791, the required mean resistance $\mu_R' =$ 907.8 με. If the girder's actual resistance is higher than this required resistance, this girder is safe. Otherwise it is unsafe. At the same time, the design point is found to be, r* = 790.1 με, d* = 100 με, l^* = 690.2 με (Nowak et al., 2000). The corresponding load and resistance factors for 2-year inspection period based on bridge 1-791 in-service data are following.

$$\phi = \frac{r^*}{R_n'} = \lambda \frac{r^*}{\mu_R'} = 1.12 \frac{790.1}{907.8} = 0.97$$

$$\gamma_D = \frac{d^*}{D_n} = \frac{100}{96} = 1.04$$

$$\gamma_L = \frac{l^*}{L_n} = \frac{690.2}{475.7} = 1.45$$

And the corresponding RF is below

$$RF = \frac{\phi R_n - \gamma_D D_n}{\gamma_L L_n} = \frac{0.97 \times 1440 - 1.04 \times 96}{1.45 \times 475.7} = 1.88 \qquad (8)$$

CONCLUSIONS

Based on the site-specific bridge response data, the extreme live load distribution is found to be Gumbel. The use of site-specific bridge response data (strain) allows us to eliminate a substantial portion of live load modeling uncertainty (i.e., site to site variation, dynamic impact uncertainty), as well as a substantial portion of bridge analysis modeling error (i.e., transverse distribution uncertainty). So we can more accurately evaluate our bridges, and provide a uniform level of safety.

ACKNOWLEDGMENTS

The authors would like to thank the Delaware Department of Transportation for their support of this work, with special thanks to Mr. Dennis O'Shea.

APPENDIX

A sequence of observations, $y_1, y_2, ..., y_n$, is said to have a *Turning Point* at *i*, $1 < i < n$, if (i) $y_{i-1} < y_i$ and $y_i > y_{i+1}$, or (ii) $y_{i-1} > y_i$ and $y_i < y_{i+1}$. If the data constitute an iid sample, then the probability of y_i, $1 < i < n$, being a turing point is 2/3. It can be shown that, *T*, the random number of turning points in an iid sample approaches the Normal distribution with mean,

$\mu = 2(n-2)/3$, and variance, $\sigma^2 = (16n-29)/90$. Hence the null hypothesis that the sample is iid can be rejected at significance α if $\Phi(|t-\mu|/\sigma) > 1-\alpha/2$, where Φ is the Normal distribution function and t is the observed number of turning points.

The *Difference-Sign Test* counts the number of points at which the above series has an increment, i.e., $y_i < y_{i+1}$, i = 2, …, n. It can be shown that, S, the random number of points with a positive increment in an iid sample approaches the Normal distribution with mean, $\mu = (n-1)/2$, and variance, $\sigma^2 = (n+1)/12$. The null hypothesis that the sample is iid can be evaluated in the same way as above.

The *Rank Correlation Test* counts the number, P, of pairs (i, j), $j > i$, such that $y_j > y_i$. There are $n(n-1)/2$ pairs (i, j) for which $j > i$. If the sequence $y_1, y_2, ..., y_n$ is iid then the probability of $\{y_j > y_i\}$ is ½. It can be shown that P approaches the Normal distribution with mean $n(n-1)/4$ and variance $n(n-1)(2n+5)/8$. The null hypothesis that the sample is iid can be evaluated in the same way as above.

In each of the above three tests, if there are consecutive values in the sample that are identical, then they should be merged and considered as only one sample point. These tests are described in the book: Brockwell and Davis (1991).

REFERENCES

AASHTO Standard Specifications For Highway Bridges (1992), Washington, D.C.

Bakht, B. and Jaeger, L.G. (1992), “Ultimate Load Test of Slab-on-Girder Bridge”, Journal of Structural Engineering, Vol.118, No.6.

Bhattacharya, B., Li, D., Chajes, M., and Hastings, J. (in review, 2004), “Reliability-based Load and Resistance Factor Rating Using In-Service Data”, Journal of Bridge Engineering.

Brockwell, P. J. and Davis, R. A. (1991). Time Series: Theory and Methods, 2nd Ed., Springer.

Castillo, E. (1988), "Extreme value theory in engineering", Academic Press, Inc.

Holloway, E.S. (1999), “A Long-Term Monitoring System for Highway Bridges”, Master Thesis, University of Delaware.

Li, D. (2004), “Reliability-Based Load & Resistance Factor Rating Of Bridges Using Site Specific Data”, Doctoral Dissertation, University of Delaware.

NCHRP report 368 (1999), "Calibration of LRFD Bridge Design Codes", TRB, National Research Council, Washington, D.C.

Nowak, A.S. and Collins, R.K. (2000), "Reliability of Structures", the McGraw-Hill companies, Inc.

Pickands, J. (1975), "Statistical inference using order statistics", Annals of Statistics, Vol. 3, pp 119-131.

Rackwitz, R., and Fiessler, B. (1978), “Structural Reliability under Combined Random Load Sequences”, Computers & Structures, 9, pp 489-494.

Reid, J.S. (1996), “Bridge Evaluation and Long-term Monitoring”, Master Thesis, University of Delaware.