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Towards actual brazilian traffic load models for short span highway bridges

Rumo a modelos de carga reais do tráfego brasileiro para pontes rodoviárias de pequenos vãos

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Abstract

New live load models for highway bridge design in Brazil are under development by assembling real traffic database, traffic simulations, analytical-numerical modeling of the dynamic interaction between vehicle and structure and statistical extrapolations. This paper presents and discusses the results obtained in the first stages of this work which includes the comparison between the static effects due to the actual traffic of heavy vehicles and those generated by the live load model given in the current national code NBR 7188. It is demonstrated that this live load model is not appropriate to represent the actual traffic effects and may be, in some cases, non-conservative. The present work deals with short span bridges for two lanes single carriageway under free flow traffic scenarios. The representative static effects in these bridges due to the actual traffic of heavy vehicles are obtained by extrapolating its probability density functions to a certain return period. To this purpose, a traffic database was constructed by gathering data from several weighing stations in Brazilian highways which was then applied to perform traffic simulations through a specially developed computational tool.

Keywords: highway bridges, live load models, heavy traffic database, traffic simulation.

Resumo

Novos modelos de cargas móveis para o projeto de pontes rodoviárias no Brasil estão em desenvolvimento com a montagem de um banco de dados de tráfego real, simulações de tráfego, modelagem analítico-numérica da interação dinâmica veículo-estrutura e extrapolações. Este artigo apresenta e discute os resultados obtidos nas primeiras etapas deste trabalho, incluindo a comparação entre os efeitos estáticos devido ao tráfego dos veículos comerciais reais e aqueles gerados pelo modelo de cargas da NBR 7188. Demonstra-se que este modelo de cargas não é adequado para representar as solicitações reais e pode estar, em alguns casos, contra a segurança. São consideradas pontes de pequenos vãos com pista simples e duas faixas de rolamento em cenários de tráfego livre. Os esforços estáticos representativos nessas pontes devido ao tráfego real dos veículos comerciais são obtidos extrapolando as suas funções densidade de probabilidade a um determinado período de retorno. Para tal, uma base de dados de trânsito foi construída através da coleta de dados em alguns postos de pesagem de rodovias federais brasileiras, os quais foram aplicados para executar simulações de tráfego através de uma ferramenta computacional especialmente desenvolvida para esta finalidade.

Palavras-chave: pontes rodoviárias, modelos de cargas móveis, base de dados de tráfego pesado, simulação de tráfego.

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1. Introduction

Highway bridges in Brazil are still designed according to the design code NBR 7188 [1], dated 1982, in which the live load model is composed of the 3-axles vehicle shown in Figure 1 plus a distributed load, multiplied by a dynamic amplification factor, function only of the span length. The configuration of this design loading follows the pattern of an older version of the NBR 7188, the NB-6 [2], dated 1960, which was based at that time on the current German design code DIN 1072 [3]. The load values have been increased over time, for example from 360 kN (NB-6) to the current heavier vehicle load equal to 450 kN. In spite of these load magnitude updates, the present live load model is not appropriate to represent the actual traffic effects on Brazilian bridges, as shown later herein.
New live load models for bridge design in Brazil are being developed by the authors so as to reproduce extreme values of bridge effects due to actual traffic (including dynamic effects) with approximately the same reliability index among the typical structural systems and through the span length range. The following steps are being performed to reach this goal [4,5,6,7]:

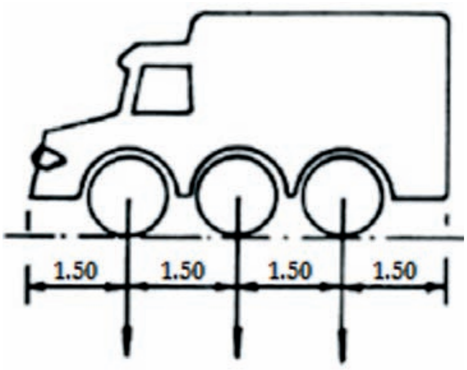
- 1) Selection of typical bridge structural systems and corresponding critical sections where internal forces are to be considered in the analysis;
- 2) Real traffic measurements and statistics;
- 3) Traffic simulation and static analysis of bridge models selected in step 1;
- 4) Statistics of the selected internal forces (bending moments and shear forces in critical sections) and extrapolation to obtain the representative values of the static effects;
- 5) Calculation of the target values of the selected internal forces by multiplying the representative static effect by the corresponding dynamic amplification factor, the latter obtained through dynamic analysis of each structural system under the load configuration which yields the largest static effect;
- 6) Search, by optimization techniques, of new live load models to reproduce the target values;
- 7) Calibration of safety load factors through reliability analysis.

This paper describes steps 1 to 4 and presents the comparison between static effects caused by the actual traffic, on typical bridge systems, and those generated by the current Brazilian load model. To this point, single carriageway and two lanes bridges are considered, composed of two types of two-girder systems, with supported span lengths ranging from 10 m to 40 m and cantilever span lengths from 2.5 m to 10 m, totalizing 24 different structures. For these cases of short spans free traffic flow scenarios are determinant [8], as opposed to longer systems for which traffic jam and mixed traffic have to be considered.
In the absence of updated weigh in motion (WIM) traffic measurements in Brazilian highways, a hybrid vehicle database containing the necessary traffic statistical information was built from five different data sources including three weighing stations in highways and traffic weighing and volume distribution research performed in many Brazilian regions by DNIT – the National Department for Transportation Infrastructure. The data base is composed of 29 classes of commercial vehicles, each one having probability density functions (PDFs) of typical traffic parameters, such as gross vehicle weight (GVW), axle and axle group weights, speed and axles spacing (wheelbases); there are also deterministic values for some vehicle dimensions and maximum axle weights.
Traffic simulations considering some possible scenarios for traffic lane distribution led to histograms of selected bridge static effects, which were extrapolated by means of the probability level, considering Weibull distributions fitted to these histograms. The obtained representative values of the static effects due to real traffic of heavy vehicles are herein compared to the static effects produced by both Brazilian load models from the past NB-6 and the current NBR 7188, when applied to the same short span bridge systems.

2. Heavy vehicles database

The data collected during 14 consecutive days in 2011 at one truck weighing station along the São Paulo State highway SP-348 (administered by the Concessionaire CCR AutoBAn), is fairly representative of the heavy traffic in Brazil. It was then taken as reference to

Figure 1 – Current Brazilian load model vehicle (NBR 7188): (a) side view; (b) plan view. Units: m



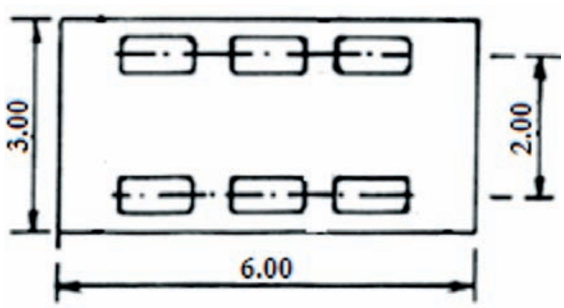
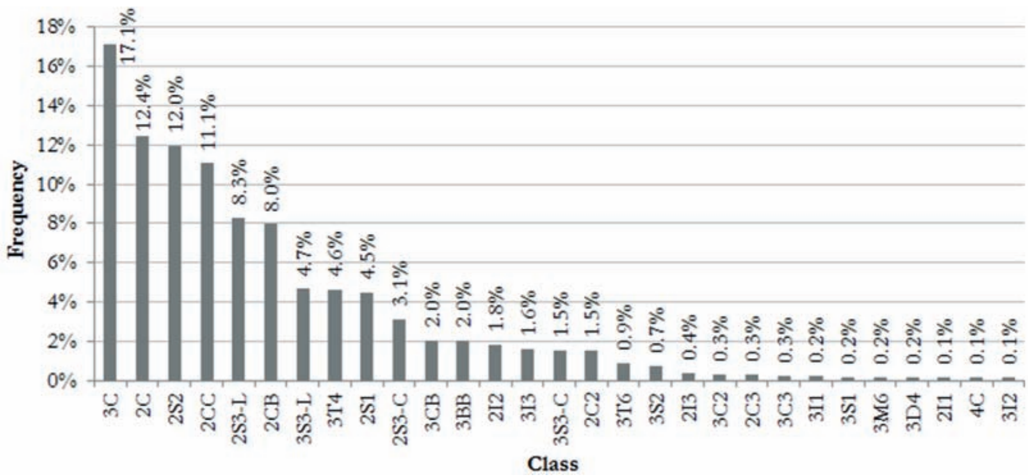


Figure 2 – Relative frequency for each vehicle class from H-2013 database



the database developed in this work, named H-2013 (H stands for hybrid; it was developed in 2013) [5]. As the available data neither include information on buses, vehicle speed, wheelbases nor allow the distinction among certain vehicle classes with the same number of axles, the H-2013 database was developed by gathering information from other 4 sources including weighing and traffic surveys carried out by Brazilian Federal Organizations. It was found that the statistical information of all these data is quite similar and that the adoption of a hybrid base would not yield important deviations.

In addition to the data provided CCR AutoBAn from 14 days measurements in 2011, the other 4 sources of data were the following:

- DNIT´s traffic survey in fifteen stations in several States of Brazil between 1999 and 2002 by means of WIM measurements;
- traffic survey performed by CENTRAN (Excellence Center in Transport Engineering, a government agency linked to the Brazilian Army) on Federal Brazilian Highways by collecting data in 109 counting stations spread all over the country in a seven days period in 2005;
- data collected at the same weighing station in SP-348 state highway consisting of a set of sheets containing daily records of a 6-days period in June 2008;
- data provided by *Ecovia* concessionaire from a weighing station located on the BR-277 highway, containing daily records of a 28-days period in June 2008.

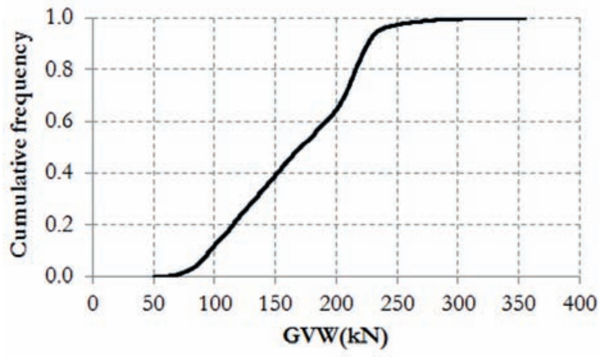
2.1 Traffic composition

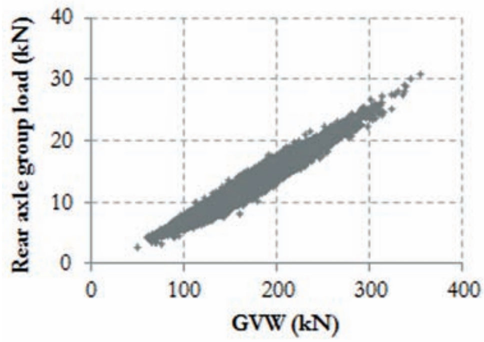
The database H-2013 traffic composition is shown in Figure 2 and comprises 29 classes of commercial vehicles which are illustrated in Table 1.

Table 1 – Brazilian commercial vehicles spectrum: classes and silhouettes

Class	Silhouette	Class	Silhouette	Class	Silhouette
2CC		2S1		3I1	
2C		2S2		3I2	
3C		2S3		3I3	
4C		2I1		3T4	
2C2		2I2		3T6	
2C3		2I3		3M6	
3C2		3S1		2CB	
3C3		3S2		3CB	
3D4		3S3		3BB	

Figure 3 – Variation of: (a) GVW cumulative distribution for class 3C ($\overline{X}$= 154.3 kN; s = 44.0 kN); (b) 3C rear axle group weight with GVW ($\overline{X}$ = 110.9 kN; s = 38.8 kN)





Classes 2S3 and 3S3 are quite frequent (see Figure 2) and their GVW distributions have more than one mode; therefore they were divided into long (L) and short (S) types according to the distance between the last axle of the tractor unit and the first axle of the trailer.

2.2 Gross Vehicle Weight (GVW) and axle weights

During the development of Eurocode 1 [9] load models, axle load records were amplified in 10% to take into account dynamic effects [10,11]. Conversely, no amplification was applied to the records for the AASHTO LRFD load model [12,13]. For the purpose of this work comparisons between GVW measured while moving at 60 km/h maximum speed and registered at a static scale showed deviations within +5% and -5% GVW [7], therefore the adopted weight was the load measured by the WIM system (from the weighing station located at SP-348, data from June-2008).

Figure 3a shows GVW cumulative distribution for class 3C, while Figure 3b illustrates the variation of the rear axles group weight with GVW for the same class; the values of mean and standard deviation are showed in their subtitles. It can be seen that there is a significant amount of records exceeding the Brazilian legal weight limit, which in this case equals 230 kN.

In order to do the traffic simulation PDFs were fitted to each one of the 29 vehicle classes GVW histogram; the best model was chosen among 18 different continuous functions. After testing these models individually or particular linear combinations of some of them (depending on the number of modes of each histogram), the most appropriate model – which is the most similar to the sample data - was elected via goodness-of-fit tests (chi-squared and Kolmogorov-Smirnov). Although not being as accurate as other methods [14], the method of moments, because of its simplicity, was used instead to perform the parameter estimation. Table 2 sum-

Table 2 – PDFs fitted to GVW histograms

Class	Distribution 1			Distribution 2			Distribution 3		
	Type	A.V.	S.D.	Type	A.V.	S.D.	Type	A.V.	S.D.
2CC	GAM	56.9	15.8	–	–	–	–	–	–
2C	GAM	91.2	20.3	–	–	–	–	–	–
3C	WEI	112.7	18.2	UNI	190.1	24.7	EXP	256.8	13.7
2S2	GUM	194.0	42.2	–	–	–	–	–	–
2S3-C	DEX	394.9	50.4	–	–	–	–	–	–
2S3-L	2MN	353.8	71.4	RAY	494.9	40.1	–	–	–
3S3-C	DEX	443.0	26.9	–	–	–	–	–	–
3S3-L	2MN	427.3	45.3	FRE	504.7	34.1	–	–	–
3T4	DEX	541.0	44.3	–	–	–	–	–	–

A.V. = average value; S.D. = standard deviation; DEX = double exponential; EXP = exponential; FRE = Frèchet (Type II max); GAM = Gamma; GUM = Gumbel (Type I max); RAY = Rayleigh; UNI = Uniform; WEI = Weibull (Type III min); 2MN = Type II min

Table 3 – Maximum axle loads recorded by scales in databases

Axle type	Registered maximum load		Legal limit (kN)	Assumed maximum load (kN)
	Value (kN)	Class		
Single	105.5	2S2	60	110
Double	173.8	2S3	85/100*	180

* 85 kN for tandem axles; 100 kN for isolated axles

marizes the information on the distributions fitted to the GVW histograms of the most frequent classes.
Axle weights were also considered as random variables. To avoid the use of correlations between axle weights, the scatter diagrams relating for each class the axle groups weights to the GVW (like the one shown in Figure 3b) were submitted to adjustment curves by means of least squares fitting. For all those diagrams it was found that linear models always display the best adjustments and were therefore the assumed curves. Besides providing the axle loads as a function of GVW these curves were also used to estimate the maximum GVWs as a function of the assumed maximum axle loads as detailed in the following.
Limitation in GVW values aims to preserve physical representation of the traffic simulation, which seeks values of bridge internal forces generated only by actual vehicles. Maximum assumed axle loads are shown in the last column of Table 3 and for which it was considered both the extreme axle weight values available in databases and the technical axle load thresholds informed by manufacturers that vary widely with the intended use of each vehicle: trucks, trailers and buses have different limits.

2.3 Wheelbases

Most wheelbases were considered random variables which were fitted to PDF models as performed for the GVWs. Table 4 shows some of these population models. For the estimation of wheelbases extreme values all the information acquired from the database as well as the manufacturers' specifications were considered. Tandem and tridem wheelbases and other distributions with small coefficient of variation were considered deterministic. Table 5 shows some of these deterministic values and ranges for wheelbases. It can be seen from Tables 4 and 5 that some wheelbases may vary within a wide range and their PDFs may have more than one mode.

2.4 Vehicle speeds

Figure 4 shows histograms describing speed distribution for single-unit trucks (such as 2C and 3C), buses and semitrailers (like 2S2 and 2S3); the values of mean and standard deviation are showed in the subtitles. In all cases the most frequent speed is 80 km/h. There are few speed records exceeding 140 km/h.

Table 4 – Population models fitted to some wheelbases; d_{ij} is the distance between axles i and j; units: m

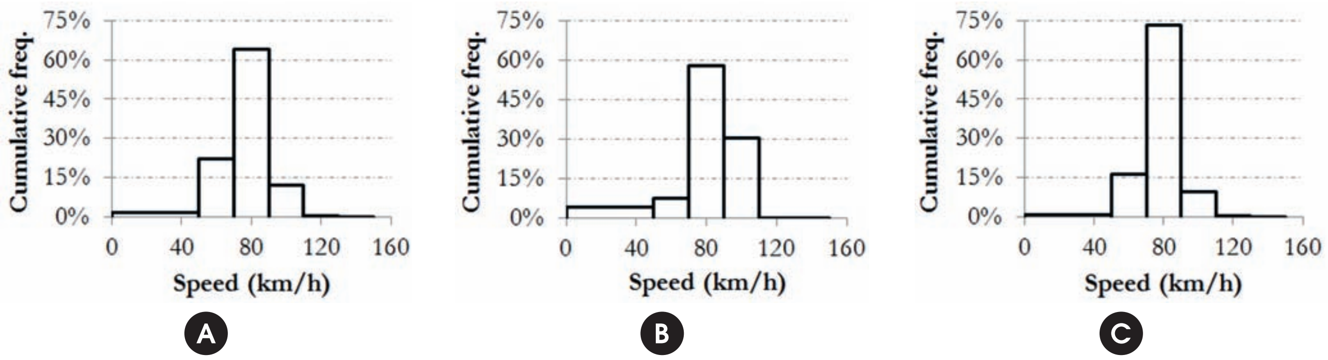
Class	Dist.	Distribution 1			Distribution 2			Distribution 3		
		Type	A.V.	S.D.	Type	A.V.	S.D.	Type	A.V.	S.D.
2CC	d_{12}	LGT	3.84	0.38	–	–	–	–	–	–
2C	d_{12}	EXP	5.31	0.80	–	–	–	–	–	–
3C	d_{12}	FRE	5.20	0.65	–	–	–	–	–	–
2S2	d_{12}	DEX	3.63	0.17	RAY	4.41	0.18	–	–	–
2S2	d_{23}	LOG	4.82	0.67	NOR	8.09	0.93	GUM	12.5	0.70
2S3-C	d_{12}	RAY	3.62	0.11	RAY	4.37	0.20	–	–	–
2S3-C	d_{12}	NOR	3.20	0.25	UNI	4.37	0.37	–	–	–
2S3-L	d_{12}	3MX	3.62	0.13	GUM	4.29	0.19	–	–	–
2S3-L	d_{23}	FRE	6.26	0.74	–	–	–	–	–	–

A.V. = average value; S.D. = standard deviation; DEX = double exponential; EXP = exponential; FRE = Frèchet (Type II max); GUM = Gumbel (Type I max); LGT = Logistic; LOG = Lognormal; NOR = Normal; RAY = Rayleigh; UNI = Uniform; 3MX = Type III max

Table 5 – Some deterministic values and ranges for wheelbases (m); d_{ij} is the distance between axles i and j

Axle type	d_{12}	d_{23}	d_{34}	d_{45}
2CC	(2.41 ; 4.05)	–	–	–
2C	(3.30 ; 14.66)	–	–	–
3C	(2.67 ; 15.81)	1.30	–	–
2S2	(2.44 ; 6.50)	(2.41 ; 23.09)	1.25	–
2S3-C	(2.44 ; 6.50)	(2.41 ; 5.00)	1.25	1.25
2S3-L	(2.44 ; 6.50)	(5.00 ; 17.81)	1.25	1.25

Figure 4 – Speed distributions: (a) single-unit trucks ($\bar{X}$= 77.1 km/h; s = 13.5 km/h); (b) buses ($\bar{X}$= 82.4 km/h; s = 16.4 km/h); (c) semi-trailers ($\bar{X}$= 78.3 km/h; s = 11.2 km/h)



3. Structural models

The bridges considered in this study are reinforced concrete Π cross-section structures (Figure 5), which were modeled as a simple grid composed of two main T-beams, a number of cross beams located at intermediate positions in its spans and also at the supports. The two bridges' Π type cross sections shown in Figure 5 represent a large portion of Brazilian highways' existing short span bridges [15], corresponding to designs originally planned for single carriageway two-lanes highways. The first one (Figure 5a) refers to old "narrow" bridge deck width, that was the standard type until the 1980s. From that decade on, it was adopted for the Brazilian highways a larger deck slab (Figure 5b); the "wide" deck that has been the standard type till today.

The schematics of the bridge' structures longitudinal profiles are illustrated in Table 6 together with the influence lines of internal forces and moments at critical sections considered in this work: shear force at support and positive bending moment for simply and two-span continuous systems and negative bending moment at the support of cantilever and continuous systems. Span lengths range from 10 m to 40 m for simply supported and continuous systems, and from 2.5 m to 10 m for cantilever span. Figure 6 illustrates a typical grid numerical model used for static analyses, where T-beams and cross beams are represented by frame elements.

4. Traffic simulation

4.1 Developed computational tool

In order to obtain the critical internal forces produced by the simulated traffic, it was developed a computational tool that works in two steps [6]:

- *Traffic simulation*: the initial stage generates information for all the vehicles composing the traffic spectrum, such as vehicle class, speed, GVW, wheelbases etc. In order to generate values for the random variables, Monte Carlo technique is employed;

Figure 5 – Two-girder RC bridges cross sections considered in this paper: (a) narrow deck (ND); (b) wide deck (WD). Units: cm

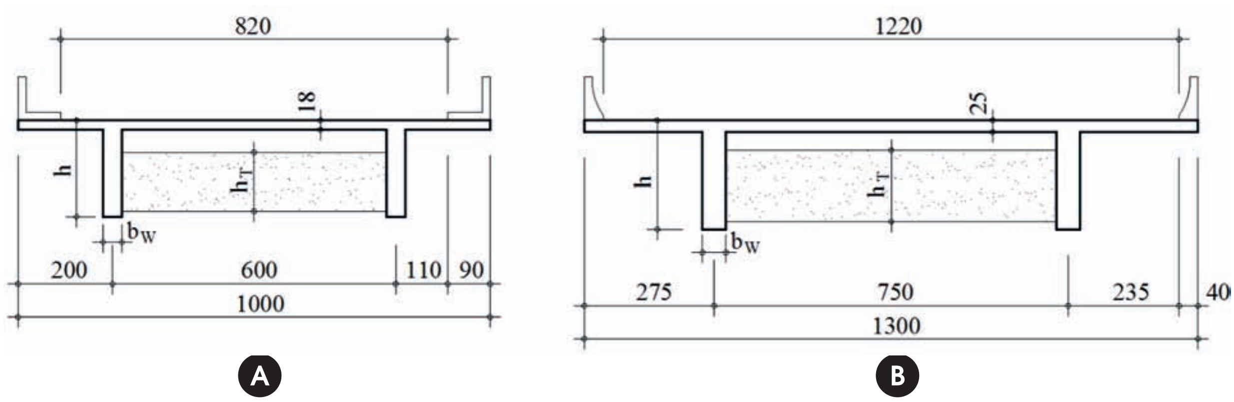
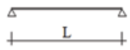
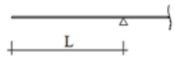
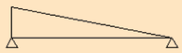
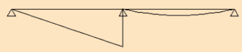
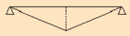
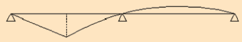
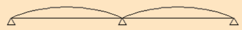
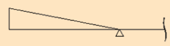


Table 6 – Structural systems, reference cross-section in girder and influence lines of critical effects considered in this paper

Structural system		Simply supported	Two-span continuous	Cantilever
Representation				
Shear force	Diagram			–
	Section	Support	Central support	–
Shear force	Diagram			–
	Section	Midspan	Approx. midspan	–
Shear force	Diagram	–		
	Section	–	Central support	Support

- *Structural analysis*: in this stage, vehicles generated by simulation travel along the model structures. Effects caused by the generated loading are recorded in certain sections (those indicated in Table 6) at each time step. The maximum effect at each loading cycle is also registered. At the end of simulation the process recorded effects are summarized in histograms.

According to O'Connor and O'Brien [16], four different traffic situations should be analyzed: traffic jams, mixed flow, free flow and emergencies. In small spans, critical load cases are due to heavy vehicles crossing and are affected by dynamic amplification factor. In large spans, however, critical load cases are due to simultaneous presence of several vehicles on structures in congested or mixed flow, with little or no dynamic amplification [11]. As the structures considered herein have maximum length of 40 m, congested and mixed flow situations were not analyzed. In free flowing, the time between vehicles is modeled as a random variable. Traffic is generated at each lane independently; correlations between random variables of the same vehicles in different lanes with same direction may be important [17], but weren't taken into account.

Traffic simulations were performed for a total of 30 days. The simulator checks if, at the current instant of time, there is at least one vehicle with at least one axle on the bridge. If so, the computational tool calculates the desired effects at the reference sections. Structural analysis ceases when all vehicles in all lanes have already travelled along the bridge; then histograms are produced for every considered effect.

For traffic simulation, speed histograms from H-2013 database were considered discarding values lower than 30 km/h or greater than 140 km/h. Time between vehicles was modeled by a gamma distribution [11]. The adoption of this distribution is suggested in the case of the process of vehicles arrival be idealized as a Poisson process. Due to lack of information, it was adopted a

Figure 6 – Typical single span bridge modeled as a 3D grid structure to perform static analysis

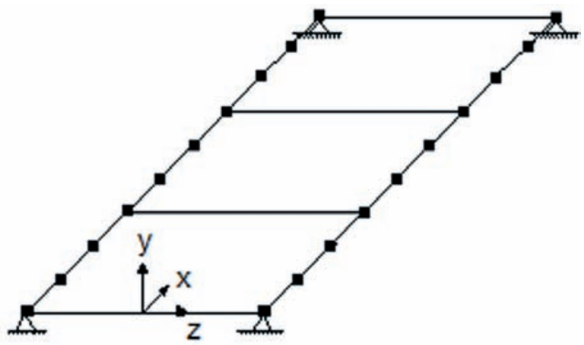
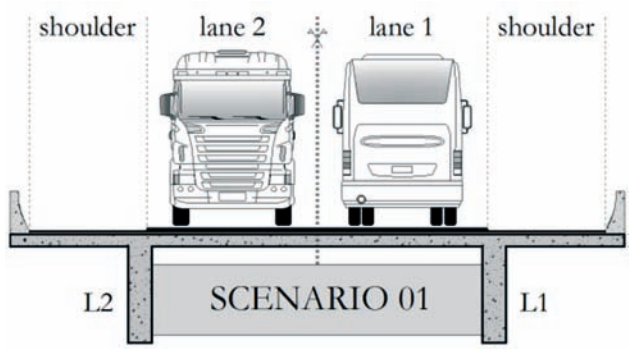


Figure 7 – Most frequent transverse position of vehicles passage



coefficient of variation equal to 0.5 for all distributions of time between vehicles. The simulation does not take into account acceleration, braking or lateral displacements.
In the structural analysis the force exerted by each tire is modeled by a concentrated load and the effects due to each load are calculated using influence surfaces. The traffic simulator was validated through some tests that demonstrated its accuracy [6].

4.2 Traffic scenarios considered

The transverse location of passing vehicles in opposite senses of traffic direction pictured in Figure 7, named scenario 1, is the most frequent scenario for one carriageway typical bridge with two traffic lanes centered along its axis; most of the heavy vehicles pass along the longitudinal axis of each lane. However, taking girder L1 as a reference for ultimate limit state design situation, the worst load case occurs for the traffic of vehicles out of the lanes marked along the pavement. Thus, many possibilities are opened for traffic on bridges. For the wider bridge deck considered in this work a variety of traffic situations are depicted in Figure 8 as scenarios 2 to 9, all of them for free flowing traffic of vehicles. It should be observed in this figure that the transverse distribution of lanes,

Figure 8 – Free flowing scenarios considered to design (ultimate limit state)

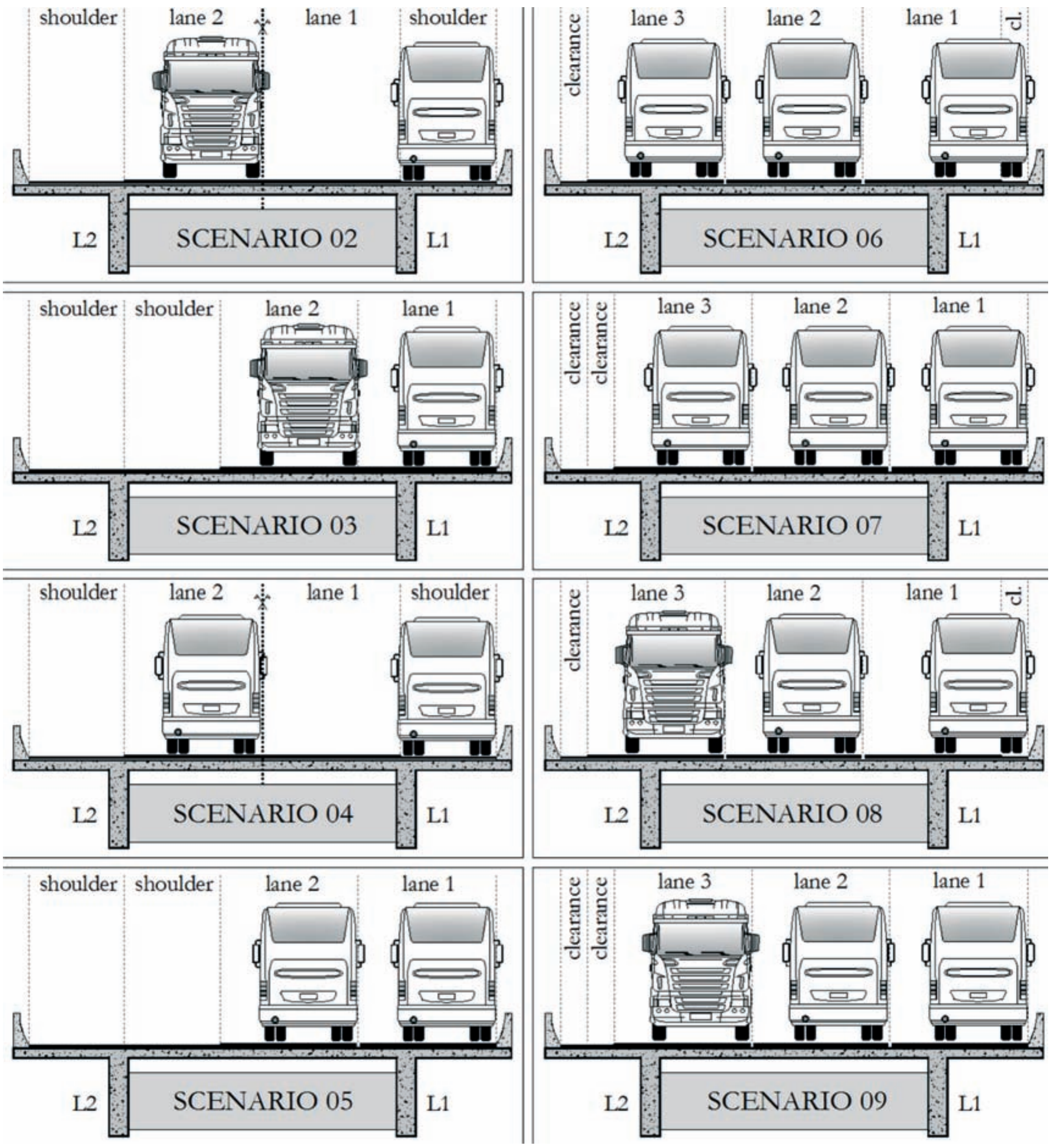


Table 7 – Proportions of total flow supported by each lane on two-lane (in same direction) carriageways

Reference	Division of the broad traffic	
	lane 1	lane 2
Prat (2001)	92.0%	8.0%
Getachew (2003)	89.7%	10.3%
O´Brien and Enright (2011) – A	92.3%	7.7%
O´Brien and Enright (2011) – B	93.8%	6.2%

shoulders and clearances differ from the actual situation in existing bridges [11]. Nevertheless it is considered in order to achieve the worst transverse load distributions, the vehicles traveling on the border lane or shoulder of the bridge deck, close to the lateral barrier or guard-rail.
In scenarios 3, 5, 7 and 9, lanes are located on the right, as close as possible to girder L1, while the shoulders are clustered on the left of the deck, to represent emergency or temporary construction situations. In scenarios 1 to 5, vehicles travel in two traffic lanes, according to original design assumption, while in scenarios 6 to 9 the carriageway is divided in 3 lanes to conform to traffic growth. All these situations are feasible only to the wide slabs (Figure 5b) since the traffic lanes are all 3.60 m wide. The scenarios considered herein intend to envelop all possible free flowing and emergency situations foreseen during lifetime of these typical RC bridges.

4.3 Adopted ADTT - Average Daily Truck Traffic

ADTT was estimated by using information from AB-2011 database whose average number of records equals 6,104 vehicles/day. However, the total number of commercial vehicles on the highway is greater than the number of records because (i) some vehicles avoid the weighing station, mainly due to legal limit weight surplus and (ii) the records were not obtained continuously since the weighing station closes in peak times until the queue of trucks is reduced to a few hundred meters. The actual number of commercial vehicles is estimated to be 15% greater than the volume measured by the weighing station, already including the presence of buses, which are not subject to weighing in this highway. Considering this "adjustment factor" the actual commercial vehicle ADTT in this database is estimated equal to 7,019 trucks and buses. This value is called *reference flow* (RF) and includes heavy traffic in all the three lanes of the considered highway.

4.4 Distribution of the traffic flow among the lanes

Due to lack of available recent traffic data collected directly in the lanes, the proportion of total traffic supported by each lane has to be estimated. Table 7 shows the distribution of total flow among lanes given by some authors, for two traffic lanes in the same direction. Values are relatively similar. In Getachew [18] and Prat [11] the proportions were estimated from detailed traffic studies performed respectively in roads of France and Sweden; O'Brien and Enright [17] refer to data collected on Netherlands (A) and Czech Republic (B) highways. For this study, in the case of two lanes in the same sense of traffic direction, it was assumed that 85% of total flow is supported by lane 1, lower than values presented in Table 7, since the reference flow RF refers to 3 lanes. The adopted proportions of total flow supported by each lane in each scenario are summarized in Table 8.

5. Determination of static effects' characteristic values

For Eurocode 1 load models calibration, the target values were taken with a return period of 1,000 years, to ensure a small probability of excess in effects' values: 0.1% per year [10]. This choice was made to limit the likelihood of several exceedances of the serviceability limit state during lifetime. The AASHTO load model HS-93 was calibrated assuming a return period of 75 years [13].
To set the return period for the extrapolations of static effects one

Table 8 – Characterization of traffic scenarios adopted for free flowing

Scenario	Number of lanes	$\bar{T}$ (years)	Lane 1		Lane 2		Lane 3	
			Direction	%RF	Direction	%RF	Direction	%RF
1	2	100	Go	85%	Return	85%	–	–
2	2	100	Go	85%	Return	85%	–	–
3	2	10	Go	85%	Return	85%	–	–
4	2	100	Go	85%	Go	15%	–	–
5	2	10	Go	85%	Go	15%	–	–
6	3	100	Go	80%	Go	18%	Go	2%
7	3	10	Go	80%	Go	18%	Go	2%
8	3	100	Go	85%	Go	15%	Return	85%
9	3	10	Go	85%	Go	15%	Return	85%

Table 9 – Wheelbases $d_{i-1,\ i}$ (m) and load of the vehicles that yielded the maximum bending moment of a simply supported 20 m span bridge with wide deck

Axle, i	Truck class 3S3-C, lane 1		Truck class 3M6, lane 2	
	$d_{i-1,\ i}$ (m)	Load (kN)	$d_{i-1,\ i}$ (m)	Load (kN)
1	0.00	55.8	0.00	58.9
2	4.80	100.2	3.46	87.2
3	1.30	100.2	1.35	87.2
4	3.09	107.6	5.32	85.8
5	1.25	107.6	1.25	85.8
6	1.25	107.6	1.25	85.8
7	–	–	5.06	86.4
8	–	–	1.25	86.4
9	–	–	1.25	86.4

should take into account that a very large return period is not representative [8] because the traffic probably will not remain with the same settings. The rapidly changing technology causes distortion of the load pattern in long-term violating the stationarity of these random processes which partially invalidates the large return periods, unlike natural phenomena such as wind speeds and river floodings.
On the other hand, the extrapolation is being held for random unmeasured but indirectly modeled quantities - the internal forces - which can generate errors, so that for safety conservatively large return periods must be adopted [11]. Considering both aspects a return period of 100 years was adopted for target values calculation in simulations according to scenarios 1, 2, 4, 6 and 8, which do not include relocation of lanes. As scenarios 3, 5, 7 and 9 refer to special situations, simulations under these configurations were carried out with a return period of 10 years. In all cases traffic growth was not taken into account for the extrapolation. These values are shown in the third column of Table 8.
For the critical static effects shown in Table 6, the following steps were accomplished in order to obtain the corresponding characteristic values:
- building of the histograms of static effects via traffic simulations for all scenarios shown in Figure 8, and fitting of a Weibull distribution for each histogram;
- calculation of the characteristic values of these distributions, using the probability level of the parent Weibull distribution and

Figure 9 – Illustration of the instant of time when the simultaneous presence of two trucks (a 3S3-C type on lane 1 and a 3M6 type on lane 2), side by side, both travelling at 80 km/h, generate together the largest bending moment at mid-span in girder L1 of the simply supported 20 m span bridge with wide deck, in scenario 3

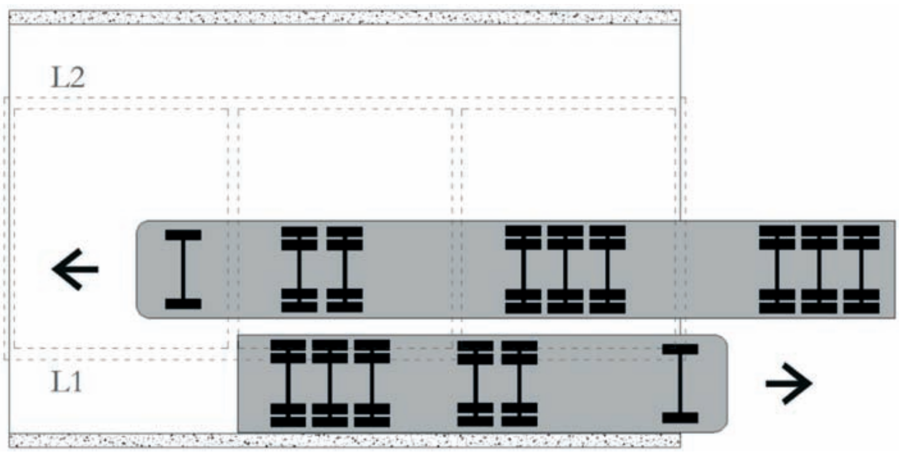
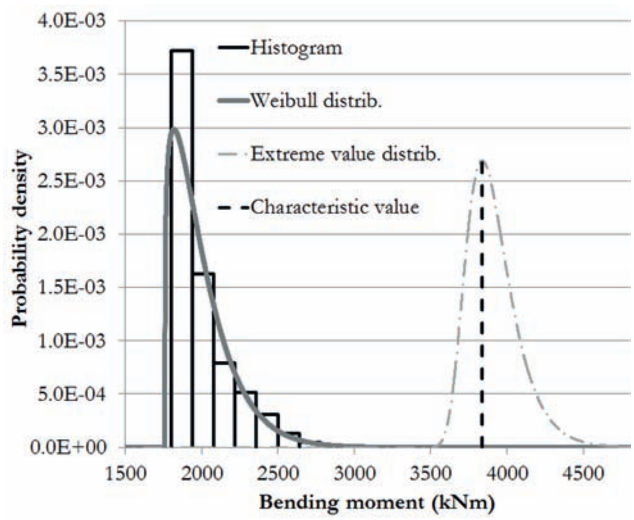


Figure 10 – Histogram and fitted Weibull distribution of the positive bending moment of the simply supported 20 m span bridge with wide deck



considering the return periods showed in Table 8. These characteristic values were took as representative of each static effect;

- adopting the scenario with the largest representative value as the reference for each considered effect in each structure.

Among all vehicle classes shown in traffic composition (Table 1 and Figure 2), those which effectively contribute to the extreme static effects are 2S3, 3S3, 3T4, 3T6 and 3I3; all of them are susceptible to high values of GVW.

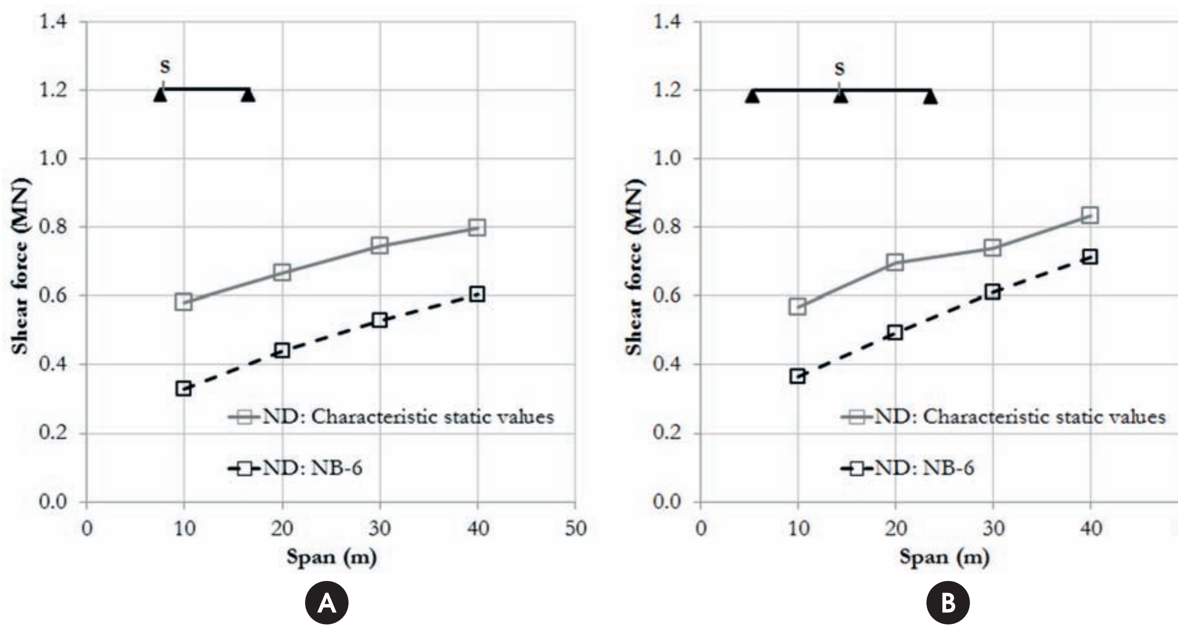
In short spans, up to 10 m, the presence of tridem axles from classes 2S3 and 3S3 govern the extreme effects. As the span lengths increase, also increase the likelihood of longer and heavier vehicles from classes 3I3, 3T4, 3T6 and 3M6 pass on the bridges with all axes simultaneously acting at multiple sections of the girders.

Although classes 3T6 and 3M6 are quite infrequent in the traffic composition, they correspond to the longest trucks and the biggest GVW among all classes from Figure 2; this could explain its importance in static effects' extreme values.

Taking as example the bending moment in a simply supported 20 m span bridge with wide deck (Figure 5b), the maximum value obtained by traffic simulation is 3059 kNm, due to simultaneous presence of two vehicles in scenario 3 - see Figure 8 - side by side, both travelling at 80 km/h: on lane 1 a 3S3-C truck with GVW equal to 578.9 kN; and on lane 2 a 3M6 truck with GVW equal to 749.9 kN. Each wheelbase and axle load of these trucks is shown in Table 9. Figure 9 illustrates the instant of time when these vehicles generate the largest bending moment at mid-span of girder L1.

Figure 10 shows the histogram of positive bending moments for the simply supported 20 m span bridge with wide deck, the Weibull distribution fitted to this histogram and its distribution of extremes, whose mode is equal to 3836 kNm. This value is equal to the extrapolated value obtained by the probability level using the parent

Figures 11 – Comparison in terms of static shear force between the extreme values produced by the real traffic and those generated by NB-6 Brazilian code load model for narrow deck (ND) bridges: (a) simply supported beam; (b) continuous beam

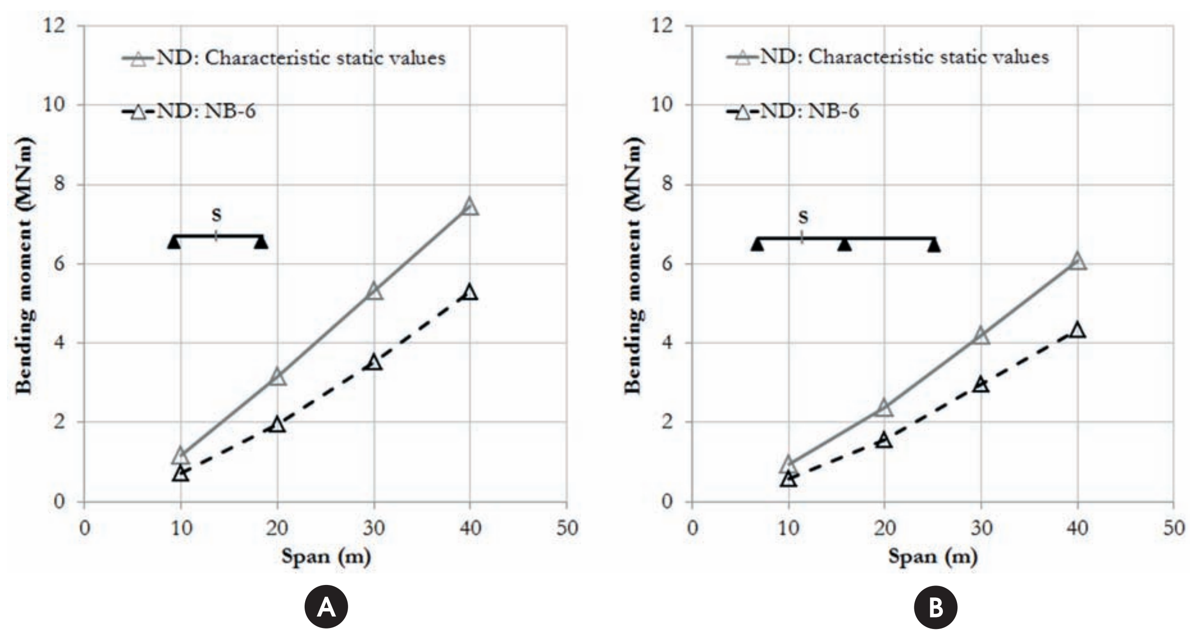


Weibull distribution and is taken as the characteristic value of the static bending moment.
The ratio between the characteristic value and the maximum value reached by the simulation in 30 days equals 1.254. One can check, assuming linear static behavior, that extrapolation leads to physically feasible results. This ratio, multiplied by the GVWs of the trucks that generated the greatest static bending moment within 30 days of traffic simulation (Table 9), results in these "new" GVWs: 726.0 kN for the 3S3-C truck on lane 1 and 940.4 kN for the 3M6 truck on lane 2. Retaining the same relative positions shown in Figure 9, this new combination of trucks would generate a bending moment equal to 3836 kNm. Both GVWs are smaller than its upper limits: 963.0 kN for 3S3-C and 1333 kN for 3M6. These values depend on the maximum axle loads (Table 3) and on the linear models considered to represent the load for each axle or group of axles, which is function of the GVW [5].

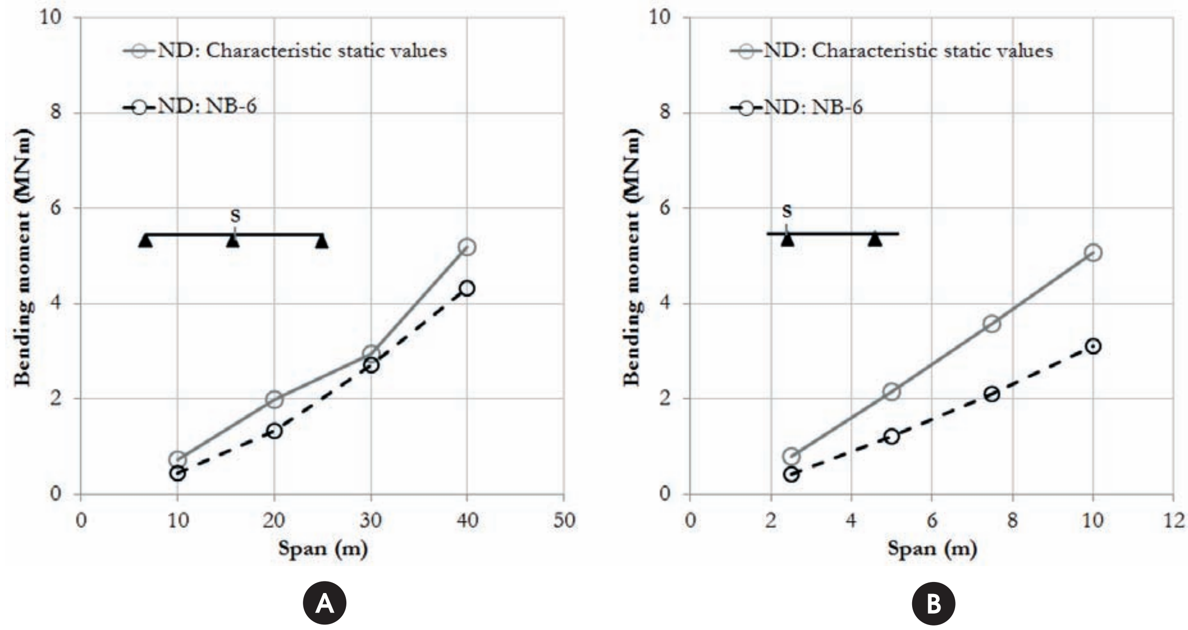
6. Comparison between the obtained characteristic static effects and those generated by the brazilian live load models

In Figures 11 to 16 the representative (extreme) values of the static effects (shown in continuous lines), which are caused by the traffic of real vehicles, are compared to those produced in the same structural models (see Figure 6) by the live load models prescribed in the Brazilian codes NB-6 and NBR 7188 (shown in dashed lines). Only the static effects produced by these live loads were considered as they were not multiplied by the impact factor. Figures 11 to 13 refer to the narrow bridge deck (ND) configuration designed with the old load model from NB-6 while Figures 14 to 16 are related to the wide deck bridge (WD) designed according to the current live load from NBR 7188.
Figures 11 and 14 illustrate the variation with the span length of the maximum shear forces in simply supported span (Figure 11*a* and 14*a*) and continuous spans (Figure 11*b* and 14*b*) bridges. Figures 12 and 15 show comparisons in terms of positive bending moments for each one of the same structural systems. Negative bending moments in continuous and cantilever spans are shown in Figures 13 and 16.
It can be seen in these figures that the static effects generated by the real traffic of vehicles as calculated according to the procedures described herein are in general greater than those produced by the past and current Brazilian standards load models. The most critical cases are related, as expected, to the narrow deck bridges: the static shear forces in simply supported bridges, the positive bending moments and the negative bending moment in cantilever bridges due to the real traffic of vehicles exceeded on average respectively 50%, 53% and 75% the static effects caused by the old NB-6 load model.
It is noticeable that the current Brazilian code gives conservative values only for the negative bending moment in 30 m and 40 m continuous spans with wide bridge deck (Figure 16*a*). The greater average exceedance of the traffic load static effect in relation to the corresponding code load model was found for the negative bending moments in cantilevers (Figure 16*b*): 48%.
These results show that Brazilian code load models may not reproduce adequately the real traffic of heavy vehicles and may, in many cases, be non-conservative. The dynamic effects and the modeling

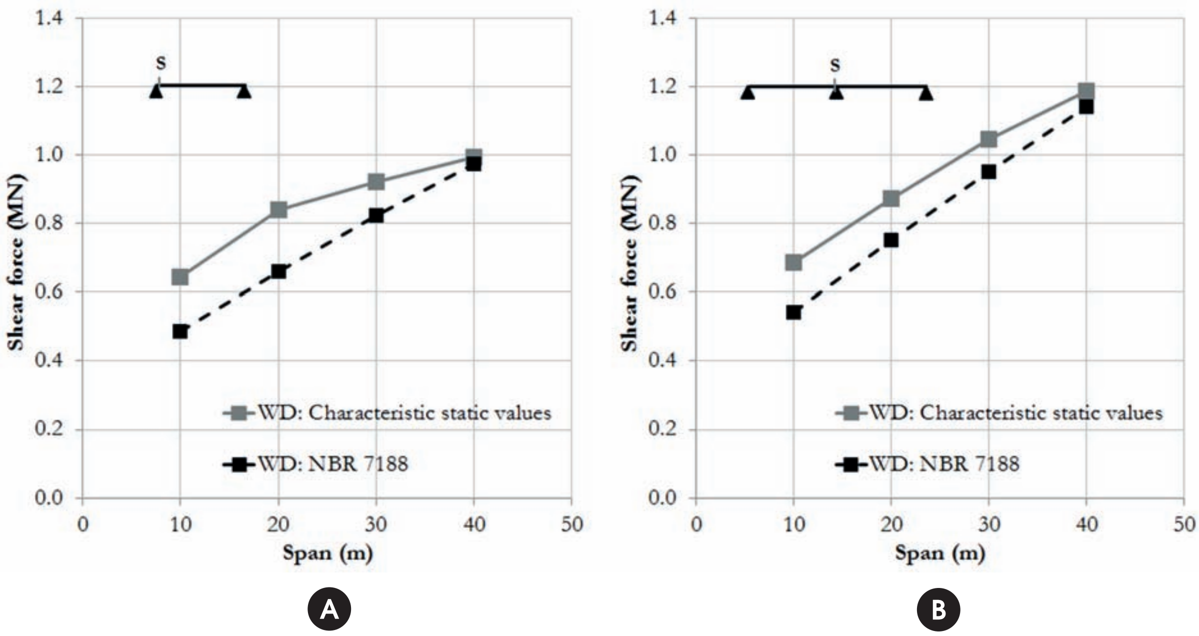
Figures 12 – Comparison in terms of static positive bending moment between the extreme values produced by the real traffic and those generated by NB-6 Brazilian code load model for narrow deck (ND) bridges: (a) simply supported beam; (b) continuous beam



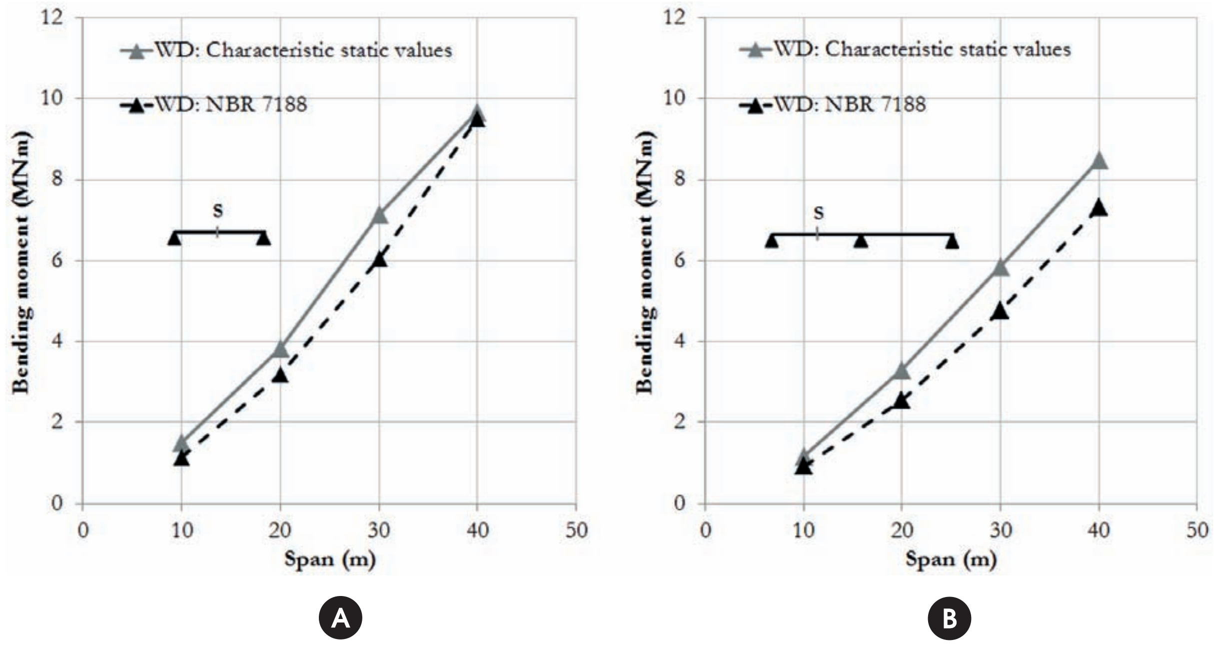
Figures 13 – Comparison in terms of static negative bending moment between the extreme values produced by the real traffic and those generated by NB-6 Brazilian code load model for narrow deck (ND) bridges:(a) continuous beam; (b) cantilever beam



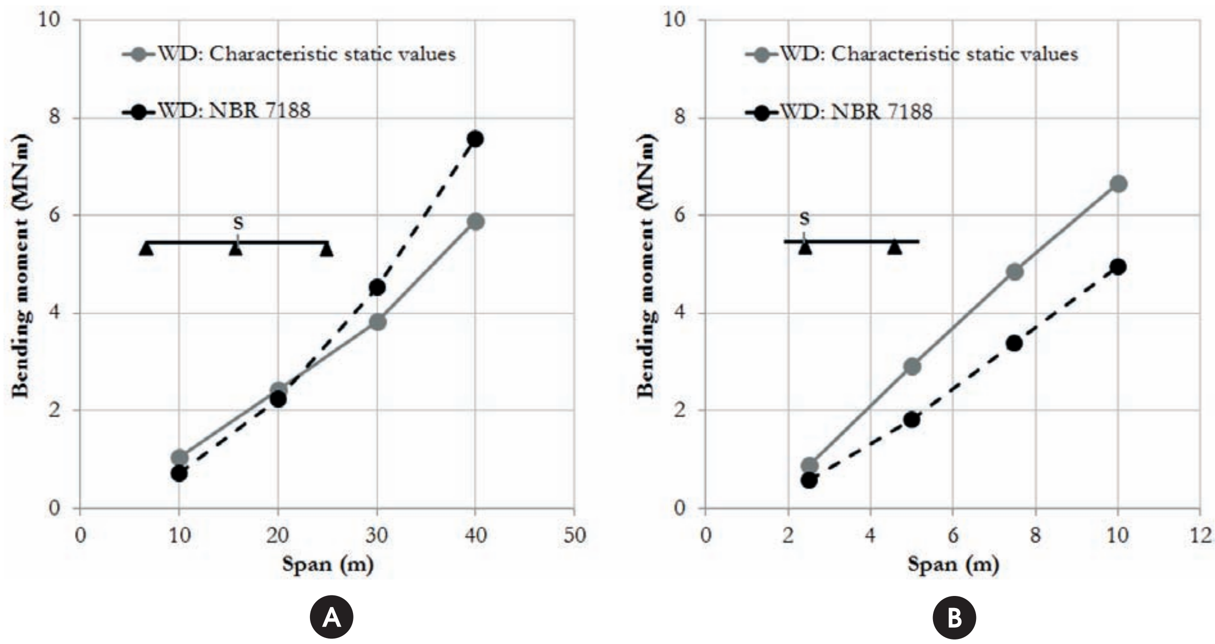
Figures 14 – Comparison in terms of static shear force between the extreme values produced by the real traffic and those generated by NBR 7188 Brazilian code load model for wide deck (WD) bridges: (a) simply supported beam; (b) continuous beam



Figures 15 – Comparison in terms of static positive bending moment between the extreme values produced by the real traffic and those generated by NBR 7188 Brazilian code load model for wide deck (WD) bridges: (a) simply supported beam; (b) continuous beam



Figures 16 – Comparison in terms of static negative bending moment between the extreme values produced by the real traffic and those generated by NBR 7188 Brazilian code load model for wide deck (WD) bridges:(a) continuous beam; (b) cantilever beam



of uncertainties must be taken into account for a final conclusion on this matter.
It can still be noted in Figures 11 to 16 that the curves related from one side to the real traffic of heavy vehicles and from the other side to load models are in general divergent with increasing span length, particularly in the case of bending moments. This indicates that the safety margin of the Brazilian load models is not uniform for all span lengths and structural systems.

7. Final remarks and conclusions

It is outlined in this paper the main results obtained in the first stage of the work performed towards the development of new live load models which aim to simulate the effects caused by the real traffic loads on existing two girders short span bridges, typical of Brazilian roadways comprising two lanes single carriageway.
The lack of a large number of WIM records was got around by establishing a database which brought together measurements from weighing stations and idealized traffic scenarios. Applied traffic simulation techniques allow for scenarios of multiple vehicles on the bridges and provided the histograms of the selected critical effects. Then Weibull distributions were fitted to these histograms from which the characteristic static effects were calculated by extrapolations according to the return period defined for the traffic scenarios. In each structure the representative values of the critical static effects were considered as the highest characteristic values among all traffic scenarios.
It was observed from the traffic simulations that in short spans (up to 10 m) the passage of tridem axles from classes 2S3 and 3S3 governed the extreme effects. In larger spans the critical effects were caused by the simultaneous presence of longer and heavier vehicles once all their axles can be located simultaneously on the bridge deck.
In most cases the static effects generated by real traffic, as calculated according to the procedures described herein, are higher than those produced by the load models from Brazilian design codes (without multiplying them by the dynamic amplification factor). These results show that Brazilian code load models may not reproduce adequately the real traffic of heavy vehicles and may, in many cases, be non-conservative.
The current Brazilian code NBR 7188 gives conservative values only for the negative bending moment in 30 m and 40 m continuous spans with wide bridge deck. The negative bending moments in cantilever spans produced by this code live load model are significantly lower than the extreme ones generated by the traffic of heavy vehicles.
A great number of bridges with narrow decks designed under the 1960 NB-6 code are still in full service. In was observed that static effects due to the load model from this code (see Figures 11 to 13) are always much lower than the extreme ones produced by the real traffic, indicating that these bridges may exhibit now a small fraction of the required safety margin and therefore should be reinforced.
It is important to mention that there are several sources of uncertainty in modeling which affect the numerical values obtained for the characteristic static effects. The uncertainties in the structural modeling do not affect the ratios between the internal forces due to the real traffic loads and those due to the design code live load models, since the same grid model is analyzed for both loading sources. On the other hand, uncertainties related to traffic statistics and simulation, in particular the need to idealize traffic scenarios and neglect of traffic growth rate may affect results to the better or to the worse. These uncertainties are to be addressed in the near future with enlarged traffic data acquisition, as they must be dealt with when performing the structural reliability analyses of the bridges.
In order to achieve fully the new load models complementary work steps 5 to 7 described in Section 1 are being carried out and results will be reported in the near future.
The configuration of the new live load models comprising concentrated and distributed loads must reproduce the target values of selected internal forces considering free, congested and mixed traffic flow. With the geometrical and physical features of the new load models, one can calibrate new related safety factors, seeking for a unique compromised reliability index to all selected types of bridges.

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