Traffic load model for road bridges in the urban road network

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Traffic load models like in the Eurocode and other design standards are mostly calibrated using traffic load measurements at heavily loaded highway locations. Therefore application of these models to existing bridges in the urban road network often leads to unnecessary strengthening. Direct measurement data for this road network are currently not available in the Netherlands. Therefore in this study, a database with traffic load measurements at a highway location is first filtered to obtain the traffic load properties of roads in the urban road network. Subsequently a reliability-based calibration method is setup to calibrate a new traffic load model. This has led to a modified Eurocode EN 1992-1 LM1 and LM2 load model for the urban road network for the Dutch design situation. It was shown that the single and tandem axle loads of the LM1 and LM2 model could be reduced. The uniformly distributed load for the first lane had to be increased for the short spans relevant for the urban road network.

Key words: *Traffic loads, urban road network, existing bridges, reliability-based calibration*

1 Introduction

The design of road bridges is based on the models for traffic loads as included design standards like the Eurocode EN 1991-2 in Europe. These traffic load models are usually calibrated using traffic load measurements at highway locations. These are mostly heavily loaded locations and representative for long-distance traffic. In Europe the measurements on the A6 in France at Auxerre have been used, see Bruls et al. (1996a and b). Auxerre traffic is characterised as very heavy and is representative of long-distance traffic in Europe. The urban road network consists of inner-city and urban access roads that are not part of the national highway network. For many of these roads, additional restrictions may be imposed by local authorities on the maximum vehicle weight. As a result, there is a difference in traffic load between the main road network and the urban road network. It is expected that, when using a traffic load model intended and calibrated for the main road network, in several cases structures in the urban road network are unjustly disapproved, whereas these structures do actually meet the requirements with regard to structural safety if account is taken of the actual traffic on these roads.

In this paper, a methodology is proposed to quantify and substantiate this reduction by means of a data set of traffic load measurements and probabilistic models adapted to the urban road network. Using this methodology this reduction is quantified for the Dutch design situation using the EN1991-2 traffic load model. This calibration of the design load values has been obtained using a full-probabilistic and reliability-based approach instead of a calibration of the characteristic value with specific return period[1]. For this purpose a dataset with the statistics for the traffic load of road bridges in the urban road network is first obtained (section 3). Direct measurement data for this road network are currently not available in the Netherlands. Therefore measurements of a highway location have been used, which have been filtered to obtain a database representative for the urban road network. Based on this database the vehicle characteristics and load effects to be accounted for have been derived as they apply to the urban road network (section 4) and a modified load model has been calibrated (section 5). The study is based on a calibration of the load design values based on a stipulated reliability level.

2 Scope of application

Several assumptions were made during the derivation of the traffic load model for road bridges in the urban road network. The main assumption is that the traffic load on the urban road network can be characterized by a maximum weight limit according to regulations. Thereby it is assumed that traffic composition on the urban road network is the same as for the main road network below a certain vehicle weight limit. Therefore the proportion of certain vehicles in the database is the same for both the main road network as the urban road network. Further investigation is needed to see if this is a conservative assumption as it can be expected that in the urban road network relatively more low weight lorries are present (e.g. Table 4.7 in EN1991-2). However the influence on the design traffic load might be limited.

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[1] In EN1991-2 a return period of 1000 years is used for traffic loads.

The bridge should not be subject to frequent loads from vehicles whose load deviates significantly (access to industrial estate or transhipment area) or by wheel configurations that deviate unfavourably from the frequent vehicles for which the traffic load model was derived. These frequent vehicles are for the European and Dutch design situations described in Table 4.6 from EN 1991-2.

In the analysis, focus was on bridges common in the urban road network with relatively short influence length (< 20 m) and with a reduced number of vehicles per year. This influence length ensures that there can be assumed to be only one vehicle on the bridge at the same time per traffic lane. Therefore vehicle distances were not considered.

3 Set of vehicles used in the analyses

The first step is to obtain a dataset with the statistics for the traffic load of road bridges in the above-mentioned scope of application. However, direct measurement data for the urban road network are not available at the moment in the Netherlands. Measurement data from the Weigh in Motion (WIM) system on the A16 motorway (RW-16 near Moerdijk) from April 2008 and 2013 were instead used as the basis for the analyses. These databases are filtered based on maximum vehicle weight to obtain databases representative for the urban road network. This filtered set of vehicles forms the basis for the analyses presented. In TNO [2012] it was shown that RW-16 is a location that is subject to relatively heavily loads in terms of national highways in the Netherlands as shown in Table 1.

Table 1: Number of vehicles in the 2008 database with a gross vehicle weight above 100 tons (TNO, 2012)

Highway	Number of vehicles in 2008 with GVW > 1000 kN
RW-16	44
RW-4	2
RW-12	36
RW-15	33

Traffic on Dutch roads can be divided into three categories due to present legal regulations: lorries and cranes without a permit with an upper weight limit of respectively 50 and 60 tonnes, lorries with a regular permit with an upper weight limit of 100 tonnes carrying an indivisible load and lorries with a special permit with an upper weight limit above 100 tonnes (which are not included in the database). These two branches of vehicles (relevant for the first two categories) can be clearly distinguished in the frequency distributions in Figure 1.

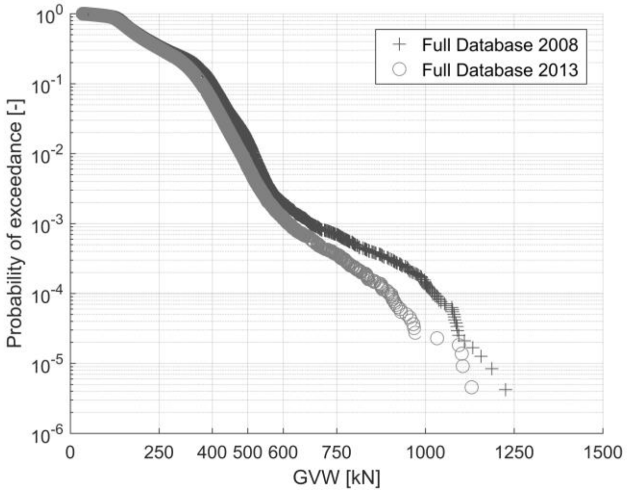

Figure 1: Vehicle weight frequency distribution for full dataset for the urban road network for the 2008 and 2013 RW-16 database

The databases for the highway location have been filtered by a maximum gross vehicle weight of 65 tonnes to consider vehicles representative for the urban road network. In TNO [2012] these 65 tonnes was found representative for the vehicles without a permit also accounting for overloaded vehicles and dynamic effects due to road-vehicle interaction.

Figure 2 shows the frequency distribution for the vehicle weights of the entire (grey) and filtered (black) database. This clearly shows that the second branch of the distribution, which belongs to the population of vehicles with a permit, is not included at all in the analyses for the underlying road network. Figure 3 and Figure 4 also show frequency distributions of the complete and filtered data set for the single and tandem axles for the 2013 database. These figures show that, in contrast to the vehicle weights, the distributions of the axle loads from the original (grey) and filtered (black) database differ little from each other.

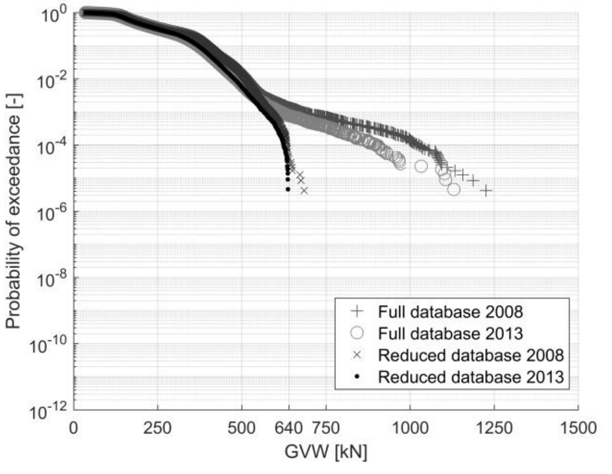

Figure 2: Vehicle weight frequency distribution for complete (grey) and filtered (black) dataset for the underlying road network for the 2008 and 2013 RW-16 database

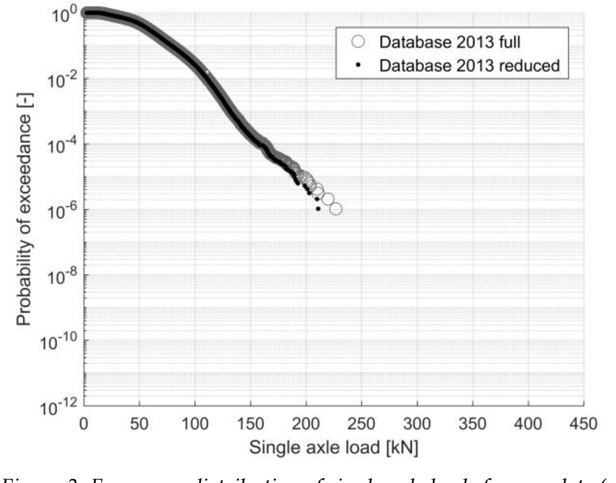

Figure 3: Frequency distribution of single axle loads for complete (grey) and filtered (black) dataset for underlying road network for the 2013 RW-16 database

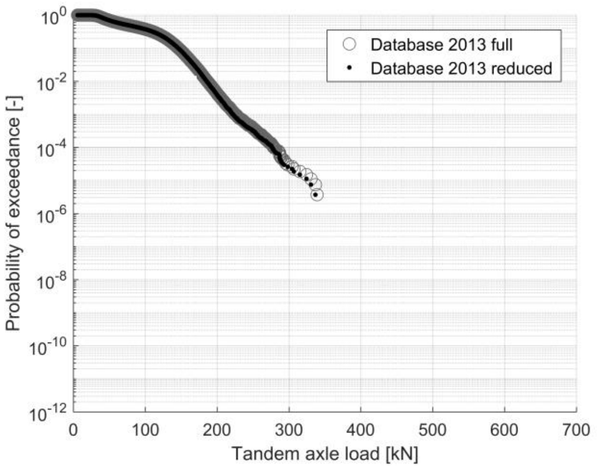

Figure 4: Tandem load frequency distribution for complete (grey) and filtered (black) dataset for underlying road network for the 2013 RW-16 database

4 Reliability based calibration of traffic load model

4.1 Methodology

Structures, like bridges and viaducts, are considered safe, when they comply with minimum reliability requirements. In the European Union these requirements are set out in the Eurocodes [EN1990]. They are expressed by a minimum reliability index β which is related to a maximum failure probability. These minimum reliability requirements in the Eurocode are defined for three consequence classes that account for the consequences of failure. Based on these consequences of failure for most road bridges in the urban road network Consequence Class 2 (CC2) of the Eurocode is applicable with a reliability index of 3.8 for a reference period of 50 years. This relates to a probability of failure of about $7 \cdot 10^{-5}$ in 50 years. To prevent the need to perform a complete reliability analysis for every individual structure, in the design codes, partial factors have been determined. The partial factor for traffic loading in the Eurocode [EN1991-2] for CC2 is γ_T = 1.35. Combined with the characteristic load parameters a load model is defined in the codes that can be used to design a new bridge or assess whether an existing bridge complies with the required reliability level.

To derive a traffic load model for bridges in the urban road network a reliability-based calibration of the load parameters (single and tandem axle loads) and load effects has been performed. For this purpose, the design values of the loads have been derived that belong to the required reliability level β considering the relevant sensitivity factor α_S:

$$P\left(S > S_d\right) = \Phi\left(-\alpha_S \beta\right) \tag{1}$$

For this research the standard sensitivity factor for load parameters according to the Eurocode [EN1990] has been used: $\alpha_S = -0.7$. The representative value of the load parameter could then be derived by dividing the design load by the partial factor for traffic loading. This in contrast to calibration of the characteristic value[2], which when multiplying with the partial factor does not necessarily lead to the load belonging to the required reliability level.

The reliability requirements have been set for newly designed structures. However the traffic load model to be derived, should also be used for existing bridges in the urban road network. Working with a reliability level of $\beta = 3.8$ for a reference period of 50 years is therefore not representative. In the Dutch design codes [NEN8700] therefore reliability requirements for existing bridges have been defined. In this paper we will also present the results for a reliability level defined for the assessment whether an existing bridge should be disapproved or not. This relates for CC2 to a reliability level of $\beta = 2.5$ for a reference period of 15 years.

To derive the design load the distribution of the load parameters with a certain reference period should be extrapolated to a probability of exceedance of $\Phi(-0.7 \cdot \beta)$. In this paper choice is made to extrapolate to the corresponding reliability level for the momentary distributions of the load parameters, accounting for the number of load events during the reference period. The number of load events depends on the number of vehicles per year and for the single and tandem axle loads on the number of axles or tandems per vehicle.

*4.2 **Distribution functions of vehicle characteristics***

The distribution functions for the different vehicle characteristics were determined from the filtered data set. Due to the multimodal characteristics of the distributions for both the vehicle weights, single and tandem axle loads a combination of a number of normal distributions has been fitted in the form of a Gaussian Mixture model. For this purpose the

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[2] For traffic loads this is defined as the load that occurs once in 1000 years, according to the Eurocode EN1991-2.

gmdistribution object of Matlab is used using the Expectation Maximizations algorithm [Mclachlan and Peel, 2000].

For the number of components in this Gaussian Mixture model a trade-off has to made between the dependence of the distribution tail on the number of components and the statistical uncertainties. The study to the required number of components in the Gaussian Mixture was evaluated using the extrapolated design value of the vehicle characteristic[3]. Increasing the number of components would decrease the dependence of the design load, but would increase the statistical uncertainties as well, as every component requires extra parameters to be fitted. Therefore the number of components is chosen at the lowest possible number of components where the influence of the number of components on the design value is low. This study is visualized in Figure 5 for the gross vehicle weight based on the database of 2013. For this case nine components have been considered appropriate.

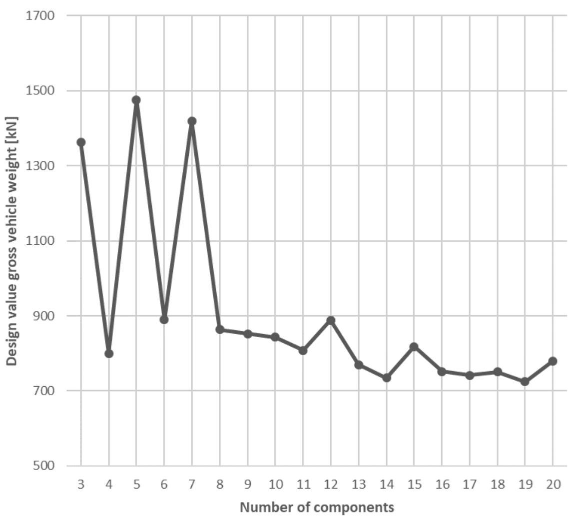

Figure 5: Effect of the number of components on the design value of the gross vehicle weight for CC2 and 125 000 vehicles per year

For the vehicle weight there are 2 branches in the frequency distributions of the original dataset. Therefore, when fitting the distribution functions for the reduced dataset, the form of first branch of the distribution of the complete dataset was considered in order not to

[3] De design value was determined by extrapolating the fitted Gaussian Mixture to the probability of exceedance belonging to Consequence Class 2 of the Eurocode and a number of 125 000 heavy vehicles per year on the urban road network.

underestimate the tail of the distribution by cutting it off at a certain vehicle weight. The specific distribution functions are shown in Figure 6. The distribution functions for the single and tandem axle loads can be applied directly to the filtered database. These distributions functions are all arbitrary point in time distributions functions for each load quantity and can be found in Figure 7 and Figure 8.

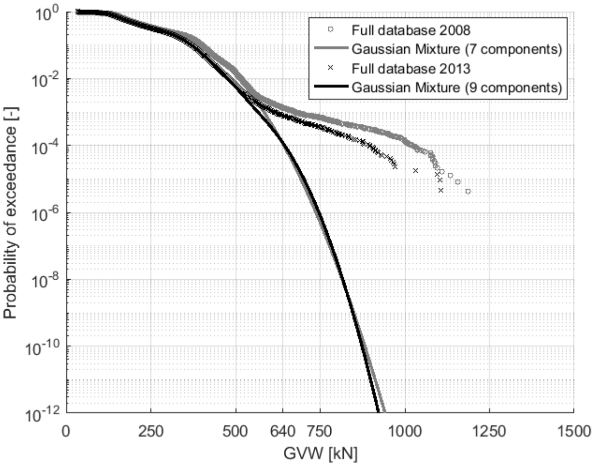

Figure 6: Vehicle weight distribution functions; frequency distribution of the full database and fitted distribution for the filtered database for the urban road network

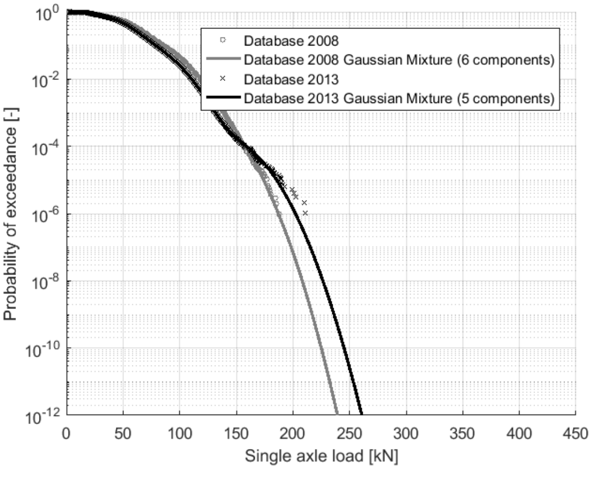

Figure 7: Single axle load distribution functions; frequency distribution and fitted distribution for the filtered database for the urban road network

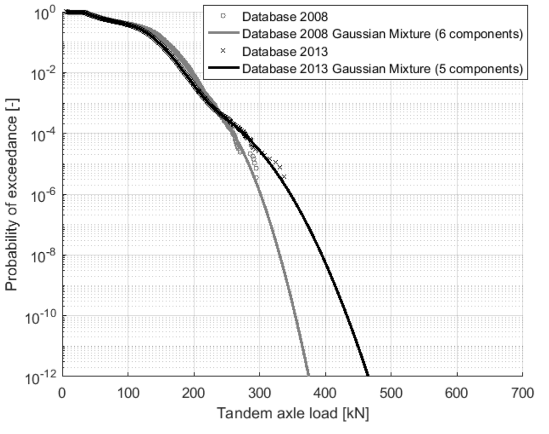

Figure 8: Tandem load distribution functions; frequency distribution and fitted distribution for the filtered database for the urban road network

4.3 ***Distribution functions for load effects***

The corresponding load effects for the vehicles in the filtered dataset have been derived for a simply supported bridge with influence lengths of 10 and 20 m respectively. The considered load effect is the bending moment in the middle of the span of simply supported bridges. Two loading situations have been considered. First the main bearing structure is loaded by only one lane of traffic and second it is loaded by one traffic lane in both directions. The distribution for the daily maxima of these load effects were obtained accounting for the expected number of vehicles per day in the urban road network and the distribution for this daily maxima has been fitted. The design value is obtained by extrapolation to the corresponding reliability level for the daily maxima and the expected number of days of traffic in the considered reference period. An average of 250 days of heavy traffic per year was assumed in this analysis.

Simulation of load effect for 1 lane

In the analyses for LM1 two influence lengths were considered: L=10 m and L=20 m. In both cases the bending moment in the middle of a simply supported beam was considered. Monte Carlo analyses were performed based on the available vehicle data from the filtered data set to derive the load effects. Because of the relatively short span, only one vehicle fits on the same lane of the bridge at the same time during the load events. Therefore the vehicles in the database could be sent across the influence line one by one and the maximum load effect per vehicle and the corresponding equivalent uniform load (q_{EUDL})

were determined. For the daily maxima of this q_{EUDL} a Generalized Extreme Value distribution was adopted for which an example is shown in Figure 9.

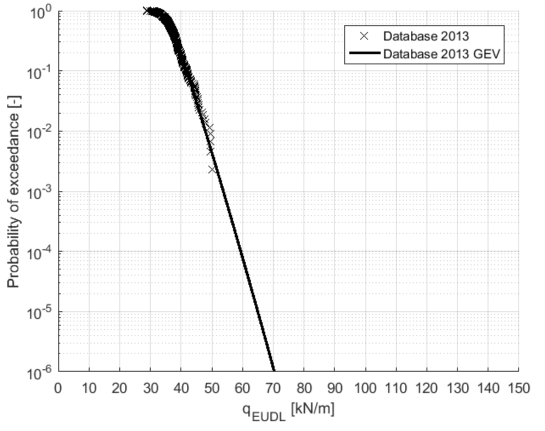

Figure 9: Frequency distribution and fitted distribution function for the daily maxima of q_{EUDL} at 125,000 heavy vehicles per year, the one lane load situation and an influence length L=20m; example for the filtered 2013 database

Simulation of load effect for 2 lanes

The situation where the main bearing structure of the bridge is loaded by two lanes, each in one direction, adopts a similar approach as in section 5.3 for one lane. However, the low probability (due to the small number of vehicles) that there are two heavy vehicles on the bridge at the same time must be included in the analysis.

The probability of two trucks being on the bridge at the same time is determined by the passage time of a vehicle on a bridge in free-flowing traffic and traffic jams. It has been assumed that the duration of congested traffic is equal to the duration of rush hours. In TNO [2014] it was found that this is about 7 hours per day for highway conditions. However it is reasonable to assume that rush hours during morning and late afternoon are common to highways and urban roads. In Keuken et al. [2012] an average speed in the urban environment for free-flowing traffic (30-45 km/h) and congested traffic (<15 km/h) has been found. Given the speed of the vehicle, the traffic intensity per day per lane I_d and the length of the bridge, the duration of bridge crossing by a vehicle t_{cross} and the number of such time intervals per day N can be easily evaluated. The probability of one vehicle on one lane in any time interval t_{cross} is equal to the ratio of the number of load events per day I_d and the number of time intervals N. Assuming the traffic flow in one direction is independent from the flow in the opposite one, the probability of two vehicles in adjacent lanes in any time interval t_{cross} is equal to:

$$P\left(2 \text{ veh in } t_{cross}\right) = P\left(1 \text{ veh in } t_{cross}\right)^2 \tag{2}$$

The probability of occurrence of a two-truck event is evaluated as the ratio of the number of two-truck events and the total number of events during a certain period. A conservative value of 10^{-1} has been chosen for the probability that two trucks are on the bridge to ensure all relevant cases for road bridges in the urban road network are being covered.

Next the frequency distribution and the fitted distribution functions for the daily maxima of the equivalent uniformly distributed load (q_{EUDL}) corresponding to the maximum load effects is determined separately for both load situations (one vehicle and two vehicles on the bridge) by means of a comparable Monte Carlo analysis as in section 5.3. Both distribution functions are then combined to one distribution function based on the probability of occurrence of both load situations.

The analyses carried out in this paper are based on two (slow) lanes (one in each direction). For bridges with more than two lanes (e.g. two lanes in each direction), the third and fourth lane are fast lanes. The analyses carried out are therefore conservative.

*4.4 **Probabilistic model input***

To derive the design values of the loads and load effects, the momentary distributions of the single and tandem axle loads (from section 4.2) and the distribution for the daily maxima of the load effects have been applied together with the parameters from Table 4 to account for dynamic amplification effects, statistical uncertainty and model uncertainty.

The dynamic amplification effects have been studied in TNO [2012] for the dataset of 2008 used in this paper as well as through a literature study. Dynamic amplification effects of the vehicles are already included in the measurement data (dynamic WIM measurements). The literature study resulted in values for the additional dynamic amplification effects due to the vehicle-bridge interaction presented in Table 4 for the single axle loads and load effects. The dynamic amplification factor for the tandem loads has been chosen based on the literature study in TNO [2012] and Dudescher and Brühwiler [2009].

The factor for statistical uncertainties is relatively low as it was assumed that the other parameters would dominate the design value and the design value of the load parameters would be close to the measured values. Therefore for the single and tandem axle loads a probabilistic analysis has been performed to check which parameters dominate the design value of the loads. This probabilistic analysis consists of a FORM calculation using the following limit state function with; X_d design load, χ_{DAF} dynamic amplification factor, χ_{trend} trend factor, θ_{stat} statistical uncertainty factor and X the load parameter:

$$Z = X_d - \chi_{DAF} \cdot \chi_{trend} \cdot \theta_{stat} \cdot X \tag{3}$$

The resulting design values for every parameter for the axle loads are presented in Table 2. The resulting design values for the tandem loads are presented in Table 3.

Table 2: Design values of the individual parameters used in the probabilistic analysis of the single axle loads (2013)

Parameter	Value	Design value
X_d	393 kN[4]	393 kN
Dynamic amplification factor – single axle loads	1.4	1.4
Trend factor 50 years	N (μ=1.0; σ=0.10)	1.26
Factor statistical uncertainty	N (μ=1.0; σ=0.05)	1.08
Single axle load	211 kN (max measured)	206 kN

Table 3: Design values of the individual parameters used in the probabilistic analysis of the tandem loads (2013)

Parameter	Value	Design value
X_d	534 kN[5]	534 kN
Dynamic amplification factor – tandem axle loads	N (μ=1.15; σ=0.1)	1.28
Trend factor 50 years	N (μ=1.0; σ=0.10)	1.20
Factor statistical uncertainty	N (μ=1.0; σ=0.05)	1.06
Tandem axle load	337 kN (max measured)	330 kN

[4] CC2 new construction database 2013 Table 5
[5] CC2 new construction database 2013 Table 6

It was found from this probabilistic analysis that the influence coefficient for the load parameters is low and the factor for trends and dynamic amplification is dominating the overall design value of the load parameters. Therefore the design value of the load parameter is close to the measured values and the effect of the statistical uncertainties is relatively limited. The design value of the load parameters is in both cases actually lower than the maximum measured one. This means that the design point is within the range of measured load parameters and not in the extrapolated part.

Table 4: Probabilistic model parameters

Factor	**Value**	**Remark**
Factor statistical uncertainty	N (μ=1.0; σ=0.05)	Effect was found to be limited as the design value of the load parameter is close to the measured values
Model uncertainty factor for load effects	N (μ=1.0; σ=0.1)	In accordance with JCSS probabilistic model code
Dynamic amplification factor for single axle loads	1.4	In accordance with TNO [2012] for local effects
Dynamic amplification factor for tandem loads	N (μ=1.15; σ=0.1)	Chosen partly based on Dudescher and Brühwiler [2009] and TNO [2012]
Dynamic amplification factor for load effects	N (μ=1.1; σ=0.05)	In accordance with the literature study in TNO [2012] and the background for the Eurocode EN1991-2 for global effects
Trend factor 50 years	N (μ=1.0; σ=0.10)	Agreed for local traffic
Trend factor 15 years	N (μ=1.0; σ=0.03)	Agreed for local traffic

4.5 Design values

In the following section the results of the reliability calibration for the single and tandem axle loads as well as the load effects for an influence length of 10 and 20 m are presented for 125,000 lorries per year assumed for the urban road network.

Single axle loads

The distributions of the single axle load from Figure 7 are combined in a probabilistic model with the factor for statistical uncertainties, the dynamic amplification factor for single axle loads and the trend factor to derive the design values of the axle loads. Table 5 shows the design values of the axle loads for a situation with 125,000 lorries per year.

Table 5: Design values of single axle loads for 125,000 vehicles per year

			Design values axle loads [kN]	
Case	**β**	**T_{ref} [yr]**	**database 2008**	**database 2013**
CC2 new constructio	3.8	50	358	393
CC2 disapproval	2.5	15	313	340

Tandem axle loads

The distributions of the tandem loads from Figure 8 are combined probabilistically with the factor for statistical uncertainties, the dynamic amplification factor for tandem loads and the trend factor to derive the design values of the tandem loads. Table 6 shows the representative values of the tandem loads for a situation with 125,000 goods vehicles per year.

Table 6: Design values of tandem axle loads for 125,000 vehicles per year

			Design values tandem loads [kN]	
Case	**β**	**T_{ref} [yr]**	**database 2008**	**database 2013**
CC2 new construction	3.8	50	468	534
CC2 disapproval	2.5	15	418	486

Load effect for 1 lane

The fitted (Generalized Extreme Value-) distributions for the daily maxima of the uniformly equivalent load q_{EUDL} for the load effect for 1 lane (for which an example is shown in Figure 9) are combined in a probabilistic model with the factor for statistical uncertainties, the factor for model uncertainties, the dynamic amplification factor for load effects and the trend factors to derive the design values of the load effects. Table 7 shows the design values of the bending moments in the middle of the span for 125,000 vehicles per year for a span length of 20 m. Table 8 shows the design values of the bending moments in the middle of the span for 125,000 vehicles per year for a span length of 10 m.

Table 7: Design values of the load effects in the middle of a simply supported beam at 125,000 vehicles per year, L=20 m, one lane

			Design values bending moment [kNm]		**Design values q_{EUDL} [kN/m]**	
Case	**β**	**T_{ref} [yr]**	**database 2008**	**database 2013**	**database 2008**	**database 2013**
CC2 new construction	3.8	50	4713	5096	94	102
CC2 disapproval	2.5	15	3891	4028	78	81

Table 8: Design values of the load effects in the middle of a simply supported beam at 125,000 vehicles per year, L=10 m, one lane

			Design values bending moment [kNm]		**Design values q_{EUDL} [kN/m]**	
Case	**β**	**T_{ref} [yr]**	**database 2008**	**database 2013**	**database 2008**	**database 2013**
CC2 new construction	3.8	50	2065	2026	165	162
CC2 disapproval	2.5	15	1650	1587	132	127

The design values of the equivalent uniformly distributed loads q_{EUDL} in Table 7 and Table 8 show that the load effects at the short span of 10 m are governing. These short spans are relevant for the urban road network as most bridges in this network have short spans. When bridges with an even shorter span are present in the network one should therefore also consider these shorter spans (e.g. L = 5 m) in the analysis of the load effects. For these shorter span lengths, a combination of several axles, more than a tandem, but not yet the total vehicle weight, could be governing, e.g. three or more axles.

Load effect for 2 lanes

Again, the fitted distribution functions for q_{EUDL} for 2 lanes are combined with the same factors as for one lane to determine the design values of the load effects. Table 9 shows the design values of the bending moments in the middle of the span and the equivalent uniformly distributed load for 125,000 vehicles per year for a span length of 20 m. Table 10 shows the results for a span length of 10 m. As expected, including the second lane considerably increases the load effects derived for one lane.

Table 9: Design values of the load effects in the middle of a simply supported beam at 125,000 vehicles per year, L=20 m, two lanes

			Design values bending moment [kNm]		Design values q_{EUDL} [kN/m]	
Case	**β**	**T_{ref} [yr]**	**database 2008**	**database 2013**	**database 2008**	**database 2013**
CC2 new construction	3.8	50	7182	6460	144	129
CC2 disapproval	2.5	15	5859	5280	117	106

Table 10: Design values of the load effects in the middle of a simply supported beam at 125,000 vehicles per year, L=10 m, two lanes

			Design values bending moment [kNm]		Design values q_{EUDL} [kN/m]	
Case	**β**	**T_{ref} [yr]**	**database 2008**	**database 2013**	**database 2008**	**database 2013**
CC2 new construction	3.8	50	2534	2361	203	189
CC2 disapproval	2.5	15	2071	1928	166	154

5 Modified traffic load model

The design values for the single and tandem axle loads and the load effects derived in previous paragraphs that belong to the required reliability levels can be used to derive a modified traffic load model for the urban road network. A load model is usually expressed in terms of representative or characteristic values of the load parameters. These can be derived from the design values by dividing them by the partial factor for loads. For the Dutch design situation these analyses have led to a modified design code for existing bridges in the urban road network [NEN8701]. This section presents a summary of the derived modified traffic load model.

It was decided to align the traffic load model for road bridges in the urban road network with the traffic load model as defined in EN 1991-2. As shown in Table 5 and Table 6, a separate test with only a single axle must be introduced because the design value of a single axle of the tandem (Table 6) is smaller than the design value of the single axle load (Table 5). It is therefore necessary to consider two different load models, like LM2 and LM1 in EN 1991-2.

In the Dutch situation separate reliability levels are defined for existing structures in NEN8700 to ensure an economic assessment. For these reliability levels corresponding partial factors have been defined as well. Most bridges in the urban road network are assessed for the CC2 disapproval reliability level. This level is defined to assess whether existing bridges are considered safe. The partial factor for traffic loading corresponding to this reliability level $\beta = 2.5$ over a reference period of 15 years is $\gamma_T = 1.1$. As most bridges are assessed for this level, choice was made to calibrate the modified traffic load model for the urban road network best with respect to this reliability level. The resulting values for the CC2 reliability level in the Eurocode used for new structures, $\beta = 3.8$ over a reference period of 50 years (and $\gamma_T = 1.35$), are also presented.

The selected LM2 load model for the assessment of an individual axle is presented in Table 11. The axle load is reduced by a factor of 0.8 compared to the EN 1991-2 LM2 load model. The choice was made to use a factor correcting for trend and reference period and a factor accounting for the number of heavy vehicles N_{obs}. The design values as Table 10 are also derived for respectively 50,000 and 5000 heavy vehicles per year and reduction factors with respect to 125,000 vehicles were derived. The smallest reduction was found for the axle loads as for this load parameter the steepest distributions were found. Choice was made to select consistent load models for both LM2 and LM1. Therefore, for both load models the reduction factors for N_{obs} are used, derived for the axle loads. This results in reduction factors of 0.98 and 0.93 for 50,000 and 5000 heavy vehicles respectively. These factors are somewhat conservative for the tandem loads and load effects.

Table 12 presents the design values for the axle load resulting from this modified and EN 1991-2 LM2 load model. The choice was made to use a reduction factor for trend and reference period consistent with the load models in the Dutch design code for existing bridges [NEN8701]. Therefore, the modified load model is most consistent with the value for the axle load corresponding the CC2 disapproval level in Table 5. For CC2 for new bridges the modified load model is relatively conservative. However, relative to the EN 1991-2 LM2 model for highway bridges there is still a reduction of approximately 17% on the design value. For the disapproval level the reduction is 25%.

Table 11: Modified LM2 traffic load model compared to the EN 1991-2 load model for road bridges in the urban road network.

LM2	Q_k single axle	Factor trend and reference period	N_{obs} = 125,000	N_{obs} = 50,000	N_{obs} = 5,000
Disapproval, T_{ref}=15 years	$Q_{i;k} = 0.8xQ_i$[6]	0.9	1.0	0.98	0.93
New construction, T_{ref} =50 years		1.0			

Table 12: Design values for the adapted LM2 traffic load model for road bridges in the urban road network and the EN 1991-2 model for N_{obs} = 125,000.

		Modified LM2 model		**EN 1991-2 LM2 model**		
	Partial factor γ_T	**Reduction factor**	**Q_d single axis [kN]**	**Reduction factor N_{obs}**	**Q_d single axis [kN]**	**Reduction**
Disapproval, T_{ref}=15 years	1.1	0.9	317	0.96	422	25%
New construction, T_{ref} =50 years	1.35	1.0	432	0.96	518	17%

The selected LM1 model is presented in Table 13. The starting point for the modified LM1 load model for the first lane are the tandem load values. Therefore the tandem axle load Q_1 was first calibrated using Table 6. It was found that a reduction factor of 0.8 could be used compared to Q_1 in the LM1 model of EN 1991-2. A lane width of 3.0 m has been assumed, which is in line with the EN 1991-2 traffic load model. Next, the distributed load on the first lane, q_1, can be derived in order to calibrate the load effects. The application of q_1 = 1.35x9 kN/m^2 is necessary to comply with the required values from Table 7 and Table 8. For both the tandem axle loads as the load effects there has been more confidence in the database of 2013. Therefore this database has been leading in the derivation of the load model.

The modified model for the first lane was then used as the starting point for the load model of the second lane. The value for Q_1 in the model has a reduction factor of 0.8 compared to EN 1991-2. For Q_2, therefore, the same reduction factor was used compared to

[6] Axle load in the EN 1991-2 LM2 load model: Q_i = 400 kN

the tandem load value in EN 1991-2 for the second lane (0.8*200=160 kN). For q_2 the same value is applied as in EN 1991-2. Using this model, values are found that fit well with the load effects from Table 9 and Table 10.

Table 14 presents the design values for the bending moment in the middle of a single span of 20 m resulting from the modified and EN 1991-2 LM2 load model. Choice was made to use a reduction factor for trend and reference period consistent with the load models in the Dutch design code for existing bridges [NEN8701]. Therefore, the modified load model is most consistent with the value for the bending moment corresponding the CC2

Table 13: Modified LM1 traffic load model compared to the EN 1991-2 load model for road bridges in the underlying road network

LM1	**Q_r, q_r**	**Factor trend and reference period**	**N_{obs} = 125,000**	**N_{obs} = 50,000**	**N_{obs} = 5,000**
Disapproval, T_{ref}=15 years	$Q_{i;r}$ = 0.8xQ_i[7]	0.9	1.0	0.98	0.93
New construction, T_{ref} =50 years	$q_{1,r}$ = 1.35xq_1[8] $q_{i,r}$ = 1.0xq_2[9]	1.0			

Table 14: Design values bending moment in the middle of the span for the modified LM1 traffic load model for road bridges in the underlying road network and the EN 1991-2 model for Nobs = 125,000, L=20m, one lane

		Modified LM1 model		**EN 1991-2 LM1 model**		
	Partial factor γ_T	**Reduction factor**	**M_d [kN]**	**Reduction factor N_{obs}**	**M_d [kN]**	**Reduction**
Disapproval, T_{ref}=15 years	1.1	0.9	4038	0.96	4404	8%
New construction, T_{ref} =50 years	1.35	1.0	5506	0.96	5404	-2%

[7] Tandem load in the EN 1991-2 LM1 load model: Q_i = 2*300 kN
[8] Uniformly distributed load for lane 1 in the EN 1991-2 LM1 load model: q_1 = 9 kN/m
[9] Uniformly distributed load for lane i with i > 1 in EN 1991-2 LM1 load model: q_i = 2.5 kN/m

disapproval level in Table 7. For new CC2 bridges the modified load model is slightly conservative. Compared to the EN 1991-2 LM2 model for highway bridges there is even a small increase of approximately 2% on the design value. For the disapproval level the reduction is still 8%.

For existing bridges in the urban road network the derived load model assessed at the Dutch reliability requirement for assessments of disapproval the derived load model leads to lower design values of the load parameters and load effects. However, for the design of a new bridge the load model leads to higher design values and the EN1991-2 LM1 model seems non-conservative. This is due to the increase of the uniformly distributed load q for the first lane. It was found that the shorter influence lengths were governing because of the effect of multiple axles (e.g. tridems and quads). It is therefore recommended to also do the analyses for bridges with an influence length less than 10 m as these can be relevant for the urban road network.

6 Conclusions

The design and assessment of road bridges is based on the models for traffic loads in codes that are usually calibrated using traffic load measurements at highway locations. For most bridges in the urban road network, additional requirements may be imposed by local authorities on the maximum vehicle weight and the load model in the codes could be reduced as it overestimates the loads on these bridges. In this paper, a methodology is proposed to quantify and substantiate this reduction by means of a dataset of traffic load measurements at a highway location and probabilistic models adapted to the urban road network. Using this methodology this reduction is quantified for the Dutch design situation and a modified load model is proposed based on the EN1991-2 LM1 and LM2 traffic load model.

It was shown that the single axle load and tandem axle load of the LM1 and LM2 model could be reduced. The uniformly distributed load for the first lane had to be increased for the short spans relevant for the urban road network. The proposed LM1 model is derived for two lanes but can be used for several lanes. This modified load model results in a significant reduction of the loads and the load effects to be considered compared to the EN 1991-2 traffic load models LM1 and LM2 in the structural assessment of existing bridges and viaducts. This model therefore offers the possibility to assess bridges that are initially disapproved by the EN 1991-2 load model with a traffic load model that does more justice to the actual traffic on roads in the urban road network.

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