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Subject review

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Modelling of traffic load effects in the assessment of existing road bridges

Traffic load models used for the design of new bridges are based on conservative assumptions and have not been proven efficient for assessing safety of existing bridges. In the case of existing bridges, it is reasonable to use load models that are based on bridge weigh-in-motion data which, in addition to axle loads and spacing of bridge-crossing vehicles, provide information on bridge behaviour under traffic load. This paper provides an overview of traffic load models, as well as guidelines on the use of weigh-in-motion data when assessing condition of existing road bridges.

Key words:

traffic load, weigh-in-motion analysis, B-WIM, condition assessment, existing bridges

Pregledni rad

Dominik Skokandić, Ana Mandić Ivanković, Aleš Žnidarič, Mladen Srbić

Modeliranje utjecaja prometnoga opterećenja u postupku ocjenjivanja postojećih cestovnih mostova

Modeli prometnoga opterećenja koji se primjenjuju pri proračunu novih mostova temelje se na konzervativnim pretpostavkama te se njihova upotreba u analizi postojećih mostova nije pokazala učinkovitom. Kod postojećih mostova preporučena je primjena modela prometnoga opterećenja temeljenog na mjerenju prometa u pokretu, koji osim osovinskoga opterećenja i razmaka, daje i podatke o ponašanju mosta pod prometnim opterećenjem. U ovome su radu dani pregled modela prometnoga opterećenja te smjernice o upotrebi podataka dobivenih mjerenjem prometa u pokretu prilikom ocjene stanja postojećih cestovnih mostova.

Ključne riječi:

prometno opterećenje, mjerenje prometa u pokretu, B-WIM, ocjena stanja, postojeći mostovi

Übersichtsarbeit

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Modellierung der Auswirkungen der Verkehrsbelastung bei der Bewertung vorhandener Straßenbrücken

Die zur Berechnung neuer Brücken verwendeten Verkehrslastmodelle basieren auf konservativen Annahmen, und ihre Verwendung bei der Analyse bestehender Brücken hat sich nicht als wirksam erwiesen. Für vorhandene Brücken wird die Verwendung eines Verkehrslastmodells empfohlen, das auf der Messung des Verkehrs basiert und neben der Achslast und dem Achsabstand Informationen zum Verhalten der Brücke unter Verkehrslast liefert. Diese Arbeit bietet einen Überblick über das Verkehrslastmodell und Leitlinien zur Verwendung von Daten aus der Messung des Verkehrs bei der Beurteilung des Zustands vorhandener Straßenbrücken.

Schlüsselwörter:

Verkehrslast, Messung des Verkehrs, B-WIM, Beurteilung des Zustands, vorhandene Brücken

1. Introduction

The assessment of existing structures, especially of road and railway bridges, constitutes a growing challenge for civil engineers due to important role of such bridges within regional and global infrastructure networks. One of main steps in the process of bridge assessment is the determination of total load effects, where the most variable loads are those induced by traffic [1]. Current design codes for new bridges [2, 3] are based on conservative assumptions in order to create safe and cost-effective new bridges, taking into account future traffic growth and reduction of their capacity over lifetime. This level of conservatism has little impact on the cost of design of new bridges, but can prove economically inefficient in the assessment of existing ones, as application of these traffic load models may show that majority of existing bridges need to be strengthened or even replaced [1, 4]. Recent research has proven that application of site-specific traffic models, obtained from monitoring data, can reveal hidden bridge reserves in terms of load carrying capacity, leading to reduction of maintenance costs and extension of remaining service life of bridges [5-7].

Basic approach in the development of traffic load models is to collect a certain amount of traffic data that include axle load and axle spacing, adjust such data to a statistical distribution, and extrapolate the distribution to estimate maximum load effects [7]. The most widely accepted procedure for the collection of realistic traffic data is the weigh-in-motion (WIM) procedure. It is defined as a system that measures axle load and gross weight data for all vehicles that drive over a measurement site, without the need for slowing down or stopping [8, 9].

Two types of these systems can be differentiated. The pavement WIM type applies bar or plate sensors that are built into the wearing course of the pavement and are in direct contact with the wheels. The portable bridge weigh-in-motion (B-WIM) type weighs vehicles with the existing instrumented bridges [8]. WIM technology has been used in Croatia for the last two decades. Two stationary pavement WIM sites were initially installed. The Croatian National Road Directorate has been collecting traffic data on various locations using B-WIM technology since 2005 [10].

Traffic load models for the design of new bridges [2] were developed using a limited amount of WIM data collected on European highways in the 1980s. Practical application of these design load models in the assessment of older bridges has proven to be inefficient. On the other hand, site-specific traffic load models, calibrated with B-WIM data, along with other applications, provide quality input for an optimized assessment of existing bridges [11-13].

This paper discusses the development of traffic load models and design codes over the years and gives a short overview

of calibration of load models in current design codes for new bridges. Furthermore, methods for development of site-specific traffic load models obtained from B-WIM data are presented. The aim of the paper is to provide guidelines for engineers dealing with modelling of traffic loads, and thus to assist them in the assessment of existing road bridges.

2. Traffic load models – overview

Traffic loading for road bridges can be divided to congested traffic, basically a traffic jam situation, and free flow traffic, involving a steady traffic flow of 60-100 km/h. Furthermore, from the engineering point of view, traffic load is divided into static and dynamic components [14]. Most of the current design codes have the dynamic part already integrated in the specified load models, but this was not always the case.

First engineering discussions regarding bridge loading and the analysis of additional impacts due to dynamic characteristics were initiated in the second part of the 19th century [15], first with moving load experiments on a beam. In our region, the first bridge code was published in 1904 [16].

Sudden increase in the number of heavy vehicles (armoured military vehicles) due to the start of World War 1 caused first acts and codes on bridge loading to be published in the UK in the 1918 [15], while in the USA first codes were published in the 1924 [17]. Over the following years, traffic load models were introduced in design codes in the majority of European countries, based on relevant engineering experience and research studies. A detailed historic overview of traffic load models for the UK and Europe can be found in the book by Dawe [15] and dissertation by Carey [14], while development of AASHTO [18] design codes in the USA is described by Kulicki and Mertz [17]. An overview and development of current design codes in the EU countries is given by Prat [19], while background studies are given in the report by Bruls et al. [20].

Traffic loads are represented in the majority of modern design codes by models based on realistic traffic data, which differ from site to site, and hence national codes also differ. Comparison of national bridge design codes was conducted by Matar et al. [21] who concluded that AASHTO [18] codes in general give lower design effects compared to Eurocodes [2]. In Croatia, design codes PTP-5 [22], with first traffic load models for the design of new bridges, were introduced after the Second World War in 1949 based on the guidelines from 1933 [23]. PTP-5 standard traffic load model consisted of four two-axle vehicles (concentrated loads), placed in two neighbouring traffic lanes, including also the corresponding continuous load distributed over the whole section of the bridge. Furthermore, both loads concentrated and distributed in the main traffic lane were magnified by the dynamic factor k_d , which changed as a function of bridge span. In addition to the standard traffic load model, PTP-5

also defined two special military vehicles, which had to be considered in the design of new bridges. The first special military vehicle M-25 consisted of seven concentrated loads with the total load of 840 kN, while the weight of the second crawler armoured vehicle, i.e. 600 kN, was distributed on two rails 5 m in length. After 1973, a new design code, based on German DIN 1072 [24], was published in Croatia, defining the new traffic load models (SLW 60 and SLW 30 and their combinations) composed of two three-axle vehicles, 600 kN and/or 300 kN in total load, along with the load distributed on the remaining area of the bridge. Similar to the previous codes, the main traffic lane and the concentrated loads were magnified by the dynamic factor, using a different formula than in PTP-5. Another difference was that the distributed load in the first lane was higher than on the remaining area of the bridge.

In the period between 1973 and 2002 there was a significant increase in the amount of daily traffic on Croatian roads and highways [25], resulting in revision of design codes and acceptance of European pre-standards ENV 1991-3 [26], which defined new traffic load models, similar to those used in the final draft of Eurocode [2], but with reduced adjustment factors. Finally, in 2012, after the transition of Eurocodes from ENV to EN standards, Croatian design code HRN EN 1991-2 [27] was officially published, prescribing the current

traffic load models for new bridges in Croatia, as described in the following section.

A more detailed historical review of development of national design codes and traffic load models in Croatia can be found in the doctoral thesis by Mandić Ivanković [28], while comparison of load effects of different codes is presented on Figure 1, and can be found in the works by Mandić Ivanković and Radić [25], and Šavor et al. [29].

3. Traffic load models - current design codes

3.1. Overview

European code EN 1991-2:2003 [2] defines imposed loads, both models and representative values, associated with road traffic, which include dynamic effects, centrifugal, braking and acceleration actions to be used for the design of new bridges. The bases for definition of EN 1991-2 traffic load models were established in the late 1980s [20, 30] according to the traffic measured on several European highways. Due to the fact that these traffic records are more than 30 years old, several authors reassessed these load models based on updated traffic monitoring data, and they concluded that these models comply with requirements of modern traffic [31, 32].

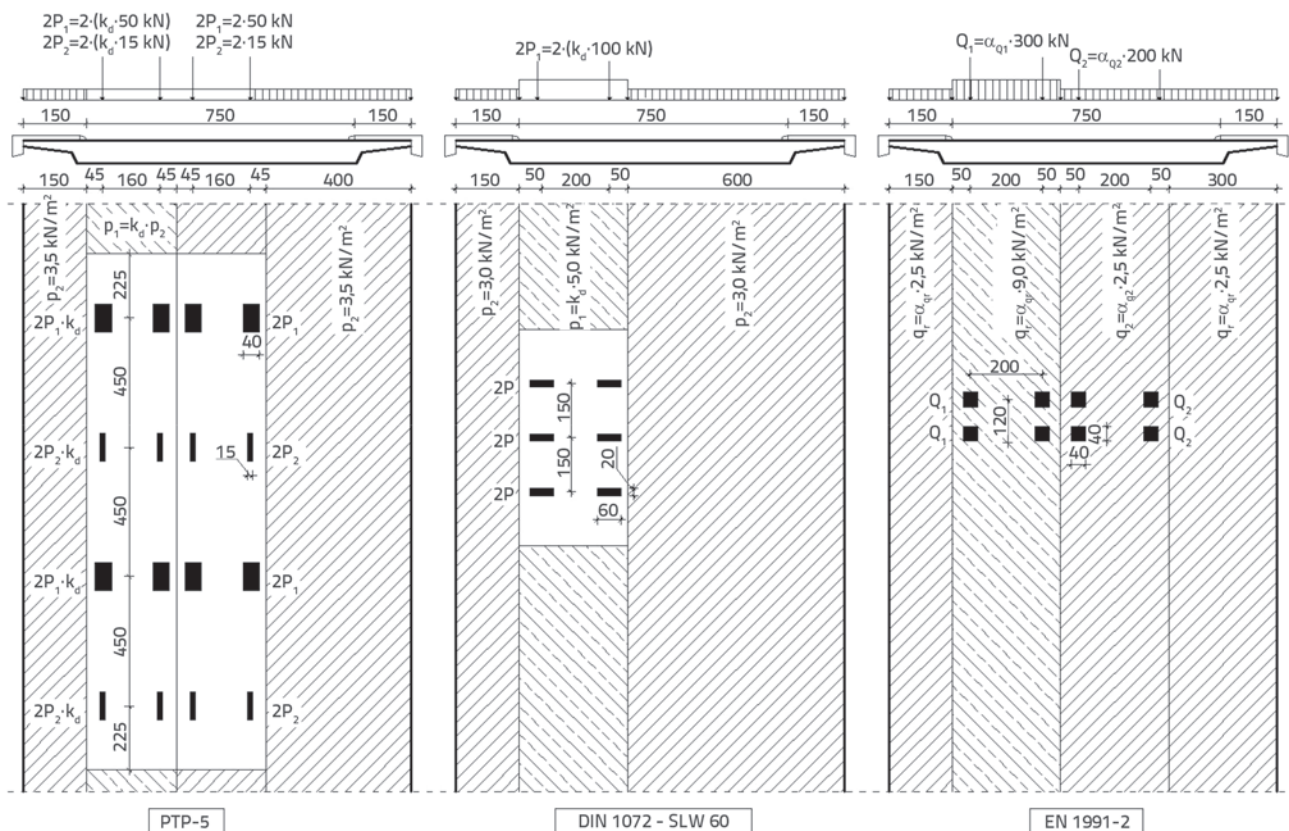


Figure 1. Comparison of new and previous traffic load models in Croatia a) PTP-5 b) DIN 1072 – SLW60 c) EN 1991-2

There are in total four load models for Ultimate Limit State (ULS) and Serviceability Limit State (SLS) verification, along with additional five load models for fatigue verifications. Vertical loads in the defined models are distributed in traffic lanes, whose number depends on the total width of the deck [2].

The most commonly used traffic load model for ULS and SLS verifications on majority of new bridges is the Load Model 1 (LM1), which is composed of two subsystems, the concentrated axle loads (tandem system TS), and the uniformly distributed loads (UDL). Graphic comparison of LM1 and design codes previously used in Croatia is given in Figure 1.

Characteristic values of LM1 for both concentrated and distributed loads are defined in [2], along with adjustment factors $\alpha_{Q1,r}$, α_{Q1} , and $\alpha_{Q1,r}$ (Figure 1c), which can be used for correction of total traffic loads, depending on the road category and expected traffic density and weight. Furthermore, these corrections can be used in the assessment of existing bridges, as explained in the next section.

Basic values of adjustment factors are given in National Annexes, or, if not specifically indicated, are taken to be equal to 1. Most EU Countries, including Croatia, have adopted the factor of 1,0 for the design of new bridges. Some countries, such as Denmark, Germany, France and the UK, recommend different values [32]. For example, the increased adjustment factors in Germany are based on the research by Maurer et al. [33] and Freundt et al. [34], in which a considerable increase in road traffic in Germany over the last two decades is reported. Further studies on adjustment factors were conducted by O'Brien et al. [35]. The latter authors have defined the technique for the determination of these factors for selected bridges, road sections, or even for the entire road network. A review of further research, along with the analysis and summary of adjustment factors for several European countries (Table 1), are given in [32]. One of the issues arising from the lack of uniformity between individual correction factors is the impossibility to design a bridge that could sustain trans-European road traffic with the same level of safety [32, 35].

Table 1. Adjustment factors for design of new bridges in some EU countries

EU member country	Adjustment factor				
	α_{Q1}	α_{Q1}	α_{Q1}	α_{Q1}	α_{Q1}
	$i=1$	$i>1$	$i=1$	$i=2$	$i>2$
Croatia [27] (and most EU countries)	1.00	1.00	1.00	1.00	1.00
Denmark [32]	1.00	1.00	0.67	1.00	1.00
France [32]	1.00	1.00	1.00	1.20	1.20
Germany [32]	1.00	1.00	1.33	2.40	1.20
United Kingdom [32]	1.00	1.00	0.61	2.20	2.20
Netherlands [36]	1.00	1.00	1.15	1.40	1.40

3.1.1. Application of adjustment factors in the assessment of existing bridges

Reduction of adjustment factors values, based on the traffic data obtained with on-site measurements, can provide a first step to site-specific assessment of existing road bridges [37]. For example, although there are no official guidelines for the assessment in Croatia, research by Mandić Ivanković et al. resulted in a reduced adjustment factor values based on the real traffic measurements [38]. Furthermore, in some European countries reduced correction factors are an integral part of official standards for assessing condition of existing structures. A good example is the Swiss national guideline SIA 269/1:2011 [39]. Unlike adjustment factors for the design of new bridges, correction values for condition assessment are defined depending on bridge length, but also on its cross section (box, slab, etc.) and, in some cases, on road category (highway, state road, etc.). Leahy et al. [37] conducted a research defining guidelines for the determination of reduced adjustment factors based on real motorway traffic measurements in the Netherlands, as compared to the traffic load model 1 from standard [2]. Similar research was conducted by O'Brien and Enright [40],

Table 2. Reduced adjustment factors for assessment of state road bridges in Croatia – values recommended in [38]

Span [m]		≤ 10	10 – 20	20 – 30	30 – 40	40 – 50
Simply supported bridge $\alpha_{q2} = \alpha_{qr} = 1.0$	α_{Q1}	0.80	0.80	0.80	0.80	0.80
	$\alpha_{Q1}; \alpha_{Q2}; \alpha_{q1}$	0.30	0.38	0.51	0.58	0.62
Continuous bridge $\alpha_{q2} = \alpha_{qr} = 1.0$	α_{Q1}	0.80	0.80	0.80	0.80	0.80
	$\alpha_{Q1}; \alpha_{Q2}; \alpha_{q1}$	0.48	0.72	0.78	0.81	0.82

Table 3. Reduced adjustment factors for assessment of existing bridges in Switzerland – values specified in [39]

Bridge type		Span [m]	α_{Q1}	α_{Q2}	$\alpha_{qt}; \alpha_{qr}$
Beam bridges	Box bridge	20 – 80	0.70	0.50	0.50
	Two webs	20 – 80			0.40
	More webs	15 – 35			
Slab bridges		10 – 30	0.80	0.53	0.40
Slab and other bridge types		5.3 – 10	0.60	0.40	0.40
		< 5.3	0.50	0.40	0.40

who calculated the same ratio for different sites in European countries. Additionally, the ratio of traffic load effects for in Slovenia is presented in [6] a case-study bridge.

Examples of reduced adjustment factors for road bridges, based on research of traffic conducted on Croatian roads [38], are given in Table 2, and an example of Swiss national codes [39] for assessment of existing bridges is given in Table 3.

3.2. Development of Load Model 1

Calibration of traffic load models for road bridges began in the late 1980s [20] based on the data collected on several European motorways between 1977 and 1987. The LM1 development process is divided into several steps, which are briefly described in the following text. A more detailed review is presented in [20, 30, 41].

The first step in the development of load models for bridges is to collect traffic data, i.e. the axle load, axle spacing, and inter-vehicle distance, for all vehicles on a representative road section. This can only be accomplished with weigh-in-motion (WIM) systems that capture this information at normal highway speed. At the time of LM1 development this type of measurement was not common and the measuring devices did not provide reliable results. Thus, the first idea to combine the available WIM records in an “European sample” [20] did not prove feasible, as extrapolation methods, required in the modelling of load effects, require homogenous data for load model calibration. Vehicle-weight data differed considerably depending on the location, but the daily maximum and axle spacing data were relatively similar [41]. As a result, a single traffic sample, collected on the motorway A6 near French city of Auxerre, was selected for further research.

Although not characterized by the heaviest traffic load compared to other available samples, it was estimated that the traffic sample of Auxerre still represents a relatively great traffic load for a single traffic lane. A detailed statistical analysis of the Auxerre traffic sample can be found in Appendix A of [41].

Complex studies involving simulation of realistic traffic situations [20, 30] resulted in two main traffic flow

conditions: “free-flow” and “congested flow”, to be combined in adjacent lanes for traffic model definition purposes. The main difference was in the recorded speed and inter-vehicle spacing, which was minimized to simulate traffic jams [31]. The next step involved evaluation of traffic load effects, i.e. calculation of bending moments and shear forces, from the recorded vehicle and axle weights. A set of influence lines and areas, in total nine of them for simply supported and continuous bridges, was selected for calibration, together with a set of different bridge spans, ranging from 5 to 200 meters.

Evaluation of target values for traffic loads was conducted based on extrapolation methods and prefixed return periods, using the histograms of load effects [30]. The return period had to be selected due to the fact that traffic data were registered in a relatively short time interval. Assuming the uniform flow of heavy vehicles, and that their weights are independent, the return period for an unlimited number of vehicles can be defined with Eq. (1), [30]:

$$R = R_{y\alpha} \cong \frac{-T}{\ln(1-\alpha)} \cong \frac{T}{\alpha} \quad (1)$$

where (R) is the return period for the corresponding design life (T) and fractile (α). For example, if the design life is 50 years, there is a 5% chance of exceedance of target values in the return period of 1000 years.

For the evaluation of extreme axle weight values according to EN 1991-2, and the corresponding traffic load effects, three different extrapolation methods were used [30]:

- Half-normal distribution – where the upper tail of extreme values is approximated with a normal distribution;
- Gumbel distribution – similar to half-normal distribution, but the extreme values are represented by the two-parameter Gumbel (type 1) distribution;
- Monte Carlo (MC) simulation – method that generates the greatest possible number of loading scenarios based on statistical properties of recorded traffic. If the number of simulations is large enough, no extrapolation is required for estimation of maximum traffic load effects.

The next step was evaluation of dynamic effects, due to the bridge-vehicle interaction, which are taken into account by multiplying the extrapolated static load effects with dynamic amplification factors [30]. Dynamic effects due to vehicle passage depend on a number of parameters, such as the structural type, span length, vehicle type, speed and weight, pavement condition, in particular its unevenness, etc.

A number of numerical simulations for various bridge types, pavement condition etc. were carried out in studies [20], [30] made before development of the standard, to establish the "impact factor", defined as a ratio between the dynamic and static values of the same fractile of the load effects. Recommended dynamic factor values are given in [41] with respect to the load effect, number of lanes, and bridge span. Beside characteristic values of traffic load, presented in Figure 1.c, other relevant values, to be used mainly in serviceability limit state verifications, were defined: infrequent, frequent and quasi-permanent. Characteristic values for the 1000-year return period allowed for increase in traffic load volume and weight in the future. In cases when values for one year reference period are required, it is allowed to use approximation and reduce 1000 year characteristic values with a reduction factor of 0,800 [41].

3.3. Reassessment of Load Model 1

In the late 1990s, the draft of the Eurocode was being prepared for conversion from ENV to EN status, and researchers emphasised the need for re-assessment of calibrated load models due to both traffic growth in the intervening period, and recent developments of WIM technologies [31]. A research group led by O'Connor [42] initiated the re-assessment process in 1998, with the study divided in two major parts, reassessment of the original calibration [20] with the original Auxerre traffic, and calculation of target values specified for LM 1 using the recently recorded traffic data from several European roads, labelled "Modern Traffic". Significant reduction in variation of traffic data was noted, which was labelled sufficient to compensate for traffic growth and increase in gross weight legal restrictions in EU. Detailed procedure can be found in preliminary report [42] and final report [31].

More recently, research from Rymsza [32] outlined the proposal to change the LM 1 in line with the concept currently discussed in EU, according to which the maximum allowable vehicle weight on European motorways would be increased to 60 tons (600 kN), as opposed to the current limit of 40 tonnes on majority of European roads.

4. Site-specific traffic load models

4.1. Traffic measurements – overview

Among several methods that are currently available for the collection of traffic information, the most commonly used

method, which is also the simplest one, involves the use of traffic counters [43]. In general, there are two types of counters. Portable ones mainly include manual counters, and pneumatic rubber tubes. The stationary ones, which are installed in or beside the road, are mainly magnetic loops infrared or, lately, laser and optical devices [44]. Traffic counters have been used in Croatia for traffic analysis since 1971 [10].

In order to obtain realistic axle loads, every vehicle needs to be weighed, either statically or in motion. Static scales, including weighbridges, axle and wheel weighers, provide the most accurate results. Unfortunately, these do not give a representative traffic sample, as the weighing process is very slow and as, due to the presence of police, heavy vehicles with excess cargo tend to avoid the weighing [43].

Weighing vehicles in motion or WIM, is a technology that records all vehicles at full speed, at uncontrolled conditions [45], providing axle loads, gross vehicle weight (GVW), number of axles and their spacing, vehicle speed, vehicle length, classification etc. In general, there are two main types of WIM systems, the pavement based system, which is permanently built into the wearing course of the road, and the bridge based system, called Bridge Weigh-in-Motion or B-WIM, which uses instrumented bridges or culverts for weighing the vehicles [43]. The main advantages of B-WIM systems are that they are fully portable and, as the sensors are placed on the soffit (underside) of the bridge, there is minimal to none traffic interruption during installation and operation [6]. Furthermore, beside the basic vehicle data (Table 4), which are equivalent to the pavement WIM system, B-WIM systems provide data on the response of the structure to the traffic loads. This information is not only crucial for proper functioning of the B-WIM system, but also ensures optimal calibration of the analytical bridge model [43, 46]. The two disadvantages of B-WIM systems are: a suitable bridge that allows proper installation and measurements is needed, and so are the experts with considerable knowledge about bridges [47].

A more detailed description of WIM and B-WIM systems can be found in COST 323 [45] and WAVE [48] reports, in the doctoral dissertation [43], and in a paper by Žnidarič [47]. The application of B-WIM traffic and structural data in the bridge assessment process can be found in the ARCHES [49, 50] project report and in a number of research papers, e.g. [6, 11, 12, 51, 52].

4.2. General approach to post-processing of WIM and B-WIM data

The main challenge in this process is proper selection of the data extrapolation method and, subsequently, estimation of expected maximum load effects on the bridge over selected time periods [7].

Prior to extrapolation, the first step of raw traffic data post - processing involves calculation of load effects from the

Table 4. Example of B-WIM data for single vehicle [46]

Time stamp	Lane	Speed [m/s]	Class	Number of axles	GSW [kN]	AW1 [kN]	AW2 [kN]	Axle spacing [m]
2007-03-22-00-39-28-955	1	17.5	41	2	123.8	37.07	86.69	6.07

collected data. These load effects, i.e. bending moments, shear forces etc. are caused by the vehicles passing over the bridge, and they vary with time, due to changing traffic intensity and composition uncertainty [53]. As load effects caused by light vehicles are irrelevant for infrastructure assessment, commercial WIM systems do not take into account vehicles lighter than 3,5 tonnes [47]. Conversion of vehicle axle loads into the corresponding load effects is in most cases done using the influence line method, which has also been used also in the development of EN 1991-2 load models [20]:

$$Q_S(x) = \sum_{i=1}^n A_i \cdot I_i(x) \quad (2)$$

Where A_i is the load of the axle i , n is the number of axles and $I_i(x)$ is the value of influence line under the axle i at location x . The two most commonly used approaches for the estimation of load effects are either extrapolation by fitting data to a statistical distribution, or the use of a very large number of Monte Carlo simulations based on recorded data [79]. Statistical extrapolation is used in many studies and was also the basis for development of EN 1991-2 Load Models, as explained in the first part of the paper. Statistical methods vary with respect to the selected distribution type, which should represent, as accurately as possible, distribution of the upper tail of traffic data. A number of authors have used normal distribution [12, 54, 55], in which measured data are being plotted on normal probability paper. However, a more common approach is to use some variations of extreme value distributions from the family of Generalised Extreme Values (GEV), like Gumbel [56, 57] or Weibull [58], in which cases the distribution is fitted to the maximum values of data, typically calculated for individual days or weeks of traffic. Statistical methods are more appropriate for practical use than the long-run simulations, but a possible disadvantage is the neglect of specific axle configurations and multiple truck events that are not captured in the recorded WIM data [43]. A detailed review of statistical methods can be found in research conducted by O'Brien et al. [53], paper by Zhou et al. [59], and report of ARCHES project [50]. A literature review of these methods indicates that they are numerous, diverse, and that it is not clear which one would be the most suitable for general use [53, 59]. Four most commonly used statistical methods are: fitting data to a normal distribution and raising it to the power using the extreme value theory [55], the Block

Maximum method [54, 60], the method involving fitting the generalized Pareto distribution to peaks over threshold (POT) [61] and the use of Rice formula for extrapolation of load effects [59, 62, 63].

4.2.1. Fitting load effects to normal (Gaussian) distribution

A common approach in extrapolation of traffic load effects for longer return periods is to collect traffic data for a certain time period, calculate traffic load effects, create histograms of these effects by fitting them to normal distribution, and using that distribution to estimate maximum load effects for a selected bridge lifetime [7, 64]. Estimation of maximum load effects is conducted by raising the initial (parent) distribution to a power of N using the extreme value theory [53, 64, 65]. If data is fitted to normal distribution with mean value μ and standard deviation σ , the maximum distribution after raising to the power of N is defined with statistical parameters μ_{Max} and σ_{Max} :

$$\mu_{Max} = \mu + \sigma \cdot \sqrt{2 \ln(N)} - \sigma \cdot \frac{\ln[\ln(N)] + \ln(4\pi)}{2\sqrt{2 \ln(N)}} \quad (3)$$

$$\sigma_{Max} = \frac{\sigma}{\sqrt{2 \ln(N)}} \quad (4)$$

Selection of parameter N in Eqns. (3) and (4) is dependent on the desired return period for estimation of traffic load effects [47]. Various types of normal distribution [59] are used to fit the upper tail of measured traffic data: standard normal distribution [64], trimodal distribution [1], bimodal distribution [66], etc.

4.2.2. Block Maxima Method

Using the extreme value theory to raise the initial function of traffic data to the certain power results in a steeper probability density function of the final function, which means that the standard deviation is decreasing. On the other hand, the higher the power, the lower is the amount of data that contributes to the maximum distribution of the final function [59]. In cases with a large number of different loading events (different truck configurations), a very large number of N has to be used in the Eqns. (3) and (4) in order to successfully estimate the maximum load effect for a certain return period. For example, Fu and You [67] defined that value of N has to be 109,5 million for a bridge with an average daily truck

traffic of 4000 and return period of 75 years, which makes it impossible to obtain accurate extremes [59].

The block maxima method is based on dividing the measurement period into non-overlapping periods of equal size, which are defined as blocks. In terms of traffic load measurements, these periods can be set as days, weeks, months, etc. The block maxima approach uses only the maximum value of the data obtained for these time periods, which have to be longer than the period of any underlying variation in statistical process, such as hourly traffic flow rates (rush hour) [53, 68].

On the assumption that loading events in defined blocks are independent, block maxima data can be fitted to three extreme value distributions: Gumbel, Fréchet and Weibull, but they result in three different tail behaviours of the initial function. The unification of these three distributions into a single family, defined as the generalized extreme value (GEV) distribution, has been used recently to avoid uncertainties arising from distribution selection [53]. The application of Block Maxima can be found in the research by Zhou et al., who fitted it to daily maximums [59].

On the other hand, an obvious drawback in using the block maxima method is that it only uses a single data maximum in each block, and so some authors have described this method as a waste of data [68]. In addition, a problem arises from the fact that the second highest value on one block can be higher than the maximum value on another block but, due to the method's basic assumption, it will not be taken into account [53].

The application of maxima method in traffic load estimation, including prediction of annual traffic growth, can be found in a work presented by O'Brien et al. [69].

4.2.3. Peaks over threshold (POT)

The peaks over threshold (POT) approach can be used to address the disadvantages of the block maxima method by taking into account all data values above the selected threshold. On the other hand, results clearly depend on threshold selection, which is subjective, and leads to the obvious disadvantage of POT approach: If the selected threshold is too high, there will be only few data points above it, resulting in high variance and unreliable results. On the other hand, if it is too low, the presence of uncritical values will affect the convergence and will cause biased results [53]. Parameters that need to be taken into account in the selection process include the length of the bridge, shape and length of influence line or surface, most frequent heavy vehicle configurations, etc.

General idea in threshold selection is to make it as low as possible, without disturbing convergence of the end result. More on the general methods for threshold definition can be found in the book by Coles [70], while Zhou et al. [71] focus on threshold selection in Mixture POT method for estimation

of extreme traffic loads on bridges. A more recent discussion on threshold selection in traffic load analysis can be found in the work by Yang et al. [72].

After selection of thresholds, the data beyond the threshold must be fitted to a probability distribution. Both Coles [70] and Crespo-Minguillon and Casas [61] stated in their research that the Generalised Pareto Distribution (GPD) is the most appropriate for traffic load modelling [53].

4.2.4. Rice formula

The Rice formula for the mean number of crossings in normal processes, introduced by Rice [73], has numerous applications in engineering. In general, this formula calculates an expected number of times a stationary process $X(t)$ "crosses" a threshold level u . In structural engineering, the Rice formula is mainly used in the analysis of structures exposed to environmental actions, such as the wind, waves and temperature fluctuations. More on applications of Rice formula in engineering can be found in the paper presented by Rychlik [74].

The Rice formula can be applied in traffic load modelling on the assumption that traffic load effects on long-span bridges can be modelled as a Gaussian random processes, as proposed by Ditlevsen [75]. This application is described in Eq. (5), where the mean rate of up-crossings ν for the threshold level $x > 0$ is defined for the reference period T_{REF} [53]:

$$\nu(x) = \frac{\sigma'}{2\pi\sigma} \exp\left[-\frac{(x-m)^2}{2\sigma^2}\right] \quad (5)$$

where m is the mean value, σ is the standard deviation and σ' is the standard deviation of stochastic process derivative \dot{x} . Cumulative distribution function can be derived from Eq. (5), as described by Cremona [63], who also suggests the method for defining an optimal number of bins, which has been adopted by Getachew in his doctoral thesis [76] with regard to the analysis of load effects on bridges. The Rice formula was also used by Jacob in his studies on the development of traffic load models for Eurocode [62].

4.2.5. Traffic simulations using Monte Carlo method

The Monte Carlo method, based on repeated random simulations of recorded data, takes into account critical combinations of heavy vehicles and axle configurations that are not recorded in the WIM data, given the large number of simulations. Extrapolations of traffic data using simulations have been dealt with by a number of authors [1, 76–81], while a detailed overview of the existing extrapolation methods is given in the report of ARCHES project [50].

The application of Monte Carlo simulation in traffic modelling is based on the data derived from traffic measurements, such as vehicle weights, configurations, axle loads, inter-

vehicle gaps etc. These parameters are used for simulations of traffic, typically over a number of years [79]. Due to large amount of data, MC simulations can be used to simulate vehicle configurations and combinations that have not been recorded during the measurement process. Unlike the traffic load models from design codes, MC approach enables taking into account special vehicles, overloaded vehicles, along with design codes and realistic traffic, and thus provides more reliable results [50, 79].

An obvious disadvantage of MC simulation approach is a certain degree of subjectivity associated with modelling of these parameters, and the fact that it requires large computational power and expertise, making it less practical for commercial use [79].

The MC model used in the ARCHES project [50], developed at the University College Dublin (UCD) by O'Brien et al. [79, 82], uses WIM data for trucks weighing more than 3,5 tonnes, collected at five European sites between 2005 and 2008. The data collected in this way were analysed through the quality-control process to remove unreliable observations, using cameras located at WIM sites. The WIM data quality control process has been described in doctoral dissertation [43] and in recent research [47] by Žnidarič.

The detailed process of MC simulation for the estimation of traffic load effects is summarized in recent work by Enright and O'Brien [79] and O'Brien et al. [69].

In addition to the above described traffic load modelling methods, some of those less frequently used can be found in the review paper by O'Brien et al. [53].

As a part of ARCHES project, and addition to the detailed MC model, a simplified procedure based on the statistical convolution method [12, 83] has been applied and will be described in the following section. This method, proposed by Žnidarič et al. [7], is also used in [6, 46, 84].

4.3. Example of B-WIM data extrapolation

The basic assumption of the proposed method is that the maximum load effect on the bridge is achieved when two vehicles, one in each traffic lane, meet side-by-side on a critical section of the bridge (e.g. at the centre of the span). This approach is justified on majority of bridges with the influence lines of up to 30 to 40 meters, where critical loading events occur during the free flow traffic, while on the long-span bridges, the maximum load effects are caused by congested traffic [43]. As the most common bridge type on the Trans-European Network (TEN-T) roads, based on the survey conducted in SKRIBIT project [85, 86], is a simply supported or continuous girder bridge with a span of less than 30 meters, this method is considered suitable for further research and application, as also stated by Žnidarič [43]. Multiple presence of vehicles in adjacent lanes of the bridge, is defined as a critical loading event [43], and is measured in time, rather than in meters. This approach is

applicable to both road bridges with two-way traffic, and highway bridges with one-way traffic, i.e. it is only required to modify calculation of multiple presence duration, as explained in [43].

The site-specific traffic load model development starts with collection of WIM data. It is important that, beside the axle loads and configurations, the data for each vehicle (Table 4) contains timestamp of its passage, with an accuracy of at least one thousandth of a second, to allow modelling of multiple presence events on the bridge [7]. The minimum amount of traffic data for reliable prediction of load effects and dynamic characteristics is at least 100 000 heavy vehicles, or at least two months of measurements [6, 43].

Due to the large number of vehicles, their static load effects, calculated using Eq. (2), are summarized into relative frequency histograms, separately for each lane. Interval (bin) sizes on the x-axis are selected to provide good resolution, while it is also imperative that values exceed the maximum calculated load effect by at least 10%, for precise modelling of distribution tails [7]. Histogram examples for a two lane bridge are presented in Figure 2 [46], where distributions are smoothed with a central moving average approximation (thick red line), which is defined as a probability mass function (PMF) for each lane.

The critical loading event is defined as a simultaneous presence of one vehicle in both lanes. Considering that the traffic in lane 1 is independent from that in lane 2, the two histograms from Figure 2 are combined using the convolution equation:

$$f_z = \sum_{k=1}^m f_x(k) \cdot f_y(z-k) \quad (6)$$

Where f_x and f_y are PMFs of load effects for lanes 1 and 2, and f_z is the PMF of load effects for an event comprising vehicles in each lane where m represents the number of intervals on the histograms, presented in Figure 3 [46].

PMF in Figure 3 is used to derive the cumulative distribution (CDF) function F_z , along with its statistic parameters, the mean value and the standard deviation, representing the predicted maximum load effect for a single loading event. Using the extreme value theory [65], CDF can be derived for any selected time period, by raising the F_z to the power of the expected multiple presence events N using Eq (7):

$$F_{max}(z) = F_z^N(z) \quad (7)$$

where N is the number of multiple presence events, obtained from WIM data, as explained in detail in [43]. As the number of multiple presence events depends on the selected time period, Eq. (7) can be used to derive CDFs from function F_{max} for different time periods. These CDFs, also called "convolution curves", are presented on Figure 4

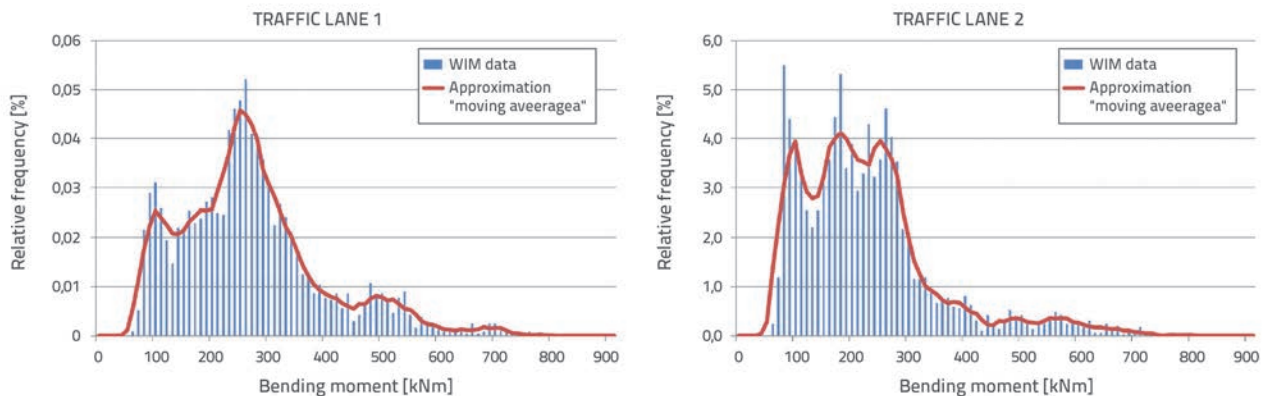


Figure 2. Relative frequency histograms for bending moments [46]

In order to account for dynamic characteristics of recorded traffic, and the bridge-vehicle interaction, the calculated static load effects are multiplied by the Dynamic amplification factor (DAF), which can be obtained directly from B-WIM measurements. The evaluation of DAF can seriously affect the ultimate traffic load effect values, and its proper selection has been investigated by a number of authors and in the scope of many research projects [43, 51, 87, 88]. An example of the DAF calculated for more than 200.000 vehicles, evaluated as 1,035, or only 17,5% of the value of 1.20 recommended in the codes [20], can be found in the previous work by the authors [6].

Results obtained with the presented method (Figure 4) for extrapolation of WIM data gave very similar results as the Monte Carlo model developed in the ARCHES project [50] and the statistical extrapolation method proposed by Sivakumar et al. [12]. On an average, the load effects were by 2,7% lower than those obtained by the Monte Carlo model, which assumed a certain increase of heavy vehicles' size and gross weights. Detailed results are presented in [7, 43]. Furthermore, in their previous papers [6, 46, 84], the authors compared the load effects on the Case Study Bridge when calculated from the site-specific traffic load model and when using the code recommended traffic load effects. These studies proved that the application of site-specific traffic models can reveal hidden bridge reserves, which consequently reduces maintenance costs and extends the remaining service life of bridges.

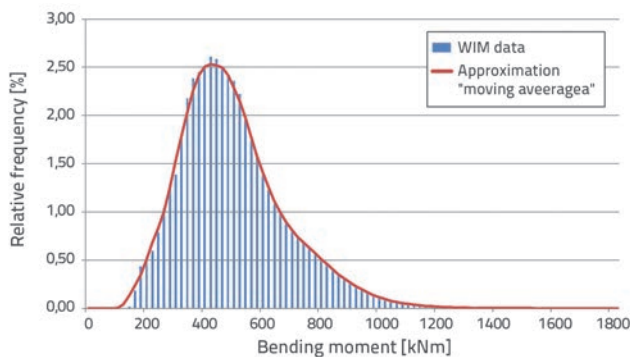


Figure 3. Convoluted histogram for both lanes – loading event [46]

5. Conclusion

The aim of this paper is to provide a review of the traffic load modelling procedures through the analysis of structural safety of road bridges, along with the background on load models from the current design codes. Due to its variable nature, modelling of traffic loads presents the key input, both in the design of new bridges and in the assessment of the existing ones. Load models defined in the current design codes are often described as conservative, due to the limited traffic data used for their calibration and their integrated safety levels. They do however provide engineers with a basis for the design of reliable and cost-effective new bridges that will sustain the expected growth in freight traffic [37]. However, despite the unified design codes for European countries, the capacities and associated structural safety of similar bridges throughout Europe vary as a result of different national adjustment factors (Table 1).

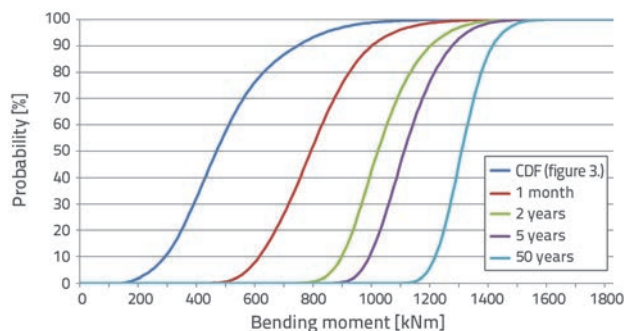


Figure 4. CDFs (convolution curves) for different time periods [46]

On the other hand, processing of realistic traffic data for the assessment of existing bridges is a more demanding process, as majority of available methods have a certain

degree of subjectivity, or are too complex for practical use. The method presented in this paper provides results comparable to those of more complex simulation and

extreme value distribution methods, and can thus be recommended for practical use in the assessment of existing road bridges in the region.

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