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► To cite this version:

Mariia Nesterova, Marcel Nowak, Franziska Schmidt, Oliver Fischer. Reliability of a bridge with an orthotropic deck exposed to extreme traffic events. The 11th International Conference on Mathematical Methods in Reliability, Jun 2019, HONG KONG, China. 8 p. hal-02316778v1

HAL Id: hal-02316778

<https://hal.science/hal-02316778v1>

Submitted on 15 Oct 2019 (v1), last revised 13 Apr 2021 (v4)

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RELIABILITY OF A BRIDGE WITH AN ORTHOTROPIC DECK EXPOSED TO EXTREME TRAFFIC EVENTS

MARIIA NESTEROVA*

*Université Paris-Est, MAST, EMGCU, IFSTTAR, F-77447 Marne-la-Vallée, France,
email: mariia.nesterova@ifsttar.fr*

MARCEL NOWAK

*Technical University of Munich, Chair of Concrete and Masonry Structures, 80333
München, Germany, email: marcel.nowak@tum.de*

FRANZISKA SCHMIDT

*Université Paris-Est, MAST, EMGCU, IFSTTAR, F-77447 Marne-la-Vallée, France,
email: franziska.schmidt@ifsttar.fr*

OLIVER FISCHER

*Technical University of Munich, Chair of Concrete and Masonry Structures, 80333
München, Germany, email: oliver.fischer@tum.de*

Predicting reliability levels for critical details of bridges based on limited statistical traffic data is a relevant topic nowadays. That is why the comparison between results from various statistical approaches based on the recorded data for applied traffic actions is the main point of interest of this work. The object of the current study is the famous Millau viaduct, a cable-stayed bridge with the steel orthotropic deck located in Southern France. Values of load effects that are used in analysis are derived from a finite element model of a part of the deck. They are based on data from traffic monitoring that is provided from the bridge Weigh-In-Motion system covering several months of axle loads, distances and speeds of heavy trucks. The methodology is based on a definition of limit state functions based on several statistical distributions in order to assess and compare reliability indexes for the ultimate limit state. It includes a comparison between different approaches of extreme values theory, the methodology proposed in background works for European standards and the design load model. Moreover, this work covers the influence of applied loads of a high amplitude, as global effects, onto stresses from axle loads, as local effects.

Keywords: Millau viaduct, orthotropic deck, extreme load effects, reliability, traffic actions

*Corresponding Author

1. Introduction

The European road infrastructure includes a significant number of bridges with orthotropic decks, which are exposed to heavy traffic loads and unfavorable environmental influence. In order to estimate if a structure operates safely, such bridges need to be assessed on a regular basis to verify the original design life. This can be done by reliability analysis (e.g. reliability index at the end of the operating life) of the most critical detail of a structure.

Various methods for reliability analysis exist nowadays, each having advantages and drawbacks. A correct choice of the most appropriate reliability method depends on the studied structure, input data, initial conditions, and computational sources [1]. In bridge engineering, the most frequently used are First and Second Order Reliability Methods (FORM and SORM) described, for instance, by D. Straub [2].

The main interest of the current study is in comparison of predicted reliability levels at the end of design life based on a limit state function (LSF) defined with various approaches of the extreme values theory (EVT) [3]. In a recent study [4] of various LSF, one of the conclusions shows that not only fatigue but also extreme values and serviceability have to be considered in the reliability analysis of bridges.

Such as for bridges with steel orthotropic decks, critical details of a deck have a shorter life than other structural elements of a bridge [5]. The current study is focused on details of the orthotropic deck of the Millau viaduct obtained using the extreme values approaches: Peaks Over Threshold (POT), Block Maximum (BM) and level crossing counting (LCC). The stresses are derived from the FE model of a part of the deck and based on 180 days of monitored by bridge Weigh-in-Motion (BWIM) traffic data.

2. Methodology

To assess the reliability of a detail, first of all, the LSF $G(x)$ is needed. It is based on one of several approaches of the EVT and is expressed as:

$$G(x) = R - L(x) > 0, \quad (1)$$

where $L(x)$ represents the probabilistic model of a load effect – equivalent stress in the critical welding between a rib and the deck plate – and $R = F_u$ is the probabilistic model of the material resistance. R is represented here by the ultimate strength of steel, described by a log-normally distributed random variable with coefficient of variance (CoV) $CoV = 0.05$.

For each described case, the reliability index β is found for the corresponding probability of failure P_f , Eq. (2) with the probability density function $f_x(x)$ of the load effect x and the cumulative distribution function (CDF) of the standardized Normal distribution Φ^{-1} (3).

$$\beta = -\Phi^{-1}(P_f) = -\Phi^{-1}\left(\int_{G(x)\leq 0} f_x(x)dx\right) \quad (2)$$

$$\Phi(x) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^x e^{-x^2/2} dx \quad (3)$$

2.1. POT approach

The POT approach has been recently proved to be a good solution for predictions of extreme traffic actions [6]. As a time-series process, "peak" values of load effect (LE), that lay above a certain threshold, are fitted to the generalized Pareto distribution (GPD). It was recently concluded [7] that the efficiency of this method is high enough under the assumption of normal distribution of return level estimators.

Let X be the sequence of n random variables X_i representing values of LE with the distribution function F . Let Y be the sequence of m random variables Y_j representing exceedance of a threshold u and defined by $Y = X - u$, for every threshold excess $X_j \in X$ so that $X_j > u$. A few assumptions for the application of the EVT include identical distribution and statistical independence of random variables X_i .

According to the general principle of the POT approach [8], the CDF of threshold exceedances tends to the upper tail of GPD (4), with shape and scale ($\sigma > 0$ and ξ). The approach is valid only if $Y_j = X_j - u \geq 0$, $X_j \geq u$ for $\xi \geq 0$ and $u \leq Y_j \leq u - \sigma/\xi$ for $\xi < 0$.

$$G(Y_j; \xi; \sigma; u) = \begin{cases} (1 - [1 + \xi(\frac{Y_j}{\sigma})]^{-1/\xi}), & \xi \neq 0 \\ 1 - \exp(-\frac{Y_j}{\sigma}), & \xi = 0 \end{cases} \quad (4)$$

For a long period, estimates can be based on the CDF of extreme values from a shorter period [9]. In other words, monitored values of a LE for a limited period can be used for extrapolation of the LE in time. The value of the p -observation return level $L_r(p)$ for the probability $P[X \leq X_i | X_i > u]$ with the probability of exceedance (PE) $\zeta_u = P\{X_i > u\}$, is a quantile that exceeds once every p observations. The procedure for selecting an optimized threshold is described

earlier by author [10]. Therefore, for a fixed period p , the LSF (1) based on the GPD takes the following form:

$$R > L_r(u, \sigma, \xi, \zeta_u) = \begin{cases} u + \frac{\sigma}{\xi} [(p\zeta_u)^{-\xi} - 1], & \xi \neq 0 \\ u + \sigma \log(p\zeta_u), & \xi = 0 \end{cases} \quad (5)$$

2.2. BM method

The BM method is based on a limiting theorem stating that the generalized Extreme Values distribution (GEVD) is an approximation for describing sample maxima for large sample sizes. This holds for the condition of the BM being (approximately) independent observations with identical distributions [3]. A BM series is generated by separating the time history for load effects X into intervals of a chosen size and determining the maxima in each interval. Different modifications of BM method are available to handle inhomogeneities in the data-set, aiming at approximately obtaining idealized conditions of independent and identically distributed observations in a remaining subset for further model fit.

One such approach [11] proposes a composite distribution model that describes bridge load effects from a mixture of different loading event types, representing different number of trucks on the bridge contributing to a maximum value of the LE. Such a model accounts for the actual physical nature of the bridge loading process, and hence allows for a higher accuracy in extreme value estimates, as also shown in a recent study [12]. Based on separate GEVD fits $G_j(z)$ with model parameters $[\mu, \sigma, \xi]$ to the corresponding BM series for each of the different loading event types, the distribution of load effects $G_C(z)$ resulting from the mixture of these loading event types asymptotically approaches the following composite distribution:

$$G_C(z) = \prod_{j=1}^N G_j(z) = \exp \left\{ - \sum_{j=1}^N \left[1 + \xi_j \left(\frac{z - \mu_j}{\sigma_j} \right) \right]^{-1/\xi_j} \right\} \quad (6)$$

Due to the small influence area of the considered load effect it was observed that the main contribution for extreme values of LE results from a single vehicle. Hence, there is no need for further classification for this study. The LSF (1) based on the BM takes the following form (with $T_{ref} = 5d$ and R_t to be inserted in $[d]$):

$$R > L_r(\mu, \sigma, \xi) = \begin{cases} \mu - \frac{\sigma}{\xi} [1 - (T_{ref}/R_t)^{-\xi}], & \xi \neq 0 \\ \mu - \sigma \log(T_{ref}/R_t), & \xi = 0 \end{cases} \quad (7)$$

2.3. Rice formula

The LCC method, where Rices formula is fitted to the upper tail of an out-crossing rate histogram (ORH) is based on the assumption of a stationary Gaussian process describing the time variations of load effects on bridges [13]. The number of times is counted at which positive values are crossed upwardly in a LE time history. By normalizing the resulting level crossing histogram with respect to the time history length, the ORH is obtained, representing for each level the mean rate of its crossing during a reference period T_{ref} . The Rices formula with its model parameters [$a_0 = \ln(\nu_0) - m^2/2\sigma^2$, $a_1 = m/\sigma^2$, $a_2 = -1/2\sigma^2$] describes the mean rate $\nu(x)$ of up-crossing for a certain level during T_{ref} , and is fitted to the significant tail regions to the ORH. The proper choice of the starting point for the fit with Rices formula is crucial. It should be as low as possible, to ensure sufficient representativeness for statistical extrapolation, but not too low to still provide a reasonable approximation of the significant tail region. In this study, optimal starting points are identified with evaluating the goodness of fit by means of a modified Kolmogorov test [13]. The LSF (1) based on the LCC takes the following form:

$$R > L_r(R_t, a_0, a_1, a_2) = -\frac{a_1}{2a_2} + \sqrt{-\frac{a_0}{a_2} + \left(\frac{a_1}{2a_2}\right)^2 - \frac{\ln(R_t/T_{ref})}{a_2}} \quad (8)$$

3. Application to the Millau viaduct

3.1. Instrumentation

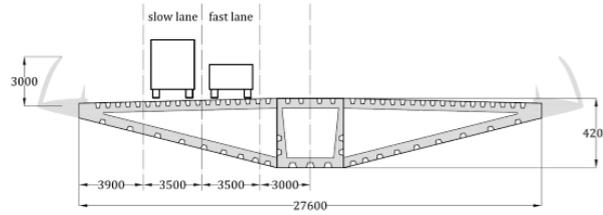


Figure 1. Cross section of the deck of the Millau Viaduct

Millau viaduct is a cable-stayed bridge that consists of 8 spans with a total length of 2460 m. The stiff orthotropic steel deck is suspended by 11 steel cables at each span. Each span of 342 m is composed of four or six meters long elements, each of them acting almost as a simply supported beam. The cross-section of the

deck, Figure 1 [14], shows that the viaduct has two lanes in each direction: (i) slow lane, where traffic is mainly composed of heavy vehicles, (ii) fast lane, that is used by lorries only to overtake.

The data on traffic actions were provided from BWIM system that was located in the middle of the first span of the viaduct. It includes axle weights [kN] and spacing [m], vehicle speed [m/s], axles configuration, pavement temperature. Recordings were made between October 2016 and June 2017 with a total 180 days of recorded traffic.

3.2. Effects of traffic actions

For the current study, the critical detail is chosen to be the welded connection between a longitudinal stiffener and the plate of the orthotropic deck that is located right under a truck wheel assuming the vehicle is passing in the middle of the slow lane.

The values of stresses that occurred in this connection were computed with a detailed finite element model (FEM) of the part of the deck. The model takes into account the self-weight of the bridge deck with its asphalt layer, the type of passing vehicle and amplitudes of axle loads from each axle, as well it considers the influence of vehicles queues and the wind [10].

The summary of random variables for each LSF based on described approaches is given in Table 1.

Table 1. Random Variables for reliability modeling

Case	Random variable		Distribution	Mean	CoV
Resistance	steel strength	F_u	Log-Normal	400 MPa	0.05
POT	threshold	u	Normal	102	0.05
	shape	ξ	Normal	-0.3	0.12
	scale	σ	Log-Normal	25.35	0.06
	PE	ζ_u	Bi-nominal	8.2×10^{-4}	0.045
BM	location	μ	Normal	100.07	0.059
	scale	σ	Normal	23.57	0.198
	shape	ξ	Normal	-1.36×10^{-2}	17.503
LCC	parameter	a_0	Normal	8.23	0.028
	parameter	a_1	Normal	-1.03×10^{-2}	0.798
	parameter	a_2	Normal	-8.06×10^{-4}	0.087

3.3. Comparison between various statistical approaches

In order to perform the reliability analysis, limit state functions are defined for every method described in Section 2, and then used in the software UQLab to

obtain reliability indexes with SORM method, based on values of load effects described in Section 3. Table 2 gives results for three time references: one year, 50 and 120 years. First three columns show comparison of three extreme values approaches, the reliability index based on design load model is shown in the fifth column, and the last column reflects minimum values of β according to European standards [15].

Table 2. Reliability index β for different studied cases

β , reference	POT	BM	LCC	EN, LM1 [16]	β , EN [15],
2017	9.8	19.7	9.6	-	4.7
50 years	7.6	12.5	8.0	8.0	3.8
120 years	7.4	11.4	7.7	-	-

4. Discussion of results

This work has been carried out on the monitored traffic data from the Millau viaduct. Three extreme value approaches have been performed in order to assess the reliability of the connection between a longitudinal stiffener and the steel plate of the orthotropic deck. The values of reliability indexes are found for the year of monitoring, the 50 years reference period and at the end of the promised operational life of the viaduct.

The results show that during the operational life, the reliability of the most critical detail of the deck is higher than the minimum value given in standards. POT approach and Rice formula show similar results that are also close to results obtained with the design load model (LM1) of standards. The reliability obtained with BM method is much higher, which could be due to the choice of a block size and high variance of distribution parameters, and should be further investigated. The choice of an appropriate method depends on available tools and on the reference data-set. For robust results, reliability levels from several methods should be compared.

Acknowledgments

This project has received funding from the European Union's Horizon 2020 research and innovation programme under the Marie Skłodowska-Curie grant agreement No 676139. The grant is gratefully acknowledged.

The BWIM measurements for the Millau viaduct were obtained with the help from CESTEL, Slovenia and EIFFAGE, France, which is highly appreciated.

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