ENSON DE LIMA PORTELA

Analysis and Development of a Live Load Model for Brazilian Concrete
Bridges Based on WIM data

São Paulo

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Area of Concentration: Structural Engineering

Advisor: Prof. Dr. Túlio Nogueira Bittencourt.

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“Prediction, not narration, is the real test
of our understanding of the world.”

Nassim Nicholas Taleb

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ABSTRACT

This thesis presents an approach to evaluate and develop a live load model. Although the main purpose of this work is with the impact of truck traffic on bridges, the data presented in this work can be used in many engineering fields that are concerned with truck characteristics of geometry and weight. Data from two different WIM stations were considered. One in Fernão Dias highway in the State of São Paulo which is comprised of a same-direction two adjacent lanes and the sample is comprised of 20 months (September 2015 to August 2017). The second station is in Rio Grande do Sul State. This road is a same-direction three adjacent lanes. The sample is comprised of 78 days (March 2014 to June 2014)

In order to evaluate and develop a new live load model, an approach to compute load effects in terms of bending moments and shear forces is proposed. It makes use of single and multiple truck presence to evaluate the live load effects for different bridge spans. Three cases of multiple presence are considered: following, side-by-side and staggered. The proposed approach to evaluate the multiple truck presence effects is compared with the approach used by AASHTO LRFD. The approach for estimating the bias factors shows that considering only full correlated trucks is too conservative, mostly for short spans where there is a lack of occurrences, especially following events. On the other hand, taking into account no correlation at all yields very low bias factors. At last, a more rational live load model was developed based on WIM data.

Another purpose of this thesis is to use existing Brazilian bridges to calibrate the live load model as in NBR7188:2013. Reliability analysis is performed with sixty existing Brazilian bridges. The bridges are taken from different states of Brazil. Out of the sixty bridges, 39 are prestressed and 21 reinforced concrete bridges. Those bridges are located in five different states: Pernambuco, Ceará, Bahia, São Paulo, Minas Gerais. Probability of failure was estimated in terms of moment and shear for interior girders and box girders. Only ultimate state limit was considered. It was found that reliability indices are higher in prestressed bridges when compared to reinforced bridges. Also, the reliability indices tend to decrease as the span length increases. This means that for larger spans the probability of failure is higher than the ones for shorter spans.

Keywords: Concrete Bridges; Live Load; Reliability Analysis; Code Calibration.

RESUMO

Este trabalho apresenta uma abordagem para avaliação e desenvolvimento de modelo de carga móvel. Embora o principal objetivo desta tese seja averiguar o impacto de caminhões nas pontes, os dados apresentados aqui podem ser usados em qualquer aplicação de engenharia que dependa das características do tráfego de caminhão. Dados de duas estações WIM foram utilizados. Uma estação fica na Autoestrada Fernão Dias no Estado de São Paulo e possui 20 meses (Setembro de 2015 a Agosto de 2017) de dados coletados. A outra estação fica no estado do Rio Grande do Sul. Esta amostra tem 78 dias e foi coletada de Março de 2014 a Junho de 2014.

Com o objetivo de avaliar e desenvolver um novo modelo de carga móvel, uma abordagem para estimar o efeito de carga em termos de momento fletor e esforço cortante é proposta. Este método faz uso de estatísticas de caminhões em múltiplas presenças e isolados. Três casos de múltiplas presenças são considerados: “Following”, “Side-by-side” e “Staggered”. A abordagem proposta é comparada com o método usado pela AASHTO LRFD. A abordagem para estimar os “bias factors” mostra que considerar apenas caminhões totalmente correlacionados é muito conservador, principalmente para períodos curtos onde há uma falta de ocorrências, especialmente para eventos “Following”. Por outro lado, não considerar a correlação de peso dos caminhões resulta em valores muito baixos de “bias factors”. Por fim, um modelo de carga móvel mais racional foi desenvolvido com base nos dados WIM.

Outro objetivo desta tese foi usar pontes brasileiras existentes para calibrar o modelo de carga móvel descrito na NBR7188:2013. Análises de confiabilidade foram realizadas em uma amostra de sessenta pontes brasileiras, sendo que destas 39 são protendidas e 21 armadas. Elas estão localizadas em cinco diferentes estados: Pernambuco, Ceará, Bahia, São Paulo e Minas Gerais. As probabilidades de falha foram estimadas em termos de momento fletor e cisalhamento para vigas internas e vigas caixa. Apenas o estado limite último foi considerado. Verificou-se que os índices de confiabilidade são maiores nas pontes protendidas quando comparadas às pontes armadas. Além disso, os índices de confiabilidade tendem a diminuir à medida que o comprimento do vão aumenta.

Palavras-chaves: pontes de concreto; carga móvel; análise de confiabilidade; calibração de norma.

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1. INTRODUCTION

1.1 OVERVIEW AND MOTIVATION

The first Brazilian live load model for bridge design was originally published during the 1940's under the name NB6 (Timerman, 2015). This code was basically an adaptation of the German code (DIN 1072) and no statistical analysis was carried out in order to consider the Brazilian fleet characteristics which, by that time, was small and insignificant from the structural point of view. After many years the current Brazilian code (NBR7188:2013) still consists of a similar live load model and only a few studies have been done to verify the probability of failure of bridges designed with this model. Although it might sound as if the Brazilian code has never significantly improved, it needs to be considered that the designed bridges are still working and many of those bridges are more than fifty years old. The important point to be addressed here is: how safe are these bridges under the current traffic? Also, how safe are the bridges that are going to be designed with the current code? Is a short span bridge safer than a long span bridge? Is the current live load model capable of safely covering the effect of volume and gross vehicle weight of nowadays traffic? The answer for those questions lies in the analysis of traffic loads and existing bridges resistance.

On the load side, Brazil has a historical problem of not keeping track of truck weights. The country barely has operational static weighing station that provides a limited dataset of truck weights. In addition, it is well-known that samples from static weighing stations have a highly probability of being biased, given the fact that the overweight trucks will tend to bypass the station. Besides, the station will hardly be taking measurements of trucks for twenty-four hours straight. In practice, the station works a few hours a day and not every day of the week. This is more than enough to conclude that samples from those stations will likely be biased and should not be used when proposing a live load model. Hence, the use of unbiased samples must be taken into account. In this case, high speed weigh-in-motion systems are strongly recommended, since they weight the trucks without the awareness of the drivers and can be operational twenty-four hours per day, with an acceptable accuracy. The need of knowing the truck weights lies in the fact that when designing long-lasting concrete structures as road bridges, information is needed about current load for estimating future loads.

On the resistance side, two problems come up when performing a reliability analysis of existing bridges. First, the National Department of Transportation (DNIT) does not offer an easy or friendly-user access to projects of Brazilian bridges. Therefore, gathering information, such as typical cross-section and material properties of existing bridge design is a “mining work”. Second, in engineering, the quantities have an inherent variability and cannot be expressed as a single value or point (that explains why we call those quantities “variables”). This means that a probabilistic approach is more recommended when setting a design method for concrete structures. However, there is no public dataset that you can have an easy access even though there is the law N° 12.527:2011 that endorses permission of Brazilian citizens to public information. Finally, adding the difficulty of finding reliable information on resistance and load sides to properly perform a reliability analysis of Brazilian live load model for bridges, one can conclude that it is so difficult to answer the questions presented in the first paragraph.

This thesis is an attempt of solving this problem. It uses WIM data from two Brazilian stations: one in Sao Paulo and the other in Rio Grande do Sul, to evaluate and propose new safety factors, as well as a new live load model. It also tackles the previously pointed problem of modeling the resistance side by evaluating sixty typical Brazilian bridges that were built in different decades (emphasizes is given to bridges built during the 2000’s). Reliability analyses are performed using statistics found for Brazilian concrete and steel used in civil construction. Probability of failure is then computed and safety factors are defined in order to uniform the reliability index.

1.2 RESEARCH OBJECTIVES

The main objective of this thesis is to evaluate and propose a live load model for Brazilian reinforced and prestressed concrete bridges using WIM data and reliability analysis that is capable of setting uniform and acceptable safety levels for those structures. Also, this work aims to evaluate the safety of existing Brazilian bridges considering the current truck traffic as registered by two WIM systems. To achieve such objectives a probabilistic approach is considered.

1.3 DOCUMENT OUTLINE

This thesis has six chapters including this introductory one. A briefly description of each chapter is presented as follows:

Chapter 1 is an introduction of the thesis and presents the main motivations of this work.

Chapter 2 discusses weigh-in-motion systems and presents both systems that are taken into account in this thesis. This chapter presents the characteristics of both WIM systems, such as type of material, length, installation procedure and calibration of the system. This chapter also discusses the quality control of the data and presents a filtering process that is based on Brazilian fleet.

Chapter 3 presents a statistical study based on WIM data from both stations. Statistics of traffic volume and truck weights are considered. Also, data for the main five classes of trucks are presented. Mean and standard deviations of axle load and spacings are shown. Finally, a study of multiple truck presence is conducted. Statistics of side-by-side, following and staggered events are shown.

In Chapter 4 a development of Brazilian live load model based on WIM data is provided. This chapter briefly presents a historical study of the Brazilian live load model currently valid. Analysis of live load for single and multiple trucks are presented in terms of bending moment and shear. An approach to compute the load effects due to multiple trucks over a bridge is also presented. Finally, a live load model based on WIM data is suggested.

Chapter 5 provides a reliability analysis for existing bridges in Brazil. Sixty bridges are used and probability of failure in terms of bending moment and shear per girder is computed. An investigation of statistics of variables on resistance side (concrete and steel strength, modulus of elasticity, cross section of rebars) is also presented. Reliability index for prestressed and reinforced concrete bridges are shown. Finally, safety factors are recommended.

Chapter six contains the conclusions and recommendations for future works.

2. CONSIDERATIONS ON WEIGH IN MOTION SYSTEM

This chapter makes a brief explanation of weigh-in-motion (WIM) systems. It describes the main advantages and drawbacks they have. It also describes both WIM systems used in this thesis and shows how to properly check the data quality. At the end, it presents the final results of applying filtering criteria adopted for Brazilian fleet.

2.1 CHARACTERISTICS OF DIFFERENT SYSTEMS

It is well-known that on national roads it is common to find overweight trucks (Albano, 2005; Peterlini, 2006; Fontenele et al, 2011). The two major reasons why this is so common in Brazil lies on the lack of surveillance and supervision by the public institutions in charge and the disrespect of truck drivers in relation to the GVW limits imposed by Brazilian regulation. Commonly, when truck drivers see the station they bypass it by taking a secondary road. In this sense, high speed weigh-in-motion (HS-WIM) systems become a potential alternative to achieve a more reliable supervision of overloaded trucks, once in order to weight the vehicle there is no need for the truck to stop or reduce speed. Also, drivers are unaware of the system and do not deviate from the route.

Although HS-WIM systems have a lot of advantages they still have some points that need to be addressed when considering implementing this kind of system. First, they need periodic calibrations due to the temperature effect. At the beginning, the installation depends on highly skilled workers. In addition, the sensors may have a limited life cycle which means that it might be necessary to change the sensors that are embedded in the pavement (Brito, 2013). Despite these disadvantages, WIM system has obtained a lot of attention for the past three decades, mainly due to its extensive possibilities of application in the infrastructure (Gindy and Nassif, 2007). They are used for statistical studies (Chapter 3), for evaluation and development of live load models for bridge designing (Chapter 4), design and maintenance of rigid and flexible pavements and evaluation of existing bridges (Chapter 5).

According to Jacob, O'Brien and Jehaes (2002) there are many WIM systems available and the selection of a given type depends on many criteria, for instance the system accuracy, road geometry, pavement characteristics and etc. Brito (2013) shortly presented the most well-known types of WIM system which includes bending plate, bridge WIM, quartz sensors, piezoelectric polyvinylidene fluoride (PVDF) sensors and etc. Bending plate systems consist

of plates with strain gauges placed under the wheels or axles to measure their static or dynamic tire force. Bridge WIM uses an instrumented bridge as a sensor; the strain is measured in some of the bridges elements and is used to determine the axle loads of vehicles crossing the bridge. Quartz and PVDF have the same base of work once both use piezoelectric principle to measure loads. Both systems have elements that are wired and arranged in a way that uniform measurements are given no matter where the truck wheel applies the force along the sensor length. In this thesis more attention is given to PVDF sensors once both WIM stations use this type of material. The layout of the WIM data is unique to the manufacturer, but the content is the same among all types of systems. The format for the data used in this study is given in Table 2. 1.

Table 2. 1 – Typical parameters of WIM systems.

Variable	Parameters - Dimension
Date	[year-month-day]
Time	[hour-minute, 1/100 seconds]
Day number	[Day]
Temperature - sensor 1	[°C]
Temperature - sensor 2	[°C]
Temperature - sensor 3	[°C]
Temperature - sensor 4	[°C]
Lane	Lane code: U1 ou U2 (left and right)
Speed	[km/h]
Aceleration	[cm/s ²]
Number of axles	Integer
Number of group of axles	Integer
Type of groups of axles	Axle number per group of axles
Length	[cm]
GVW	[kg]
Total spacing	Distance between first and last axle [cm]
Each Axle spacing	[cm]
Each axle weight	[kg]
Sensos type	PVDF

Both systems follow the recommendation of ASTM E1318 (2002). This document set specification for highway WIM systems classification as follow:

a) Type I

According to ASTM E1318 (2002), Type I WIM Systems correspond to those installed on highways with one or more lanes with the purpose of counting and classifying vehicles with speeds that range from 16 to 130 km/h. This type of system is capable of collecting data as in Table 2. 2.

b) Type II

Type II WIM Systems are those installed on highways of one or more lanes in order to count and classify road vehicles with speeds ranging from 24 to 130 km/h. This type of system is also capable of collecting data as in Table 2. 2.

c) Type III

The Type III WIM Systems are those installed on highways with one or more lanes where it is desired to pre-select trucks suspected of being overweight. This system is able to record trucks with speeds ranging from 16 to 130 km/h and it collect data as in Table 2. 2.

d) Type IV

The Type IV WIM Systems are those installed on highways with one or more lanes where it is intended to collect data of trucks traveling at speeds ranging from 3 to 16 km/h.

Table 2. 2 – Type of WIM systems according to ASTM E1318 (2002).

Data Item Produced by WIM Systems	Type I	Type II	Type III	Type IV
1. Wheel Load	•		•	•
2. Axle Load	•	•	•	•
3. Axle-Group Load	•	•	•	•
4. Gross-Vehicle Weight	•	•	•	•
5. Speed	•	•	•	•
6. Center-to-Center Spacing Between Axles	•	•	•	•
7. Vehicle Class (via axle arrangement)	•	•		
8. Site Identification Code	•	•	•	•
9. Lane and Direction of Travel	•	•	•	
10. Date and Time of Passage	•	•	•	•
11. Sequential Vehicle Record Number	•	•	•	•
12. Wheelbase (front-most to rear-most axle)	•	•		
13. Equivalent Single-Axle Loads (ESALs)	•	•		
14. Violation Code	•	•	•	•

Both WIM systems used in this thesis are Type I and present accuracy as in Table 2. 3.

Table 2. 3 – Functional Performance of WIM Systems in this thesis.

Data Item Produced by WIM Systems	Type I
Gross-Vehicle Weight	$\pm 10\%$
Individual Axle weight	$\pm 15\%$
Group Axle Weight	$\pm 15\%$
Traffic Volume	$> 99.5\%$
Speed	$\pm 1.5\%$
Length	$\pm 8.0\%$

2.1.1 The HS-WIM in Fernão Dias (BR-381)

The first WIM station is located at BR-381 also known as Fernão Dias Highway. This is a Federal road that connects São Paulo to Belo Horizonte, two of the most important cities in Brazil from the economic point of view. This road is currently under concession to Arteris S.A. who supports the work in this thesis. Fernão Dias highway consists of a same-direction two adjacent lanes. For each of the two lanes, the HS-WIM system is comprised of two lines of PVDF sensors, two inductive loops (Figure 2. 1) and temperature sensor. Hence, the final layout of the WIM stations is L-P-L-P (Loop-Piezo- Loop-Piezo). The PVDF sensor is 2.73 m (9 ft) long and 1.27 cm (0.5 in) wide. The inductive loops detect the vehicles, measure the distance between axles and the vehicle speed. The WIM System was installed in July 2015, and so far four calibrations were conducted. This HS-WIM allows the acquisition of raw generic data, which ultimately allows the management of the records as the user wishes. In this project, the system is connected to the concessionaire's network and the data are directly sent to the Structures and Structural Materials Laboratory at the University of São Paulo – LEM/USP. The author is currently affiliated to this laboratory.

Figure 2. 1 – Definition of layout and sensors positions – BR 381

Figure 2. 2 to Figure 2. 6 show the installation of WIM system in São Paulo. The installation basically consists of the following steps:

1. Clean the pavement surface. If necessary, use air-blower to remove excessive dust (Figure 2. 2);
2. Measure and mark WIM sensors position (Figure 2. 2);
3. Cut the pavement for the sensor slots (Figure 2. 3);
4. Clean and dry the sensor slots (Figure 2. 3);
5. Wipe WIM sensors;
6. Place the installation brackets on the sensor at an interval of 15 to 20 cm (Figure 2. 4);
7. Place the sensor with brackets and/or cables into the cut slot, and check the sensor depth with a depth gage (Figure 2. 4).
8. Pour the grout mixture over the sensor and cables (Figure 2. 5);

Figure 2. 6 shows the station after all steps of installation.

Figure 2. 2 – Definitions of sensors position – Fernão Dias BR 381

Figure 2. 3 – Cutting the pavement – Fernão Dias BR 381

Figure 2. 4 – Placing the brackets and sensors – Fernão Dias BR 381

Figure 2. 5 – Pouring grout over the slots – Fernão Dias BR 381.

Figure 2. 6 – Final appearance of the pavement – Fernão Dias BR 381.

As mentioned before, WIM systems provide data that include the essential characteristics of truck population: date and time of arrival, speed, class, axle weight and spacing, length, etc. The raw WIM data is recorded in a binary format. Thus, the data compression makes it possible to store a large amount of truck data. Typically, for a single truck all the information is stored as in Figure 2. 7. For São Paulo WIM station the average day has 13000 records and the .txt file is 3MB in size which corresponds to 0.23 KB per record.

```
@3f,BRT-FD0001,0000,2015-07-14 12:24:19.020,206,20.00,20.00,0.00,0.00$VH:B1,62.54,
37.52,29900,29900,T2R3,10,5,3,113,-200,0,16205,95$AD:1309,443,621,122,123
$AL:1936,3404,2736,3973,4156$GA:1936,3404,10865$EF:076620,2000,1$WD:0,0,0,0,0,0,N
$WL:0:0.00:0,0,0,0,0&0,0,0,0,$SCL:PVDF:2:1734,2967,2228,3700,4163&
2138,3842,3246,4246,4150
```

Figure 2. 7 – Example of typical record by WIM system – Fernão Dias BR 381

2.1.2 The HS-WIM in BR290 – Rio Grande do Sul

The second WIM station is located at BR-290. This is also a Federal road that connects Porto Alegre to Osorio in Rio Grande do Sul. The WIM sensors are installed nearby the km 72. This road is currently under concession to Concepa S.A. who supports the work done by Brito (2013) and (2015) and currently used in this thesis. BR-290 is a freeway that consists of a same-direction three adjacent lanes. The HS-WIM system is comprised of two lines of PVDF sensors and one inductive loop (Figure 2. 8). Although the freeway has three lines, only the two far right have sensors. Hence, the final layout of the WIM stations is P-L-P (Piezo- Loop- Piezo). The PVDF sensors are 3.64 (12ft) long. The inductive loops detect the vehicles, measure the distance between axles and the vehicle speed. The WIM System was installed and calibrated in 2013.

Figure 2. 8 – Definition of layout and sensors positions – BR-290/RS.

Figure 2. 9 to Figure 2. 12 show the installation of WIM system in Rio Grande do Sul. The installation basically consists of the same eight steps as in São Paulo station and previously discussed.

Figure 2. 9 – Setting sensors position – BR290/RS (Brito, 2013).

Figure 2. 10 – Cutting the pavement – BR290/RS (Brito, 2013).

Figure 2. 11 – Pouring grout over the slots – BR290/RS (Brito, 2013).

Figure 2. 12 – Final appearance of the pavement – BR290/RS (Brito, 2013).

2.2 CALIBRATION OF WIM SYSTEMS

As piezoelectric sensors are strongly influenced by the temperature variation, a compensation of these effects must be taken into account when using WIM systems to accurately measure the weight of trucks. Accordingly, one common procedure is to perform a calibration test with a known weight truck. In this methodology, the truck crosses the WIM station several times in different speeds and time of the day. The system response is recorded and compared with the weight of the truck (measured in a static scale) and then adjustments of the system are made if needed. One major disadvantage of this practice is that, in order to obtain a reliable set of correction factor (also known as calibration factors) the truck must cross the WIM sensors in a different range of temperature; preferably the range of temperature should cover the temperature variations for one year.

2.2.1 The HS-WIM in Fernão Dias (BR381/SP)

So far, five calibrations have been performed on the WIM system currently in operation in São Paulo. These calibrations were carried out in July/2015, October/2015, February/2016, February/2017 and May/2017. Performing these tests over different months of the year is a good way of calibrating the system considering a wider range of temperature. This process provides a good estimate of the calibration factors that will be used to help offset the effects of the rapid changes in daily and seasonal temperature of pavement. Figure 2. 13 shows the three-axle truck used during the calibration. The truck was loaded with gravel and weighed in a static scale. Figure 2. 14 show the truck crossing the WIM station during calibration.

Figure 2. 13 – Truck used for calibration – Fernão Dias BR 381.

Figure 2. 14 – Truck crossing the WIM system during calibration – BR 381/SP

These calibration events are as follows:

1. The first calibration test was performed on July 14-15, 2015 when a Engineer from NewConstech (NCT) was present. The temperature range recorded was between 15.6°C and 32.6 °C. The calibration tests were performed from morning to late afternoon to cover various daily temperature ranges that would be typically observed during winter season;
2. The second calibration test was performed on September 29 and October 9, 2015 when the NCT Engineer was not present. The temperature ranges recorded were between 31.4 °C and 42.7 °C. The test was performed at around noon only and did not cover the daily temperature range;
3. The third calibration test was performed on February 2-3, 2016 when the NCT Engineer was present. The temperature ranges were between 25.5 °C and 46.3 °C that covered daily temperature ranges that would be typically observed during summer;
4. The forth calibration test was performed on February 17, 2017. The temperature ranges were between 16.9 °C and 24.7 °C that covered daily temperature ranges;
5. The third calibration test was performed on May 16, 2017. The temperature ranges were between 33.5 °C and 43.3 °C that covered daily temperature ranges that would be typically observed during summer;

Figure 2. 15 and Figure 2. 16 show the typical results of a calibration test. Each of empty squares in Figure 2. 15 and the empty circles in Figure 2. 16 correspond to the truck crossing

the WIM system. The empty shapes show the results of GVW error x Temperature when the system is not calibrated. It is worth noting that the system reach errors of +/- 40%. The truck weight is then adjusted by the known weight of the truck used during the calibration procedure and the results can be plotted again. The filled shapes in Figure 2. 15 and Figure 2. 16 represent the final response of the system, i.e., the results after applying the correction due to temperature effects. It is worth noting that the error is now within +/- 10%.

Figure 2. 15 – GVW error versus Temperature – Left Lane – Fernão Dias BR 381.

Figure 2. 16 – GVW error versus Temperature – Right Lane – Fernão Dias BR 381.

The calibration for Rio Grande do Sul WIM system was performed on February/2013 and is well described in Brito (2013). They used two trucks in their calibration. One is similar to the one used to calibrate São Paulo WIM system, which is a three-axle truck weighing 20.5 tons. The second vehicle is a five-axle truck weighing 34.5 tons. The truck crossed the system with speeds that ranged from 40 to 100 km/h and the temperature was found to vary from 20 °C to 50°C. For further details see Brito (2013) and Brito (2015).

2.2.2 *Quality Control of WIM data*

It is highly recommended to check the quality of WIM data before using them to perform any kind of analysis. As pointed out by Davis (2007) there are many causes for poor quality WIM data. For example, the system might be considering two or more close trucks in following position as one single vehicle. There is also the possibility of improper sensors installation or settings, the pavement can suffer excessive cracks and spalling which will affect sensors response. Finally, according to Huang (2004) for hot-mix asphalt (HMA), at higher temperatures, the pavement is more viscous. In this case the load of a truck transmitted to the sensors is larger than the load for lower temperature. Since piezoelectric material use the mechanical stress generated by the tire of a truck to measure the axle weight, the stiffness of the pavement has a great influence on the final estimation of the load.

With the objective of checking the quality of WIM data the test proposed by Southgate (2001) and applied by Davis (2007) is considered in this thesis. In this well-known approach, Southgate (2001) used the front axle weight (FAW) of Class 9 (according to FHWA classification) truck as in Figure 2. 17 to check the WIM data using a logarithmic regression. He used these trucks not only because they are common commercial trucks in USA but also because they have a stable relation between the FAW and the steering axle spacing. Southgate (2001) established a logarithmic relation between those two variables and used this relation to check the quality of WIM data. In Brazil the correspondent trucks to Class 9 are the trucks Class 3S2 according to the National Department of Transportation (DNIT) classification system. In Brazil these trucks are not common and due to their low daily percentage the sample size is defined based on a period of 15 days, since the test needs a certain number of 3S2 trucks – in this thesis at least 500 trucks were considered.

Figure 2. 17 – Configuration of truck Class 3S2.

It is important to consider that the Southgate approach to check the WIM data quality was developed based on USA trucks. Thus, new minimum and maximum limits must be defined in relation to Brazilian 3S2 fleet. In order to find the characteristics of trucks class 3S2 an investigation of seventy-six catalogs from the main truck manufactures (Scania, Volvo, Mercedes, Volkswagen) in Brazil was carried out. The spacing between first and second axles varies from 300 to 370 centimeters while the minimum front axle weight varies from 3897 to 4767 kgf. Table 2. 4 summarizes the final relation between minimum FWA and axle spacings of 3S2 trucks. The maximum weight for front axle of these trucks is 7100 kgf no matter the axle distance.

Table 2. 4 – Minimum front axle weight and axle spacing for Brazilian trucks 3S2.

First to second axle spacing (cm)	Front Axle Weight (kg)
300	4767
310	4622
320	4302
330	3897
350	4392
355	4471
360	4487
370	4293

After converting the data in Table 2. 4 to feet and pounds and applying a logarithmic regression as recommended by Southgate (2001) Figure 2. 18 to Figure 2. 22 were built and used to check the quality of WIM data. The method consists of comparing the logarithmic regression of the WIM data (red lines) with a reference logarithm curve (blue lines). The results are considered acceptable when the regression of the data generates a smooth curve that approaches the reference curve respecting the upper and lower limits and also when most data points are within the Maximum Minimum limit curves.

Figure 2. 18 – Southgate test for São Paulo WIM data – Right Lane – Oct/2015.

Figure 2. 18, Figure 2. 19 and Figure 2. 20 show the results for São Paulo WIM data. As we can see the regression line (red one) is almost identical to the reference line. Also, most the data points are within the thresholds curves. Therefore, the weight calibration is satisfactory. Figure 2. 21 and Figure 2. 22 show the results for Rio Grande do Sul WIM data. Again, the significant majority of data points are within the upper and lower limits curves and the regression line is close to the reference line. WIM data that was found to be roughly out of calibration was removed from further studies. After passing the check for quality, the data is then examined by applying filtering routines as will be shown in the next section.

Figure 2. 19 – Southgate test for São Paulo WIM data – Right Lane – Jun/2016.

Figure 2. 20 – Southgate test for São Paulo WIM data – Right Lane – Jun/2017.

Figure 2. 21 – Southgate test for Rio Grande do Sul WIM data – March/2014.

Figure 2. 22 – Southgate test for Rio Grande do Sul WIM data – June/2014.

2.2.3 WIM data Filtering

Once the quality of WIM data has been checked there is another important step to minimize the possibility of erroneous and irrelevant data. An example of erroneous truck is a record that shows a vehicle with a GVW or axle weight that is physically impossible. On the other hand, an irrelevant record would be motorcycles or very light vehicles as the ones with less than 6.2 tons. They are irrelevant from the structural point of view once the load effects induced by those vehicles are negligible.

It is important to mention that truck records that not fit Brazilian legislation not necessarily mean erroneous data. Data from the WIM system must be filtered to meet the analysis scenario and settings. With that in mind, the following are the criteria that were considered in filtering the data acquired by the WIM system. Vehicles that fit any of the filters shown below were not considered in any further analysis. The filters were set after a research that includes consultation to DNIT and investigation of main truck manufacturers catalogs.

- 1) Minimum GVW ≤ 6.2 tons. This lower bound limit is set based on the lightweight two axle (four tires) truck (veículo semi-leve in Portuguese). It should be noticed that this limit is not based on the legal GVW of any given lightweight truck, but it represents the sum of the physical limits of both front axle (2.2 tons) and rear axle of 4.0 tons. This criterion has the objective of removing irrelevant light trucks.
- 2) Simple axles weight ≤ 2.2 tons. This value represents the steering axle load of the lightweight trucks.
- 3) Tandem axle weight (double) > 32 tons. This limit is set based on the heaviest axle weight of a double tandem found in the catalogs of truck manufactures in Brazil. For the sake of comparison the legal limit for these types of axles according to Brazilian regulations is 17 tons.
- 4) Axle spacings ≤ 0.92 metros. In Brazil, the minimum wheel diameter for a tandem was found to be 0.87 meters. Adding to this diameter the wheel spacing of 0.05 meters the final minimum distance between axles is 0.92 meters. Unrealistically short axle spacing can induce overestimated load effects and consequently must be removed.
- 5) Total length > 36 meters. The maximum length allowed is 30 meter. Then, it was considered a margin of error.
- 6) Length > 15.4 meters and GVW ≤ 10.43 tons. This filter screens out the unrealistic long and light trucks and also possible considerations of two trucks in following position as a single unit. The length is defined based on the 14-meter trucks (common in Brazil) and a

minimum weight of 10.43 tons which is set as the sum of 4.23 tons (axle with lower carrying capacity for this type of truck) + 6.2 tons (see criterion 1).

7) Single axle weight > 18 tons. Brazilian legislation established the weight limit of single axles in 10 tons. However, this is not the physical weight limit of the axle. Among all catalogs that were investigated the maximum carrying capacity of a single axle was found to be 18 tons.

8) $GVW \neq 10\% \sum (\text{axle weights})$. In Brazil there are no available studies that could possible define this percentage. Thus, in this thesis, the 10% established in NCHRP 12-76 report 683 is considered.

9) Sum of axle spacings > total truck length.

10) Total truck length < 5 meters.

11) Speed > 170 km/h. For the sake of comparison in NCHRP 12-76 report 683 this limit is 100 mph (161 km/h). As in Brazil there are records of trucks traveling at 160 km/h the 170 km/h was then set. It should be mentioned that the occurrence of such events is rare.

12) Steering axle weight > 10 tons.

13) Maximum GVW > 93 tons. This limit is set based on the maximum carrying capacity of 9-axle trucks. For the sake of comparison in Brazil until December 2017 the maximum GVW allowed by law was 80 tons. Since December 2017 trucks with 91 tons and eleven axles are now permitted.

After evaluating each truck record with the thirteen filters a qualified sample is considered for further analysis. The disqualified records are then stored in a file that saves all the characteristics of each vehicle. The whole process follows the flowchart in Figure 2. 23.

Figure 2. 23 – Steps for defining qualified and discarded truck.

Table 2. 5 and Table 2. 6 show the average number of vehicles that were filtered out by each filter for São Paulo and Rio Grande do Sul WIM stations respectively. Table 2. 5 shows the data for six random months. It can be noted that the first filter is responsible for filtering out 68.36% to 75.91% of the records depending on which month is under consideration. Similar results are found in Table 2. 6. An interesting point is that criterion eight did not remove a

single record which shows that the measurements of axle weights and GVW are consistent. Also, filter eleven removes only one truck per month in São Paulo dataset. In practice it means that this filter could be removed from the analysis without further change to the results. Interestingly enough, criterion number thirteen filtered out only one record a day in both stations which is not expected to cause major impact in further analysis. The statistics for the remaining months are similar.

Table 2. 5 – Daily average exclusions per each filter – São Paulo WIM data.

Filters	Months					
	Oct - 15	Jan - 16	Jul - 16	Oct - 16	Jan - 17	Jul - 17
F1	7806	10065	9715	9320	11738	10322
F2	481	441	393	492	470	583
F3	1	1	0	0	0	0
F4	23	17	7	7	5	9
F5	2	2	1	1	10	14
F6	2	2	2	2	4	5
F7	2	1	0	1	0	1
F8	0	0	0	0	0	0
F9	30	32	64	79	144	249
F10	1	1	1	1	1	2
F11	0	0	0	0	0	0
F12	0	0	0	0	0	0
F13	1	1	1	1	1	1
Total Exclusions	8349	10562	10183	9902	12374	11187
Total Records	12212	13825	13430	13785	15628	14737
Discarted (%)	68.36%	76.40%	75.83%	71.83%	79.18%	75.91%

In São Paulo, a total of 8.35 million vehicles were recorded by the WIM system from September 2015 to August 2017 (data from February 2015 to May 2015 are not considered) which means a daily average of 13292 records. Out of those, 6.08 million vehicles were filtered out. Hence, 3631 vehicles are the average daily qualified trucks. Figure 2. 24 shows a pie chart with the statistics for the whole sample. It can be noted that only 27% of the entire sample is considered valid trucks and will be used for evaluating and developing the live load model for Brazilian bridges. In Rio Grande do Sul WIM station a total of 927174 vehicles were recorded during March to June (79 days) 2014. 83% of those records were filtered out

(see Figure 2. 25) and only 154995 vehicles were considered valid for further analysis. 1962 vehicles is the average daily qualified truck number.

Table 2. 6 – Daily average exclusions per each filter – RS WIM data.

Filters	Months			
	Mar - 14	Apr - 14	May - 14	Jun - 4
F1	8968	9824	9461	7664
F2	589	507	578	1099
F3	15	16	29	31
F4	7	11	23	38
F5	0	0	0	0
F6	32	31	32	20
F7	12	5	13	16
F8	0	0	0	0
F9	36	37	33	30
F10	93	91	88	110
F11	8	7	8	44
F12	14	23	42	57
F13	0	1	1	1
Total Exclusions	9773	10554	10309	9110
Total Records	11821	12744	12302	10729
Discarted (%)	82.68%	82.81%	83.80%	84.91%

Figure 2. 24 – Statistics of filtered data – São Paulo WIM data.

Figure 2. 25 – Statistics of filtered data – Rio Grande do Sul WIM data.

3. SINGLE AND MULTIPLE PRESENCE STATISTICS FOR BRIDGE LIVE LOAD BASED ON WEIGH-IN-MOTION DATA

In this chapter a statistical study is presented based on data from São Paulo and Rio Grande do Sul WIM stations. The statistical analysis performed here only considers the data after the filtering process carried out in the previous chapter. Also, this chapter is an extended and more complete version of the paper published by Portela *et al.* (2017).

3.1 INTRODUCTION

It is well-known that truck traffic characteristics in Brazil have changed over the years (Luchi, 2006; Ferreira, 2007; Rossigali, 2013, 2015; and Junges, 2017). The advantages of knowing traffic composition have applications that go beyond the definition of a reliable live load model for bridges. The knowledge of daily traffic distribution provides information that can be used for properly defining a pavement design model, geometry of roads, the toll prices and cities logistic. Although the main concern of this thesis is with the impact of truck traffic on bridges, the data present in this chapter can be used in many engineering fields that are concerned with truck characteristics of geometry and weight.

Luchi (2006) states that trucks, in Brazil, are not all standardized and although there are classifications of the number and type of axles, there is a lack of information about the load distribution and spacing of these axles. He also raised the problem that Brazilian highway network receives millions of trucks with different axle configurations, which make the analysis of live load models more difficult. Rossigali (2013) mention that a reliable representation of current truck traffic by a calibrated live load model favors safety of the bridges, since the calibration of safety coefficients can be performed in accordance with the real traffic, and using probabilistic methods which guarantee a known safety margin. Junge (2017), in his thesis about fatigue in reinforced concrete bridges using B-WIM data, found that real traffic presents trucks with GVW that are much larger than the truck used by the Brazilian code. He argues that for studies of fatigue in reinforced concrete bridges it is imperative to know the GVW, frequency and transversal distribution of the truck over the bridge. In other words, truck traffic characteristics must be well known.

In Brazil the public agency CONTRAN (in Portuguese stands for National Traffic Council) is responsible for defining and enforcing the trucks geometry and weights. This public

institution rules the trucks characteristics by issuing Resolutions. The history of those Resolutions helps us to understand how Brazilian truck traffic has developed for the past twenty years. Resolution 12 published in 1998 established weight limits and dimensions for vehicles traveling on Brazilian roads. This resolution set the maximum weight of a single unit in 45 tons and for multiple units as 57 tons. Additionally, CONTRAN defined weight limits for each type of axles, as follow: single axle up to 10 tons, a tandem of two axles up to 17 tons, tandem with three axles up to 25.5 tons and a two axles but no tandem up to 15 tons. No tolerance for excessive weight was initially established in percentage terms. In 1999 a new resolution was published. Resolution 102 set the maximum gross vehicle weight – GVW – tolerance in 7.5%. In 2000 a new resolution changed this tolerance to 5%. In 2004 the Resolution 164 removed the need of permit for a truck with weight between 45 tons and 57 tons.

The Resolution 526 published in 2015 changed the acceptable level of excessive GVW as follows:

- I - 5% (five percent) on the regulatory weight limits for total gross weight (GVW) and total;
- II - 10% (ten percent) on the regulatory weight limits per axle of trucks.

This rule is still valid as this date. In 2017 a new resolution was published allowing the traffic of trucks with up to 91 tons. According to Resolution 663, multi-trailer trucks (known in Brazil as CVCs) with total GVW more than 74 tons and less than 91 tons are allowed to travel as long as they have a minimum length of 28 meters and a maximum of 30 meters with a height up to 4.40 meters.

Figure 3. 1 – Typical multi-trailer truck allowed to carry 91 tons.

In order to properly characterize the current Brazilian truck traffic composition, this chapter presents a statistical study based on WIM data collected from two stations. One is located on Fernão Dias highway in the State of São Paulo (SP). This road is a same-direction two

adjacent lanes. The sample is comprised of two years data, from September 2015 to August 2017. The WIM station provided an average of 11000 records per day, which were reduced to 3600 trucks per day after the filtering process. This station is currently in operation. The second station is located in Rio Grande do Sul State (RS). This road is a same-direction three adjacent lanes, but only the middle and right lane have sensors installed. The sample is comprised of 78 days data, from March 2014 to June 2014. The effects of bridge span length and the daily volume of truck are also examined.

3.2 STATISTICS OF TRAFFIC VOLUME

After applying the filtering process as previously described, the remaining data were considered for statistical analysis. Table 3.1 shows the results of Average Daily Truck Traffic (ADTT) from São Paulo station on a monthly basis considering the trucks of both lanes it also shows the maximum daily truck traffic DTT_{max} and minimum daily truck traffic DTT_{min} . Each row contains information as follow: the month that the ADTT is referring to, the ADTT value, the standard deviation of the monthly sample, the minimum and maximum daily traffic volume of the month. The table shows that the ADTT varies from 3043 to 4053 with a mean value of 3631 trucks and a coefficient of variation ranging from 21% to 39%.

Table 3.1 - Statistics of traffic volume for both lanes - SP.

Month	ADTT	STD_{ADTT}	DTT_{min}	DTT_{max}
Sep-15	3489	1320	788	5079
Oct-15	3863	1169	436	5054
Nov-15	3646	1175	1177	5367
Dec-15	3043	1193	611	4853
Jan-16	3263	1072	631	4626
Jun-16	3832	811	2376	4676
Jul-16	3246	1279	276	4845
Aug-16	3999	886	2433	4856
Sep-16	4053	909	2529	5063
Oct-16	3883	877	2313	4855
Nov-16	4024	897	2470	5091
Dec-16	3771	1121	1212	5101
Jan-17	3254	823	1079	4361
Feb-17	3542	973	1825	4698
Mar-17	3474	840	1847	4598
Apr-17	3379	1160	1495	4712
May-17	3553	942	1800	4823
Jun-17	3619	948	1859	4911
Jul-17	3712	951	1796	5014
Aug-17	3981	1038	1778	5586
Mean	3631	1019	1537	4908

Table 3.2 shows the results of Average Daily Truck Traffic (ADTT) for Rio Grande do Sul data, also on a monthly basis considering the trucks of both lanes. This table shows that the ADTT varies from 1619 to 2190 with a mean value of 1962 trucks and a coefficient of variation ranging from 30% to 53%. The ADTT for São Paulo data is much larger than the one from Rio Grande do Sul. As a matter of fact, the average value of daily truck traffic in São Paulo is 85% larger.

Table 3.2 – Statistics of traffic volume for both lanes - RS.

Month	ADTT	STD_{ADTT}	DTT_{min}	DTT_{max}
Mar-14	2047	851	790	3024
Apr-14	2190	656	1001	2986
May-14	1993	818	562	3212
Jun-14	1619	864	225	3156
Mean	1962	797	645	3095

Table 3.3 and Table 3.4 shows the volume statistics for left lane in São Paulo and Rio Grande do Sul WIM stations, respectively. Table 3.3 reveals that on average 17% of the ADTT goes on left lane, despite the fact that there is no obligation for the trucks to travel on any given lane. For Rio Grande do Sul (Table 3.4), the mean ADTT is a little bit higher (24%). As expected, the minimum GVW in both dataset is close to 6.2 tons, which corresponds to the limit imposed during the data filtering. The average maximum GVW is 92.65 tons (SP) and 91.65 tons (RS).

Table 3.3 – Statistics of traffic volume for left lane - SP.

Months	[%]	ADTT	GVW _{avg} [ton]	GVW _{std} [ton]	GVW _{min} [ton]	GVW _{max} [ton]
Sep-15	16.22	566	23.78	13.93	6.20	92.50
Oct-15	13.09	506	24.01	14.04	6.20	90.72
Nov-16	13.91	507	23.64	13.63	6.20	83.01
Dec-16	17.45	531	23.49	13.96	6.20	91.57
Jan-16	12.88	420	23.27	13.92	6.20	86.28
Jun-16	13.03	499	24.02	14.23	6.20	83.66
Jul-16	15.24	495	23.74	14.03	6.20	84.58
Aug-16	13.41	536	24.58	14.90	6.20	88.32
Sep-16	14.55	590	25.24	15.55	6.20	91.67
Oct-16	13.77	534	24.69	14.77	6.21	89.77
Nov-16	14.02	564	24.20	14.75	6.20	89.47
Dec-16	20.91	789	24.87	15.30	6.20	89.17
Jan-17	15.50	504	23.71	14.50	6.20	86.04
Feb-17	14.67	520	24.79	15.47	6.20	91.78
Mar-17	31.24	1085	27.01	16.13	6.20	92.75
Apr-17	13.62	444	25.33	15.67	6.20	91.87
May-17	19.45	652	25.53	15.48	6.20	90.70
Jun-17	14.02	478	23.89	14.24	6.20	88.28
Jul-17	36.66	1302	25.18	14.76	6.20	92.53
Aug-17	14.52	539	24.33	14.71	6.20	86.93

Table 3.4 – Statistics of traffic volume for left lane - RS.

Months	[%]	ADTT	GVW _{avg} [ton]	GVW _{std} [ton]	GVW _{min} [ton]	GVW _{max} [ton]
Mar-14	22.23	455	21.24	12.76	6.30	79.80
Apr-14	22.18	486	21.40	12.75	6.30	80.30
May-14	21.36	426	21.22	12.81	6.30	90.30
Jun-14	31.50	510	27.76	14.45	6.30	92.50

Table 3.5 and Table 3.6 shows the ADTT and GVW statistics on a monthly basis for right lane in São Paulo and Rio Grande do Sul WIM stations, respectively. In both tables the minimum and maximum GVW follow the limits imposed by the filtering process. The percentages of trucks on right lanes are 83% (SP) and 76% (RS) and the mean GVWs are 26.52 tons and 25.53 tons, respectively. Additionally, Figure 3. 2 shows the GVW histograms for both lanes. Two peaks can be observed: one around 17 tons, and the other around 45 tons. This is due to the leading presence of trucks class 3C and 3S3, which have mean GVWs of 17.3 tons and 43.0 tons. This observation is addressed in more details later on in this chapter.

Table 3.5 – Statistics of traffic volume for right lane - SP.

Months	[%]	ADTT	GVW _{avg} [ton]	GVW _{std} [ton]	GVW _{min} [ton]	GVW _{max} [ton]
Sep-15	83.78	2923	27.75	16.79	6.20	92.55
Oct-15	86.91	3358	28.03	16.71	6.20	92.90
Nov-16	86.09	3139	26.38	15.51	6.20	92.99
Dec-16	82.55	2512	26.01	15.68	6.20	92.37
Jan-16	87.12	2843	26.98	16.17	6.20	92.87
Jun-16	86.97	3333	26.08	15.09	6.20	91.87
Jul-16	84.76	2752	26.34	15.25	6.20	92.68
Aug-16	86.59	3463	26.72	15.64	6.20	92.87
Sep-16	85.45	3464	26.52	15.66	6.20	92.77
Oct-16	86.23	3348	25.51	14.69	6.20	92.39
Nov-16	85.98	3460	24.96	14.54	6.20	92.76
Dec-16	79.09	2983	24.74	14.48	6.20	92.57
Jan-17	84.50	2749	24.60	14.34	6.20	92.69
Feb-17	85.33	3023	26.23	15.49	6.20	92.44
Mar-17	68.76	2388	28.74	17.29	6.20	92.60
Apr-17	86.38	2819	27.38	16.05	6.20	92.61
May-17	80.55	2700	26.42	15.31	6.20	92.55
Jun-17	85.98	2929	26.38	15.36	6.20	92.90
Jul-17	63.34	2249	27.67	16.15	6.20	92.96
Aug-17	85.48	3174	26.91	15.73	6.2	92.76

Table 3.6 – Statistics of traffic volume for right lane - RS.

Months	[%]	ADTT	GVW _{avg} [ton]	GVW _{std} [ton]	GVW _{min} [ton]	GVW _{max} [ton]
Mar-14	77.77	1592	25.14	13.84	6.30	90.80
Apr-14	77.82	1705	24.94	14.44	6.30	92.50
May-14	78.64	1568	25.34	14.64	6.30	90.30
Jun-14	68.50	1109	26.70	15.42	6.30	93.00

Figure 3. 2 – Histogram for of trucks GVW (a) left lane and (b) right lane.

3.3 STATISTICS OF TRUCKS ACCORDING TO DNIT CLASSIFICATION

There is a large number of different vehicles traveling on Brazilian highway bridges. Those vehicles are classified by the Brazilian laws according to their axle spacings, number of axles, and axle distribution. As previously mentioned, in Brazil CONTRAN is the public agency responsible for defining the rules that governs the maximum weight of trucks. However the trucks classification used by DNIT is the one that is normally used. There are 90 trucks classifications, as considered by DNIT. For the sake of comparison, USA classification is based only on thirteen standard Federal Highway Administration (FHWA) classes. In this thesis, some of Brazilian truck classes are going to be merged with respect to the number of axles and axle spacing to make the statistical analysis simpler without losing important information. Table 3.7 shows the remaining thirty-three classes of trucks. NC stands for Not Classified trucks. The lines in light gray represent the five most frequent classes of trucks. Interestingly enough, Table 3.8 shows that for Rio Grande do Sul WIM data these five classes of trucks are also the most frequent ones.

From Table 3.7 and Table 3.8 it can be verified that trucks classified as 3C/3CD/3DC/3BB/3BC are counted for almost 24% of the whole truck traffic. This group represents single unit trucks and buses with three axles, where the rear axle is a tandem axle. Figure 3. 3 shows a typical 3C and 3BB vehicles, respectively. Appendix A shows lateral views and characteristics of trucks belonging to classes that reach at least 1% of ADTT found in the WIM data.

Figure 3.3 – Typical lateral of view for (a) 3C truck and (b) 3BB bus.

The vehicle group comprising of two simple axle trucks (2C and 2CB) and six axle single trailer trucks each represent close to 15% of the whole qualified truck sample for São Paulo WIM data. For Rio Grande do Sul WIM data the values for two axle trucks are slightly higher, reaching 18.21%, but the average gross vehicle weights is still within 10 tons for both samples. In fact, the average gross vehicle weights for the trucks are identical, no matter what class of truck is under analysis. The average difference is only 5% and trucks from São Paulo sample are heavier. Also, the lengths of the trucks from São Paulo are larger than the trucks from Rio Grande do Sul. However, the differences are negligible. Appendix C shows the statistics presented in Table 3.7 and Table 3.8 for the five main classes, on a monthly basis.

Table 3.7 – Statistics of GVW per classes of trucks for both lanes – São Paulo.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	15.36	10.36	3.23	6.20	27.51	10.24
2CC/2BC	0.23	8.30	1.62	6.20	19.71	6.51
2S1/2B1	1.86	15.93	3.28	6.99	35.98	17.01
3C/3CD/3DC/3BB/3BC	23.40	17.32	5.60	6.68	41.65	10.95
4C	0.03	26.56	6.78	10.41	50.93	10.85
4DD/4DC/4CD/4DB/4CB	2.73	25.28	6.64	9.60	51.34	11.40
2S2/2B2/2BD/2DB	12.96	19.91	5.44	9.67	58.50	18.45
2I2/2C2/2DL/2BI/2BR	1.87	20.53	5.71	9.67	54.83	18.28
3S1/3D1/3B1/3I1B/3BD	0.08	19.66	4.34	10.55	40.24	17.23
2S3	9.46	33.76	9.02	12.42	71.22	17.53
2I1/2C3/2JD	0.14	34.86	9.55	14.08	63.76	18.62
2I3/2N3	0.28	36.20	9.36	12.82	69.76	19.99
3I2/3DL/3D2/3DD/3DJ	0.99	24.98	8.14	12.00	68.38	18.82
3S2/32D	2.00	27.19	7.66	12.45	67.13	19.93
2LD	0.29	30.64	9.78	12.79	67.58	24.28
3S3/3D3	14.75	43.02	9.70	13.99	89.94	17.21
3I1/3DI/33D/3C3/3JD	0.69	46.91	10.02	16.59	80.43	17.90
3I3/3ID/3DN/3N3	3.13	46.88	10.59	14.31	84.87	18.15
2R4/2D4	0.29	32.22	10.42	14.00	73.51	30.22
2N4	0.17	27.97	5.63	17.27	59.95	30.79
3LD/3DT	0.98	46.64	10.68	15.78	80.10	18.54
4DI/4R3/43D	0.01	44.29	7.93	29.53	69.90	18.03
4D3/4DT	0.00	51.56	8.24	30.30	76.42	17.23
3D4/34D/3T4/3TD	4.55	51.92	12.39	17.92	92.94	20.02
3ND/3N4/3QD	0.01	36.13	14.07	21.65	71.38	29.97
3Q4	0.01	62.18	10.60	27.63	80.04	20.13
3DQ	0.01	31.03	6.47	20.42	66.11	28.88
3J5/3DS	0.00	40.41	15.97	25.10	69.41	29.21
3V5	0.10	51.77	14.04	22.27	92.97	23.57
3P5	0.03	57.07	12.45	25.15	90.57	20.83
3T6/3R6	0.59	68.39	12.19	23.99	92.99	27.05
3M6/3Q6	2.48	67.00	11.99	24.08	93.00	26.13
NC	0.54	35.36	20.54	7.05	92.82	15.32

Trucks with four axles present the largest difference between the two samples. In São Paulo they count for 12.96% of total qualified trucks, while in Rio Grande de Sul they count only for 7.45%. This is a difference of 74%. Although this difference is significant, the GVW of both samples are still close to the mean value of 19.91 tons in São Paulo and 19.21 tons in Rio

Grande do Sul. Finally, the trucks with five axles comprised of a two axle tractor and three axle trailers represent 9.46% and 10.56% of the whole qualified sample, respectively. The five most frequent groups of trucks represent 75% of the whole sample of qualified trucks; and they will receive special attention in the statistical analysis presented later in this chapter. The statistics presented here show that traffic characteristics of Brazil are quite different from the interstates highway in USA, as presented by Davis (2007). According to his work the five-axle single trailer trucks (FHWA Class 9) are dominant in the US traffic. He reported WIM stations where 61% of qualified trucks were Class 9 (equivalent to 3S2 trucks in Brazilian classification). In both Brazilian WIM stations considered in this thesis, the presence of 3S2 truck is less than 2%.

Table 3.8 – Statistics of GVW for main classes of trucks for both lanes – RS.

Classes of Trucks (DNIT)	[%]	GVW _{avg} [ton]	GVW _{std} [ton]	GVW _{min} [ton]	GVW _{max} [ton]	L _{avg} [m]
2C/2CB	18.21	10.12	3.11	6.30	27.30	9.19
3C/3CD/3DC/3BB/3BC	23.37	16.62	5.08	6.72	42.30	9.76
2S2/2B2/2BD/2DB	7.45	19.21	5.23	10.00	54.70	16.67
2S3	10.56	31.85	7.61	12.10	66.00	17.40
3S3/3D3	15.55	40.04	7.77	15.40	80.70	16.57

Although the traffic characteristics of Brazil and USA are different, the results for Brazilian WIM stations are coherent with the data presented by Rossigali (2013). In his work, he analyzed five sources of data from static scale. Comparisons of mean and standard deviations of GVW are shown in Table 3.9. With the exception of 2C class, the results are similar with the differences within 11%.

Table 3.9 – Means and standard deviations of GVW based on Rossigali (2013).

Classe	DNIT (1999-2002)		Ecovia (2008)		AutoBAn (2008)		AutoBAn (2011)		Fernão Dias WIM (2015-2017)	
	μ [kN]	σ [kN]	μ [kN]	σ [kN]	μ [kN]	σ [kN]	μ [kN]	σ [kN]	μ [kN]	σ [kN]
2C	65.60	36.20	79.00	31.70	74.00	27.20	76.60	25.60	103.64	32.25
3C	151.00	59.30	169.00	52.30	171.00	51.80	157.00	45.00	173.17	56.04
2S1	129.00	44.50	157.00	30.80	164.00	25.20	166.00	23.40	159.26	32.76
2S2	169.00	54.00	183.00	42.80	199.00	48.20	198.00	43.10	199.09	54.39
2S3	341.00	98.00	373.00	68.60	376.00	72.10	320.00	93.00	337.62	90.15
3S3	428.00	99.40	449.00	35.40	451.00	47.70	435.00	49.70	430.17	96.96
3T4	-	-	550.00	15.90	573.00	28.70	552.00	45.20	519.22	123.89
3T6	-	-	718.00	19.40	709.00	53.00	-	-	683.93	121.89

μ = GVW_{mean} = mean of gross vehicle weight

σ = GVW_{std} = standard deviation of Gross vehicle weight

Figure 3. 4 to Figure 3. 13 show the gross vehicle weight distribution for the five groups of classes with the highest frequencies for both WIM stations. As observed in figures below, they are unimodal distribution, which is a characteristic of interstate traffic, with longer distances. It is important to note that the shape of the weight histogram of the dominant classes provides an indication of the function of the highway, as stated in Davis (2007). According to this author, shorter distance routes used in local transport and/or located near ports and distribution centers tend to present a mixture of light and heavy vehicles, which generates bi-modal histograms (two peaks of frequency). Such behavior tends to be smoothed on interstate routes, as a consequence of logistic and economic characteristics. This is because the freight companies generally arrange the trips in way that the trucks are always loaded.

Figure 3. 4 – Histogram for GVW for trucks Class 2C/2CB – SP.

Figure 3. 5 – Histogram for GVW for trucks Class 2C/2CB – RS.

Although the shape of Figure 3. 4 and Figure 3. 5 resembles a lognormal distribution, this is also related to the fact that the filtering procedures cutoff the trucks that weight less than 6.2 tons. This criterion truncates the histogram at 6.2 tons. Also, both figures depict similar shapes which show that the traffic characteristics of these trucks are identical. This same statement cannot be said for trucks belonging to class 3C/3CD/3DC/3BB/3BC. The histogram in Figure 3. 6 shows that the GVW distribution for São Paulo WIM data has two peaks with mean at 17.32 tons, while Figure 3. 7 shows a unimodal distribution with mean at 16.62 tons. Standards deviations are 5.6 and 5.08 tons respectively.

Figure 3. 6 - Histogram for GVW for trucks Class 3C – SP.

Figure 3. 7 - Histogram for GVW for trucks Class 3C – RS.

Figure 3. 8 - Histogram for GVW for trucks Class 2S2 – SP.

Figure 3. 9 - Histogram for GVW for trucks Class 2S2/2B2/2BD/2DB – RS.

Figure 3. 8 and Figure 3. 9 depict the distributions of GVW for 2S2/2B2/2BD/2DB trucks. The histogram for São Paulo WIM stations presents a mean value of 19.91 tons with a coefficient of variation of 27.32%. The mean value of GVW for Rio Grande do Sul 2S2/2B2/2BD/2DB trucks is 19.21 tons. The maximum GVW allowed by Brazilian rules for this group of trucks is 19.21 tons. The maximum GVW allowed by Brazilian rules for this group of trucks is 34.65 tons. This is more than three standard deviations away from the mean value, in both datasets.

Figure 3. 10 - Histogram for GVW for trucks Class 2S3 – SP.

Figure 3. 11 - Histogram for GVW for trucks Class 2S3 – RS.

Figure 3. 12 and Figure 3. 13 show the distribution for GVW of Class 3S3/3D3 trucks. This is the group of trucks with the smallest coefficient of variation. For São Paulo and Rio Grande do Sul data, they are 22.54% and 19.40% respectively. One crucial point to address here is that, according to Brazilian regulation, this class of trucks is not allowed to travel with more than 51 tons. Based on the results from both WIM sites, it is shown that not all the truck drivers obey the law. The maximum GVW in São Paulo WIM station was found to be 89.94 tons, while in Rio Grande do Sul it was 80.70 tons. Both values are much higher than the limit imposed by the law.

Figure 3. 12 - Histogram for GVW for trucks Class 3S3/3D3 – SP.

For the sake of comparison, Rossigali et al. (2015) showed that trucks with 6 or more axles represent about 6% of traffic, and some of those trucks presented a GVW higher than 70 tons. He used one sample that comprises of data collected during 14 consecutive days in 2011 at a static weigh station in São Paulo state highway SP-348. These findings are different from the ones in both WIM stations used in this thesis. Although trucks weighing more than 70 tons have been found in both WIM stations, six-axle trucks represent more than 20% of traffic.

Figure 3. 13 - Histogram for GVW for trucks Class 3S3/3D3 – RS.

Since each truck has its own unique configuration and axle loads, the gross vehicle weight alone does not precisely depict the demand on a given bridge. Thus, it is necessary to check the load effect that each truck may cause to the structure. In this case, axle spacing and

frequencies are also necessary when evaluating and calibrating a live load model. With that in mind, a statistical study of axle load and axle spacing was performed. Table 3.10 and Table 3.11 show the mean and coefficient of variations of axle weight and axle spacing, for each of the five most frequent classes of trucks in both WIM stations. Figure 3. 14 to Figure 3. 25 show the histogram for axle spacing and axle load for two classes of trucks 3C/3CD/3DC/3BB/3BC and 2S2/2B2/2BD/2DB. The histogram for axle weight and spacing for the other three classes are shown in Appendix B.

Table 3.10 – Statistics of the five main classes of trucks for both lanes – SP.

Classes of Trucks (DNIT)		Fisrt Axle Weight (ton)	First Axle Spacing (cm)	Second Axle Weight (ton)	Second Axle Spacing (cm)	Third Axle Weight (ton)	Third Axle Spacing (cm)	Forth Axle Weight (ton)	Forth Axle Spacing (cm)	Fifth Axle Weight (ton)	Fifth Axle Spacing (cm)	Sixth Axle Weight (ton)
2C/2CB	Mean	4.03	562.3	5.93	---	---	---	---	---	---	---	---
	COV (%)	28.1	21.7	33.8	---	---	---	---	---	---	---	---
3C/3CD/3DC/3BB/3BC/3CB	Mean	4.74	561.0	6.45	129.2	5.17	---	---	---	---	---	---
	COV (%)	25.2	16.6	36.4	5.4	39.3	---	---	---	---	---	---
2S2/2B2/2BD/2DB	Mean	4.77	366.6	5.76	887.5	4.48	126.9	3.74	---	---	---	---
	COV (%)	17.7	7.6	31.3	19.9	34	12.9	38.1	---	---	---	---
2S3	Mean	5.06	379.2	8.09	668.5	5.98	124.2	6.47	124.3	6.27	---	---
	COV (%)	18.6	8.9	24.0	17.7	30.8	7.1	31.9	4.7	32.5	---	---
3S3/3D3	Mean	5.31	364.8	7.68	128.6	6.12	602.3	6.82	125.3	7.49	125.5	7.04
	COV (%)	18.6	8.8	21.5	3.5	22.8	21.0	23.6	11.2	23.2	10.1	24.5

Table 3.11 – Statistics of the five main classes of trucks for both lanes – RS.

Classes of Trucks (DNIT)		Fisrt Axle Weight (ton)	First Axle Spacing (cm)	Second Axle Weight (ton)	Second Axle Spacing (cm)	Third Axle Weight (ton)	Third Axle Spacing (cm)	Forth Axle Weight (ton)	Forth Axle Spacing (cm)	Fifth Axle Weight (ton)	Fifth Axle Spacing (cm)	Sixth Axle Weight (ton)
2C/2CB	Mean	4.12	528.4	6.00	---	---	---	---	---	---	---	---
	COV (%)	32.8	20.7	38.4	---	---	---	---	---	---	---	---
3C/3CD/3DC/3BB/3BC/3CB	Mean	4.80	541.8	6.51	128.3	5.31	---	---	---	---	---	---
	COV (%)	32.0	19.5	39.5	6.9	42	---	---	---	---	---	---
2S2/2B2/2BD/2DB	Mean	4.40	371.3	6.40	801.2	4.48	136.6	3.93	---	---	---	---
	COV (%)	29.2	10.8	31.9	18.0	38.2	27.3	40.4	---	---	---	---
2S3	Mean	4.79	383.7	8.39	626.1	6.06	125.6	6.16	123.3	6.44	---	---
	COV (%)	27.9	9.5	28.1	18.5	33.3	15.7	32.4	12.0	32.4	---	---
3S3/3D3	Mean	5.04	365.5	8.00	128.9	5.73	571.1	6.94	134.7	7.10	133.8	7.23
	COV (%)	27.8	10.2	25.6	4.7	26.9	20.7	27.4	26.8	26.0	26.5	25.5

Figure 3. 14, Figure 3. 15 and Figure 3. 16 present the axle weights for trucks class 3C/3CD/3DC/3BB/3BC. The Brazilian regulation defines the limit of the weights of first axle as 6.0 tons and the second and third axles are a tandem with weight limit at 17 tons. According to Brazilian regulation the trucks are allowed to travel above the weight limit as long as the excess weight is not higher than 10% for axles or 5% of GVW, as previously pointed out. In the present case, the maximum weight per axle would then be 6.6 tons for the first axle and 18.7 tons for the second axle. Keep in mind that both axles could not simultaneously present this overweight, since it would be more than 24.5 tons, which represent the 5% weight excess of GVW allowed by the law. For the sake of comparison, the data in Table 3.11 shows that for Rio Grande do Sul these trucks present a first axle weight mean of 4.8 tons with a standard deviation of 1.536 tons. Thus, considering that the axle weight distributions follows a Normal distribution the probability of exceeding the front axle weight (6.6 tons) for this group of trucks is 12.1%. For São Paulo data, this probability of failure is 11.5%.

For the tandem, mean and standard deviation according to São Paulo WIM data are 11.62 tons and 3.11 tons, respectively. This will result in a probability of exceeding the 18.7 tons (tandem) of 1.13%. For Rio Grande do Sul data the probability is 2.15%.

Figure 3. 14 – First axle weight for trucks Class 3C: (a) SP data (b) RS data.

Figure 3.15 – Second axle weight for trucks Class 3C: (a) SP data (b) RS data.

Figure 3.16 – Third axle weight for trucks Class 3C: (a) SP data (b) RS data.

Figure 3.17 – Front axle spacings for Class 3C trucks: (a) SP data (b) RS data.

The National Department of Transportation – DNIT – also defines the trucks axle spacing. For Class 3C the axle spacing between first and second axle is set to be larger than 2.40 meters, while the spacing between the second and third axle is between 1.20 meters and 2.40 meters. As shown in Figure 3. 17 and Figure 3. 18, both stations show that trucks obey this rule. It should be noted that this variable is controlled by the companies that manufacture the trucks. From São Paulo WIM station, the mean value for first and second axle spacings are 5.61 meters and 1.29 meters, respectively, while the standard deviations are 93 and 6.96 centimeters. For Rio Grande do Sul, the means are 5.42 meters and 1.28 meter and standard deviations are 105 and 8.89 centimeters for first and second axle spacings, respectively.

Figure 3. 18 – Second axle spacings for Class 3C trucks: (a) SP data (b) RS data.

Figure 3. 19 to Figure 3. 22 present the axle weights for trucks class 2S2/2B2/2BD/2DB. For this group of trucks, Brazilian regulation defines the weight of the first axle as 6.0 tons and the second as 10 tons, while the third and fourth are a tandem axle with weight limit at 17 tons. The trucks are allowed to travel above the weight limit, as long as the excess weight is not higher than 10% for axles or 5% of GVW. In the present case, the maximum weight per axle would then be 6.6 tons for the first axle, 11 tons for the second and 18.7 tons for the tandem axle, as long as the total truck weight do not exceed the limit of 5% increase in GVW allowed by the law. For the sake of comparison, the data in Table 3.10 shows that for Rio Grande do Sul these trucks present a first axle weight mean of 4.4 tons, with a standard deviation of 1.29 tons. Thus, considering that the axle weight histogram follows a Normal distribution, the probability of exciding the front axle weight (6.6 tons) for this group of trucks is 4.41%. For São Paulo data, the probability of failure is 1.49%.

For the second axle, the mean and standard deviation are 6.40 tons and 2.04 tons, respectively, for RS WIM station. Thus, a probability of a truck exceeding the second axle weight limit is 1.21%. For SP WIM station, the mean and standard deviation are 5.76 tons and 1.80 tons. This results in a probability of 0.18% of any truck exceeding the limit imposed by the law. The tandem has a very low probability of exceeding the limit of 18.7 tons. The mean weight for according to São Paulo WIM data is 8.21 tons and the standard deviation is 1.78, which means that the limit weight is 5.9 standard deviations away from the mean. For Rio Grande do Sul the limit is 4.4 standard deviations away from the mean weight.

Figure 3.19 – Front axle weight for trucks Class 2S2: (a) SP data (b) RS data.

Figure 3.20 – Second axle weight for trucks Class 2S2: (a) SP data (b) RS data.

Figure 3. 21 – Third axle weight for trucks Class 2S2: (a) SP data (b) RS data.

Figure 3. 22 – Forth axle weight for trucks Class 2S2: (a) SP data (b) RS data.

As mentioned before the DNIT is the public agency that defines the trucks axle spacing for each class. For Class 2S2 the axle spacing between first and second axles and between second and third axles is set to be larger than 2.40 meters. The spacing between the third and forth axles is to be between 1.20 meters and 2.40 meters. As shown in Figure 3. 23, Figure 3. 24 and Figure 3. 25 both station shows that trucks obey this rule. It should be noted that this variable is controlled by the companies that manufacture the trucks.

Figure 3.23 – Front axle spacings for Class 2S2 trucks: (a) SP data (b) RS data.

Figure 3.24 – Second axle spacings for Class 2S2 trucks: (a) SP data (b) RS data.

Figure 3.25 – Third axle spacings for Class 2S2 trucks: (a) SP data (b) RS data.

3.4 MULTIPRESENCE ANALYSIS

The consideration of truck multiple presence on concrete bridges is mandatory to understand and characterize live load models because its occurrence directly affects future extrapolation of live load effects, as well as code calibration. According to Nowak (1999), when dealing with multiple presence, there are two main factors to consider: the frequency and the GVW of occurrences on the bridge. The frequency is related to the probability of two trucks over the bridge at the same time, while the GVW is related to the load effect. The frequency of the two heavy trucks has a large influence on the maximum expected load effect during the service life of a concrete bridge. These two factors ultimately affect the extrapolation of live load effects.

Multiple presence statistics are reported only for qualified trucks, *i.e.* the remaining trucks after the filtering process. Therefore, this analysis excludes all passenger vehicles, light trucks and erroneous records. The simultaneous occurrence of trucks on a bridge can occur in many different disposals. In this chapter, a very similar arrangement (Figure 3.26) as in Gindy (2006) will be considered. In this case, four patterns are considered:

- a) Single event: only one truck is on the bridge;
- b) Following: Two trucks in the same lane with the distance from the first axle of the first truck to the rear axle of the second truck less than a bridge span;
- c) Side-by-side: two trucks in adjacent lanes with an overlap of at least half the body length of the first truck and with the distance from the first axle of the first truck to the rear axle of the second truck less than bridge span;
- d) Staggered: two trucks in adjacent lanes with an overlap of less than half the body length of the first truck and with the distance from the first axle of the first truck to the rear axle of the second truck less than a bridge span;

Figure 3.26 - Traffic loading patterns - Nassif and Gindy (2006).

The multiple presence statistics for each case: following, side-by-side and staggered are needed to quantify the trucks superposition in a given concrete bridge. The computation of those statistics is based on a definition of a range of fictitious span lengths. This allows studying multiple presence for long, medium and short bridges. The span lengths comes into consideration when specifying an upper bound for the event criteria, for example, two trucks that are in the same lane such that the distance from the first axle of the leading truck to the rear axle of the second truck is 42 meters, they would be considered as a following occurrence on a fictitious span length of 50 meters. However, the same two trucks would not be qualified as following on a 35 meters span length because trucks would not fit on the bridge span. The percentage of each occurrence is computed as the number of occurrence, divided by the total number of qualified trucks on the bridges.

One important aspect of this computation is the accuracy of the WIM system. In order to have a reliable modeling process, the system must be capable of recording the trucks with a resolution of 0.01 second. This accuracy is necessary because trucks can travel at high speeds, for instance, a truck at a speed of 50 km/h will travels about 13.9 meters in one second. If the truck position is known each 0.01 second this means that the truck position is known every 13.9 centimeters, which is a very acceptable value to define multiple presence occurrence. Thus, only three pieces of information are needed to compute the relative position of trucks to each other:

- i) The travel lane;
- ii) Truck speed;
- iii) Time truck goes over the sensor (resolution of 0.01 sec).

Additional information such as GVW and truck length is used to refine the qualifications for each type of multiple presence occurrences.

3.4.1 Algorithm for Calculating the Multiple Presence on Concrete Bridges

As both WIM systems provide information about vehicle speed, travel lane as well a time stamp with an accuracy of one hundredth of a second, it is possible to construct computational routines to perform the multiple presence analysis for different span lengths. With the input data, each truck is compared with the six next ones. This is important because by definition multiple presence only consider the event of two trucks simultaneously over a concrete bridge. The cases of three or more trucks together are considered of very low probability of occurrence and are not the subject of this thesis.

Figure 3.27 shows the flowchart structure of the multiple presence routine used in this thesis developed in MATLAB (Mathworks, 2011). The routine checks each possible case of multiple presence. Then it eliminates trucks that are qualified for more than one case. Initially, for the following event, the algorithm firstly checks whether the trucks are in the same lane, then, it checks if the distance from the first axle of the leading truck to the rear axle of the second truck is smaller than the current span length in analysis. This ensures that the vehicles fit in the fictitious bridge span. If these two criteria are satisfied, then the event is classified as following.

Figure 3.27 - Flowchart of multiple presence algorithm.

For the side-by-side event, first the program checks if the trucks are in different lanes. Then, it checks if the second truck has an overlap of at least half the body length of the first truck as shown in Figure 3.26-c. Finally, the algorithm checks if the length from the first axle of the first truck until the last axle of the second truck is less than the length defined by the span of the bridge. If all these criteria are met, then the event is counted as a side-by-side. Finally, for the staggered case, the procedure is almost the same as the side-by-side. The only difference is that the trucks overlap is less than half the body length of the first truck. As by definition the multiple presence is only the simultaneously occurrence of only two trucks. If a truck is, for example, followed by a second truck, but at the same time is side-by-side with a third truck, as Figure 3.28, these three trucks are not considered as multiple presence. Figure 3.29 shows a case of three trucks following each other. This case would not be considered as multiple presence because there are more than two trucks over the bridge.

Figure 3.28 - Traffic loading patterns - Nassif and Gindy (2006).

Figure 3.29 - Traffic loading patterns - Nassif and Gindy (2006).

3.5 RESULTS

The entire sample of qualified truck is simulated over a bridge span lengths ranging from 6.1 meters (20 ft) to 91.4 meters (300 ft) meters by using recorded time stamp (accuracy of 0.01 second), vehicle speed (assumed as constant), truck length and travel lane. The multiple presence statistics for the patterns of loading as in Figure 3.26 are computed through the algorithm described in Figure 3.27. The main results for São Paulo and Rio Grande do Sul WIM stations are shown through Figure 3.30 to Figure 3.47.

Figure 3.30 depicts the change of multiple presence statistics in relation to bridge span length for São Paulo WIM station. With the exception of the side-by-side case, the frequency of multiple truck presence is significantly affected by span length. The increase of bridge span lengths gives more opportunities for trucks to occur simultaneously. However, for side-by-side loading pattern, when the span length is already above 30 meters, increasing the bridge span will slightly decrease the frequency of occurrence. This happens because for larger spans more trucks have the opportunity to be on the bridge. The side-by-side trucks could be declassified if more vehicles enter the bridge at largest values of spans length. The frequency of side-by-side events has an average value of 0.70% for all bridge spans and volume conditions, which is in accordance with the results presented by Nassif and Gindy (2006) and Davis (2007).

Figure 3.30 – Multiple presence statistics – SP.

The comments stated in the previous paragraph are also valid for Figure 3.31. However the frequency of side-by-side events has an average of 0.274%. Such a low value is probably related to the low ADTT found in Rio Grande do Sul WIM station. As it was previously found ADTT for São Paulo WIM station is on average 3597 while for Rio Grande do Sul is 1962.

Figure 3.31 - Multiple presence statistics – RS.

Figure 3.30 and Figure 3.31 also show that the rate of increase in the frequency of following loading events is lower for bridge spans up to 30 meters as compared to the longer spans. This is mainly due to the fact that bridges with spans of less than 30 meters commonly will not accommodate two trucks in the same lane, once the most common truck is the six-axle semi-trailer with a body length of around 15 meters. The results also show that the occurrence of following events keeps increasing as the bridge spans increase. This is because as the span increases the opportunity for following loading pattern to happen also increases. In the case of staggered events, the frequency of occurrence also increases as the span length increases. But on the contrary of what happen to following case, the percent occurrence of staggered trucks grows at a faster rate for shorter bridge spans and at a steadier pace for longer spans - above 60 meters.

Figure 3.32 to Figure 3.40 shows the variation of the three different multiples presence cases in relation to the daily truck traffic volume. Three different span lengths are considered: 12.2 meters, 45.7 meters and 79.3 meters. As shown in these figures, truck volume considerably impacts the multiple presence statistics for all load patterns. The results show that the general tendency of the occurrence of all multiple presence events is to increase as the truck volume increases. This is due to the fact that a truck is more likely accompanied on a given bridge span as the number of trucks increases. It is also observed that the frequency of staggered (Figure 3.38 to Figure 3.40) occurrences increases at a higher rate than for following (Figure 3.32 to Figure 3.34) or side-by-side (Figure 3.35 to Figure 3.37) loading patterns.

Figure 3.32 – Multiple presence for Following case – Span length = 12.2 meters.

Figure 3.33 - Multiple presence for Following case – Span length = 45.7 meters.

Figure 3.34 - Multiple presence for Following case – Span length = 79.3 meters.

Figure 3.35 - Multiple presence for Side-by-Side case – Span length = 12.2 meters.

Figure 3.36 - Multiple presence for Side-by-Side case – Span length = 45.7 meters.

Figure 3.37 - Multiple presence for Side-by-Side case – Span length = 79.3 meters.

Figure 3.38 - Multiple presence for Staggered case – Span length = 12.2 meters.

Figure 3.39 - Multiple presence for Staggered case – Span length = 45.7 meters.

Figure 3.40 - Multiple presence for Staggered case – Span length = 79.3 meters.

Since the frequency of multiple truck presence can greatly influence the extrapolation of loading effects on bridges, it is important to know the distribution of the sample as the spans increase. Figure 3.32 to Figure 3.40 show that, in general, as the span length increases the measurements tends to disperse. In the case of side-by-side event the change in the dispersal is very small while for following and staggered occurrences the change is quite noticeable. This also has relation with the fact that increasing bridge spans lengths also gives more opportunities for trucks to occur simultaneously with more diversified trucks combinations.

Another important aspect of developing a live load model is to know the correlation between the total truck weights for multiple presence occurrences. These relations are shown in Figure 3.41, Figure 3.42 and Figure 3.43 for each of the three types of events considering the span length of 85.3 meters (280ft) and the lower bound for GVW is defined as 36287 Kg (80kips) which corresponds to the truck recommended by AASHTO code when designing a bridge. For the following case, the GVW Left Lane corresponds to the leading truck weight where GVW – Right Lane corresponds to the second truck. Although an unassuming 45-degress zone can be noted in Figure 3.41 they show very strong correlations between the GVWs of both trucks. For the staggered and side-by-side cases, the correlations are very small. This is in accordance with Brazilian traffic regulation for federal highways, which allows heavy trucks in any lane of the road.

Figure 3.41 - Relation of GVW 1 and GVW 2 in tons for following event - only heavy trucks.

Figure 3.42 - Relation of GVW 1 and GVW 2 in tons for side-by-side event- only heavy trucks.

Figure 3.43 - Relation of GVW 1 and GVW 2 in tons for staggered event- only heavy trucks.

Table 3.12 and Table 3.13 show the correlations between GVWs for heavy trucks that have been classified as an occurrence of multiple presence for several span lengths for São Paulo and Rio Grande do Sul data. In general, the correlations are very small. The stronger

correlation between heavy trucks when they are classified as following might be due to the fact that drivers from the same company tends to form a convoy when traveling together.

Table 3.12 – Correlation between GVW from lane 1 and lane 2 for each case of multiple presence - SP.

Span Length (m)	Correlation of GVW 1 and GVW 2		
	Following	Side-by-side	Staggered
12.2	None	-0.035	None
45.7	0.341	0.122	-0.021
79.2	0.373	0.111	0.025

Table 3.13 – Correlation between GVW from lane 1 and lane 2 for each case of multiple presence - RS.

Span Length (m)	Correlation of GVW 1 and GVW 2		
	Following	Side-by-side	Staggered
12.2	None	-0.092	None
45.7	0.613	-0.135	0.055
79.2	0.445	-0.185	0.034

When developing a live load model for concrete bridges, the occurrence of heavy trucks over the bridge is a very important aspect that needs to be considered. Figure 3.44 shows the probability of two heavy trucks occurring together if they are classified as a multiple presence case for São Paulo WIM station. The probabilities are shown for each case of multiple presence and for a range of span lengths from 6 meters to around 91 meters. For the side-by-side case one can see that above the span length of 20 meters the probabilities are about 9.5%. This means that out of all side-by-side case 0.095 cases are of two heavy trucks. The same behavior can be observed for the staggered events after the span length of 40 meters. In this case, most probabilities are around 8.0%. For the following cases the probabilities stay close to zero up to span lengths of 35 meters and then it starts to climb up. This is strictly related to the results of Figure 3.30 in which following cases only start happening for span lengths larger than 35 meters.

Figure 3.44 - Probability of two heavy trucks occurrence - SP.

Figure 3.45 shows the probability of multiple presence of two heavy trucks for Rio Grande do Sul WIM data. Although the curves are more inconsistent when compared with São Paulo WIM data, the curves behavior are similar to the ones in Figure 3.44. The more inconsistent behavior is indicative that the sample did not get into a flow yet. Additional data may be necessary to achieve a more consistent behavior. Most values of side-by-side occurrences of two heavy trucks lies on 6.5% while for staggered events the curve lies over 5.9% after span length of 50 meters.

Figure 3.45 - Probability of two heavy trucks occurrence - RS.

Finally, the probability of two heavy trucks occurring together in relation to the total truck traffic can be computed by multiplying the results from Figure 3.44 and Figure 3.30. In this work, heavy trucks are the ones weighing over 36.3 tons, which correspond to the HS20 truck approved by AASHTO for designing bridges in USA. As shown in Figure 3.46 the general behavior of the lines for each case of multiple presence are strongly attached to the ones in Figure 3.30. This is due to the fact that the occurrences of multiple presence events dictate the possibility of occurrence of heavy trucks together. Figure 3.46 and Figure 3.47 show that the probabilities for two side-by-side heavy trucks are in average 0.079% and 0.0186% for SP and RS WIM data, respectively. The highest probability in both datasets occurs for following event at span length of 91.4 meters. The probability value of 0.84% (SP) implies that for a highway with an ADTT of 3631 trucks (Table 3.1) it is expected at least 30 occurrences of two heavy trucks following each other in any given day. Knowing these probabilities allows computing and extrapolating the load effects that those trucks have over a given bridge. Not only this ultimately allows to evaluate the live load model proposed by the current code, but also to develop new models.

Figure 3.46 - Probability of two heavy trucks occurrence in relation to total truck traffic – SP.

Figure 3.47 - Probability of two heavy trucks occurrence in relation to total truck traffic – RS.

3.6 FINAL REMARKS

Concrete bridges are considerably affected by trucks loads, their frequency of occurrence, and their geometric features, and also by their GVW in two situations: single and multiple presence events. Data collected from a WIM station at Fernão Dias highway in Brazil from September 2015 to August 2017 and Rio Grande do Sul from March to June 2014 were used to determine both single and multiple presence statistics for various loading patterns. The São Paulo WIM station provided a daily average traffic of 13292 vehicles. However, after the filtering process, its daily average traffic reduces to 3630 trucks per day. Similarly, Rio Grande do Sul station provided a daily average traffic of 9000 vehicle. After the filtering process, the number reduces to 1962 trucks per day on average.

Prior to multiple presence analysis, traffic characteristics were surveyed in order to detect the main types of trucks and statistically investigated important parameters for the definition of a representative live load model for concrete bridges. The results showed that trucks of 2C,3C3 and 3S3 are the most common ones and combined they represent 53.51% and 57.13% of total valid records for São Paulo and Rio Grande do Sul WIM data. The results also showed that the GVW statistics stabilize in a relatively short monitoring time – about a month.

Bridges with span length from 6.1 to 91.4 meters were considered on this study for statistical evaluation of single and multiple presence occurrences. In general, multiple presence events are notably affected by the bridge span length, with the exception of side-by-side case. For the side-by-side case, the effects are almost insensitive to variations of span length. Based on the results in this study, side-by-side occurrence slightly decreases as the span length increases. This happens because more trucks will have the opportunity to fit the larger span, *i.e.*, increasing bridge span length will increase the probability of more than two trucks to occur simultaneously. The results also show that the percentage occurrence of side-by-side stays steadily at 0.95% (Sao Paulo WIM data), until it reaches to span length of 50 meters and then starts to slowly decrease.

On the other hand, both staggered and following event have a considerable change in their percentage of occurrence. In the staggered loading event, the rate of increase becomes lower for bridges with span length up to 40 meters while the following event is the opposite. For bridges with span length larger than 80 meters, the percentage occurrence of following event overcomes the staggered values. The results presented in this paper are consistent with the ones presented in Gindy and Nassif (2006).

Finally, when designing a live load model for concrete bridges, it is important to know the probability of multiple presence of heavy trucks. It is important to note that heavy truck, in this work, means the trucks that weight more than 36.3 tons (80 kips). The results regarding occurrence of two heavy trucks in adjacent lanes show that the probabilities of a side-by-side events are very small, less than 0.02% for any given span length. It also shows that this value does not change according to the span length for the following event, where the maximum probability for two heavy trucks was 0.185%.

4. DEVELOPMENT OF BRAZILIAN LIVE LOAD MODEL BASED ON WEIGH-IN-MOTION DATA

In this chapter a historical view of the current Brazilian live load model is presented. WIM data is used to compute load effects in terms of moment and shear per girder and the results are compared with the code (bias factors are estimated). Both single and multiple presence of trucks are taken into account. A live load model based on the most common trucks is developed.

4.1 INTRODUCTION

A lot of effort has been put into developing live load models based on real traffic data. Amidst this effort, the work by Nowak and Nassif (1993) and Nowak (1993) have shown significantly interesting results. In the former, WIM data was used to determine the unbiased truck loads in Michigan. In the later, the normal probability paper technique was used to extrapolate traffic load effects for 75 years return period. The work presented by Nassif and Gindy (2007) performed a statistical analysis to determine the truck load spectra for bridges in New Jersey. They used data from 25 WIM sites from 1993 to 2003, this illustrates how advanced they are on this subject.

In Europe, the analysis performed by O'Brien & Enright (2011) also used Weigh-In-Motion data in order to model bridge loading. WIM data from 2 countries (Czech Republic and Netherlands) was used to model same-direction two-lane traffic for live loads on bridges. In the latter, they used WIM data to show the effects of non-stationary traffic data on maximum traffic load effects. Miao and Chan (2002) used ten years of Hong Kong WIM data to estimate extreme daily bending moments in different simply supported bridge within a span length ranging from 5 to 40 m.

In Brazil, there is no tradition in using WIM data to characterize bridge live load, since the majority of Brazilian states have no WIM stations. Investigations of truck loads on bridges are all made based on data from stationary scales. However, there is a reasonable concern that those records are biased. The work presented by Ferreira (2006) proposed limits for the truck weights passing over reinforced and prestressed concrete bridges. He used data from São Paulo state to estimate the load effects on bridges considering 50 years return period. He used

normal probability paper to extrapolate trucks GVW and typical axle distances were determined through automakers' catalog.

Luchi (2006) also used data from the state of São Paulo to propose a model to verify the design traffic load model of the Brazilian code. He used 121,078 records throughout the year of 2005. The truck passages were simulated over bridges with span lengths varying from 30 to 200 meters, assumptions were made regarding the axle distances and axle weights distributions, since the available data did not contain this information. Rossigali (2013) presented a similar study for short span bridges. He took into account simple supported bridge with span lengths from 10 to 40 meters and cantilevers from 2.5 to 10 meters under free flow situation. In addition, extreme value theory was used to extrapolate the load effects and compared the results with the ones caused by Brazilian live load model.

The study presented in this chapter is based on the same WIM data presented in the previous chapter. As stated before, one station is located on Fernão Dias highway in the State of São Paulo (SP) and the sample is comprised of 20 months, from September 2015 to August 2017. The second station is located in Rio Grande do Sul State (RS). This road is a same-direction three adjacent lanes, but only the middle and right lanes have sensors installed. The sample is comprised of 78 days, from March 2014 to June 2014. Load effects are computed in terms of bending moment and shear per girder in both scenarios single and multiple presence trucks.

4.1.1 Background of Brazilian live load model for highway bridges

The first national code (NB-6) for highway bridges in Brazil was published in 1943. The live load model in this code was based on an old version of the German code DIN 1072 (Rossigali, 2013). The code NB-6 classified the roadway bridges into three classes: Class I represented the most important roads where the heavy trucks were supposed to travel, such as Federal and State roads, Class II represented the secondary State roads and Class III that basically represented the local roads (Timerman, 2015). The live load model in NB-6 consisted of three sorts of load: a three-wheel roller, a truck and a distributed load as indicated in Figure 4.1. The distributed load depended on the span length and the kind of structural element that one was trying to design, but in general it was between 0.4 tf/m^2 and 0.5 tf/m^2 . Class I bridges were designed with truck and roller type B and verified with type C. Class II bridges were designed using the trucks and roller type A and verified with type B while Class

Figure 4.2 - Live load model as in NB-6: 1960 (Dimensions in meter)

In 1984 the Brazilian code for live load model changed its name to NBR7188. The bridge classes and loads were as in Table 4.2. The Class 12 was still the same as in NB-6, but now there were Classes 45 and 30, the wheel loads are 75 kN and 50 kN, respectively, and both of them used the same truck with three axles as depicted in Figure 4.2. The live load model used only one truck no matter how many lanes the bridge has. Outside the truck projection there is a distributed load as in Table 4.2. Needless to say that the truck must be placed over the most unfavorable positions to the structural element.

Table 4.2. Loads of Live load model as in NBR7188:1984.

Bridge Class	Truck GVW (kN)	Distrib. Loads (kN/m ²)
45	450	5
30	300	5
12	120	4

The current version of the NBR7188 (new version of NB-6) which is valid since 2013 divides the bridges into only two classes: Class 450 and Class 240 which are represented by a truck with a GVW of 450kN and 240kN, respectively. The live load model for Class 450 (Figure 4.3) is known as TB-450 and is going to be used in this thesis, since it is the most used in Brazil. The live load model also has a distributed load of 5 kN/m² for Class 450 and 4 kN/m² for Class 240; in both cases the distributed load is positioned outside the truck projection over the bridge. The code also specifies that the loads must be placed where they cause the largest load effect over the structural element. Also, no matter how many lanes the bridge has, only one truck must be considered.

Figure 4.3 - Live load model as in NBR7188: 2013 (Dimensions in meter)

Design according to NBR7188 requires that the final load effect be multiplied by three different coefficients: one accounting for the moving vehicle vertical impact on the structure, another to consider the number of lanes over the bridge; and lastly one coefficient considers the effects of expansion joints on the bridge live loading. The first one is given by Equation (4.1).

$$CIV = 1 + 1.06 \left(\frac{20}{Liv + 50} \right) \quad (4.1)$$

in which Liv is the span length in meters (for isostatic girder). If the span length is less than 10 meters, then the CIV is taken as 1.35. The second coefficient is as in Equation (4.2).

$$CNF = 1 - 0.05(n - 2) > 0.9 \quad (4.2)$$

in which the n represents the number of lanes. The third coefficient is the CIA and has two values 1.25 for concrete structures and 1.15 for steel structures. The final live load model will be the product of all those three coefficients by the loads in the truck load model.

4.2 LIVE LOAD ANALYSIS

4.2.1 Single Trucks

The moments and shears determined with the WIM data are variables that reflect the randomness of the traffic. However, WIM data are recorded over a relatively short period of time, compared to bridge design life and thus, constitute only a sample of the vehicle population that travels over the bridge in its expected lifetime. Therefore, an extrapolation technique is used to estimate the future load effects that trucks will cause in the structure. Many methods to extrapolate the load effects have been presented: Ghosn and Moses (1986),

Nowak (1999), Crespo-Minguillón and Casas (1997), Hwang and Koh (2000). This thesis follows a similar approach described by Nowak (1999). In his work, Nowak (1999) used the Normal Probability Paper (NPP) method to predict the maximum effects of live loads for return periods up to 75 years. The NPP method is also used in this chapter to extrapolate the load effect of the trucks in terms of moment and shear, but in this thesis a period of 100 years is considered for extrapolation. The central idea of this technique is to represent the upper tail of the maximum load effects histograms by an adjusted normal distribution and to use the Average Daily Truck Traffic (ADTT) statistics of the vehicles of interest to make future projections, under the assumption that the sample of monitored vehicles can properly represent the traffic.

The first step when applying the NPP method is to determine the cumulative probability distribution of load effect due to each truck (or the respective load effect ratios considered in relation to the load effect determined according to live load model, such as HL-93 or TB-450). This can be done by organizing the observed variable in ascending order and assigning to each point an index m ranging from 1 to N , where N is the total number of points in the sample. An estimate of the cumulative probability (p) can be determined, according to the following equation:

$$p = \frac{m}{N+1} \quad (4.3)$$

The cumulative probability is used to determine the variable Z , which represents the inverse of the standard normal distribution ($\mu = 0$ and $\sigma = 1$). By calling the random variable under analysis by X , a graphical representation in the space $Z \times X$ (Figure 4.4) must provide a linear correlation if the variable population is physically modeled by a Normal distribution. In general for load effect due to trucks, this behavior is expected for the upper tail of the load effect distribution. A linear fit can be applied to extrapolate X . The number of trucks (N_{future}) and their corresponding cumulative probability (p_{future}), can be estimated according to the following expressions:

$$N_{future} = 365 \cdot ADTT \cdot Time \quad (4.4)$$

$$p_{future} = \frac{N_{future}}{N_{future} + 1} \quad (4.5)$$

where $ADTT$ only considers the remaining trucks after filtering, and $Time$ is the return period in years, whereby the future load effects is evaluated. Therefore, the variable X in the future (X_{future}) can be easily estimated by knowing the inverse $Z_{future} = \Phi^{-1}(p_{future})$ and the linear extrapolation curve, as shown in Figure 4.4.

Figure 4.4 - NPP for extrapolation of load effects.

In this work, for single trucks, the variable X is represented by the following dimensionless ratios:

M_i / M_{NBR} , where M_i is the absolute maximum moment in simply supported bridges or the extreme negative moment in continuous bridges, associated to each truck after filtering the WIM data; M_{NBR} is the maximum moments calculated according to the live load model of NBR7188:2013;

V_i / V_{NBR} , where V_i is the absolute maximum shear in simply supported bridges, associated to each truck after filtering the WIM data; V_{NBR} is the maximum shear calculated according to the live load models of NBR7188:2013;

Since the interior and exterior girders in a bridge are subjected to different moments and shears, this work uses the girder distribution factors as proposed by AASHTO (2012). When the structure has only one lane load, the moment and shear on an interior girder can be computed as:

$$LE_{Single} = LE_{Truck} \frac{DF_1}{1.2} \quad (4.6)$$

Where LE_{Single} is the load effect on an interior girder, LE_{Truck} is the load effect of a single truck on one lane and DF_I is the girder distribution factor. Since DF_I already has a multiple presence factor of 1.2, to account for the probability of two trucks side-by-side, this factor is divided by 1.2 in order to consider only the load effect for one truck (AASHTO, 2012).

4.2.2 *Statistics of multiple presence events of trucks*

Chapter 3 showed that the consideration of multiple truck presence on the bridge is essential to understand and characterize live load models, since its occurrence directly affects future extrapolation of live load effects (Davis, 2007). Also, the number of occurrences of multiple truck presence when both trucks are heavy allows investigating what might be a prevailing event from the load effect point of view. Multiple truck presence statistics are reported only for qualified trucks, defined as the trucks that pass the filtering process, excluding all passenger vehicles, light trucks and erroneous records. The simultaneous occurrence of trucks on a bridge can occur in many different patterns. In this chapter, the results presented in the previous chapter are taken into account.

As mentioned before, the multiple presence statistics for each case: following, side-by-side and staggered are used to estimate the load effect due to multiple trucks for a given bridge. The percentage of each event is computed as the number of occurrences divided by the total number of qualified trucks, as shown in the previous chapter. The case of three or more trucks together is considered to be rare and not much attention is given to this event. Thus, by definition multiple trucks presence can happen only between two trucks. It should be kept in mind that the presence of three or more heavy trucks on a bridge is only possible for long and/or multi-lane structures. In this type of structure self-weight is normally dominant and the load effect due to live loads plays a secondary role. For short bridges with few lanes there is not even enough space for three trucks.

4.2.3 *Load effect of multiple presence effect using AASHTO LRFD approach*

With the statistics of truck multiple presence, as presented so far, Table 4.3 to Table 4.6 can be constructed. The two trucks are described as T1 and T2 and N_{future} represents the number of future trucks for the period T (the considered truck for computing the load effect is the

maximum within period T). Nowak (1999) computed these statistics under the assumptions that each 15th truck would be a side-by-side case, each 450th would be a side-by-case event with both trucks heavy, each 50th truck would be a following case and each 500th truck is followed by a fully correlated truck. He then used these statistics to compute the bias factor for different load effects. This method for obtaining the bias ratio for multiple presence events was later adopted by the AASHTO LRFD. In this chapter, two versions of this method are used for comparison with the proposed approach. Herein, Approach 1 refers to the method used by Nowak (1999), with the statistics assumed in the original paper, while Approach 2 is also based on Nowak's method, however uses the statistics in Figure 3.46 and Figure 3.47.

Table 4.3 depicts the parameters for one lane load based on Sao Paulo WIM data, while Table 4.4 shows the results for Rio Grande do Sul WIM data. The statistics are presented in terms of range and not in single-value, since the percentage of following occurrences changes for each span length. Table 4.5 and Table 4.6 show the parameters for two lanes load. The Side-by-side occurrences correspond to the two-loaded lane design case in the AASHTO-LRFD (AASHTO 2012). The results in these tables will be compared with the results from the proposed approach of this work.

Table 4.3. Truck parameters for one lane load (Approach 2) SP.

One Lane Loaded		N_{future}	z	T
One truck		132531500	5.66	100 years
Two trucks	$\rho = 0$	T1	1.04e5 - 1.07e7	4.27 - 5.21
		T2	1	0
	$\rho = 1$	T1	1770 - 1.14e6	3.26 - 4.78
		T2	1770 - 1.14e6	3.26 - 4.78

Table 4.4. Truck parameters for one lane load in RS (Approach 2).

One Lane Loaded		N_{future}	z	T
One truck		71175000	5.55	100 years
Two trucks	$\rho = 0$	T1	4.5e5 - 4.1e6	4.59 - 5.03
		T2	1	0
	$\rho = 1$	T1	9.6e3 - 3.6e5	3.71 - 4.54
		T2	9.6e3 - 3.6e5	3.71 - 4.54

Once the statistics presented in these tables are defined, it is needed to compute the load effect caused by the corresponding trucks to each number z , and then find the corresponding final bias factor for each case.

Table 4.5. Truck parameters for two lanes in SP (Approach 2).

Two Lanes Loaded			N_{future}	z	T
One truck			132531500	5.66	100 years
Two trucks	$\rho = 0$	T1	1111939	4.77	11 months
		T2	1	0	average
	$\rho = 1$	T1	106262	4.28	1 month
		T2	106262	4.28	1 month

Table 4.6. Truck parameters for two lanes in RS (Approach 2).

One Lane Loaded			N_{future}	z	T
One truck			71175000	5.55	100 years
Two trucks	$\rho = 0$	T1	204842	4.42	4 months
		T2	1	0	average
	$\rho = 1$	T1	13571	3.80	10 days
		T2	13571	3.80	10 days

4.2.4 Load effect of multiple presence effect using the proposed approach

Assessment of any live load model requires the evaluation of simultaneous occurrences of trucks over the structure. In this sense, the load effects resulting from the application of the live load model on the structure must be compared to the load effects generated by the simultaneous presence of trucks on the bridge. One important issue when computing the moments and shears for these occurrences is that the load effect of each truck on the girders needs to be properly addressed. It means that the total effect on a given girder is a combination of the effects that each truck imposes to the girder itself. If the bridge has two or more lanes, then a different girder distribution factor needs to be considered as in Equation (4.7). The distribution factor needs to produce an increase in the load effects caused by the new lanes.

$$LE_{Mult} = LE_{Truck} DF_2 \quad (4.7)$$

Section 4 of the AASHTO LRFD Specification gives DF_1 and DF_2 as below:

$$DF_1 = 0.06 + \left(\frac{S}{14} \right)^{0.4} \left(\frac{S}{L} \right)^{0.3} \left(\frac{K_g}{12Lt_s^3} \right)^{0.1} \quad (4.8)$$

$$DF_2 = 0.075 + \left(\frac{S}{9.5} \right)^{0.6} \left(\frac{S}{L} \right)^{0.2} \left(\frac{K_g}{12Lt_s^3} \right)^{0.1} \quad (4.9)$$

where L is the span length in ft, S is the girder spacing in ft, t_s is the deck thickness in inches and K_g represents the longitudinal stiffness. Equations (4.8) and (4.9) can be applied for structures, such as concrete deck, composite deck with reinforced concrete slab over steel or concrete beams, concrete T-beams etc. This thesis uses both distribution factors to make an approximation of the net effect that two trucks in adjacent lanes can cause to an interior girder. The net load effect will be given as below:

$$LE_{Mult} = \frac{DF_1}{1.2} LE_{Trk_1} + \left(DF_2 - \frac{DF_1}{1.2} \right) LE_{Trk_2} \quad (4.10)$$

In order to take into account the largest net load effect, the LE_{Trk_1} is considered to be the load effect due to the heaviest truck and LE_{Trk_2} is the accompanying truck. Since DF_1 already has a multiple presence factor of 1.2 to account for the probability of two trucks side-by-side in Equation (4.10) this factor is removed by dividing DF_1 by 1.2. The final moment on an interior girder caused by two trucks is the sum of load effect of each truck, weighed by the distributions factors. For following cases, however, the load effects of both trucks are multiplied only by DF_1 , since both trucks are over the same girder.

Since the frequency and magnitude of load effects due to two trucks in a bridge are the most important aspects of evaluating a live load model, two different situations of multiple truck presence are addressed in this paper. The first one considers all the occurrences of two trucks over the bridge, no matter the GVW of those trucks; while the second one considers only the heavy trucks (80 kips). The average daily occurrence for both cases is the number of daily occurrences of each event. These statistics have been presented in Chapter 3. With those values, moments and shears can be computed and extrapolated.

4.3 RESULTS AND DISCUSSION

4.3.1 Sensitivity analysis of the proposed approach

When estimating the bias factor for the load effects it is important to define a reliable percentage of the upper tail for the probability distribution for each load effect. The quality of fit is strongly related to the behavior of the cumulative distributions itself, this means that the better the data resembles a Normal distribution, the better the fit of the upper tail with a linear function. If the data were following a Normal distribution the results of extrapolation for different percentages of the upper tail would be the same, since in a normal probability paper the data would be a straight line. In practice that is not the case and a sensitivity analysis is necessary to properly select the percentage that returns the most reliable results.

The percentage of the upper tail to be extrapolated also depends on the amount of data that is available, but since it is impractical to make any decisions based on multiple curves, a sensitivity analysis can help to reduce the number of cases to be considered when developing a live load model. In this work, different percentages (25% to 1%) of the upper tail were examined for all cases under consideration: Single trucks, Following, Side-by-side and Staggered cases. A range of span lengths from 6.1 (20 ft) to 91.4 (300 ft) meters were also considered. It was found that the best results are obtained when considering the upper 5% of the data plotted on a normal probability paper. Therefore, this percentage will be the one taken into account in this paper.

Any percentage above 5% (25%, 20%, 15% and 10%) will yield similar results since the upper portion of the data approaches a straight line as can be seen from Figure 4.5 and Figure 4.6. The behavior of the data in these figures is found for the other span lengths and load effects as well. Any percentage below 5% (3% and 1%) will result in unstable extrapolation mainly for shorter span due to the lack of data to extrapolate. Furthermore, the upper tail of the distribution resembles a straight line and the fit for 5% of the data has a regression coefficient R^2 between 0.96 and 0.99, indicating that upper tail of the truck load effect histograms convincingly approaches a normal distribution. A similar result was also reported by Sivakumar *et al.* (2011).

Figure 4.5 - Extrapolation curve for 100 years for positive moment – (a) Following, (b) Side-by-side and (c) Staggered. (Span length = 45 meters).

Figure 4.6 - Extrapolation curve for 100 years for single trucks – (a) Positive Moments, (b) Shear and (c) Negative Moments. (Span length = 45 meters).

Figure 4.7 to Figure 4.10 show the bias factors for positive moments for both multiple presence and single trucks considering full correlation between the GVWs of the trucks (curves with “Heavy”) and no correlation. Figure 4.11 to Figure 4.14 depict the same results for shear while Figure 4.15 to Figure 4.18 show for negative moment. Figure 4.7, Figure 4.9, Figure 4.11 and Figure 4.13 show that for following case for span lengths up to 30 and 40

meters, respectively, there is not enough data to reliably estimate the bias factors. This is mainly because, for short spans, the length of two trucks does not fit the bridge and consequently not many Following events will occur. Side-by-side cases with no correlation between the GVWs of trucks are considered as governing the response. This means that for the majority of cases side-by-side events are in control or close to the curves that governs the response. Therefore, for practical reasons, side-by-side is the only case considered to evaluate the live load model.

Figure 4.7 - Bias factors for positive moments for one lane - SP.

Figure 4.8 - Bias factors for positive moments for two lanes - SP.

Figure 4.9 - Bias factors for positive moments for one lane - RS.

Figure 4.10 - Bias factors for positive moments for two lanes - RS.

From Figure 4.7 to Figure 4.18 one can see that the bias factors for multiple presence occurrences of two heavy trucks will never dominate the bias factor results when compared with the other cases. It indicates that the chances of having one extremely heavy truck with another extremely heavy truck are too small. Therefore, this case will not appear in the upper tail in twenty months of data, but it might appear for longer periods. Also, when defining the multiple presence of two heavy trucks it may be the case that the occurrences of two trucks in which one is heavy and the other is not are filtered out. On the other hand, for single trucks it is believed that the extremely heavy trucks have already occurred and are present in the upper tail. This explains why the upper tail in Figure 4.6 starts to verticalize. Notice that there are no occurrences of side-by-side trucks when they are both heavy for Rio Grande do Sul WIM data.

Figure 4.11 - Bias factors for shear for one lane - SP.

Figure 4.12 - Bias factors for shear for two lanes – SP.

Figure 4.13 - Bias factors for shear one lane - RS.

Figure 4.14 - Bias factors for shear for two lane - RS.

Figure 4.15 and Figure 4.17 show the bias factors for negative moments. They show that the results for following cases are slightly above the values for single trucks. This was expected once following trucks will have the chance to occur in different spans creating a combined load effect of two trucks each in one span. Staggered and side-by-side trucks produce similar results for bias factor of negative moment.

Figure 4.15 - Bias factors for negative moment for one lane - SP.

Figure 4.16 - Bias factors for negative moment for two lane - SP.

Figure 4.17 - Bias factors for negative moment for one lane - RS.

Figure 4.18 - Bias factors for negative moment for two lane - RS.

4.3.2 Comparison of load effect based on different approaches

With the results from previous section it is now possible to evaluate the live load model for roadway bridges recommended by the Brazilian code NBR 7188 (2013) and compare the results of each approach. Although, the method presented here is applied to the Brazilian code, it could also be applied for any other code. Also, the results are compared with the method proposed by Nowak (1999) in both scenarios: with his multiple presence statistics (Approach 1) and with the statistics as in Table 4.3 to Table 4.6 (Approach 2). Figure 4.19 to Figure 4.24 show the bias factors for two loaded lanes for positive moments, shear and negative moments, respectively. The horizontal solid line crossing the vertical axis in 1.5 represents the load safety factor that the Brazilian code requires to be used when computing the load effects for the live load model. This line is to be seen only as a reference. Safety factors are not to be confused with bias factors, more on that next chapter.

Figure 4.19 - Bias factors for positive moment - SP.

Figure 4.20 - Bias factors for positive moment - RS.

Even though the multiple presence statistics for Approach 1 and 2 are different, the bias factors for both approaches are similar within each case: full and no correlation between the GVWs of the trucks for all the three load effects under consideration. This is mainly related to the nonlinear nature of inverse normal distribution value Z , for example, the probability of side-by-side occurrences in Approach 1 is $1/15$ while for Approach 2 is $1/115$. For these two probabilities Z is 5.10 and 4.76, respectively. Thus, although one probability is 7.67 larger than the other, the Z values are only 7% apart. The proposed approach which is represented by the extrapolation of the upper 5% of load effect distribution for side-by-side trucks lies between the curves for full and no correlation between the GVW of the trucks. This is meaningful since it implies that considering only full correlated trucks is too conservative, mainly for the shorter spans where not enough occurrences can be observed, while considering no correlation at all is not safe enough once it produces low values of bias factors which may create the idea that the live load model under analysis is conservative.

Figure 4.21 - Bias factors for shear - SP.

Figure 4.22 - Bias factors for shear - RS.

The bias factors of all the three cases of load effects for short spans present higher values. This does not necessarily mean that the designing method has a high probability of failure for short span bridges. In order to check the probability of failure, the resistance requirements must be looked into. However, from these results it can be said that the current Brazilian live load model results in very distinct bias factors over the span lengths which make the choice of a single safety factor uneven and inconsistent.

Figure 4.23 - Bias factors for negative moment - SP.

Figure 4.24 - Bias factors for negative moment - RS.

4.4 DEVELOPMENT OF RATIONAL BRAZILIAN LIVE LOAD MODEL

As can be seen from Figure 4.19 to Figure 4.22 the bias factor for the current Brazilian live load model presents an unsteady curve for different span lengths. To develop a more horizontally uniform curve, this chapter proposes a more rational model based on the WIM data. Thus, an investigation over the sample of trucks was made to be able to detect the prevailing ones. The four most common trucks (according to Chapter 3) were considered when defining a new live load model. They are trucks classes 2C, 3C, 2S2, 2S3, 3S3. Based on these trucks, different live load models were evaluated, considering bias factor for three load effects: positive and negative moments and shear. Span length were ranging from 6.1 meter 91.4 meters.

Figure 4.25 and Figure 4.26 depict the final live load model that gives the steadiest results. For positive moment and shear the bias factors are taken as the maximum value between the current live load model (Figure 4.3) and the one in Figure 4.25. For negative moment the model is represented by two equal trucks with a minimum distance of 5 meters from each other and a uniformly distributed load of 3.60 kN/m (Figure 4.26) when a lane 3.6 meters wide is under consideration. Figure 4.27 and Figure 4.28 show the bias factors for shear, positive and negative moment per girder. In this figure, the bias factor for negative moment based on the current version of the Brazilian live load model is added for the sake of comparison. The load effects were calculated based on the approach presented in this chapter, resulting on stable and uniform results. Based on the results, a bias factor of 1.4 would safely cover all the load effects generated by the trucks.

Figure 4.25 - Proposed live load model: one truck with three axles for shear and positive moment.

Figure 4.26 - Proposed live load model: two trucks for negative moment.

Figure 4.27 - Bias factor for the proposed live load model per girder - SP.

Figure 4.28 - Bias factor for the proposed live load model per girder - RS.

4.5 FINAL REMARKS

Bridges are considerably affected by truck loads, their frequency of occurrence, their geometric features, and also by their GVW in both single and multiple presence events. Data collected from two WIM stations in major federal highways in Brazil were used to determine the load effect on fictitious bridges with span lengths ranging from 6.1 to 91.4 meters. The load effect in terms of positive and negative moments and shear per girders were estimated using girder distribution factors (AASHTO, 2012). The load effects caused by trucks were compared with the ones generated by the current Brazilian code. The ratios are then extrapolated for the bridges service life of 100 years. The results show that for the majority of cases, side-by-side will govern the response, *i.e.*, it will present the highest value of bias factor.

The comparison of the proposed approach with the method developed by Nowak (1999) and latter adopted by AASHTO LRFD shows that considering only full correlated trucks is too conservative, mostly for short spans, where there are not enough occurrences of trucks multiple presence. On the other hand, the results also show that considering no correlation at all yields very low bias factors. This could create the false idea that the Brazilian live load model under analysis is conservative. At last, a more rational live load model is developed based on WIM data. The new model is a combination of the current Brazilian model with a new truck and new distributed load of 900kg/m for positive moment and shear. For negative moment two trucks are recommended. The new model yields more uniform results for bias factor for different span lengths.

5. RELIABILITY ANALYSIS OF BRAZILIAN BRIDGES

5.1 INTRODUCTION

In 2001 the National Department of Transportation – DNIT – started the SGO, a system to manage the Brazilian bridges under the Federal Government control. In 2004 they published a report showing the results of 1210 bridges that were inspected. Not much of progress has been done since that day. In Brazil, there is still no dataset with reliable information about Brazilian bridges. There is no information that could possibly indicate the correct number and situation of the bridges in the country. There is a weak estimation that the country has around 120 thousands bridges. DNIT states that under Federal Government control there are approximately 6600 bridges. In a study presented by Mendes (2009) he stated that 60% of bridges in Federal highways have a span length shorter than 50 meters and 90% of those are made of reinforced or prestressed concrete. This means that the majority of bridges in Brazil are short and made of concrete. This is important information to be considered when performing a reliability study of the safety of current Brazilian bridges in relation to live load. For short span bridges, the effect of live load is more significant than that of dead load. Many of these existing structures were designed between 1940's and 1970's. During this period, the Brazilian Live Load Model was taken as an adaptation of German code. As mentioned before, the current Brazilian live model is still based on that code.

With that in mind, the main purpose of this chapter is to use existing Brazilian bridges to calibrate the live load model, as in NBR7188:2013. In Brazil, only a few studies have been done in this area. Among them, the works performed by Rossigali (2013), Luchi (2006) and Ferreira (2006) are the most important ones. Those works have been mentioned throughout this thesis, because they represent the best work in this field. Rossigali (2013) presented a work that analyzed five data sources of static scale to simulate traffic on typical bridges of Brazilian highways, aiming at the evaluation and calibration of the Brazilian live load model for short span bridges. The main traffic data in his analysis came from one station in Espírito Santo state. Although his work is from 2013, the data is from 1999 to 2002. In his analysis, he considered simple supported bridges, continuous two-span and cantilever bridges. The structures had spans lengths of 10, 20, 30 and 40 meters and the traffic was in free flow situation. Figure 5. 1 shows the typical cross-section considered by Rossigali (2013). One important aspect of his thesis is that he makes no consideration in relation to resistance of the

bridges. His conclusions are solely based on the load side. His main objective is to compare the load effect due to the traffic data with the load effect produced by the live load model.

Figure 5. 1 – Typical reinforced concrete section in Rossigali thesis - Rossigali (2013).

Luchi (2006) presented a work in which box-girder bridges with span length varying from 30 to 200 meters are considered. As in Rossigali (2013), Luchi only checked the problem from the load side. Thus, no consideration of resistance was made. He analyzed bending moment at the middle span and shear at the support on simply supported bridges. Traffic jam situations on the bridge were taken into account. In his thesis, he also presented a study with multi-girder cross section bridge. Span length of 30 meters was considered. The traffic data he used was based on a single static scale in the state of Sao Paulo. He had access to 121,078 vehicles that were weighted during 204 days. There were no information regarding the axle spacing and total length of the trucks.

In the work performed by Ferreira (2006) bridge formula was developed using traffic data from static weight station to check the safety level of existing bridges designed by the valid live load model during that time. Ferreira (2006) designed 60 bridges and checked the safety of those bridges using Nowak (1999) approach. He used reliability indices for moment capacity and obtained truck weight limits in order to assure the structural integrity of Brazilian bridges. He considered reinforced and prestressed structures with different structural arrangements: simple supported and continuous bridges. The ultimate limit state was considered in sections subject to critical positive and negative moments. His results indicated that limitations should be applied to the traffic of some vehicles, especially the 740 kN and 19,80 meters length.

After reading the work presented by those authors, some important consideration can be made. First, there is no work, so far, that is completely based on nonbiased traffic data. All the works studied are based on static scales, that are well-known to produce biased information once only a small fraction of the daily real traffic is measured and the overweight truck can

easily bypass the station. Also, the works presented by Rossigali (2013) and Luchi (2006) do not perform analysis of the structures resistance. They did not draw conclusions on the safety level of the current live load model based on comparisons between actual load and resistance. They only compared the code with their traffic data. Although Ferreira (2006) used several bridges for checking the safety level, he used bridges that he designed instead of existing ones. Besides, his traffic data is also biased, once it comes from static scale. In this chapter, a reliability analysis is performed with sixty existing Brazilian bridges. The load effect is computed as shown in the previous chapter. The bridges are taken from different states of Brazil. The next section describes them in more detail.

5.2 SELECTED BRAZILIAN BRIDGES

In order to perform a calibration of the current Brazilian live load model, a dataset of selected structures is needed. The ideal set should have existing bridges from different states in the country, different material and structural types. The main problem when performing this work is that in Brazil there is no well-organized dataset of those structures. The Brazilian Department of Transportation – DNIT – does not keep an easy-accessed set of data that could help. Thus, a request was sent to several designers around Brazil asking for drawings and relevant information of concrete bridges. At the end, 60 bridges were obtained, in which 39 were prestressed concrete bridges and 21 were reinforced concrete. As a reference, when Nowak (1999) performed his study he obtained 107 existing bridges (including steel bridges), while Ferreira (2007) used 60 bridges that were designed by himself.

Table 5. 1 shows the main characteristics of the prestressed bridges taken into account in this thesis. The span length varies from 14 to 60 meters. Girder spacing varies from 215 cm to 340cm. The set is composed of girder bridges with 3,4,5,6,7 and 10 girders and also box girders. The bridges are from the following states: Pernambuco, Ceará, Bahia, São Paulo and Minas Gerais. The majority of the bridges have a concrete strength of 40 MPa. It is considered that those 60 bridges are representative of the current practice of Brazilian bridges.

Table 5. 1 – Selected prestressed bridges

Bridge Number	Type	Span Length (m)	Fck (MPa)	Girder Spacing	Number of Girders	Year	State
1	Prestressed	19.4	40	250	5	2000	PE
2	Prestressed	39	40	235	7	2000	BA
3	Prestressed	27	40	250	7	2000	PE
4	Prestressed	30	40	260	5	2000	PE
5	Prestressed	20	40	250	5	2000	PE
6	Prestressed	25.5	40	250	6	2013	PE
7	Prestressed	41.5	40	340	3	2013	MG
8	Prestressed	16.2	40	340	3	2013	MG
9	Prestressed	28.33	40	340	3	2013	MG
10	Prestressed	30	40	270	5	2013	MG
11	Prestressed	22	40	236	5	2013	MG
12	Prestressed	37.8	40	225	5	2013	MG
13	Prestressed	19	40	220	5	2013	MG
14	Prestressed	34.2	30	225	5	2012	MG
15	Prestressed	24.8	30	270	5	2012	MG
16	Prestressed	34.7	30	270	5	2012	MG
17	Prestressed	30.5	30	215	6	2012	MG
18	Prestressed	35.3	40	220	5	2012	MG
19	Prestressed	18.3	40	220	5	2012	MG
20	Prestressed	29.5	35	260	5	2012	MG
21	Prestressed	37.3	40	237.5	4	2013	MG
22	Prestressed	16.2	40	340	3	2013	MG
23	Prestressed	18	40	340	3	2013	MG
24	Prestressed	14	40	340	3	2013	MG
25	Prestressed	39.4	40	290	5	2013	MG
26	Prestressed	18.4	40	240	10	2013	MG
27	Prestressed	39.4	40	237.5	4	2013	MG
28	Prestressed	17.5	40	285	5	2013	MG
29	Prestressed	28.6	40	284	5	2013	MG
30	Prestressed	19.2	35	286.5	5	2010	SP
31	Prestressed	34.9	35	286.5	5	2005	SP
32	Prestressed	39.2	35	280	5	2014	SP
33	Prestressed	36	30	280	6	2010	CE
34	Prestressed	24.3	25	240	6	2002	CE
35	Prestressed-Box	22	30	-	Box Girder	1974	SP
36	Prestressed-Box	60	30	-	Box Girder	1974	SP
37	Prestressed-Box	31	30	-	Box Girder	2012	MG
38	Prestressed-Box	34.2	30	-	Box Girder	2012	MG
39	Prestressed-Box	36.4	40	-	Box Girder	2013	MG

Table 5. 2 shows the characteristics of the reinforced concrete bridges under consideration in this thesis. The span length varies from 10.80 to 35 meters. The set is composed of mainly two girder bridges, but there are bridges with 4,5,10 and 13 girders and also box girders. The bridges are from the following states: Pernambuco, Ceará, São Paulo and Minas Gerais.

Table 5. 2 – Selected reinforced bridges

Bridge Number	Type	Span Length (m)	Fck (MPa)	Girder Spacing	Number of Girders	Year	State
40	Reinforced	15	40	250	5	2011	PE
41	Reinforced	16.6	40	235	5	2011	PE
42	Reinforced	11.5	30	Slab	0	2012	MG
43	Reinforced	10.8	30	250	10	2012	MG
44	Reinforced	17.5	40	180	13	2013	MG
45	Reinforced	21	20	250	4	1995	CE
46	Reinforced	30	15	0	4	1975	SP
47	Reinforced-Box	24	18	-	Box Girder	1974	SP
48	Reinforced-Box	25	15	-	Box Girder	1969	SP
49	Reinforced-Box	12	15	-	Box Girder	1969	SP
50	Reinforced	16	20	620	2	1958	SP
51	Reinforced	21	20	575	2	1963	SP
52	Reinforced	20	15	750	2	1977	SP
53	Reinforced	17.6	22	635	2	1957	SP
54	Reinforced	26	15	570	2	1967	SP
55	Reinforced	12	20	550	2	1960	SP
56	Reinforced	22.4	20	570	2	1963	SP
57	Reinforced	18	25	570	2	1966	SP
58	Reinforced	35	15	775	2	1968	SP
59	Reinforced	25	15	775	2	1968	SP
60	Reinforced	26	15	760	2	1974	SP
61	Reinforced	20	20	450	2	2001	PE

For all the selected bridges, the moment and shear are estimated for dead load, live load and dynamic effect as required by the Brazilian code (NBR7188:2103). The resistance is estimated in terms of flexural capacity (resistant moment) and shear capacity. The resistance is computed considering that the bridge was constructed exactly as in the drawings and no deterioration process is considered. Section 5.3 shows the computation of the load carrying capacity.

5.3 RESITANCE AND LOAD COMPONENT FOR SELECTED BRIDGES

5.3.1 Moment and Shear Capacity Computation

In order to estimate the probability of failure of a structure due to bending moment it is necessary to estimate the flexural capacity. To do so, the program Response2000 (Bentz and Collins, 2001) was used. Response2000 is a non-linear 2D sectional analysis program for reinforced concrete elements, mainly beams and columns. It was written in C++ by Evan Bentz, a PhD candidate at the University of Toronto, between 1996 and 1999. The program is very intuitive and user-friendly.

Response2000 uses the moment-curvature relationships to estimate the flexural capacity. This method uses the sectional analysis combined with material properties to compute the equilibrium within the section. The method divides the whole section into many small individual layers and it also considers the reinforcement as a layer (see Figure 5. 2). Each layer has its own force, and then the method adds up those forces to find the equilibrium. The assumptions in this method are the traditional ones plane sections will remain plane, also strains and stresses are constant within each layer. For more detail on this method see Bentz (2000) and Colombo (2016).

Figure 5. 2 – Typical reinforced concrete section under sectional analysis (adapted from Colombo, 2016).

Another step of the method is to define the material properties. In this work, the concrete behavior under compression is based on the NBR6118 (2014), as in Figure 5. 3. The behavior for reinforced steel and prestressed steel under tension are as in Figure 5. 4 and Figure 5. 5, respectively. Parameters $\mu_{\sigma c}$, $\mu_{\sigma s}$, ϵ_c , ϵ_s , $\mu_{E s}$ and $\mu_{E p}$ are the mean value of concrete strength, mean of steel strength, concrete strain, steel strain, mean of elastic modulus of reinforced steel

and mean of elastic modulus of prestressed steel, respectively. The procedures for computing the bending moment for prestressed structures are the same as for reinforced structures, only adding the pre-strain. Similar approach was also applied by Ferreira (2006).

Figure 5. 3 – Stress X Strain relation for concrete under compression.

Figure 5. 4 – Stress X Strain relation for reinforced steel under tension.

Figure 5. 5– Stress X Strain relation for prestressed steel under tension.

Figure 5. 6 to Figure 5.8 show a typical cross-section of a bridge under analysis. Figure 5. 9 shows how the cross-section of bridge 12 is modeled inside Response2000. Special attention must be given to the fact that the stirrups are always adapted when the trapezoidal shape (see bar N22 in Figure 5. 7) is not possible. However, the estimation from the program for moment and shear capacity is still very good (Bentz, 2000). For shear, the Modified Compression Field Theory – MCFT - (Vecchio and Collins, 1986 and 1993) is used to estimate girder shear capacity. This is the same method applied by Nowak (1999). In the model, the cracked concrete is treated as a new material, with its own stress-strain relationship. MCFT uses compatibility, equilibrium and stress-strain relationship in terms of average stress and average strain. The method considers tensile stresses in the concrete between the cracks and employs experimentally verified average stress-strain relationships for the cracked concrete. A perfect bond between concrete and steel is considered.

Figure 5. 6 – Girder section of bridge 12 under analysis.

Figure 5. 7– Reinforced girder section of bridge 12.

Figure 5.8 – Prestressed Girder section of bridge 12.

Figure 5.9 – Typical Page of Response2000

5.4 RELIABILITY ANALYSIS OF SELECTED BRIDGES

5.4.1 *Assessment of Resistance of Selected Bridges with 2K+1 method*

When performing any kind of reliability analysis, one unavoidable step is to define and statistically describe the random variables related to the problem. In order to properly assess a reliability problem of concrete structures, the correct distribution type of variables must be defined. Also, those variables need to be properly modeled. This is important because the accuracy of the mean, standard deviation and probability distribution will dictate the accuracy of the estimated probability of failure. In order to estimate the statistical moments for resistance distribution of each bridge under analysis in this thesis the method presented by Rosenblueth (1975) will be considered.

This method consists of a approximate technique for the number of simulations is $N = 2k+1$, k is the number of random variables in the problem. It should be noted that this methodology requires smaller number of simulations when compared with, for example, Monte Carlo Method. The $2k+1$ method consists of assessing a given function of random variable at fundamental points and then uses the results to estimate the statistical moments of the function itself (Nowak, 2012). One advantage of the method is that there is no need for defining the probability distribution of the variables. To schematically present the method, consider the following function of random variables Nowak and Collins (2000):

$$Y = f(X_1, X_2, X_3, \dots, X_k) \quad (5.1)$$

Determine the mean (μ_x) and standard deviation (σ_x) of each random variable;

Compute the function value (y_0) considering each variable as their mean value;

For each variable (X_i) compute Equation 5.1 at two points: $\mu_{x_i} \pm \sigma_{x_i}$ keeping the remaining variables at their mean value. Two values for each variable will be found as:

$$y_i^+ = f(\mu_{x_1}, \mu_{x_2}, \mu_{x_3}, \dots, \mu_{x_i} + \sigma_{x_i}, \dots, \mu_{x_k}) \quad (5.2)$$

$$y_i^- = f(\mu_{x_1}, \mu_{x_2}, \mu_{x_3}, \dots, \mu_{x_i} - \sigma_{x_i}, \dots, \mu_{x_k}) \quad (5.3)$$

For each variable, compute the following two quantities:

$$\bar{y}_i = \frac{y_i^+ + y_i^-}{2} \quad (5.4)$$

$$V_{y_i} = \frac{y_i^+ - y_i^-}{y_i^+ + y_i^-} \quad (5.5)$$

Estimate the mean and coefficient of variation of the function by:

$$\bar{Y} = y_0 \prod_{i=1}^k \left(\frac{\bar{y}_i}{y_0} \right) \quad (5.6)$$

$$V_Y = \sqrt{\left[\prod_{i=1}^k (1 + V_{y_i}^2) \right]} - 1 \quad (5.7)$$

Even though the method presents some advantages, Christian and Baecher (2002) alerted that if the number of variables are too many then this method starts to become unpractical. They also stated that if the variables are strongly correlated the method might give unrealistic outputs. In this work, this method is used to only estimate the mean and standard deviation of flexure and shear capacity of the bridges under consideration. For this purpose, the following random variables are taken into account:

f_{ck} ultimate compression strength;

f_{pu} ultimate tensile strength of prestressing steel;

f_{yu} ultimate tensile strength of reinforcing steel;

A_s total cross section area of rebars;

A_p total cross section area of tendons;

E_p modulus of elasticity of prestressing steel

E_y modulus of elasticity of reinforcing steel

d_p effective depth of tendons;

d_y effective depth of rebars;

This list of variable means of reinforced concrete structures when estimating the flexure capacity five variables are taken into account (f_{ck} , f_{yu} , A_s , E_y and d_y), while for prestressed bridges the variables under considerations are: f_{ck} , f_{pu} , A_p , E_p and d_p . Thus for each bridge, a total of twenty-two analysis had to be performed, eleven for flexure capacity and eleven for

shear capacity. With the estimated mean and standard deviation of the resistance a Monte Carlos Simulation can then performed to find the reliability index. A description of each variable involved in this process is given next.

5.4.1.1 Concrete compressive strength

When performing a code calibration, the standard deviation of the concrete strengths should reflect the uncertainty of the concretes produced and used in the region that the code is being applied. The best dataset of concrete strength currently available in Brazil is from the work published by Santiago and Beck (2011, 2017). In Santiago and Beck (2011) the authors presented a statistical study of Brazilian strength of structural concretes. Their investigation was based on experimental results of around five thousand concrete samples produced in loco at different construction sites from different Brazilian states. In their work, they divided samples accordingly with the characteristic strength of concrete – C20, C30, C40 and C50 in which C stands for concrete and the following number represents the strength in MPa. The results found by the two authors can be seen in Table 5. 3 and will be considered in this work when modeling bridges that were built between the years 2008 - 2013.

Table 5. 3 – Statistical results for the concrete strength as in Santiago and Beck (2011).

f_{ck} (MPa)	Mean (MPa)	Std. Deviation (MPa)	CV	Bias Factor
20	26	1.85	0.071	1.30
30	33.8	3.55	0.105	1.13
40	41.7	3.57	0.086	1.04
50	47.2	2.91	0.062	0.94

Nowak (2003) and Melchers (1999) presented bias factors that range from 1.12 to 1.38. Ferreira (2007) presented bias factors of 1.15, 1.12, 1.09 and 1.07 for concrete C20, C25, C30 and C35 respectively. Ferreira (2007) also presented the following coefficient of variation 0.103, 0.093, 0.089, 0.088, respectively.

5.4.1.2 Ultimate tensile strength of prestressing steel

In the work presented by Santiago (2017), 4200 samples of strands were tested to determine the ultimate load capacity of four different types of strands used in Brazil: CP190 RB 9.5, CP190 RB 12.7, CP190 RB 15.2, CP 210 RB 12.7. CP stands for prestressed concrete whereas RB stands for low relaxation in Portuguese, the numbers 190 and 210 represent the minimum value of tensile strength in kgf/cm^2 and the number after RB represents the diameter of the strand in mm. The final bias factor and CV recommended by the author were 1.07 and 0.015, respectively. The author also presented bias factor and CV for cross section

area of strands based on 1600 samples. The final results were 1.03 and 0.01. Based on those results the final bias factors and CV for ultimate tensile strength of prestressing steel are 1.04 and 0.19; these values are going to be used in this thesis.

In relation to yielding stress of steel used in reinforced concrete, the author also performed a statistical analysis of 8700 samples for 5 different rebar diameters. Those samples came from different states in Brazil and were analyzed in 2016. The final results show that the bias factors range from 1.17 to 1.29 and the CV ranges from 0.03 and 0.05. For the sake of comparison, Nowak (2012) found values that ranged from 1.12 to 1.18 and CVs that ranged from 0.02 to 0.04. Based on his results, in this thesis, the bias factor for ultimate tensile strength of reinforcing steel is taken as 1.19 with a CV of 0.04.

5.4.1.3 *Cross section area of rebars*

As mentioned before, the bias factor and CV for cross section area of tendons were found to be 1.03 and 0.01. Since there is no information available about those two values, for this thesis the bias factor and CV for cross section area of rebars are going to be considered the same as for tendons.

5.4.1.4 *Modulus of elasticity of prestressing steel and reinforcing steel*

Santiago (2017) also performed an evaluation of 4200 samples of strands in order to estimate the bias factor and CV of elastic modulus of prestressing steel. The samples were collected between 2011 and 2015 in different states of Brazil. The sample was composed of the three most used diameters in Brazil: 9.5mm, 12.7mm, and 15.2mm. He determined that the bias factor and CV as in Table 5. 4. In this work, the modulus of elasticity of prestressing steel is considered 1.03 and 0.02, respectively. Under the lack of data related to modulus of elasticity for reinforcing steel in Brazil, this thesis is going to consider the values of bias factor and coefficient of variation for prestressing steel as representative of those for reinforcing steel.

Table 5. 4 – Statistical results for modulus of elasticity of prestressing steel Santiago (2017).

Variable	Diameters (mm)	Samples	B.F	CV
E_p	9.5	948	1.03	0.02
	12.7	358	1.02	0.01
	15.2	2855	1.03	0.02

5.4.1.5 *Effective depth of tendons and rebars*

In Brazil, there is no data available that could be used to determine the bias factor and coefficient of variation for effective depth of tendons and rebars. Therefore the values presented by Nowak and Szerszen (2001) will be considered as representative of Brazilian structures. In this thesis, the bias factor for effective depth in concrete is 1.00 while the CV is 0.025.

5.4.1.6 Summary of statistical parameters

Finally, Table 5. 5 shows a summary of the statistical parameters used in this thesis. One important aspect of these values is that, out of seven variables considered in the analysis performed in this thesis, six of them were based on Brazilian data.

Table 5. 5 – Summary of statistical parameters used in this thesis.

Variable	Bias Factor	CV
f_{ck} (C20)	1.30	0.071
f_{ck} (C30)	1.13	0.105
f_{ck} (C40)	1.04	0.086
f_{ck} (C50)	0.94	0.062
f_{yk}	1.19	0.040
f_{pu}	1.04	0.019
E_y	1.03	0.020
E_p	1.03	0.020
Area	1.03	0.010
d_p	1.00	0.025

5.4.2 Estimation of Loads Acting on the Selected Bridges

5.4.2.1 Live Load and Impact Load

Since there is no studies in Brazil about the dynamic effects on concrete bridges due to live loads, in this thesis the impact live load is considered as in Nowak (1999). In this experimental work, Nowak determined the mean dynamic load as percentage of the mean live load. For two trucks on the bridge the percentage is 0.10 and the coefficient of variation is 0.80. To compute the maximum 75-year combination of live load and dynamic load, Nowak (1999) assumed that the live load effect is a combination of two parameters, the static value of live load and the live load analysis factor with mean m_p of 1.00 and coefficient of variation V_p of 0.12. The final statistical parameters of live load are then given by:

$$V_{LP} = \sqrt{V_L^2 + V_P^2} \quad (5.1)$$

In which V_L is the coefficient of variation of static live load taken from the WIM data. The statistical parameter of maximum live load effect including the dynamic effect for two trucks can be written as:

$$m_{LP+I} = 1.1 \cdot m_{LP} \quad (5.2)$$

$$\sigma_{LP+I} = \sqrt{\sigma_{LP}^2 + \sigma_I^2} \quad (5.3)$$

$$V_{LP+I} = \sigma_{LP+I} / m_{LP+I} \quad (5.4)$$

in which $m_{LP} = m_L$ (mean static live load from WIM data computed as described in the previous chapter), $\sigma_{LP} = V_{LP} m_{LP}$ and $\sigma_I = V_I m_I$ from Nowak (1999) $m_I = 0.10m_L$ and $V_I = 0.80$. The previous chapter showed that the coefficient of variation and mean of live load depend on the span length. In this thesis, the combined value of live load and dynamic effects are modeled as Normal distribution with mean and standard deviation as in Equations (5. 2) and (5. 3), respectively.

5.4.2.2 Dead Load

There are many uncertainties related to modeling load effects on concrete structures. Dead loads are also a source of uncertainty when estimating the total load acting on a given structure. Since in a structure as a bridge there are different materials working, such as: asphalt, concrete, steel and also considering that those materials have different processes of fabrication, it is expected that the level of uncertainty involved in modeling those loads is different (Nowak, 1999). Therefore, the construction of probabilistic model of dead loads has to consider different variation for different kinds of dead load. Melchers (1999) suggests that when constructing those probabilistic models three steps need to be observed:

- i) Identification and definition of the loads with different uncertainties;
- ii) Selection of probabilistic distribution for each variable defined on the previous step;
- iii) Estimation of distribution parameters by available data;

One important aspect that should be considered when applying those three steps is the feasibility of considering the dead load uncertainty of different structural elements. For example, the weight of concrete will change over time, considering the change of uncertainty

on the concrete weight might be realistic, but it also implicates in a lot of difficulties to properly define this change in uncertainty over time.

In this thesis, the dead load is divided into three variables: asphaltic pavements, precast concrete elements and cast-in-place concrete elements. The bias factor and coefficient of variation for cast-in-place concrete elements are taken from Santiago (2017). He performed a statistical study with engineers from different states in Brazil. His study concluded that the self-weight of concrete follows a Normal distribution with a bias factor of 1.06 and coefficient of variation of 0.12. This is very close to the values (1.05, 0.10) stated in Ellingwood *et al* (1980). In Brazil there is no data available to estimate the probabilistic distribution parameters for asphaltic pavements and precast concrete elements. Thus, the values (Table 5. 6) proposed by Nowak (1999) are going to be considered representative of Brazilian bridges.

Table 5. 6 – Statistical parameters of dead load.

Variable	Bias Factor	CV	Distribution
Precast Elements	1.03	0.08	Normal
Asphaltic Pavements	1.10	0.25	Normal

The mean self-weight of reinforced and prestressed concrete are estimated considering the specific weight of concrete as 25 kN/m^3 , while the specific weight of asphalt is considered to be 24 kN/m^3 . The geometry of each element is taken as indicated by the project. For asphaltic pavement, if the thickness is not available, it is assumed to be 10 cm. This standard value is commonly used by the designers in Brazil. When available, New Jersey barriers are considered to distribute their weight evenly over the girders. The self-weight of girders are taken from the total volume of the girder divided by its length. This is mainly due to the fact that the cross section of the girder is enlarged at the supports, when compared with the middle span section. The dead load due to the asphalt is computed considering the distance between girders as the width.

The dead load due to the slab is taken as the minimum of two values: the distance between girders and the effective slab width. This width is computed following the recommendations based on the Brazilian code NBR6118:2014, in which Section 14.6.2.2 states that the effective width can be considered as a sum of the width of web and up to 10% of the span length (simple supported girders). There are some limitations to use of this recommendation. For only two of the bridges studied in this thesis, the distance between girders will not prevail. Figure 5. 10 shows a typical case of that. The dark green box represents the asphalt weight, while the light green box represents the self-weight of concrete slabs over the interior girder.

Figure 5. 10 – Typical reinforced concrete section under sectional analysis (adapted from Colombo, 2016).

5.4.3 Estimated Load Components of Selected Bridges

Following the procedure presented in the previous session, the nominal load components for moment and shear were estimated and are presented in Table 5. 7 to Table 5. 10. Three categories of dead load are considered, to account for differences in the degree of variation of the material. The analysis of the selected bridges gives information about actual values of resistance and load components. All the values presented are given per interior girder, with the exception of box girder bridges. In this case, the load components are given in terms of the whole bridge. The impact load is taken as a fraction of the live load component. The nominal resistance is computed using the 2K+1 method, as previously explained. The ultimate purpose of estimating the load is to compute the reliability indices. In this case, no coefficients are applied when computing these loads. The beta is about comparing the actual load with the actual resistance, thus, the moment capacity already accounts for all the factors that the code demands and since this thesis analyzes the designed section, it is understood that those factors were taken into consideration.

Table 5. 7 – Load and resistance components for selected prestressed bridges (Moment).

Bridge Number	D1 (KN.m)	D2 (KN.m)	D3 (KN.m)	LL (KN.m)	Impact (KN.m)	Resistance (KN.m)
1	1041	191	282	1876	188	8697
2	6618	963	1501	5258	526	30461
3	2721	635	547	2632	263	16431
4	3705	594	693	3317	332	18968
5	1052	222	285	1679	168	8665
6	1932	715	488	2419	242	11105
7	8783	1098	878	5447	545	32505
8	1224	167	268	1503	150	14190
9	3742	512	409	3047	305	18687

10	3061	683	365	3597	360	17493
11	1483	321	171	2040	204	9531
12	5873	904	482	4270	427	27072
13	1079	223	119	1507	151	9535
14	4369	493	790	3499	350	20327
15	2297	427	498	2470	247	14575
16	4761	835	975	4129	413	20681
17	3195	625	420	2848	285	20095
18	5900	514	576	3686	369	23462
19	1586	138	155	1519	152	12304
20	3087	813	475	3143	314	13345
21	5991	516	496	3598	360	25898
22	1150	139	134	1461	146	10302
23	1420	172	165	1662	166	10317
24	859	104	100	1227	123	9062
25	7059	703	675	6065	607	25995
26	940	127	122	1360	136	5713
27	6685	576	553	3954	395	24892
28	962	136	131	1539	154	5839
29	2989	832	348	3104	310	12997
30	1285	165	317	1865	187	6926
31	4405	733	1005	4463	446	18556
32	5984	941	904	5407	541	24945
33	5391	813	762	3629	363	21418
34	1635	221	213	2225	223	8961
35	15274	1591	2381	7088	709	77100
36	96480	4450	13284	32918	3292	217031
37	14793	341	1816	7558	756	50290
38	18005	376	2211	8615	862	50639
39	9514	0	1144	9247	925	48958

Table 5. 8 – Load and resistance components for selected reinforced bridges (Moment).

Bridge Number	D1 (KN.m)	D2 (KN.m)	D3 (KN.m)	LL (KN.m)	Impact (KN.m)	Resistance (KN.m)
40	601	105	169	1106	111	5538
41	745	121	194	1196	120	6151
42	132	3306	397	1346	135	10225
43	279	55	87	695	70	2596
44	697	86	83	1026	103	4504
45	1577	79	232	1729	173	7150
46	3354	165	340	4411	441	12808

47	5249	242	976	4364	436	25957
48	12844	1389	2344	6975	698	40128
49	2509	511	540	2047	205	17536
50	582	221	171	1993	199	5356
51	1072	360	261	2985	299	8274
52	1236	272	240	3658	366	6223
53	753	147	214	2350	235	12974
54	2042	250	394	3907	391	3889
55	251	134	79	1343	134	8296
56	1386	100	335	3292	329	7537
57	894	336	189	2372	237	27190
58	5912	898	1133	8873	887	15240
59	2669	1039	578	5160	516	18884
60	2406	650	856	5446	545	1381

Table 5. 9 – Load and resistance components for selected prestressed bridges (Shear).

Bridge Number	D1 (KN)	D2 (KN)	D3 (KN)	LL (KN)	Impact (KN)	Resistance (KN)
1	215	35	58	413	41	2227
2	679	84	84	684	68	4096
3	403	94	81	487	49	2717
4	497	80	93	543	54	3287
5	216	41	59	414	41	2845
6	475	1024	140	480	48	4121
7	303	101	77	471	47	2985
8	847	106	85	702	70	4699
9	302	41	66	403	40	2902
10	528	72	58	542	54	3421
11	408	91	49	602	60	3354
12	270	58	31	450	45	2601
13	621	96	51	571	57	4005
14	227	47	25	378	38	7342
15	511	58	92	516	52	3583
16	371	69	80	480	48	3679
17	549	96	112	595	60	3096
18	419	75	55	474	47	4337
19	669	58	65	529	53	4251
20	347	30	34	385	39	4252
21	419	100	64	522	52	2847
22	642	55	53	515	52	4323
23	284	34	33	411	41	2915

24	315	38	37	424	42	2750
25	245	30	29	376	38	3031
26	717	71	69	794	79	4073
27	204	28	26	386	39	2223
28	679	58	56	530	53	4045
29	220	31	30	439	44	2372
30	418	116	49	542	54	3108
31	268	34	66	456	46	2252
32	515	86	118	647	65	3703
33	611	96	92	705	71	4040
34	599	77	85	529	53	4114
35	269	36	35	476	48	2745
36	2405	151	375	1143	114	7900
37	6432	197	886	2219	222	14800
38	1909	22	234	1000	100	10415
39	2106	22	259	1068	1.56	12708

Table 5. 10 – Load and resistance components for selected reinforced bridges (Shear).

Bridge Number	D1 (KN)	D2 (KN)	D3 (KN)	LL (KN)	Impact (KN)	Resistance (KN)
40	160	28	45	358	36	1993
41	180	29	47	368	37	2193
42	46	1150	138	486	49	7872
43	103	20	32	298	30	1976
44	159	20	19	318	32	1645
45	300	8	44	399	40	1594
46	630	24	64	660	66	5358
47	1193	55	222	820	82	5509
48	2055	122	375	1143	114	6400
49	836	70	180	714	71	6436

5.4.4 Reliability Indices for Current Brazilian Live Load Model

Using the nominal load components as in Table 5. 7 to Table 5. 10 and the Monte Carlo Simulation technique (Melchers, 1999) the reliability index can then be estimated for each bridge. Only ultimate limit states are considered in the form of the following equation:

$$R > D1 + D2 + D3 + LL + \text{Impact} \quad (5. 5)$$

in which R is the resistance. The final results can be seen in Table 5. 11 (prestressed bridges) and Table 5. 12 (reinforced bridges) for moment and shear. For prestressed bridges, the empty

cells mean that the reliability index was so high that the value could not be computed. In Table 5. 12 the probability of failure for shear was not checked for bridges 50 to 60. Those are two girder bridges and the moment prevails. Also, these bridges are old, mainly from 1960's and 1970's and there is a trend to not build those bridges anymore.

Table 5. 11 –Reliability indices for current Brazilian Live Load Model. Prestressed bridges (Moment and Shear).

Bridge Number	Span Length (m)	Moment (KN.m)	Shear (KN)
1	19.4	5.38	6.77
2	39	5.30	6.53
3	27	6.35	6.09
4	30	5.62	6.65
5	20	6.28	-
6	25.5	4.35	7.20
7	41.5	5.06	7.15
8	16.2	-	7.09
9	28.33	5.88	6.82
10	30	5.33	6.78
11	22	5.38	4.95
12	37.8	6.26	7.76
13	19	7.85	-
14	34.2	5.51	6.64
15	24.8	6.32	-
16	34.7	4.33	5.47
17	30.5	7.39	-
18	35.3	5.72	8.29
19	18.3	-	-
20	29.5	3.53	5.69
21	37.3	6.56	-
22	16.2	-	-
23	18	7.40	7.52
24	14	-	-
25	39.4	3.78	6.04
26	18.4	4.85	7.21
27	39.4	5.43	8.29
28	17.5	4.30	7.04
29	28.6	3.60	6.55
30	19.2	3.69	5.71
31	34.9	3.48	6.38
32	39.2	4.25	6.32

33	36	5.04	8.29
34	24.3	4.39	6.88
35	22	8.80	4.09
36	60	3.68	3.22
37	31	5.14	6.58
38	34.2	3.85	-
39	36.4	6.03	6.99

The reliability index results show that the probability of failure for reinforced concrete bridges is larger than for prestressed concrete bridges, in terms of ultimate limit state. This is mainly due to the fact that for prestressed structure the statistical parameters vary less than for reinforced concrete structures, for example, prestressed girders are normally factory-made structures, while reinforced girders are normally casted in place. Casting concrete elements in a factory not only can help to control the variation of dimensions, but also the material parameters. This finding was also reported by Ferreira (2006).

Another important aspect of these results is that the reliability indices tend to decrease, as the span length increases (Figure 5. 13). This means that for larger spans, the probability of failure is higher than the one for shorter spans. Also, Figure 5. 11 shows that the ratio of live load to dead load is higher for shorter span which indicates that the current live load model is too conservative for shorter span. As reference, the reliability indices for prestressed girder bridges presented by Ferreira (2006) had a mean value of 4.22 and for box girder bridges 4.76. On the other hand, for reinforced girder bridges, the mean value was 3.49 and for box girder bridges was 4.36. It should be noted that the work performed by Ferreira (2006) was based on bridges designed by himself, and also based on the minimum resistance.

Figure 5. 11 – Live load/dead load ratio for reinforced and prestressed bridges.

Table 5. 12 –Reliability indices for current Brazilian Live Load Model. Reinforced bridges (Moment and Shear).

Bridge Number	Span Length (m)	Moment (KN.m)	Shear (KN)
40	15	7.26	7.03
41	16.6	7.02	7.45
42	11.5	5.20	-
43	10.8	4.85	7.50
44	17.5	5.62	6.53
45	21	4.21	3.93
46	30	2.57	6.28
47	24	5.12	4.74
48	25	4.06	4.02
49	12	6.28	5.98
50	16	3.21	-
51	21	3.09	-
52	20	4.67	-
53	17.6	3.18	-
54	26	3.97	-
55	12	4.24	-
56	22.4	2.52	-
57	18	3.86	-
58	35	2.93	-
59	25	2.63	-
60	26	4.29	-

The computation of reliability indices for Brazilian existing bridges based on the current live load model serve as a basis for the definition of a target reliability index. Nowak (1999) set the target beta by assuming that the safety level for simple supported bridge with span length of 60ft and girder spacing of 6ft was acceptable. His target beta was defined as 3.5. Moses (1999) recommended a minimum value of 3.5. Gosh (2000) defined a beta target of 3 when developing the bridge formula. Ferreira (2006) established the beta as 4 when developing equations to limit the truck weight in reinforced and prestressed bridges in Brazil. As previously mentioned, his results were based on bridges designed by himself, the bridges were not built, which means that he is mostly dealing with minimum values of resistance. In this thesis, the target beta is defined as the lowest mean value of the computed betas for flexure capacity for prestressed and reinforced bridges. This avoid considering very high and unpractical values of beta. The final target reliability index is 4.7 which is equivalent to a probability of failure of 1.3×10^{-6} .

5.4.5 Reliability Indices for Calibrated Brazilian Live Load Model

To properly create the margin of safety for the selected bridges considering the closeness to the target beta safety factors are going to be defined as in the following equation:

$$\gamma L_n = \mu + k\sigma \quad (5.6)$$

in which γ is the safety factor, L_n is the nominal load, μ is the mean value of the load, σ is the standard deviation and k is a integer that set the distance from the mean in terms of standard deviation. Using the definition of bias factor and coefficient of variation and doing the proper arrangements the final safety factors for load can be defined as:

$$\gamma_i = \lambda_i(1 + kV_i) \quad (5.7)$$

in which λ_i is the bias factor and V_i is the coefficient of variation. This thesis follows the recommendation by Nowak (1999) and takes $k = 2$. Considering the bias factors and coefficient of variation presented in the previous chapter, the final safety factors based on WIM data and current Brazilian live load model for different span lengths would be:

Table 5. 13 –Safety Factors based on WIM data and NBR7188:2013.

Span Length (m)	Partial Factors Moment	Partial Factors Shear
6.1	1.51	1.49
9.1	1.46	1.55
12.2	1.47	1.62
15.2	1.46	1.57
18.3	1.49	1.56
21.3	1.49	1.55
24.4	1.49	1.53
27.4	1.49	1.51
30.5	1.48	1.49
33.5	1.46	1.47
36.6	1.45	1.44
39.6	1.43	1.41
42.7	1.41	1.38
45.7	1.38	1.35
48.8	1.35	1.32
51.8	1.33	1.30
54.9	1.30	1.26
57.9	1.27	1.24
61	1.25	1.21
64	1.22	1.18
67.1	1.20	1.16
70.1	1.17	1.13
73.2	1.15	1.11
76.2	1.12	1.08
79.2	1.10	1.06
82.3	1.08	1.04
85.3	1.05	1.02
88.4	1.03	1.00
91.4	1.01	0.97

For practical reasons, only the largest value is taken into account when calibrating the current model. In this case, for moment and shear the safety factors are 1.5 and 1.6, respectively. Using Equation (5. 7), the safety factor for dead load is 1.4 (D1 and D2) and for pavement (D3) is 1.5. Figure 5. 12 shows how the safety factors change with k for dead and live load considering the proposed method. The minimum resistance and the resistance factor can then be calculated according to Equation (5. 8).

Figure 5. 12 – Safety factors versus k for dead and live load.

$$\frac{R}{\gamma_{\text{Res}}} = 1.4DL + 1.5D3 + \gamma_{LL}(LL + \text{Impact}) \quad (5.8)$$

in which R is the minimum required resistance, γ_{Res} is the resistance factor, LL is the live load, Impact is the load due to dynamic effects taken as a fraction of LL , DL is dead load ($D1+D2$), $D3$ is the pavement self weight. The safety factors for live load (γ_{LL}) is given in Table 5. 14.

Table 5. 14 – Live load partial factors.

	Limit State	Current Model	Proposed Model
Reinforced Concrete	Moment	1.50	1.80
	Shear	1.60	1.80
Prestressed Concrete	Moment	1.50	1.80
	Shear	1.50	1.80

After analyzing different values of resistance safety factors using the closeness to the target reliability index as the main criteria, the final factors for the current live load model, and the proposed model in the previous chapter are:

Table 5. 15 – Resistance partial factors.

	Limit State	Current Model	Proposed Model
Reinforced Concrete	Moment	1.00	1.00
	Shear	1.10	1.10
Prestressed Concrete	Moment	1.10	1.05
	Shear	1.15	1.10

Using the load components shown in Table 5. 7 to Table 5. 10, combined with Equation (5. 8), the new reliability indices for moment and shear were estimated. Table 5. 16 shows the beta values for flexure capacity, for both calibrated NBR7188:2013 and the proposed live

load model, for prestressed bridges. Table 5. 17 shows the betas for moment for reinforced concrete bridges. Table 5. 18 and Table 5. 19 show the reliability indices for shear of both types of bridge.

Table 5. 16 –Reliability indices for calibrated Brazilian Live Load Model and suggested new live load model. Prestressed bridges (Moment).

Bridge Number	Span Length (m)	Calibrated NBR 7188	New Live Load Model
1	19.4	4.69	4.64
2	39	4.43	4.25
3	27	4.48	4.50
4	30	4.47	4.46
5	20	4.68	4.78
6	25.5	4.49	4.54
7	41.5	4.52	4.31
8	16.2	4.65	4.73
9	28.33	4.32	4.35
10	30	4.53	4.55
11	22	4.67	4.75
12	37.8	4.67	4.51
13	19	4.76	4.84
14	34.2	4.44	4.38
15	24.8	4.51	4.56
16	34.7	4.38	4.32
17	30.5	4.51	4.51
18	35.3	4.53	4.41
19	18.3	4.71	4.77
20	29.5	4.49	4.50
21	37.3	4.52	4.38
22	16.2	4.85	4.86
23	18	4.75	4.83
24	14	4.80	4.82
25	39.4	4.57	4.41
26	18.4	4.76	4.85
27	39.4	4.49	4.34
28	17.5	4.76	4.86
29	28.6	4.58	4.60
30	19.2	4.63	4.74
31	34.9	4.49	4.44
32	39.2	4.58	4.43
33	36	4.48	4.39
34	24.3	4.67	4.77

35	22	4.99	4.69
36	60	4.98	4.20
37	31	4.33	4.28
38	34.2	4.31	4.23
39	36.4	4.60	4.55

Table 5. 17 –Reliability indices for calibrated Brazilian Live Load Model and suggested new live load model. Reinforced bridges (Moment).

Bridge Number	Span Length (m)	Calibrated NBR 7188	New Live Load Model
40	15	4.77	5.05
41	16.6	4.60	4.96
42	11.5	3.90	3.96
43	10.8	4.75	5.03
44	17.5	4.77	5.27
45	21	4.52	4.87
46	30	3.71	3.86
47	24	3.65	3.81
48	25	3.51	3.58
49	12	3.80	3.86
50	16	4.01	4.13
51	21	3.85	4.06
52	20	3.88	4.10
53	17.6	3.93	4.12
54	26	3.77	3.98
55	12	4.10	4.21
56	22.4	3.82	4.04
57	18	3.90	4.08
58	35	3.69	3.83
59	25	3.75	3.95
60	26	3.76	3.96
61	20	3.86	4.06

Figure 5. 13 shows the reliability indices based on the current live load model. It is possible to visually notice that the indices reach values ranging from 3.48 to 7.85. This pattern can also be observed for shear in prestressed and reinforced concrete bridges. On the other hand, Figure 5. 14 and Figure 5. 15 show the reliability indices for the same bridges using the proposed calibration factors . It can be noticed that the beta values are less dispersed. The minimum and maximum reliability index for calibrated NBR7188:2013 are 4.48 and 5.04 respectively, while for the proposed new model the betas range from 4.20 to 4.86.

Figure 5. 13 – Reliability Indices for Prestressed Bridges based on current Brazilian Live Load Model - Moment.

Figure 5. 14 – Reliability Indices for Prestressed Bridges based on calibrated Brazilian Live Load Model - Moment.

Figure 5. 15 – Reliability Indices for Prestressed Bridges based on Proposed Live Load Model - Moment.

Table 5. 18 –Reliability indices for calibrated Brazilian Live Load Model and suggested new live load model. Prestressed bridges (Shear).

Bridge Number	Span Length (m)	Calibrated NBR 7188	New Live Load Model
1	19.4	4.93	5.04
2	39	4.78	4.51
3	27	5.00	4.97
4	30	5.03	4.95
5	20	5.04	5.07
6	26	4.55	4.44
7	25.5	4.90	4.93
8	41.5	5.08	4.74
9	16.2	4.35	4.40
10	28.33	4.89	4.86
11	30	4.99	4.93
12	22	4.59	4.66
13	37.8	5.09	4.82
14	19	5.03	5.14
15	34.2	4.34	4.20
16	24.8	4.86	4.88
17	34.7	5.02	4.87
18	30.5	5.00	4.93
19	35.3	5.02	4.81
20	18.3	4.98	5.05
21	29.5	4.66	4.59
22	37.3	5.07	4.81
23	16.2	5.01	5.12
24	18	5.02	5.11
25	14	4.53	4.62
26	39.4	5.09	4.82
27	18.4	5.04	5.17
28	39.4	4.52	4.27
29	17.5	5.02	5.16
30	28.6	5.03	4.97
31	19.2	4.86	4.94
32	34.9	4.86	4.72
33	39.2	4.77	4.51
34	36	5.04	4.84
35	24.3	4.47	4.53
36	22	4.67	4.60
37	60	4.78	4.25
38	31	3.90	3.76
39	34.2	4.84	4.62

Table 5. 19 –Reliability indices for calibrated Brazilian Live Load Model and suggested new live load model. Reinforced bridges (Shear).

Bridge Number	Span Length (m)	Calibrated NBR 7188	New Live Load Model
40	15.0	5.03	5.17
41	16.6	4.97	5.08
42	11.5	3.68	3.76
43	10.8	4.03	4.16
44	17.5	4.89	5.00
45	21.0	4.83	4.91
46	30.0	4.21	4.15
47	24.0	4.07	4.09
48	25.0	4.00	3.97
49	12.0	4.15	4.22

6. CONCLUSIONS AND FUTURE WORKS

6.1 CONCLUSIONS

A total of 9.3 million trucks were recorded by the two WIM stations presented in this thesis. São Paulo WIM station recorded a total of 8.35 million vehicles from September 2015 to August 2017. A daily average of 13292 trucks was found. Out of those, 6.08 million vehicles were filtered out. Hence, 3631 vehicles are the average daily qualified trucks. Only 27.2% of the entire sample are considered valid trucks. On the other hand, in Rio Grande do Sul WIM station, a total of 927174 vehicles were recorded from March to June (79 days) 2014. 83% of those records were filtered out. 1962 vehicles are the average daily qualified trucks. In both samples the filter responsible for removing the majority of trucks is criterion 1, which filters out any records with GVW less than 6.2 tons. This filter alone removes on average 72% of the whole sample. The filters used in this research were defined based on Brazilian fleet. From Chapter 2 is also possible to conclude that the temperature plays an important role on the accuracy level of WIM systems. Thus, performing calibration is a must-do activity in order to record reliable data.

From Chapter 3, it can be concluded that trucks belonging to classes 2C, 3C and 3S3 are the most common ones, and combined they represent 53.51% and 57.13% of total valid records for São Paulo and Rio Grande do Sul WIM data, respectively. Also, 17% and 24% of the trucks travel on left lane for SP and RS WIM data, respectively. Bridges with span length from 6.1 to 91.4 meters were considered in the statistical study for evaluation of single and multiple presence occurrences. The number of multiple presence occurrences is notably affected by the bridge span length, with the exception of side-by-side case. The results show that the percentage of side-by-side events stays steadily at 0.95% (Sao Paulo WIM data) until it reaches span length of 50 meters, and then starts to slowly decrease. The frequency of side-by-side events has an average value of 0.70%, for all bridge spans and volume conditions. The results demonstrated that the occurrence of two heavy trucks in adjacent lanes have a small probabilities of occurrence, reaching less than 0.02% for any given span length.

The analysis of current Brazilian live load model demonstrated that the bias factors are not uniform in relation to bridges span length. The bias factors presented values that go from 0.5 to 2.0. The current live load model presents bias factors that are higher for shorter spans no matter which kind of load effect is under consideration (shear, positive or negative moment). Also, the extrapolation of 5% of the upper tail has been shown to be enough. The upper tail of the distribution resembles a straight line, and the fit for 5% of the data has a regression coefficient R^2 between 0.96 and 0.99.

The proposed approach for estimating the bias factors shows that considering only full correlated trucks is too conservative, mostly for short spans, where there is a lack of occurrences, especially following events. On the other hand, taking into account no correlation at all yields very low bias factors. At last, a more rational live load model was developed based on WIM data. The new model is a combination of the current Brazilian model with a new truck and new distributed load of 900kg/m for positive moment and shear. For negative moment two trucks are recommended. The new model yields more uniform results for bias factor, for different span lengths. The bias factor ranges from 0.97 to 1.35.

Finally, in Chapter 5 a total of sixty existing Brazilian bridges were analyzed in which 39 are prestressed and 21 reinforced concrete bridges. Those bridges are located in five different states: Pernambuco, Ceará, Bahia, São Paulo, Minas Gerais. Probability of failure was estimated in terms of moment and shear for interior girders and box girders. Only ultimate state limit was considered. It was found that reliability indices are higher in prestressed bridges, when compared to reinforced bridges. Another important aspect of these results is that the reliability indices tend to decrease as the span length increases. This means that for larger spans, the probability of failure is higher than for shorter spans; this is the opposite conclusion found by Ferreira (2006). After computing the reliability index for all bridges in the sample, a target beta of 4.7 was defined, which is equivalent to a failure probability of 1.3×10^{-6} . After redesigning the bridges and performing a new reliability analysis, the safety factors were found to be as in Table 5. 14 and Table 5. 15. The final reliability index where found to be more uniform, varying from 4.20 to 4.86 for prestressed bridges, and 3.81 to 5.27 for reinforced concrete bridges.

6.2 FUTURE DEVELOPMENTS

Given the fact that many subjects need investigation, when dealing with concrete structures and development of a new live load model, there are countless topics that need further addressing. First, the dataset of truck loads needs to be increased with more WIM stations, preferably from different Brazilian states. Also, in this research there were no studies related to fatigue and dynamic effects. The computation of these two load effects need to be considered when developing a live load model. The dynamic effect considered in this thesis, although valid, is still basic and was not developed based on Brazilian bridges. Future work should estimate reliability indices for fatigue and dynamic effects.

Another important aspect that should be addressed in future works are the calculus of reliability indices for service limit states. In this research, only ultimate limit state was considered. In addition, a future work might include more bridges of different span lengths and numbers of girder and it might include steel bridges. For future work, the extrapolation technique used in this thesis can be replaced by using FORM. Also, the new research can consider extrapolating the occurrences of maximum loads that can physically happen. Finally, more research including more statistical parameters of all variables included in the analysis are needed, as for example, it would be recommended to have statistical parameter of workforce quality in Brazilian bridge construction.

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APPENDIX A

Side View (Illustration)	Axle Spacing (meters)	Axle Load (tons)	Classes
$d_{12} \geq 3.5$	$E1 \leq 6.0$ $E2 \leq 6.0$	2C
$d_{12} > 2.40$ $1.20 < d_{23} \leq 2.40$	$E1 \leq 6.0$ $E2E3 \leq 17$ (tandem)	3C
$d_{12}, d_{23} > 2.40$	$E1 \leq 6$ $E2 \leq 10$ $E3 \leq 10$	2S1
$d_{12}, d_{23} > 2.40$ $1.20 < d_{34} \leq 2.40$	$E1 \leq 6$ $E2 \leq 10$ $E3E4 \leq 17$ (tandem)	2S2
$d_{12} > 2.40$ $d_{12} > 2.40$ $d_{12} > 2.40$	$E1 \leq 6$ $E2 \leq 10$ $E3 \leq 10$ $E4 \leq 10$	2I2
$1.20 < d_{12} \leq 2.40$ $d_{23} > 2.40$ $1.20 < d_{34} \leq 2.40$	$E1E2 \leq 12$ $E3E4 \leq 13.5$ (tandem)	4DD
$d_{12}, d_{23} > 2.40$ $1.20 < d_{34} \leq 2.40$ $1.20 < d_{45} \leq 2.40$	$E1 \leq 6$ $E2 \leq 10$ $E3E4E5 \leq 25.5$ (tandem)	2S3
$d_{12}, d_{34} > 2.40$ $1.20 < d_{23} \leq 2.40$ $1.20 < d_{45} \leq 2.40$	$E1 \leq 6$ $E2E3 \leq 17$ (tandem) $E4E5 \leq 17$ (tandem)	3S2

$d_{12}, d_{34} > 2.40$ $d_{45}, d_{56} > 2.40$ $1.20 < d_{23} \leq 2.40$	$E1 \leq 6$ $E2E3 \leq 17$ (tandem) $E4, E5, E6 \leq 10$	3I3
$d_{12} > 2.40$ $1.20 < d_{23} \leq 2.40$ $d_{34} > 2.40$ $1.20 < d_{45} \leq 2.40$ $d_{56} > 2.40$ $1.20 < d_{67} \leq 2.40$	$E1 \leq 6$ $E2E3 \leq 17$ (tandem) $E4E5 \leq 17$ (tandem) $E6E7 \leq 17$ (tandem)	3D4
$d_{12} > 2.40$ $1.20 < d_{23} \leq 2.40$ $d_{34} > 2.40$ $1.20 < d_{45} \leq 2.40$ $1.20 < d_{56} \leq 2.40$ $d_{67} > 2.40$ $1.20 < d_{78} \leq 2.40$ $1.20 < d_{89} \leq 2.40$	$E1 \leq 6$ $E2E3 \leq 17$ (tandem) $E4E5E6 \leq 25.5$ (tandem) $E7E8E9 \leq 25.5$ (tandem)	3M6

Figure A. 1 - Vehicle types according to DNIT classification

APPENDIX B

Axle load and axle spacings histograms for trucks Class 2C, 2S3 and 3S3

Class 2C:

Figure B. 1 - Axle weight for trucks Class 2C: first axle – SP.

Figure B. 2 - Axle weight for trucks Class 2C: second axle – SP.

Figure B. 3 - Axle spacing Class 2C: first to second axles – SP.

Class 2S3:

Figure B. 4 - Axle weight for trucks Class 3S2: first axle – SP.

Figure B. 5 - Axle weight for trucks Class 3S2: second axle – SP.

Figure B. 6 - Axle weight for trucks Class 3S2: third axle – SP.

Figure B. 7 - Axle weight for trucks Class 3S2: fourth axle – SP.

Figure B. 8 - Axle weight for trucks Class 3S2: fifth axle – SP.

Figure B. 9 - Axle spacing for trucks Class 3S2: first to second axle – SP.

Figure B. 10 - Axle spacing for trucks Class 3S2: second to third axle – SP.

Figure B. 11 - Axle spacing for trucks Class 3S2: third to fourth axle – SP.

Figure B. 12 - Axle spacing for trucks Class 3S2: fourth to fifth axle – SP.

Class 3S3:

Figure B. 13 - Axle weight for trucks Class 3S3: first axle – SP.

Figure B. 14 - Axle weight for trucks Class 3S3: second axle – SP.

Figure B. 15 - Axle weight for trucks Class 3S3: third axle – SP.

Figure B. 16 - Axle weight for trucks Class 3S3: fourth axle – SP.

Figure B. 17 - Axle weight for trucks Class 3S3: fifth axle – SP.

Figure B. 18 - Axle weight for trucks Class 3S3: sixth axle – SP.

Figure B. 19 - Axle spacing for trucks Class 3S3: first to second axles – SP.

Figure B. 20 - Axle spacing for trucks Class 3S3: second to third axles – SP.

Figure B. 21 - Axle spacing for trucks Class 3S3: third to fourth axles – SP.

Figure B. 22 - Axle spacing for trucks Class 3S3: fourth to fifth axles – SP.

Figure B. 23 - Axle spacing for trucks Class 3S3: fifth to sixth axles – SP.

APPENDIX C

GVW statistics of five most important truck classes for SP WIM data.

Table C. 1 – GVW statistics from September 2015.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	15.95	10.42	3.09	6.2	23.87	11.16
3C/3CD/3DC/3BB/3BC	24.68	17.35	5.49	6.8	38.47	11.78
2S2/2B2/2BD/2DB	13.21	19.94	5.11	10.46	46.87	12.16
2S3	10.35	34.29	8.48	13.21	68.89	18.29
3S3/3D3	17.43	43.66	8.51	15.67	81.46	17.91

Table C. 2 – GVW statistics from October 2015.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	15.18	10.41	3.05	6.20	24.33	11.19
3C/3CD/3DC/3BB/3BC	23.96	17.30	5.35	6.97	38.50	11.79
2S2/2B2/2BD/2DB	13.22	20.01	5.09	10.31	44.79	19.12
2S3	10.25	34.26	8.17	12.05	66.61	18.26
3S3/3D3	15.06	43.39	8.10	16.03	80.21	17.81

Table C. 3 – GVW statistics from November 2015.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	14.59	9.97	2.73	6.20	24.52	11.30
3C/3CD/3DC/3BB/3BC	24.16	16.36	4.88	6.87	37.56	11.81
2S2/2B2/2BD/2DB	13.10	18.79	4.61	10.45	46.23	19.21
2S3	10.55	31.89	7.40	12.35	63.37	18.29
3S3/3D3	14.83	40.59	7.19	15.89	75.84	17.90

Table C. 4 – GVW statistics from December 2015.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	15.80	10.14	2.93	6.20	23.28	11.41
3C/3CD/3DC/3BB/3BC	24.14	16.32	4.92	6.87	38.78	12.07
2S2/2B2/2BD/2DB	14.22	19.09	4.83	10.43	45.29	19.37
2S3	9.76	32.47	7.69	12.59	64.00	18.23
3S3/3D3	13.89	41.13	7.58	15.16	73.13	17.92

Table C. 5 – GVW statistics from January 2016.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	15.59	10.48	3.20	6.20	22.60	11.37
3C/3CD/3DC/3BB/3BC	25.53	16.95	5.04	6.71	39.83	11.97
2S2/2B2/2BD/2DB	12.67	19.35	4.80	10.28	50.55	19.20
2S3	9.50	33.40	7.89	13.02	65.00	18.27
3S3/3D3	14.24	42.42	7.64	16.21	73.40	17.86

Table C. 6 – GVW statistics from June 2016.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	14.23	9.67	2.56	6.20	21.07	10.37
3C/3CD/3DC/3BB/3BC	23.73	16.13	4.78	6.89	35.04	10.80
2S2/2B2/2BD/2DB	12.80	18.47	4.61	10.28	46.81	18.48
2S3	9.91	31.34	6.91	13.53	60.12	17.44
3S3/3D3	15.08	39.33	6.97	15.33	72.89	17.01

Table C. 7 – GVW statistics from July 2016.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	14.60	9.82	2.76	6.20	21.99	10.38
3C/3CD/3DC/3BB/3BC	24.33	16.40	4.79	6.78	35.44	10.86
2S2/2B2/2BD/2DB	12.25	18.60	4.48	10.67	44.03	18.36
2S3	9.88	31.69	7.16	13.46	62.90	17.47
3S3/3D3	14.85	40.06	7.02	16.00	78.80	17.04

Table C. 8 – GVW statistics from August 2016.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	14.74	9.89	2.70	6.20	24.04	10.28
3C/3CD/3DC/3BB/3BC	23.13	16.39	4.88	6.93	39.66	10.83
2S2/2B2/2BD/2DB	12.62	18.70	4.47	10.23	50.59	18.39
2S3	9.92	32.18	7.37	13.35	66.14	17.42
3S3/3D3	14.98	40.79	7.18	15.31	81.01	17.08

Table C. 9 – GVW statistics from September 2016.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	14.83	9.90	2.71	6.20	23.33	10.23
3C/3CD/3DC/3BB/3BC	23.49	16.45	4.91	6.76	37.07	10.84
2S2/2B2/2BD/2DB	12.83	18.83	4.48	9.93	45.41	18.34
2S3	9.65	32.29	7.64	12.67	65.66	17.46
3S3/3D3	14.79	41.12	7.47	15.59	82.61	17.16

Table C. 10 – GVW statistics from October 2016.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	13.80	9.72	2.65	6.20	24.14	10.43
3C/3CD/3DC/3BB/3BC	23.61	16.02	4.66	6.77	36.68	10.90
2S2/2B2/2BD/2DB	12.82	18.06	4.25	10.44	43.35	18.36
2S3	9.89	30.61	7.01	12.79	63.87	17.51
3S3/3D3	15.09	39.05	7.15	15.50	80.61	17.25

Table C. 11 – GVW statistics from November 2016.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	14.70	9.66	2.59	6.20	24.57	10.37
3C/3CD/3DC/3BB/3BC	23.46	15.88	4.64	6.87	36.93	10.88
2S2/2B2/2BD/2DB	13.05	18.05	4.17	10.08	43.67	18.33
2S3	9.55	30.55	7.13	13.34	63.57	17.51
3S3/3D3	14.45	38.93	7.19	14.56	84.75	17.28

Table C. 12 – GVW statistics from December 2016.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	15.03	9.84	2.74	6.20	24.00	10.44
3C/3CD/3DC/3BB/3BC	23.17	15.97	4.67	6.90	38.52	11.12
2S2/2B2/2BD/2DB	14.03	18.34	4.41	10.49	47.24	18.49
2S3	9.15	31.01	7.30	12.75	64.92	17.52
3S3/3D3	13.84	39.51	7.31	15.49	83.26	17.32

Table C. 13 – GVW statistics from January 2018.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	14.61	9.75	2.83	6.20	22.94	10.58
3C/3CD/3DC/3BB/3BC	23.91	15.57	4.40	6.68	38.42	11.16
2S2/2B2/2BD/2DB	12.77	17.59	4.05	10.45	39.73	18.46
2S3	9.06	30.22	6.97	13.22	68.87	17.55
3S3/3D3	14.65	38.15	7.02	15.39	86.04	17.31

Table C. 14 – GVW statistics from February 2018.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	14.92	9.90	2.77	6.20	23.78	10.36
3C/3CD/3DC/3BB/3BC	23.08	16.15	4.83	6.85	39.29	10.92
2S2/2B2/2BD/2DB	13.09	18.72	4.47	10.59	49.74	18.53
2S3	9.52	31.81	7.52	12.51	67.47	17.53
3S3/3D3	14.89	40.30	7.48	15.99	78.37	17.24

Table C. 15 – GVW statistics from March 2018.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	15.55	10.36	3.07	6.20	24.17	10.26
3C/3CD/3DC/3BB/3BC	23.01	17.55	5.39	6.96	39.95	11.03
2S2/2B2/2BD/2DB	12.82	20.17	5.06	10.52	48.18	18.51
2S3	9.79	34.58	8.40	12.42	68.26	17.57
3S3/3D3	15.10	44.20	8.39	14.56	83.47	17.28

Table C. 16 – GVW statistics from April 2018.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	14.79	10.23	2.95	6.20	22.12	10.34
3C/3CD/3DC/3BB/3BC	23.15	16.68	5.00	6.80	39.68	11.07
2S2/2B2/2BD/2DB	13.08	19.39	4.55	9.69	44.48	18.55
2S3	9.19	32.77	7.65	13.67	61.48	17.56
3S3/3D3	15.62	41.88	7.66	14.21	81.00	17.22

Table C. 17 – GVW statistics from May 2018.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	14.73	9.93	2.74	6.20	21.72	10.34
3C/3CD/3DC/3BB/3BC	23.28	16.32	4.90	6.93	40.62	11.05
2S2/2B2/2BD/2DB	12.60	18.79	4.38	10.45	46.93	18.58
2S3	9.39	31.87	7.26	12.96	63.36	17.64
3S3/3D3	15.44	40.48	7.19	15.06	79.16	17.29

Table C. 18 – GVW statistics from June 2018.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	15.05	9.95	2.76	6.20	21.45	10.30
3C/3CD/3DC/3BB/3BC	23.56	16.38	4.88	6.77	38.24	11.05
2S2/2B2/2BD/2DB	12.85	18.88	4.41	10.47	42.97	18.51
2S3	9.27	31.99	7.36	12.90	67.97	17.59
3S3/3D3	15.05	40.55	7.18	15.39	71.10	17.27

Table C. 19 – GVW statistics from July 2018.

Classes of Trucks (DNIT)	[%]	GVW_{avg} [ton]	GVW_{std} [ton]	GVW_{min} [ton]	GVW_{max} [ton]	L_{avg} [m]
2C/2CB	14.73	10.26	3.10	6.20	22.60	10.27
3C/3CD/3DC/3BB/3BC	23.95	17.26	5.24	6.93	41.00	10.96
2S2/2B2/2BD/2DB	12.35	19.58	4.94	9.38	45.94	18.52
2S3	9.26	32.59	8.45	11.98	65.00	17.48
3S3/3D3	15.48	40.95	9.08	15.04	78.14	17.08

APPENDIX D

PRESTRESSED BRIDGE (BRIDGE 21) - Example of how the load effects were computed in this thesis. The cross section below shows a 4 girder bridge (Figure D.1). This is a simple supported bridge with a span length of 37.3 meters. The load effects are computed only in terms of interior girder.

i) Loads

Moment due to dead load is considered in terms of self weight of the girder, slab, pavement (5 cm) and barriers. For the moment due to the live load the TB450 combined with the girder distribution factors were considered. The nominal moments were found to be:

$$M_{DL} = 7003 \text{ kN.m} \quad (M_{D1} = 5991, M_{D2} = 516, M_{D3} = 496)$$

$$M_{LL} = 3598 \text{ kN.m (no impact)}$$

For the sake of comparison, this bridge was also modeled in STRAP and the results were similar.

$$M_{DL} = 6820 \text{ kN.m (result from STRAP)}$$

$$M_{LL} = 3264 \text{ kN.m (result from STRAP - no impact)}$$

Considering the bias factor as presented in chapter 5*, the mean and standard deviation for flexure are:

Table D. 1 – Mean moments due to dead and live load

Load	Mean (kN.m)	Standard Dev. (kN.m)
D1	6171	494
D2	542	54
D3	545	136
LL + Imp	4527	888

*the bias factor for positive moment and a span length of 37.3 meters is 1.14.

ii) Resistance

Figure D.1 shows the cross section of the bridge analyzed in this example. This bridge used a concrete with a $f_{ck} = 40\text{MPa}$, reinforcement CA-50 ($f_{yk} = 500\text{MPa}$) and four tendons with a nominal area of 1540mm^2 each. The position of each tendon follows Figure D.2.

$$\mu_{\sigma c} = 41.7 \text{ MPa (CV = 0.086)}$$

$$\mu_{\sigma p} = 1976 \text{ MPa (CV = 0.019)}$$

$$\mu_{Es} = 218.4 \text{ GPa (CV = 0.02)}$$

$$\mu_{Ep} = 206 \text{ GPa (CV = 0.02)}$$

For the concrete, a tension strength of 2MPa was considered.

Figure D. 1 – Cross section of bridge 31

Figure D. 2 – Interior girder: (a) geometry (b) tendons

Figure D.3 shows the reinforcement as the original project while Table D.2 shows the diameter of each bar.

Figure D. 3 – Reinforcement of the interior girder.

Table D. 2 – Mean moments due to dead and live load

Bar Number	Diameter
2	16
3	6.3
8	12.5
10	16
17	6.3
14	16
25	6.3
27	8

With all the data present above a model of the cross section was built within Response2000 as in Figure D.4. The final resistant moment was found to be 25898 kN.m. For resistance, the bias factor of this bridge and the coefficient of variation were 1.044 and 0.094, respectively. Then, the final mean and standard deviation of resistance are:

$$\mu_{\text{res}} = 27038 \text{ kN.m} (25898 \times 1.044)$$

$$\sigma_{\text{res}} = 2529 \text{ kN.m}$$

Resistance is considered to follow a lognormal distribution.

Considering the values μ_{res} , σ_{res} , and the values in Table D.1 the final beta was found to be

$$\beta = 6.56.$$

Geometric Properties

	Gross Conc.	Trans (n=5.20)
Area (mm ²) x 10 ³	1233.5	1293.9
Inertia (mm ⁴) x 10 ⁶	740097.0	812266.8
y _t (mm)	755	790
y _b (mm)	1495	1460
S _t (mm ³) x 10 ³	980532.0	1028403.4
S _b (mm ³) x 10 ³	494979.1	556283.4

Crack Spacing

$$2 \times \text{dist} + 0.1 d_b / \rho$$

Loading (N,M,V + dN,dM,dV)

$$0.0, 0.0, 0.0 + 0.0, 1.0, 0.0$$

Figure D. 4 – Response2000 model for resistant moment.

Figure D. 5 – Resistant moment of the girder.