ADDIS ABABA UNIVERSITY

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Development of Live Load Model for Ethiopian Short to Medium Span Bridges

A Thesis in Structural Engineering

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A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

The undersigned have examined the thesis entitled '**Development of Live Load Model for Ethiopian Short to Medium Span Bridges**' presented by **Binyam Wesenseged**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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UNDERTAKING

I certify that research work titled "Development of Live Load Model for Ethiopian Short to Medium Span Bridges" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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ABSTRACT

This thesis assesses the development of a new live load model for Ethiopian bridges that reflects the actual developments of the growing traffic. To have a clear idea of the research idea different countries' experiences and models were explored to have the best benchmark practice that helps to replicate to our context.

The subject of interest of this paper is to explore the existing traffic loading model in Ethiopia and have contributed to its role in identifying the strength and weaknesses of this model. A Quantitative research design used to be applied for the study and efforts were made to use data from Static Weighing Stations obtained from the Ethiopian Roads Authority from three sites, which consists of 35,000 truck loading data in total. A simulation program was developed for the calculation of static load effects for simply supported bridges spanning up to 50m. Extreme 75-year loads obtained using different statistical extrapolation methods and the data was used to compare the loading effects generated from the actual trucks with the loading effects of the trucks used in the Ethiopian Bridge Design Manual. The result shows that the traffic loading effects obtained from the analysis vary from the currently adopted practice of the Ethiopian bridge design manual that demands to increase either the design load or the factor of safeties indicating the need for further research to reflect the existing reality.

KEYWORDS: Bridge, Traffic Load, Load Effect, Static Weighing Stations.

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Roman Upper Case

A Concentrated axle load

Fn (x) The empirical distribution function for a discrete random variable $X = x_1, x_2, \ldots, x_n$,

FX(x) Probability distribution function or cumulative distribution function of the stochastic variable X.

L Bridge span

N The average number of vehicle queues that is assumed to occur per year.

P(X ≤ x) The probability that $X \leq x$

R_T The return period

T The reference time

GVW The gross vehicle weight

Roman Lower Case

fx(x) Probability density function of the random variable X

Greek Upper Case

$\Phi(\cdot)$ The standard normal distribution function

$\phi(\cdot)$ The standard normal density function

Greek Lower Case

μ The mean value of X

σ The standard deviation of X.

Abbreviations

ERA The Ethiopian Roads Authority

EN The European Standard

HeDS Headway Distribution Statistics

LM1 The Load Model 1 according to the Eurocode

WIM Weigh-In-Motion

cdf Cumulative distribution function

pdf Probability density function

UDL Uniformly Distributed Load

1. BACKGROUND

Road infrastructures development contributes a significant role for the development of a country in many ways, because it does enhancement the transportation facilities which will create efficiency and effectiveness of the business and the economy. Especially the road transportation sector is vital for developing countries like Ethiopia as other means of transportations are expensive to explore. *"Road transportation has played a dynamic role in developing economy, commercial life, and trade, traditions, and generally shaped the thought, the culture, and the socio-economic life of people of sub-Saharan countries particularly in Ethiopia, where other means of transportation is not easily available " (Carruthers, et al., 2008; Gwilliam, et al., 2008).*

The economic growth of the country brings a challenge which deserves to give a due attention for the road deteriorations encored by the flow of heavy vehicles and a new policies was induce to control over-weight vehicles in order to protect these roads against misuse and damage. Ethiopia has promulgated a road traffic act that stipulates permissible maximum axle and vehicle mass and dimensions Act 11/1990 by the Council of Ministers. The limits are meant to ensure that roads last for their full design life with normal maintenance expenditures.

The role of proper road management and maintenance is next to none in priority for the economic development of any country. It does create an opportunity to make the supply chain more effective and efficient. For a country like Ethiopia where vast majorities of the population leads their life in agriculture it does create a market opportunities for agricultural products. *"The issue of market access is more relevant for a country like Ethiopia where the rural population accounts for about 78.8% of the national population" (World Bank, 2019).*

According to Strock (2013), in recent years data-driven decisions for infrastructure maintenance have been encouraged by the American Association of State Highway and Transportation Officials (AASHTO), Federal Highway Administration (FHWA), and the Transportation Research Board (TRB). To make effective decision on the area requires to collect data's on truck weights, types, and the number of trucks traveling on a highway network, in chronological order. Such types of data can be collected at weigh Stations, which require trucks to pull over and weighted statically. In Ethiopia, there are nine stationary weigh stations operating at key locations covering the main routes of the country. *"To aid those stations the use of mobile weighbridges for random axle load control activities is undertaken. Two mobile teams are dedicated to this task, operating in different areas of the country and covering those routes missed by the stationary weighbridges" (ERA 2016).* All these data's collected from Dec 2017 till Dec 2019 were used for the study.

1.1 Statement of the problem

Many decisions made with regards to infrastructure management, such as configuration and the number of freight trucks passing over the network, are based on approximations and assumptions (Christenson, 2014). In most cases, historical data is used, while local and recent data is rarely available. Traffic loading is one of the factors that affect bridges as bridges age. Damage caused by heavy traffic results in a higher repair and perhaps a reduced load-carrying capacity

Currently, the development of live load models is being emancipated over several countries. According to Zhou (2013), *"The HL93 is a combination of HS20 truck and lane loads that was developed using the 1975 truck data from the Ontario Ministry of transportation to project a 75-year live load occurrence".* However, these did not show up in the Ethiopia traffic loading context.

Although the use of weigh in motion data collection is more prefer to use, it is not available in our country context, thus using adequate representative sample collected from weighing stations could be used to update the bridge live load models.

Gross-Vehicle mass and Axle spacing, configuration vary from country to country depending on the legal requirements as a result several live load models have been developed in different countries. The traffic load models in Ethiopia are taken from AASHTO standards which are based on old collected traffic data from Ontario (Canada), which is used in the AASHTO Manual as well as the ERA manual.

Therefore, the development of live load models would have a use in the design and assessment of traffic loads, on performance and lifecycle cost of bridges.

1.2 Objectives

The paramount objectives are the following:

- Undertaking literature review to replicate best practices.
- Reviewing the traffic load models based on a representative, traffic loads induced by current vehicles in Ethiopia in relation to the existing load model HL-93 trucks.
- Investigate how the data obtained from the weighing station and field survey could be utilized in appropriate modeling of traffic loadings in bridges.
- To show the application of different statistical tools to make the analysis
- Critical analysis loading situations imposed by traffic, moments and shear forces induced in several types of bridge configurations.
- To develop appropriate software to make simulations on a huge database.

1.3 Scope and limitation of the study

The study explores a development of live load models in the Ethiopian context. In this work both primary and secondary data was found from the ERA from three weighing stations located at Modjo, Semera, and Sululta. These sites are selected considering that they are the major gateway for Ethiopia's import-export logistics. After collection of data methods of influence lines have been used in the calculation of loading effects for bridges spanning from 5 to 50m, and appropriate statistical tools and methods have been applied.

Accurate headway information and trucks bypassing the weigh stations was not included data found from fixed weigh stations, can be considered as a challenge and factors such as dynamic effects, lateral brunching and multilane loading have not been considered.

1.4 METHODOLOGY

The Methodology adopted for the study was done by critically reviewing the HL-93 live load model given in the Ethiopian Bridge design manual. The studied codes comprise of the south African TMH7 the south African code which has got similarity to the British code the American Load Resistance Factor Design (LRFD), "Bridge Design

Specifications" (2002) and, the British code of practice BS5400, "Steel, Concrete and Composite Bridges", CAN\CSA-S6-00, "Canadian Highway Design Code" (2000) , the Eurocode, ENVI991-3:2000, "Basis of design and action on structures - Part 3: Traffic loads on bridges" the Ethiopian Bridge Design Manual could be mentioned as some of the examples of live load models used in the study.

1.5 Analysis of traffic data obtained from the weighing stations

A. Processing of the weighing station data

The traffic data obtained from the weighing stations were organized, filtered from erroneous records. The data were grouped and classified as per axle, and then a field survey was undertaken in the determination of axle configurations. Using the data from the population the maximum loading effects i.e. the moments and shears, caused by the trucks were computed based on simply supported bridge spans ranging from 5m to 50m. A python program was developed for the calculation of the loading effects induced by the moving vehicles. The python program developed was able to generate and calculate out the maximum loading events induced by the vehicles.

B. Statistical Properties

The data obtained from the Modjo, Sululta, and Semera weighing stations was a collection of 35,000 heavy vehicles. For the recorded vehicles their corresponding statistical properties were calculated to understand the nature and distribution of heavy trucks traveling in the rout where the weighing stations are located.

C. Statistical Distributions

As many authors had mentioned live loading is a random variable that is dependent on time, and fitting a probability distribution function to the observed events will help in the prediction of extreme events with a given non-exceedance probability (Castillo et. al., 2004).From the theoretical distribution, the maximum load effects which could occur within a 75 year return period (for spans ranging from 5 to 50m) are extrapolated

In this thesis, two approaches were used for the extrapolation of extreme events. The first method, used was of these was the use of normal distribution in the extrapolation as per Nowak (1991) in the calibration of the LRFD (1994). The second method comprises the application of an extreme distribution. A contrast of the results generated by both methods is presented.

1.6 Structure of the Thesis

The following outline describes the general structure of the thesis: in Chapter 2 an extensive literature research on the current international bridge design codes of practice, historical developments and methodologies was highlights in order to replicate best practices for the formulation of live load models, Besides fundamental statistical and probability concepts that were also explored to develop the live load models to apply.

In Chapter 3, Presents the methods methodologies used in the computation of bridge traffic loadings, and the steps used in the calculation are also enumerated.

Chapter 4 highlights methods of filtration of data in-depth and the detail statistical properties of the sample of 35,000 vehicles that used to be recorded at the weighing stations.

The development of the model is presented in Chapter 5, for simply supported spans ranging from 5m to 50m, the loading effects caused by the trucks collected from weighing stations are calculated. Methods of extrapolation of extreme loading effects were also discussed with a given return period of 75 years.

Chapter 6, highlights a summary, recommendation, and conclusion of the research conducted in this thesis.

2. REVIEW OF LITERATURE

This chapter aims to assess' current codes of practice, as well as methods used on the formulation of live load models that simulate traffic loading on bridge structures and tries to outline the history of their development.

Traffic loads on bridge decks are used to simulate the properties of vehicles and pedestrian loads. Some traffic loads denote the weight of real vehicles that can travel over the bridges; other standards and distributions are chosen in such a way that they produce maximum internal forces in bridge structures similar to the ones produced by actual vehicles. *"Though the loading and spacing of heavy vehicles have changed significantly since the development of the first live loading curves, the basic form of the live load models used by design engineers has stayed relatively unchanged. This is because traffic loading could be simulated with reasonable accuracy by the use of a uniformly distributed load and point loads" (Buckland, 1978).* The axle spacing and the gross vehicle mass (GVM) of heavy vehicles vary from country to country depending on their road traffic regulations. Numerous national and international authorities have the responsibility for drafting design codes to be employed by engineers in designing structures. several codes are presented in this section such as the American Load Resistance Factor Design (LFRD), British Design Codes For Highway Bridges, The Euro code ENV1991-3:2000, the South African Code THM7, The ERA code 2013 is also used. Although the selection is partial, it tries to give fair and unbiased models of several countries with variation gross vehicle mass (GVM) and axle spacing depending on their countries legal requirements.

A highway bridge is a structure that carries a highway over an obstruction (Mark Hurt, Steven D.Schruck, 2016), and for the structure to remain functional one of the main requirements is the load-carrying capacity must exceed the demand. The loading which is encountered in the bridge structure is the determining factor for the safety and serviceability of the structure as a result the service life and bridge structure could be affected by heavy vehicle loads. The load effects, such as moments and shears that result due to the loading cause higher damage rather than Gross Vehicle Weights. (Eugene J. O'Brien, Andrzej S. Nowak, Anjan Ramesh Babu, 2018)

2.1 Bridge Live Load Models

A. South Africa Model

South Africa's Bridge design code, TMH7, was first issued in 1981. In the derivation of the code in 1978, Liebenberg noted that a study of the extreme truck event was not viable due to a lack of statistical information at that time (Anderson 2006). In 1988, limitations with regards to the live load model for normal traffic conditions on narrow and short span bridges were identified (Ullmann,1988).Due to those reasons in 1991 it was proposed to increase the axle load from 144kN by 25% to a value of 180kN, but the code was not amended (Oosthuizen et al,1991). In the recent code, WIM data was used to derive an imposed live load model for bridge design in South Africa. According to Van der Spuy (2014) South Africa's bridge design codes have historically been based on British Codes.

A.1. TMH7 LOAD MODELS

A.1.1 NA loading

NA loading is normal traffic loading which consists of a distributed component (Figure 2-1) and an axle load in each lane. A dynamic allowance is included according to the Swiss formula (CSRA 1981). The distributed component of NA loading, Q_{uist}, is 36 kN/m for the first 36 m loaded length and then reduces according to Equation (2.1) below.

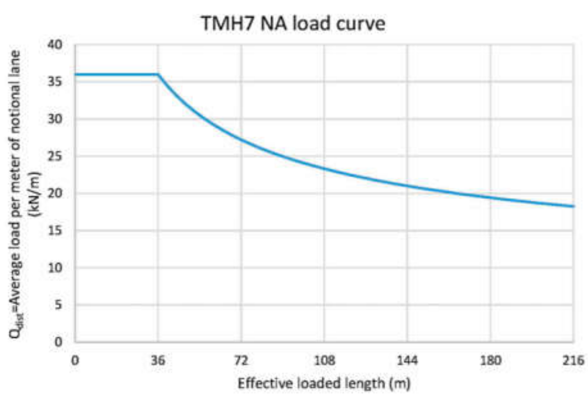


Figure 2-1: NA Distributed Load

$$Q_{dist} = \frac{180}{\sqrt{L}} + 6 \qquad \text{Equation (2.1)}$$

Where L=total aggregate loaded length.

The axle load component of NA loading, Q_{axle}, is given by Equation (2.2) in kN

$$Q_{axle} = \frac{144}{\sqrt{n}} \qquad \text{Equation (2.2)}$$

Where n = the number of the notional lane in which the axle load occurs. The axle load, therefore, decreases as the number of loaded notional lanes increases.

A.1.2 NB loading

Nominal NB loading is a unit loading representing a single abnormally heavy vehicle. Thirty-six units of NB loading are typically applied and referred to as NB36. This corresponds to a wheel load of 90 kN, an axle weight of 360 kN, and a total weight of 1440 kN. The NB load configuration is shown in Figure 2-2

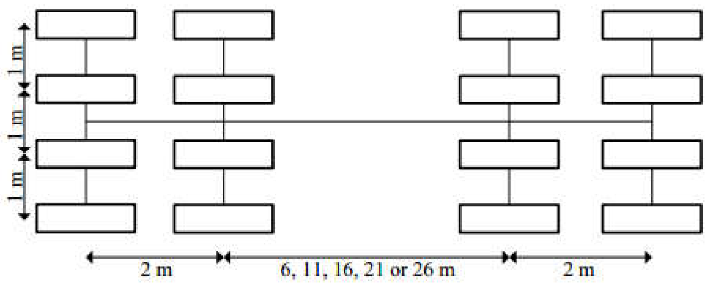


Figure 2-2: NB load configuration

Only one NB vehicle is allowed on a bridge at a time without any other traffic loading acting in conjunction. No allowance is made for impact.

A.1.3. NC loading

Nominal NC loading is a loading representing multi-wheeled trailer combinations with controlled hydraulic suspension and steering intended to transport very heavy indivisible payloads. The applied loading is 30 kPa with a configuration as shown in Figure 2-3

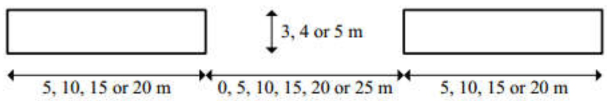


Figure 2-3: NC load configuration

NC vehicles shall move along the centerline of a bridge only with an allowance of 1 m either side for moving offline. No allowance is made for impact.

B. Bridge Live Load Models in North America and Europe

In the USA, the design service life of a bridge is 75 years as specified by AASTHO LRFD (Nowak 1999, AASHTO LRFD 2016). In Europe, the default service life of bridges is 100 years as dictated by the Euro code although some variation is allowed in the National Annexes

B.1 The AASHTO BRIDGE LIVE LOAD MODEL

The development for the live load model In North America, started in the 1913's by the US Department of Agriculture, it used the provision for 15-ton road roller loading for the computation of live load stress. With the substantial growth of technology and industrialization, the trucking industry was introducing new axle configurations to maintain the heavy gross truck weight these changes demanded new live load specification for the design of highway bridges. In 1924, the first live loads were based on 10 and 15-ton trucks were followed by a 20-ton truck, these trucks were named as H10, H15, and H20 trucks respectively (the H- stands for highway and the number following specifies the weight intones). Thus, the first notational model (trucks that do not represent physical configuration or similarity to any particular type of truck) for the highway bridge design model was developed.

The variations in the existing load model, H15 loading and H20 loading, were imposed by the loads imposed by heavy tanks and the need for superior roadway width to permit military convoys over the highways without interference with the normal traffic (ENR 1941, ASCE 1958). In the 1940s, this led to the introduction of H20-S16 loading, a system of truck and lane loading which functioned as the live load model for AASHTO Standard Specifications (AASHTO 2002). The load description, H stands for highway and S for semitrailer.

These live load models were collected and first published in the 1944 AASHO Standard Specifications and came to be known as H15-44, H20-44, HS15-44, and HS20-44 loadings characterized as follows: For many years in the loading of the HS20 truck and semi-trailer combination was included as shown in Figure 2-4

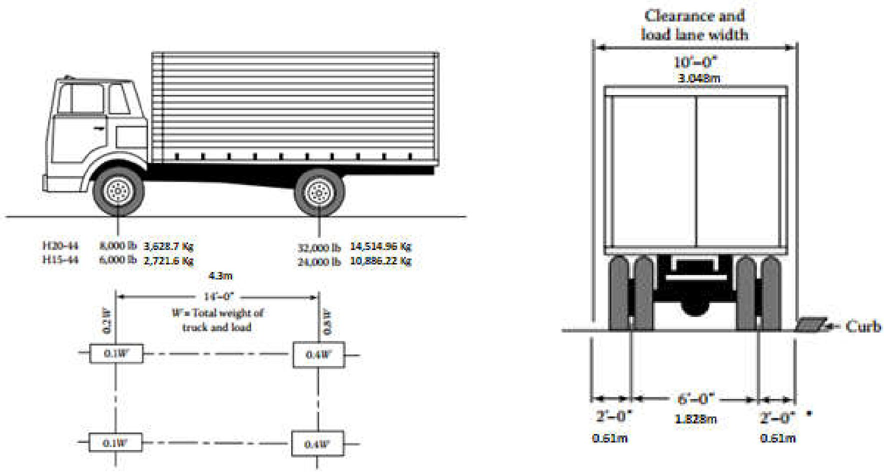


Figure 2-4: AASHTO H15 and H20 trucks. Axle loads of H15 truck are three-fourths of HS 20 truck

Then, the second notational model (ideal model), currently in use was formulated in the 1990s. The AASHTO LFRD notational model for American Highway Bridges, labeled as HL-93, (HL meaning highway loading and 93 indicating the year 1993) (AASHTO 2002). The HL-93 load model is the calibration of the AASHTO LFRD specifications, is based on the top 20% of the trucks in an Ontario truck weight database assembled in 1975. Nowark & Hong (1991) have used truck measurements, which consist of 9,250 trucks representing two weeks of traffic collected in 1975 in Ontario, to develop a probability-based live load model for bridge design.

The HL-93 design live load consists of a combination load placed contemporarily in each design lane as follows; the more critical of these two load combinations should be chosen as the basis for design:

a. The design truck (concentrated loads, Figure 2-5) combined with the design lane load (uniform load, Figure2-5).
b. The design tandem (concentrated loads, Figure 2-5) and the design lane load (uniform load, Figure 2-5). The tandem load, also known as the alternate military loading, is specified to simulate military loading and typically governs the design of spans approximately shorter than 12m.

Tandem loading is defined as two closely spaced axles, usually connected to the same undercarriage, by which the equalization of load between the axles is enhanced.

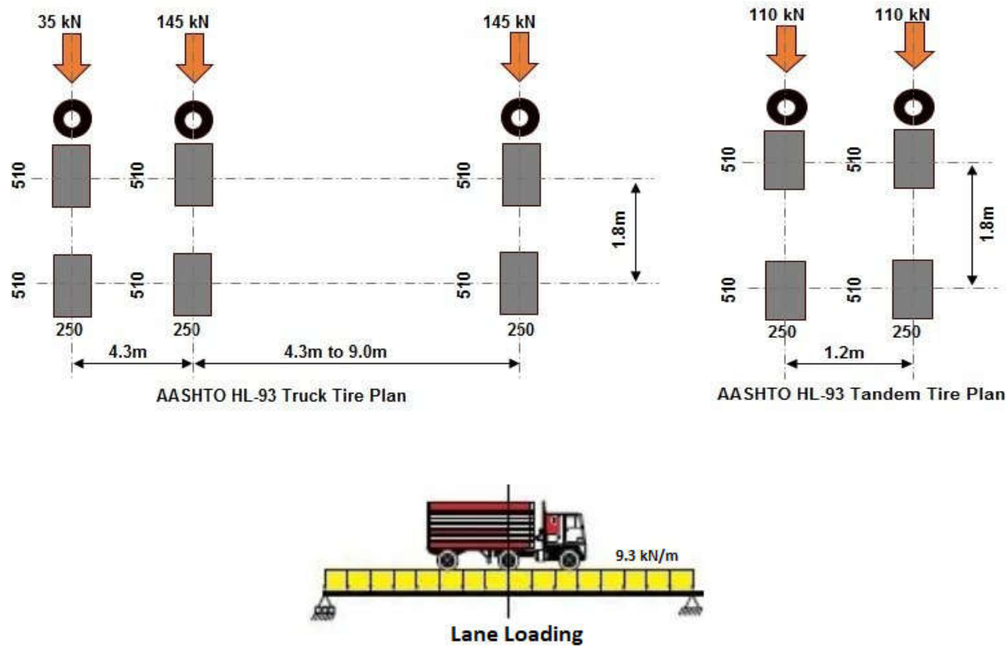


Figure 2-5: AASHTO HL-93 Truck and Tandem Loadings

B.2 British Codes for Highway Bridges.

The first vehicle loading for highway bridges in Great Britain was the ministry of transport standard loading train, in 1922 which incorporated a tractor and four trailers with a major axle load of 219 kN, followed by a succession of 100 kN axles, and included an impact value of 50% as shown in Figure 2.6. Traffic live load requirements were introduced in British Standards in BS153 part3 in 1923, which were later revised in 1925 and 1937.

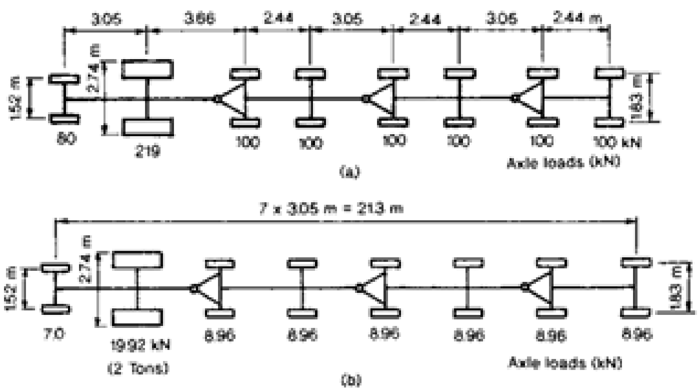


Figure 2-6 (a): Ministry of transport loading train (1922); (b) BS 153 unit loading train (1923)

In the early stages, the British Codes were developed based on allowable working stress design. The given loads were quite heavy relatively from some other countries. In the discussion of Henderson's paper (1954:355–9), it was mentioned that the standard American AASHO loading, had effects that were equal to about two-thirds of the British loading over the range of spans. Before Henderson also, in 1933, Chettoe mentioned, 'The average intensity of traffic in Great Britain is heavier than in any other country in the world, regarding loading intensity.'

The first British limit states design code was BS 5400, which was issued in 1978. As the preceding code BS 153, is composed of normal traffic and an abnormal vehicle load, which are identified as HA and HB loading. HA represents a normal traffic loading in the United Kingdom, while HB loading presents the abnormal vehicle unit loading.

As per BS 5400 [2006], the highway loading consists of HA and HB loading in which both loadings include impact. The structure as well as its elements are required to resist more severe effects induced by design HA loading, or design HB 45 units loading, or design HA loading combined with design HB 30 units loading.

B.2.1. HA loading contains:

- uniformly distributed load (HA-UDL) and a knife-edge load (HA-KEL) in combination, or
- A single-wheel load

Value of Nominal uniformly distributed load (UDL)

- For loaded length up to and including 50m. The UDL expressed in kN per linear meter of notional lane shall be derived from the equation:

$$w = 336 \times \left(\frac{1}{L}\right)^{0.67} \qquad \text{Equation (2.3)}$$

- For loaded length above 50m but less than 1600m, the UDL shall be:

$$w = 36 \times \left(\frac{1}{L}\right)^{0.1} \qquad \text{Equation (2.4)}$$

Nominal Knife Edge Load (KEL)

- The knife-edge load per lane shall be taken as 120 kN

Single nominal wheel load alternative to UDL and KEL

- One 100kN wheel placed on the carriageway and uniformly distributed over a circular contact area assuming an effective pressure of $1.1\frac{N}{mm^2}$shall be considered.

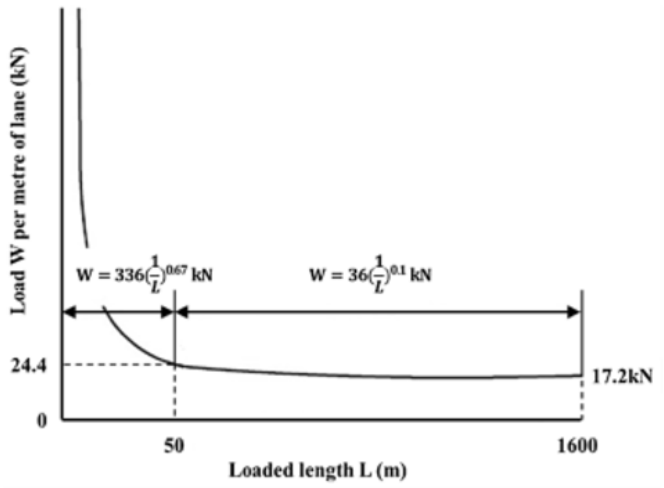


Figure 2-7: BS 5400 Live Loading curve HA UDL (2006)

B.2.2 HB Loading

As shown BD 37/01, exceptional industrial loads such as generators, electrical transformers, machine presses, pressure vessels, etc. were addressed by using HB loading (BD 37/01 Appendix A).

The vehicle loading is represented by a four-axle vehicle with four wheels equally spaced on each axle. The load on each axle is defined by several units which are dependent on the class of road generally considering for public highway bridges, the number of units of type HB loading that shall be considered are:

- 30 when acting together with HA (HB 30) – lane share between normal vehicle & truck
- 45 when HB alone (HB 45) – truck only

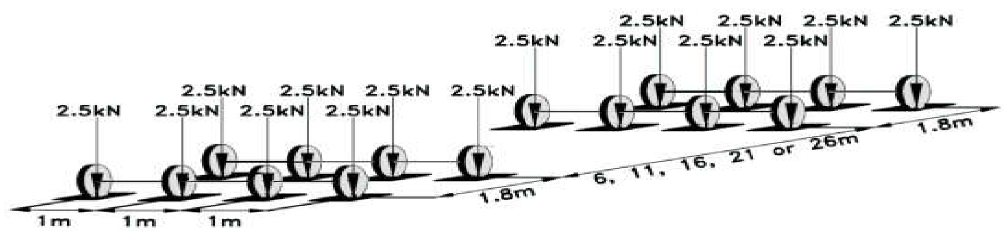


Figure 2-8: Dimensions of HB loading.

B.3. Ontario and Canadian Highway Bridge Design Codes

By the end of 1975, the Ontario Ministry of transportation and communication started the development of the code for Ontario's highway bridges. In 1979 the OHBDC was first published. The Ontario bridge design code is based on the existing AASHTO specification, what makes this code unique is that it eliminated the use of the working stress design method and introduced an important concept which is the use of the Limit State Design Method (Dorton and Csagoly). The Ontario Highway Bridge Design Code was suppressed by the CAN\CSA-S6-00, "Canadian Highway Bridge Design Code" (2000). The main difference between the two codes is that CSA-S6-00's live load represents actual vehicle loads permitted on Canadian highways, while the OHDBC truck came from the Maximum observed load (MOL) in Ontario.

B.3.1. OHBDC (1991)

The Ontario Highway Bridge Design Code (Third edition) models the live load based on two aspects as a truck or a combination of truck and lane load, whichever produces the maximum load effect:

1. The OHBD Truck, which is of a 5-axle truck as seen in Figure 2.9
2. The OHBD Lane Load consists of an OHBD Truck with each axle reduced to 70% and superimposed centrally within the width of a 3.0 m wide uniformly distributed load of 10.0 $\frac{kN}{m}$ as seen in Figure 2.10.

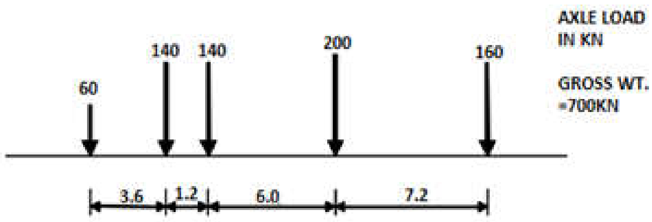


Figure 2-9 (a): OHBDC Live loading (1991).OHBD Truck

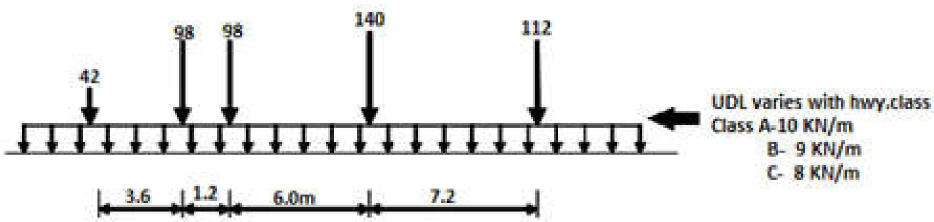


Figure 2-9 (b): OHBD Live Loading (1991). OHBD Truck and Lane Load.

B.3.2. CAN/CSA-S6-00 (2000)

Canadian Highway Bridge Design Code uses CL-W loading comprises of the truck or the lane loading which is used to represent loading from lighter traffic:

1. The CL-W is a 5-axle truck. The letter "W" denotes the gross vehicle weight of the truck in kN. In the designing of national highway networks, loading CL-625 is used to simulate effects induced by heavy vehicles. Figure 2-10.
2. The CL-W Lane Load consists of CL-W Truck with each axle reduced to 80%, and a superimposed with a uniformly distributed lane load of $9.0\frac{kN}{m}$. As shown in Figure 2-11.

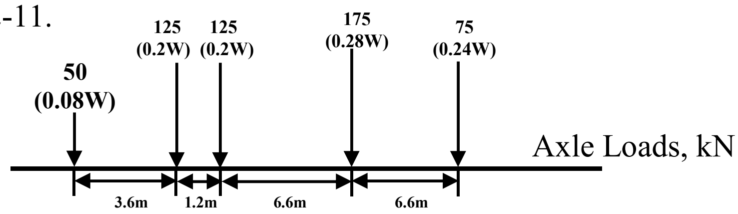


Figure 2-10: CAN/CSA-S6-00 Live Loading [2000].CL-W Truck

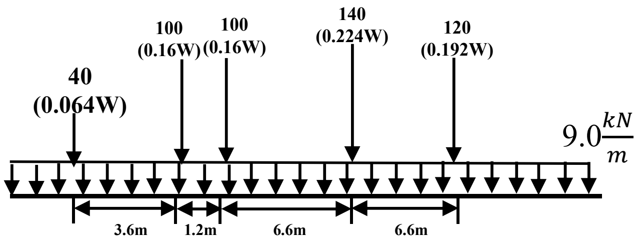


Figure 2-11: CAN/CSA-S6-00 Live loading [2000].CL-W Lane Load

B.4. Eurocode 1 (2002)

Recent bridge codes such as Eurocode and OHBDC codes specifically have attempted to move in the direction of a more lucid statistical approach on the prediction of lifetime maximum imposed loads. They are renowned that the characteristics of vehicles vary extensively to their gross vehicle weight, corresponding axle spacing, and distribution of load to axles, lane location, speed, and the probability of multiple presences of vehicles on the structure.

The Eurocode 1 Part 2 is used for the design of bridges that span from 5m up to 200m, and carriageway width spanning to 42 m. In the Eurocode four different load models are presented:

1. **Load Model 1 (LM1)** consists of concentrated and uniformly distributed loads (Figure 2-12) which cover most of the effects of the traffic of trucks and cars. The code specifies live load to be used on each traffic lane, therefore there is no need to introduce multilane factors. It is used for general and local verifications.

Table 2-1: Characteristic values of load for successive road lanes

Position	Axle Load Q_{ik}	UDL Q_{ik}
Lane number 1	300KN	9KN/m^2
Lane number 2	200KN	2.5KN/m^2
Lane number 3	100KN	2.5KN/m^2
Other lanes	0	2.5KN/m^2
Remaining area	0	2.5KN/m^2

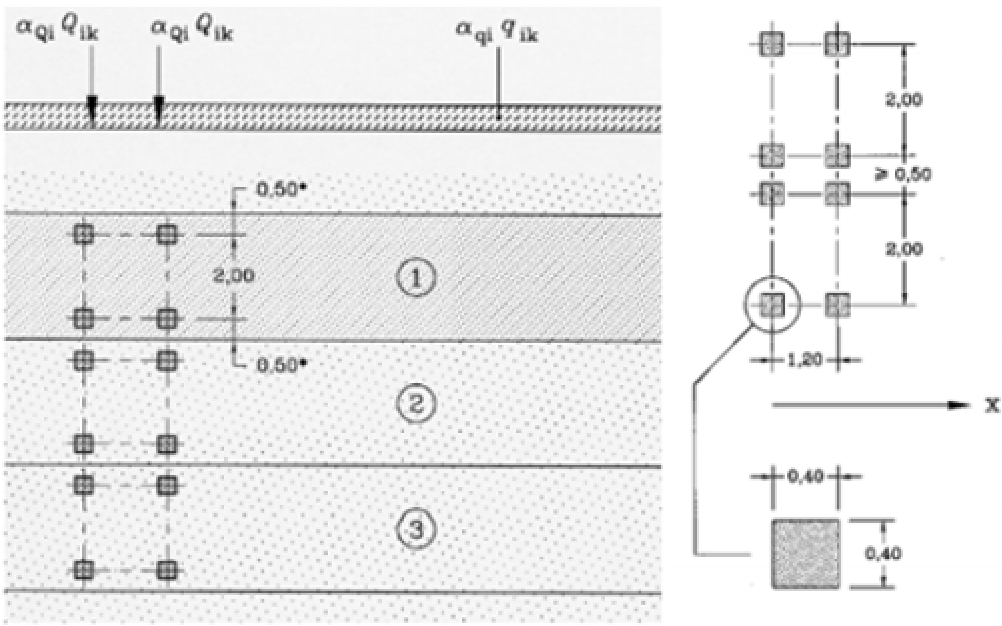


Figure 2-12: Eurocode 1 [2002]. Load Model 1

2. **Load Model 2 (LM2)** comprises of a single axle load of 400 kN, which covers the dynamic effects of the normal traffic on short structural members. The distance between wheels is 2 m. The contact surface of each wheel should be taken as a rectangle of sides 0.35 m and 0.60 m. When relevant, only one wheel of 200 kN may be taken into account.
3. **Load Model 3 (LM3)** consists of sets of axle loads representing special (carrying heavy loads) vehicles, which can travel on routes permitted for abnormal loads. And are considered only when requested in a transient design situation. The axle

loads as well as the geometry of these special vehicles to be considered are assigned by the bridge owner.

4. **Load Model 4 (LM4)** represents a crowded loading of $5.0\frac{kN}{m^2}$. The main intention is only for general verifications and it is mainly applicable for bridges in or located near town areas.

C. Ethiopian Vehicular Live Loads

The Ethiopian Roads Authority (ERA) has adopted the use of AASHTO by adopting the specification of our local condition for highway bridge design. Vehicular live loading on the roadways of bridges structures designated HL-93, and shall consist of a combination of the:

- Design truck or design tandem, and
- Design lane load as discussed and highlighted in section B.1 above

2.2 Probability Theory

As the name implies the word 'probability' or 'chance' is a frequently used word in a day to day conversations and generally, people have a rough idea about its meaning. The probability of a given event is an expression of the possibility of occurrence of an event. An event is the outcome of an experiment while an experiment is a process of making observations and taking measurements. And thus, a probability of an event is a number which ranges from 0(zero) to 1(one) - zero for an event which may no chance of occurrence and a value of 1 for an event which certainly occurs. From the definition of classical probability theory P (A) is given by:

$$P(A) = \frac{\text{Number of favorable cases}}{\text{the total number of equally likely cases}} \qquad \text{Equation (2.5)}$$

A. Importance of The concept of probability

Since the creation of the term probability in the seventieth century, probability theory has advanced, evolved, and employed to treat and solve many substantial problems. It serves as a foundation for classical decision processes estimating as well as testing. Probability models can and have been very important tools for making predictions. We shall consider that the concepts of probability are not only for many scientific investigations but also for many problems that are encountered in every day to day life.

Highlighting the significance of probability theory, Ya-Lun Chou has strongly pointed out that in statistics, as a method of decision making under uncertainty, is founded on probability theory, since the probability is the measure of uncertainty and the risks in association with it. In dealing with problems of probability, there are chances that various events will occur, based on an assumed model, whereas in dealing with the problems in statistics observed data is available and if one wishes to determine a model which can be used to describe the data (Hahn & Shapiro 1967). Both of these situations are observed in engineering problems. The uncertainty in the problem must be combined into the analysis, and this can be achieved through the use of Theory of Probability, which is a branch of mathematics dealing with uncertainties.

B. Random Variables and Distributions

A random variable is a variable in which the numerical quantity is subject to variations in value determined from the outcome of a random (chance) Experiment.

Two types of random variables are given in statistics: discrete and continuous. If the set of values defined over the sample space is finite or countable infinity we call it a discrete random variable. The probability function of a discrete random variable is known as the probability mass function, $P_X(x_i)$, and the distribution as 'discrete probability distribution. *"Alternatively, if it can assume any real value in an interval, then the random variable is said to be a continuous random variable and it can take on a continuous range of values, and its probability function is known as the probability density function (PDF), f X (x) and the resulting distribution as 'continuous probability distribution" (Gupta 2000).*

B.1 Cumulative distribution function

The cumulative distribution function (CDF) of a continuous random variable X with probability density function f(x) is denoted by F(x) and defined by:

- Cumulate the pdf to produce a CDF
- Cdf describes the probability that a random variable is less than or equal to the specified value of x

$$F(x) = \Pr(X \le x) = \int_{-\infty}^{x} f(t)dt \qquad \text{Equation (2.6)}$$

In this case;

1. $F(x)$ is defined for all $x \in R$ and $0 \le F(x) \le 1$
2. $F(x)$ is non decreasing function of x.
3. $\lim_{x \to -\infty} F(x) = 0$ and $\lim_{x \to \infty} F(x) = 1$.
4. $\dfrac{dF(x)}{dx} = f(x)$
5. $\Pr(a \le X \le b) = F(b) - F(a)$

B.2 Expected values (Mean) and variances

Let X be a discrete or continuous random variable. If f(x) is the pdf of X, then the mean and variance of X are given by:

$$\text{Mean} = \mu = E[X] = \begin{cases} \sum_i x_i f(x_i) & \text{,for a discrete distribution} \\ \int_{R_X} x f(x) dx & \text{,for a continuous distribution} \end{cases} \quad \text{Equation (2.7)}$$

$$Var(X) = E[(X-\mu)^2] = \begin{cases} \sum_i (x_i-\mu)^2 f(x_i) & \text{, for a discrete distribution} \\ \int_{R_X} (x-\mu)^2 f(x) dx & \text{, for a continuous distribution} \end{cases} \quad \text{Equation (2.8)}$$

B.3 Standard Deviation

The standard deviation, denoted by σ**,** was first introduced by Karl Pearson in 1893 (Gupta). It is the most widely used measure of studying dispersion. σ measures the spread of X about x = μ**.**

$$\sigma = \sqrt{Var(X)} \quad \text{Equation (2.9)}$$

B.4 Coefficient of Variation

The standard deviation is an absolute measure of dispersion. The corresponding relative measure is known as the coefficient of variation developed by Karl Pearson (Gupta). The coefficient of variation (*V)* is a dimensionless quantity defined as:

$$\text{Coefficient of Variation or C.V} = \frac{\sigma}{\overline{X}} \times 100 \quad \text{Equation (2.10)}$$

C. Probability distributions

There are several kinds of discrete and continuous distributions. The most commonly used are continuous distributions: uniform, normal, lognormal, exponential, and gamma.

In this segment, several distributions are presented, because they were found to be relevant in this paper. Additional information about the distributions can be found in statistical books, for instance, in Nowak and Collins (2000).

C.1. The Normal Distribution

The normal or bell-shaped distribution is used in most cases where aggregation of values over the smoothened process. The normal distribution has a significant role as an approximation to certain discrete and continuous distributions but is also useful as an approximate density curve for certain populations.

The normal curve that approximates a population histogram is indexed by the mean and standard deviation of the population, labeled μ and σ, respectively.

Note that normal curves are centered at μ and the spread of the distribution is controlled by the size of σ (i.e. larger σ implies less probability near μ).

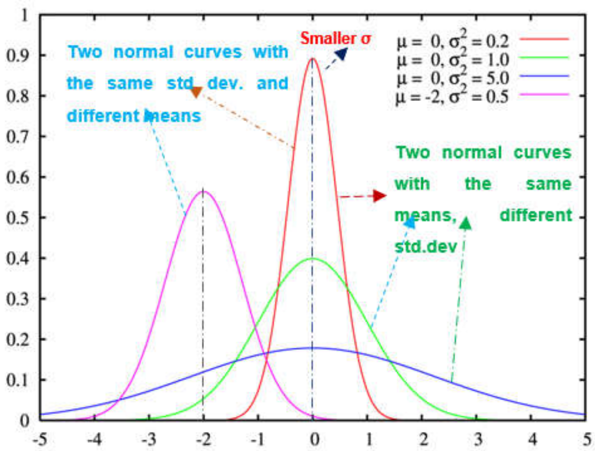


Figure 2-13: PDF of Normal distribution for various mean values and constant standard deviation

The normal curve can be presented in several forms. The following is the basic form relating the curve with the mean μ and standard deviation σ (Montgomery, Runger 2003).

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2} \qquad -\infty < x < +\infty \qquad \text{Equation (2.11)}$$

The figure below tries to present some of the most important areas in approximation.

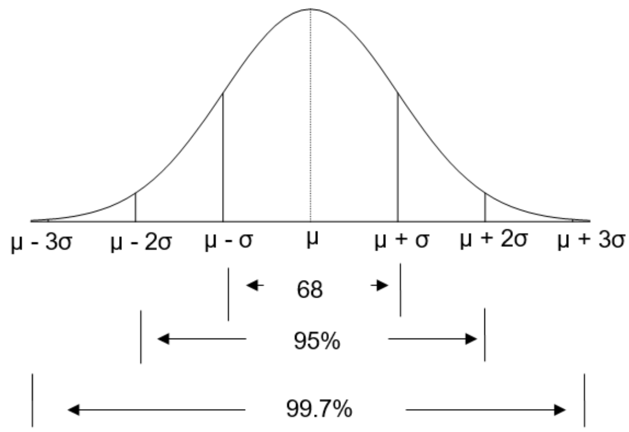


Figure 2-14: Areas of approximation in Normal Distribution.

C.2 Log-Normal Distribution

A Random variable is a log-normally distributed when y= ln(x) the logarithm of the sample is normally distributed (Saporta, 1990). The probability density function (PDF) of the lognormal distribution is given by:

$$f(x) = \frac{1}{\sqrt{2\pi}\sigma_x} e^{\frac{-.(\ln x - \mu_x)^2}{2\sigma_x^2}} \quad 0 < x < \infty; 0 < \mu < \infty; \sigma_x > 0 \qquad \text{Equation (2.12)}$$

Examples of lognormal distributions are presented in Figure 2-15

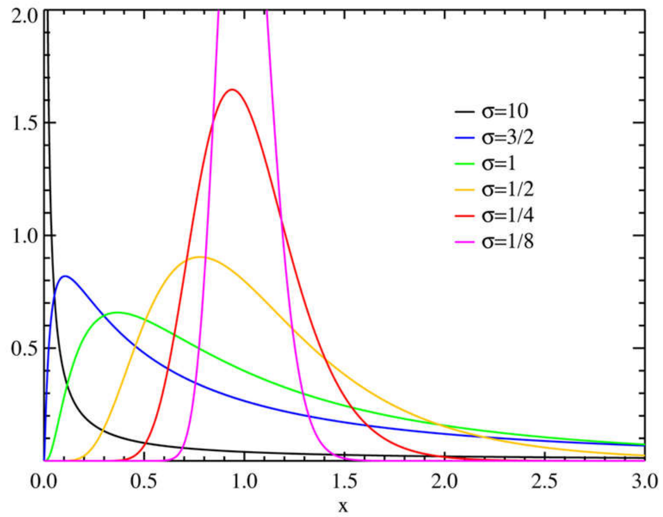


Figure 2-15: PDF of Log-normal distribution for various mean values and constant standard deviation

C.3. The Gamma Distribution

Gama Distribution is a continuous distribution. Similarly to the lognormal distribution, the gamma distribution is also unbounded on the right, defined for only positive X, and tends to yield skewed distributions (Drapela 2010)

The probability density function of the Gamma distribution is given by:

$$f(x;\alpha,\beta) = \frac{\lambda^{\eta} x^{\eta-1} e^{-\lambda x}}{\Gamma(\eta)} \quad , \quad x,\lambda,\eta > 0 \qquad \text{Equation (2.13)}$$

Two parameters $\lambda \& \eta$

Where $\Gamma(\eta)$ is the gamma function $\int_0^\infty t^{\eta-1} e^{-t}\, dt, \eta > 0$.

When η is a positive integer,

$$\Gamma(\eta) = (\eta - 1)! \qquad \text{Equation (2.14)}$$

The exponential distribution is a special case of gamma distribution with η=1

- λ is the Scale parameter
- η is the Shape parameter

The mean, variance, skewness is given by the following formula Mean = $\frac{\eta}{\lambda}$

Variance = $\frac{\eta}{\lambda^2} \rightarrow \sigma = \frac{\sqrt{\eta}}{\lambda}$, Skewness coefficient $\gamma = \frac{2}{\sqrt{\eta}}$

Gama distribution is a group of distribution, or it can be said it is a family of distributions and is made possible by varying the parameters λ and η.

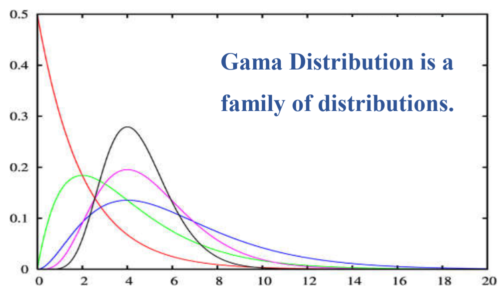


Figure 2-16: PDF of Gama distribution for various λ and η values.

C.4. Extreme value theory

According to Friederichs (2007) the extreme value theory focuses on the extreme and rare events encountered. Extreme Value Theory states that the maximum of independent identically distributed random variables which are drawn from a parent distribution converges to one of the three possible tailed distributions, depending on the shape parameter ξ of the parent distribution: type I Gumbel distribution $\xi = 0$, type II Frechet distribution $\xi > 0$, or type III reversed Weibull distribution $\xi < 0$. The extreme value theory is dependent on the Fisher-Tippett theorem. What we are doing is that we are essentially fitting the distributions to the extreme values and then talking about the probability of a random variable X being greater than X_T . to characterize the data of extreme values two methods are available the block maxima and the POT (the peak over threshold) in the Block Maxima method we take the time and cluster the data in blocks, then the maximum values from the blocks are taken for further analysis. In the Block Maxima method, there is a risk that important data is discarded. Whereas when using the POT method a series of data are selected so that their magnitude is greater than the selected threshold value. An important disadvantage of the POT approach is the issue concerning selecting the threshold value.

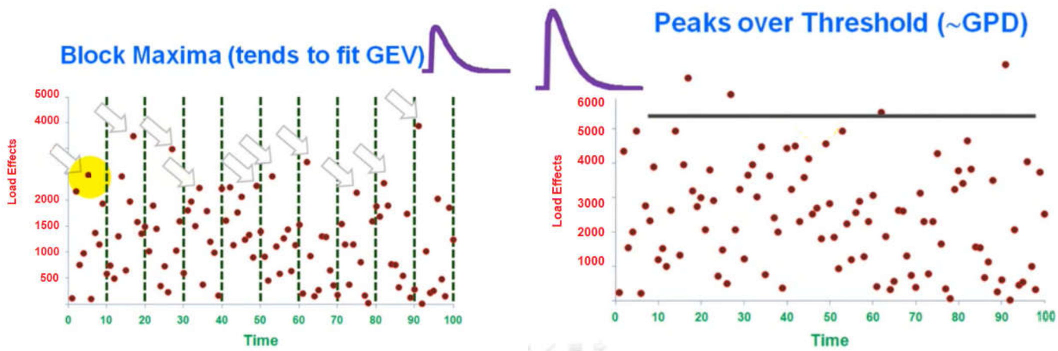


Figure 2-17: Illustration of the Block Maxima and the POT methods

The three families of the Extreme Value Distributions are:

- **Gumbel:** $G(z) = \exp\left\{-\exp\left[-\left(\frac{X-\mu}{\sigma}\right)\right]\right\}$ $-\infty < x < \infty$ Equation (2.15)
- **Fréchet:** $G(z) = -\exp\left\{-\left(\frac{x-\mu}{\sigma}\right)^{-\alpha}\right\}$ for $x>\mu$ Equation (2.16)
- **Weibull:** $G(z) = -\exp\left\{-\left(\frac{x-\mu}{\sigma}\right)^{-\alpha}\right\}$ for $x<\mu$ Equation (2.17)

For parameters μ,σ>0 and in case of typeII and typeIII and $\xi > 0$

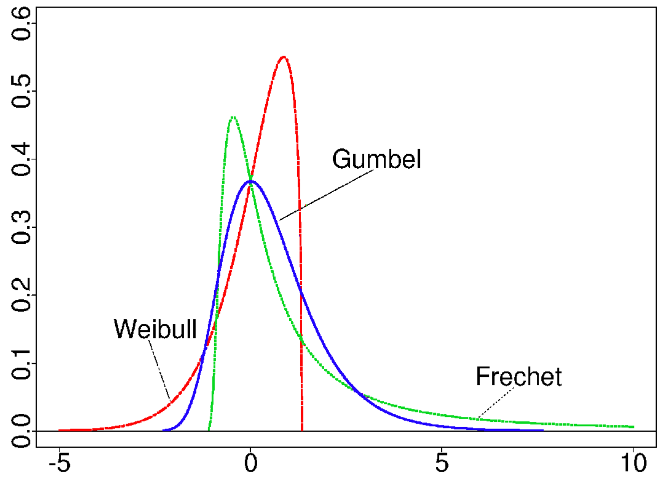


Figure 2-18: Three types of GEV distributions

Cole (2001) simplified the three separate equations by combining the three models into a single-family and came up with the following distribution function:

$$G(z) = \exp\left\{-\left[1+\xi\left(\frac{z-\mu}{\sigma}\right)\right]^{-1/\xi}\right\} \qquad \text{Equation (2.18)}$$

The equation above is known as the Generalized Extreme Value (GEV) family of distributions incorporating three parameters: μ the location parameter, σ the scale parameter, and ξ the shape parameter.

Different values of ξ lead to the three extreme value distributions as follows:

$\xi > 0$ corresponds to a Fréchet distribution;

$\xi < 0$ corresponds to a Weibull distribution;

$\xi \rightarrow 0$ corresponds to a Gumbel distribution.

This equation eases statistical application by the removal of the uncertainty involved in making a prior judgment as to which family to adopt. The parameter ξ defines the most suitable type of tail behavior.

The domain of attraction of the most common Distributions are given by Castillo et.al. 2004 the table is shown below.

Distribution	Domain Attraction	
Distribution	Maximum	Minimum
Exponential	Gumbel	Weibull
Lognormal	Gumbel	Gumbel
Gamma	Gumbel	Weibull
Gumbel	Gumbel	Gumbel
Uniform	Weibull	Weibull
Pareto	Fréchet	Weibull
Weibull	Weibull	Gumbel
Fréchet	Fréchet	Gumbel

Table 2-2: Domains of attraction (Castillo, 2004)

D. Return Period

Let's consider a random variable X and let X_T be the threshold value when an event x is greater than X_T at any time when this event occurs it is called an extreme event. The elapsed time between two events $X>X_T$ is called the recurrence interval. $E(\tau)$ is the return period 'T' of the event $x \geq x_T$. The probability of occurrence of an event in any observation is the inverse of its return period.

$$P\,(X \geq X_T) = \frac{1}{T} \qquad \text{Equation (2.19)}$$

Therefore the T year return period event is $X \geq X_T$ and it occurs on an average once in T years.

E. Probability paper

The probability plot is a graphical method for assessing whether or not a data set follows a given distribution. It is the simplest way to find the distribution type. For different distributions, there are different probability papers and if the data plotted is a straight line, it fits that distribution. Several empirical methods are used in the determination of plotting positions. There are two procedures for plotting in Normal Probability Paper given in NHCRP059 known as the manual and computer procedures which are discussed in the following sections

Manual Procedure for plotting on Probability Paper:

Step 1

- Collect the data to be plotted
- X_1, X_2, X_3…………………………………………………………x_n

Step 2

- Rank Xi's, from the minimum to the maximum value. the rearranged Xi's, are represented by
- Y1, Y2,…………………………………., Yn
- so that,
- $Y_1 \leq Y_2 \leq Y_3$……………………………….$\leq Y_n$

Step 3

- For each Yi, calculate the probability corresponding to i,
  - $P_i = \frac{i}{n+1}$ Weibull's formula, the most commonly used formula
  - Plot Yi vs. p_i, for i: 1,………………n

Computer Procedure:

Step 1

- Prepare the file with original data to be plotted
  - X_1, X_2, X_3…………………………………………………………x_n

Step 2

- Use the rank command to rearrange X_i's, from the minimum to the maximum value. The rearranged X_i's is represented by
- Y1, Y2,…………………………………., Yn
- so that,
- Y1≤Y2≤Y3……………………………….≤Yn

Step3

- For each Yi, calculate the probability corresponding to i,
- $P_i = \frac{i}{n+1}$

Step 4

- For each pi calculate the corresponding value of the inverse standard normal function
  - $F_i^{-1} = \phi^{-1}(P_i)$

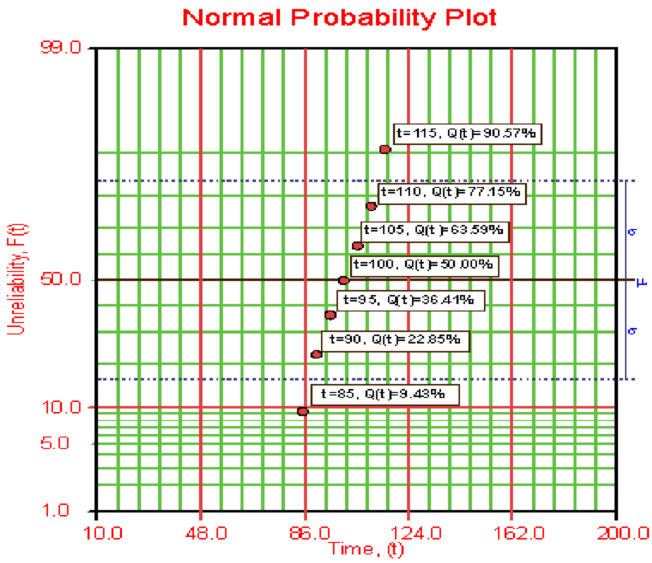


Figure 2-19: Example of data plotted on normal probability paper

2.3 Extreme Values in Bridge Traffic Load Effects

Applications of extreme values has come up these days a due attention in the development of live load model used for bridge design and evaluations.

Statistical methods were introduced to model traffic load effects on bridges including historical methods (Nowak and Hong, 1991) and modern methods that are based on extreme value theory (Bailey and Bez,1999; O'Brien et al., 2003, 1995; Siegert et al., 2008). A review of those methods used in the modeling of extreme bridge loading effects are given below:

2.3.1 Tail Fitting of Loading Events

In working with the current research, several methods of extrapolation of extreme events were studied and compared several works of literature name it as tail fitting. The first of those methods were developed by Nowark in 1991 in the calibration of the live load model for AASHTO LRFD code, (Nowak, 1993; Nowak and Hong, 1991; Nowak et al., 1993). Nowark assumed that Normal Distribution best fitted the Load Effects such as the shear forces and moments induced from the populations of trucks which were identified as being overloaded. They plotted the empirical CDF on normal probability paper, in fitting the upper tail they calculated maximal bending moment and shear force i.e., by fitting the data to a Normal distribution and then applies extrapolation to find the characteristic maximum. The calculations were carried out for different span bridge length varying up to 60m from simple span and two-span continuous bridges. The CDF functions were plotted on the normal probability paper as shown in Figure 2-20.

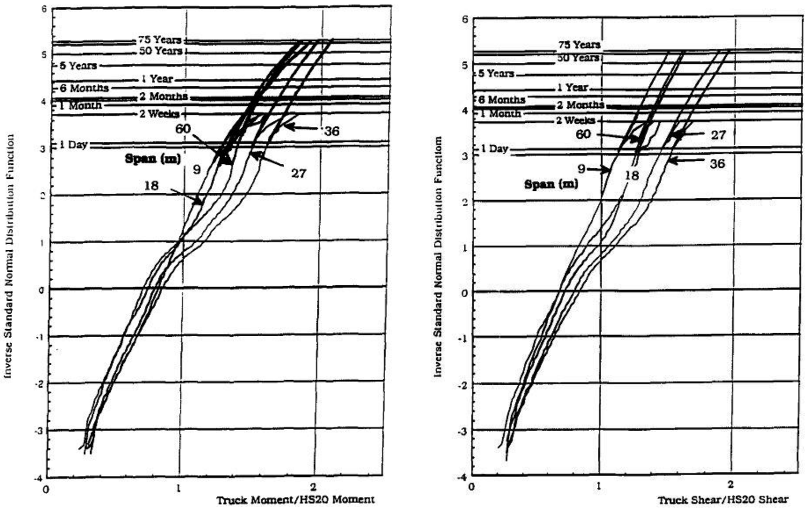


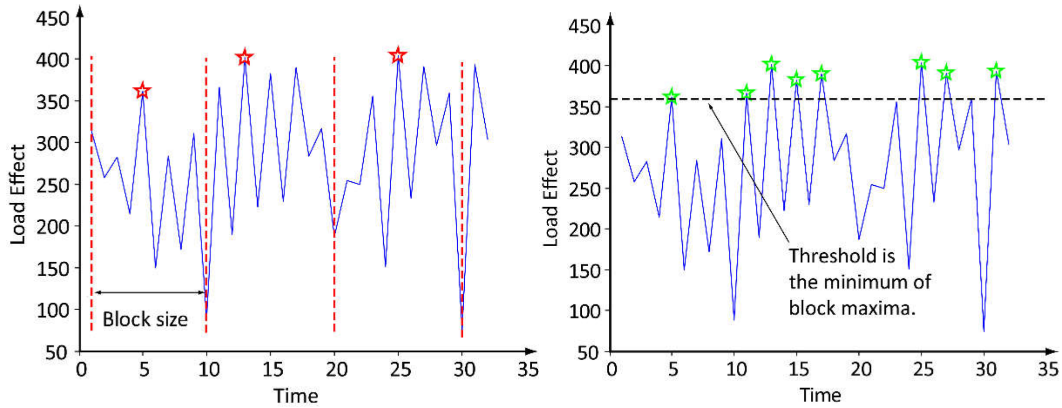
Figure 2-20: CDF of Moment and Shears for different Spans

Other researchers have mentioned that the tail could be selected by engineering judgment when the CDF changes at a particular probability level (OBrien, 2015). Some authors like Castillo have fitted the top $2\sqrt{n}$ of distribution of n data, built on theoretical consideration (Castillo, 1988), and some researchers like Enright based on sensitivity analyses have fitted the top 30% of data(Enright, 2010).

Two of the tail fitting methods which are popular when using the extreme value theory are the POT and Block Maximum methods.

Peaks-Over-Threshold considers a series of data in which the random variable x greater than x_t the threshold value. Whereas in considering the Block Maximum approach, only the maximum loading events occurring in the given blocks of time are usually used. This method gives the advantage in referencing the time of the data which is essential in calculating lifetime maximum probabilities of exceedance.

The Block Maximum approach and The POT approach are widely used and applied in finding the extremes .although the methods are found to be useful, the selection of the block of time which is considered in the block maximum method and the selection of the threshold value in the POT approach below which Loading Events are discarded, are found to be subjective.



(a) Block Maxima **(b) Peak over Threshold**

Figure 2-21: Extreme Value Modelling Methods the Block Maxima and the POT

In this study, three methods of extrapolation were used and compared for a return period of 75 years which is as per the AASHTO LFRD code. The methods used were extrapolation using the normal distribution as per Nowark's(1991) method, and the Gumbel Distribution using the POT and the Block Maxima and in the calculations, we did use the equations which were formulated by (chow,1988) and Sivakumar et al (2011). For the selection of the data frame, we did select for the block maxima method the blocks were chosen monthly, and in using the POT methods the approaches used by Castillo which fit the top $2\sqrt{n}$ of distribution of n data and like Enright top 30% of data have been used and are compared.

2.3.2 Normal Distribution

In the calibration of the traffic load model used in the AASHTO, Nowak, and others use Normal probability paper to extrapolate the maximum loading effects for the time, i.e., he fitted the load effects to a Normal distribution and extrapolates to find the characteristic maximum.

In the case of Nowak (1991), the number of vehicle combinations will remain unchanged as of the data collection period. Assume that N is the number of vehicle combinations in period T if a site has 240,000 vehicle combinations monthly, this will result in N = 216 million vehicle combinations in 75 years. Then the probability level for N is 1/N. For 216 million it is $\frac{1}{216{,}000{,}000}$=4.6x10^{-9}, which would be equal to 5.74 on the vertical scale of the CDF plot. The number of combinations N, probabilities$\frac{1}{N}$, and inverse normal distribution corresponding to 75 years periods.

When assuming a normal distribution, the magnitude of the event occurring at time T is taken as the mean μ plus the deviation of ΔX_t of the variate from the mean.

$$X_t = \mu + \Delta X_t \qquad \text{Equation (2.20)}$$

The deviation is taken to be equivalent to the product of standard deviation and frequency factor k_t (chow, 1988).

$$X_t = \mu + K_t \sigma \qquad \text{Equation (2.21)}$$

The frequency factor k_t in using normal distribution is equal to the normal deviate z. z value for a return period T, with P(exceedance probability) equal to $\frac{1}{T}$, could be approximated using an intermediate value w and can be inputted in the formula used for calculating z.

$$W = \left[\ln \frac{1}{p^2}\right]^{\frac{1}{2}} \qquad \text{Equation (2.22)}$$

$$z = w - \frac{2.515517 + 0.80853w + 0.010328w^2}{1 + 1.432788w + 0.189269w^2 + 0.111308w^2} \qquad \text{Equation (2.23)}$$

P= exceedance probability

W= intermediate Variable

Z= standard normal distribution value

2.3.3 Extreme Value Distribution

Extreme value Type I distribution gives numerous advantages. The first requirement in using extreme value theory the extreme values in our case which are truck loadings are independent events.

In general, independence is assured when the event-type is not dependent on the event which is preceding it. Naturally, it is rational that incidences of bridge loading events are found to be independent. When using the Gumbel (Extreme value Type I) distribution parameters are estimated directly from recorded maxima of the events without the need in the use of numerical methods or tables. This extreme distribution is found to be useful in the extrapolation of extreme events. Two methods will be discussed.

(a) Chow method

In plotting the distribution graph of the loading effects, the reduced variate, y, of the exceedance probability the equation developed by Chow is used.

$$y_t = -\ln\left[\ln\left(\frac{T}{T-1}\right)\right]$$ Equation (2.24)

where T is the return period and is given by $T=\frac{1}{p}$

as it was discussed earlier the frequency factor K_T for the Normal Distribution was found to be equivalent to the normal deviate z, which is the standard normal distribution variable. when finding the frequency factor K_T, for Gumbel(EVI) distributions. The following formula was used which is developed by (Chow,1988)

$$K_T = \frac{\sqrt{6}}{\pi}\left\{K_T = \frac{\sqrt{6}}{\pi}\left\{0.5772 + \ln\left[\ln\left(\frac{T}{T-1}\right)\right]\right\}\right.$$ Equation (2.25)

Where T = Return Period

In the calculation of the return period, finding the population size of the extreme events was found to be necessary. The same procedure was used in the selection of the population the Block Maxima, Castillo, And Enright methods of selecting populations were used.

For example, if 30 extreme events were recorded in a month i.e. by extreme events we are saying that a certain threshold value has been exceeded. When considering a return period of 75 years we are going to expect, 27,000(30x 12 x 75) effective periods were considered to have occurred.

(b) Sivakumar et al. [2011]

Sivakumar et al. (2011) assessed the performance of the normal fit of the tail method on estimation of the maximum loading effects for long return periods the results display the method could be able to obtain a good estimate for a short duration of the return period. Periods less than let's say a month, but have got flows in obtaining maximum loading effect for obtaining a longer return period.

Due to this factor, an alternate a more analytic, and better method is proposed by Sivakumar in 2011. By raising the normal distribution to power to obtain the maximum distribution, which is the Extreme Value Type I (Gumbel) distribution according to the domain of attraction. Then the mean μ_{max} and the standard deviation, σ_{max}, could be found analytically using the equation below:

$$\mu_{max} = \mu + \sigma\sqrt{2\ln(N)} - \sigma\frac{\ln[\ln(N)] + \ln(4\pi)}{2\sqrt{2\ln(N)}}$$ Equation (2.26)

$$\sigma_{max} = \frac{\sigma}{\sqrt{2\ln(N)}}$$ Equation (2.27)

3. METHODOLOGY

A qualitative research methodology was adopted and both a primary and secondary data sources were also used for the study .Generation of a database, simulation of the loading events, extrapolation of results, and creation of a live load model were also be discussed in depth as follows .

3.1 Collection of Traffic Data

This step is the foundation for the analysis. The first step in this stage is the selection of the study area the requirement of the selection process was the site which has the highest amount of traffic loading. Due to this fact, the road which connects Addis Ababa to Djibouti which is a major gateway of Ethiopia's import-export logistics, and it is anticipated that heavy vehicles will remain a common sight on this road. Due to this fact, the collection of the vehicle weights and axle related data was done in weighing stations located on this road path. Then, Axle load related data was obtained from the Ethiopian Roads Authority from weighing stations found from Modjo, Sululta, and Semera this data could be categorized as secondary data. The Axle Load Collection format is presented in the next chapter which is chapter 4. Then a field survey was undertaken in two sites namely the Modjo and Sululta Sites to record several types of truck configurations as well as inter-axle spacing.

3.2 Generation of Database

The data obtained from ERA and field survey was analyzed and the first procedure involved the removal of data records which were found to be erroneous using data filtration techniques. It is found to be important to eliminate such erroneous records as they could ruin the findings of this study. Then the organized categorized and filtered data could be used in the determination of loading effects.

3.3 Simulation of Loading Effects

For the created traffic database, bypassing the trucks in bridges we were able to capture the maximum loading effects such as the mid-span moment and shear force occurring in a simply-supported beam with span lengths varying from 5m to 50m span bridges using the concept of influence lines. In the calculation of those load effects, it was found to be highly computational. Therefore, a simulation program was developed in Python for the calculation of the loading effects. A comprehensive description of the program developed to calculate the load effect is presented in section 5.5.

3.4 Extrapolation of loading events

Once finding the bridge loading effects we like to see the extreme events which occur in the bridge. In reality, it is found to be impossible and demanding in the computation for simulating for longer periods. Thus, the Load Effect data consistent with a statistical distribution which helps to extrapolate the characteristic loading values.

3.5 Live Load Models

Notional models differ from actual truck configuration models, because they are invented to signify a group with multiple types of trucks. To be functional for a range of conditions, the notional load model is found in which envelopes reflect load effects.

4. DATA COLLECTION AND ANALYSIS

The use of statically weigh stations to collect traffic survey and control in Ethiopia has taken place since the period of 1990 the axle load limits were put in place by the Council of Ministers. The data provided in this master's thesis is a secondary data found from the Ethiopian Roads Authority (ERA) tries to give insight into the complexity and the random nature of the traffic in Ethiopian roads.

A total of 35,000 truck load data including number of axles The data is gathered from three selected sites namely Modjo, Semera, and Sululta. The data from those sites is selected due to the reason that there is a large amount of movement of heavy vehicles and most of the vehicles are recorded in those sites and since those routs are the routs where most of the country's economic transactions occur the data is taken from those locations.

In this work, accuracy of investigation of vehicle data from a measurement series is introduced and results and methods were also filtered based on data. The filtration process removes the data which are found to be inaccurate and then the data both before and after were evaluated to the method discussed. Finally, a total of 35,000 trucks loading data was evaluated to quantify the amount of overloading present in our national roads.

4.1 Measurements of Traffic Loading

Traffic counters are mostly used in the measurement of traffic data (Ales Znidaric, Maja Kreslin, Jan Kalin,2016). In the method of determining the axle loading and the corresponding gross vehicle weights, it needs to be weighed. And several weighing methods are available

4.1.1 Static Weighing

Is the most accurate and most commonly used weighing technology, but mainly used for direct enforcement.

Figure 4-1: Static Weighing station in Modjo and sululta

4.1.2 Low-speed WIM

It runs in controlled conditions usually at speeds less than $10\frac{km}{h}$. It is used for accurate pre-selection and sometimes direct enforcement. The low-speed weighing system is installed and is being used at the toll gates of the Addis Ababa Adama Expressway

Figure 4-2: Low-speed WIM

4.1.3 High-Speed WIM or WIM Systems

Are used in measuring the dynamic axle loads in highways in unrestrained conditions and normally provide the exact time of event occurrence, the individual and group axle loads, the number of axles, the axle spacing, the speed, headway distance between trucks, and also the gross vehicle weight as well as vehicle classification.

Figure 4-3: High-Speed WIM systems embedded in the pavement

4.1.4 Bridge Weigh In Motion

Uses an existing road structure either a bridge or culverts to weigh the moving vehicles.

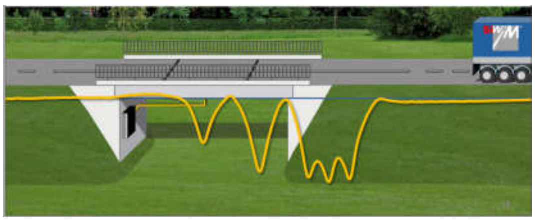


Figure 4-4: Measurement of passing vehicles using B-WIM

4.1.5 Analysis of GVW Data

In this thesis, three sets of weighing station secondary data were collected from the Ethiopian Roads Authority (ERA) which were recorded between periods of November 2017 to December 2019 at the three locations from Modjo, Semera, and Sululta were used. Though there were other parameters which were found from the data extracted, only the recorded axle weights were used in this study.

Figure 4-5: Location of Sites where data is collected (image: Intercontinental Consultants and Technocrats Pvt. Ltd, 2011)

The recorded amount of data during this period was around 35,000 heavy vehicles. The data from these sites was received in an Excel format as shown in Table 4.1 below and from that, the classification of vehicles was based on the total number of axles.

Veh	Date	A-1	A-2	A-3	A-4	A-5	A-6	A-7	GVW	Total No Axles
1	6/12/2017	5.68	6	5.96	0	0	0	0	17.64	3
2	6/12/2017	4.7	3.58	3.46	7.16	0	0	0	18.9	4
3	7/12/2017	7.36	13.7	13.46	0	0	0	0	34.52	3
4	12/7/2017	7	13.24	14.2	8.64	9.3	9.22	0	61.6	6
5	12/7/2017	8.42	12.76	13.52	8.56	10.2	11.3	0	64.76	6
.	12/7/2017	8.42	12.76	13.52	8.56	10.2	0	0	53.46	5
.	25/7/2017	6.82	7.08	10.26	10.24	11.3	7.84	7.82	61.36	7
.	25/7/2017	6.64	13	13.38	9.54	9.32	9.86	0	61.74	6
.	25/7/2017	6.58	13.74	14.08	0	0	0	0	34.4	3
.	25/7/2017	7.38	13.68	13.38	10.46	9.84	10.06	0	64.8	6
.	25/7/2018	8.22	12.54	13.08	0	0	0	0	33.84	3
.	25/7/2018	7.04	13.22	13.34	0	0	0	0	33.6	3
.	25/7/2018	8.22	12.96	13.06	11.48	7.62	8.16	0	61.5	6
.	25/7/2018	6.74	13.92	13.46	0	0	0	0	34.12	3
.	25/7/2019	6.44	13.44	14.38	10.58	9.62	10.1	0	64.56	6
35000	**25/7/2019**	**7.22**	**13.38**	**13.68**	**0**	**0**	**0**	**0**	**34.28**	**3**

Where **: Veh** Vehicle number**, A-1 to A7** weight of Axles in tons with their corresponding locations. **GVW** in tons.

Table 4-1: - classification of vehicles

"A common rule to define the class of truck is the number of axles" (Caprani, 2005; Enright and O'Brien, 2012). Assigning the axle masses was crucial to classify the numerous configurations of vehicles that were present in the mentioned location sites.

The classifications of vehicles data presented in Table 4-1 above highlighted the weight data used on the calculation of the load effects induced by the recorded "actual" across a range of simply supported spans these also been discussed and presented in depth in chapter 5.

From Council of Ministers Regulations No. 11/1990 from NEGARIT GAZETA (1990). The legislation with regards to axle weight limits was taken. The purpose of this was to show the gap between the recorded or "actual" and the "legal" the extent of overloading.

4.2 Cleaning (Filtration) of Unreliable weight Data

Even though, we are using static weighing machines to collect data in our country in this thesis it is intended that best practices and methodologies used by several authors and other countries could be used in our context. Most of the literature is written for the use of weigh-in-motion techniques we have slightly modified so that they can be applied for the use in cleaning our data. Defective recordings regarding weigh in motion systems have been written several kinds of literature by several authors. *“It is important to examine the WIM data to remove unreliable data containing unlikely trucks to ensure that only quality data is used to model traffic load” (Enright and O'Brien, 2011, Sivakumar et al.,2011).* WIM data cleaning rules have been recommended by Enright and O'Brien [2011]; Sivakumar et al. [2011]. However, the feature of traffic data, the type of WIM system differs in the country's regulations. The cleaning techniques used in this thesis by prominent authors (Enright and O’Brien, 2011; Getachew, 2003; Sivakumar et al., 2011, Xiao Yi Zhou, 2013) are presented in Table 4-2 below:

Getachew,2003	Enright and O'Brien,(2011)	Sivakumar et al., 2011	Xiao Yi Zhou,2013 Thesis	This thesis
Accept one axle vehicles with length less than 12 m	At least 2 axles; and no upper limitation	Total number of axles less than 3 or greater than 12	Exclude vehicles with less than 2 axles; and the observed maximum number of axles is 8.	Exclude vehicles with less than 3 axles; and the observed maximum number of axles is 7.
–	Exclude if the speed below 20 to 60km/h varies with the site.	Speed below 16 km/h	Clean if the speed below 40 km/h	–
–	Exclude if speed above 120 km/h.	Speed above 160 km/h	Clean if speed above 120 km/h	–
–	Exclude if the sum of axle spacinggreater than the length of the truck	Sum of axle spacing greater than length of truck	Sum of axle spacing greater than the length of a truck or less than 65% of truck length	–
–	Exclude if the sum of the axle weights differ from the GVW by more than 0.05t	The Sum of the axle weights differs from the GVW by more than 10%	Exclude if the sum of the axle weights differ from the GVW by more than 10%	–
–	GVW less than 3.5 t are excluded	GVW less than 5.4 t	–	GVW less than 3.5 t are excluded. As per Enright and O'Brien
–	Vehicles with individual axles greater than 40 t are excluded.	Individual axle weight greater than 32 t	Exclude vehicles if the proportion of axle weight is greater than 85% of GVW	Individual axle weight greater than 32 t.As per Sivakumar et al., 2011
–	–	Steer axle greater than 11.3 t	–	–
–	–	Steer axle less than 2.7 t	–	–
Exclude if the distance between the first axle and last axle is less than 3 meters for four-axle vehicles	–	First axle spacing less than 1.5 m	–	–
Any axle spacing less than 1 m for five or more axle vehicles	Any axle spacing less than 0.4 m	Any axle spacing less than 1 m	–	–
–	–	Any individual axle less than 1 t	Keep only vehicles with axle weight greater than 0.6 t	–

Table 4-2: Data Cleaning Techniques

4.2.1 Three Axle Vehicles

From the data obtained from the Ethiopian Roads, authority 15,058 vehicles (41.2 % of the population) were recorded with three axles and Gross Vehicle weights (GVW) ranging from 10.98 tons to 52.7 tons. Vehicles with three axles that were observed during the site visit include cars with trailers, dump trucks, Isuzu's also known as (FSR), and other forms were observed. In this event, it was the dump trucks that were of interest in defining the upper limits of the weights. This class of the data investigation resulted in approximately 99.6 % of the entire number of vehicles recorded with three axles that were accepted and kept for additional evaluations.

4.2.2 Four Axle Vehicles

1,581 vehicles were comprising of (4.32 % of the population) were recorded with four axles the vehicles were registered with GVW between 14.7 tons and 66.25 tons. From the investigation, around 97.3 % of the whole data from this group was found to be adequate and saved for further evaluations.

4.2.3 Five or more Axle Vehicles

These vehicles consist of 49.3 % of the population, the maximum and minimum GVW of those vehicles registered with five to seven axles are shown in Table 4.3 below. In the filtration of unreasonable data of vehicles in this group, among the 17,991 vehicles that were registered with five to seven axles. All vehicles registered were kept except for 29 vehicles that were found in the six-axle truck group. The smallest vehicle recorded in this group had a GVW of 19.78 tons and the highest record was found to be 70.3 tons, see Table 4-3.

Number of axles	Number of vehicles	Gross Vehicle Weight (GVW) in a ton	
		Min	Max
5	1,718	23.82	69.06
6	15,840	19.78	70.16
7	433	48.72	70.28

Table 4-3: The maximum and minimum weights and lengths of vehicles registered with five to seven axles.

4.2.4 Result of Filtration of Unreasonable Data for the Series

From December 2017 and December 2019, around 99 % of the total vehicle data collected were found to be acceptable for further evaluation.

Measurement period	Number of axles	Number of vehicles	Number of vehicles after filtrations	Percentage of unreasonable vehicle data, [%]
06/12/2017-26/12/2019	3	15,058	15,007	0.67
06/12/2017-31/12/2019	4	1,581	1,581	0.00
07/12/2017-18/12/2019	5	1,718	1,718	0.00
06/12/2017-31/12/2019	6	15,840	15,819	0.13
12/12/2017-18/12/2019	7	433	433	0.00
Total		34,630	34,558	0.99

Table 4-4: Results of Filtration

Throughout the filtration process, the vehicle's upper limits were closely investigated. We were able to see negative recordings from the data. Vehicles with reasonable weights were kept for further analysis. This work was done using Excel templates.

4.3 Statistical Approach

4.3.1 Accuracy of Data

It appears that errors in weighing systems are caused due to two factors, one is due to the scale itself the way it is installed the level of the scale, site unevenness, and other factors and the other is due to external factors such as the way the truck entered the scale, the oscillation of the truck, the friction caused by the truck and the scale, the suspension system of the truck, the braking system, etc.. Could be mentioned. Although the static scales have a high percentage of accuracy they are subjected to some errors. Due to those reasons*, "Tolerance in the weighing measurement is made as to the form of a weighing tolerance. The magnitude of such an allowance is based on the assumed scale error (scale type dependent) plus that estimated for external factors" (Intercontinental consultants and Technocrats Pvt.Ltd, 2011).*

In giving a reasonable basis for setting a tolerance in weighing scales, a National Weighbridge Survey was carried out in South Africa in 2002, in which 57 weighbridges were used (Intercontinental Consultants and Technocrats Pvt. Ltd, 2011). From the survey, it was seen that:

- Gross Vehicle Mass: All readings fell within the range –0.88% to +0.76% of the average combination mass;
- Axle Unit: All readings fell within the range –2.14% to +2.78% of the average combination mass; and
- Steering Axle: All readings fell within the range –5.12% to +4.96% of the average steering axle mass.

Based on the above results, a recommendation was made so that the tolerance on GVM should be set at +2% and that on axles at +5%. It was also agreed at a three-party (SADC/COMESA/EAC). Regional Workshop on Harmonization of Key Elements of Best Practice in Overload Control (Infra Africa (Pty) Ltd, 2008) recommends that, as a short-term measure, a mass tolerance of 5% on axles, axle units, and GVM would be adopted. In this study, we were not able to find a legal document with regards to tolerance on axle or vehicle weight in Ethiopia.

4.3.2 General Statistical Properties of the Data

Statistical properties of trucks traveling on the route to Modjo, Semera, and Sululta has been presented in this section. All heavy vehicles, ranging from 3 to 7 axle vehicles that were recorded for two years were considered for investigation. The population of vehicles to analyze the data was 34,558 truck events. The computation with regards to the vehicle axle masses, shown in Table 4-5, shows a significant number of axle masses are bypassing the legal limits put in the regulation which is 8 ton for axle 1 and 10 ton for the remaining axles. This gives an indication for the possible errors in the estimated static axle loads and the result indicate that predominant on Axle-2 and Axle 3 that deserves to have a look for further study.

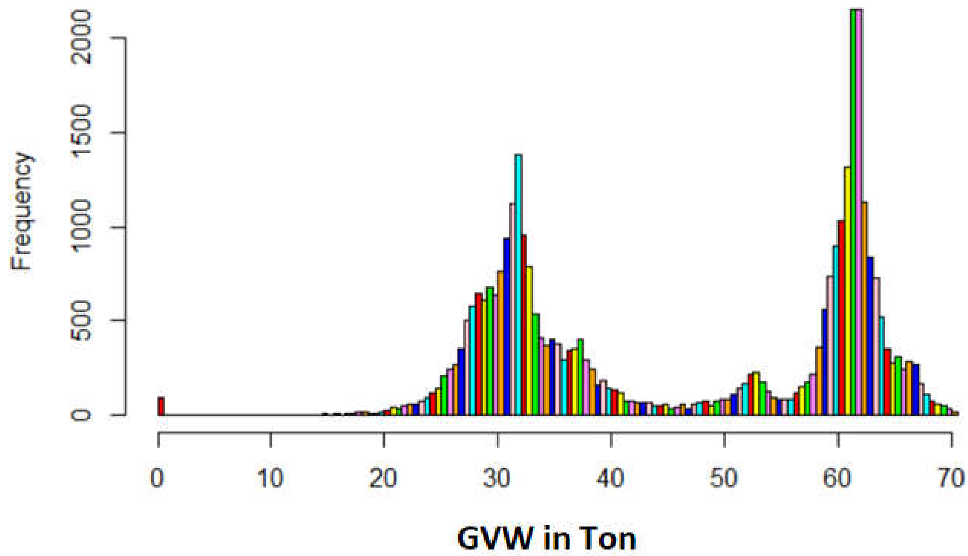
Table 4-5: General Statistical properties of the weighing station data.

MEAN	**7.13**	**12.35**	**11.97**	**9.99**	**9.13**	**9.55**	**5.75**
MAX	20.8	21.74	26.24	31.46	27.26	27.04	15.82
STANDARD DEVIATION	0.99	1.72	2.00	2.06	1.35	1.68	2.38
TONNES	**Axle 1**	**Axle 2**	**Axle 3**	**Axle 4**	**Axle 5**	**Axle 6**	**Axle 7**
0.0-4.0	7	5	18	484	9	75	62
4.1-5.0	57	30	129	111	9	28	163
5.1-6.0	3,430	101	318	261	345	315	68
6.1-7.0	13,249	217	374	401	620	291	28

7.1-8.0	12,567	390	626	554	1,818	1,095	33
8.1-9.0	4,164	708	1,228	1,994	4,763	3,745	22
9.1-10.0	818	1,497	2,483	5,994	6,926	5,843	22
10.1-11.0	148	3,400	3,765	5,166	2,632	2,631	22
11.1-12.0	49	5,236	5,016	2,304	530	1,010	8
12.1-13.0	11	11,053	11,121	1,112	146	570	4
13.1-14.0	15	8,097	6,333	624	67	359	-
14.1-15.0	6	2,313	1,821	288	30	176	-
15.1-16.0	11	910	786	109	20	70	1
16.1-17.0	5	398	323	24	7	25	-
17.1-18.0	7	132	124	7	10	3	-
18.1-19.0	4	48	49	4	10	1	-
19.1-20.0	5	13	19	1	8	1	-
20.0-21.0	5	6	8	-	2	-	-
21.1-22.0	-	4	10	1	2	-	-
22.1-23.0	-	-	2	1	1	-	-
23.1-24.0	-	-	3	-	-	1	-
24.1-25.0	-	-	-	-	-	-	-
25.1-26.0	-	-	1	-	1	-	-
26.1-27.0	-	-	1	1	-	-	-
27.1-28.0	-	-	-	3	1	1	-
28.1-29.0	-	-	-	5	-	-	-
29.1-30.0	-	-	-	2	-	-	-
TOTAL	34,558	34,558	34,558	19,451	17,957	16,240	433

The histogram plot of the gross vehicle masses which is shown in Figure 4.6 below, displays a twin-peaked bimodal distribution of gross vehicle mass ranging from 11 ton to 70 ton. This type of distribution is common for gross vehicle weights as it has been said by (O'Connor et al., 2001). The first mode contains mostly the 3 axle vehicles and partially loaded vehicles which are above the three axles. The second mode involves the fully loaded 4 to 7 axle vehicles.

Figure 4-6: Histogram Plot of the Gross Vehicle Weight



4.3.3 Statistical properties corresponding to axle configuration

From the data obtained from Ethiopian Road's authority, the statistical properties and theoretical distributions are presented below.

Weights in [Tons]

Vehicle	Statistical properties	A-1	A-2	A-3	A-4	A-5	A-6	A-7	GVW
3-Axle Vehicle	Mean	7.17	12.26	12.14	–	–	–	–	31.47
	Mode	6.70	12.38	12.42	–	–	–	–	31.54
	Standard Deviation	1.03	1.92	2.12	–	–	–	–	4.61
	Skewness	1.25	0.12	1.39	–	–	–	–	-0.79
4-Axle Vehicle	Mean	6.71	10.88	8.13	6.58	–	–	–	31.16
	Mode	6.72	12.78	7.80	2.68	–	–	–	0.02
	Standard Deviation	1.18	2.68	2.38	3.87	–	–	–	8.10
	Skewness	3.77	-0.08	0.59	2.05	–	–	–	-0.91
5-Axle Vehicle	Mean	7.18	12.01	11.61	9.99	9.77	–	–	50.56
	Mode	6.92	13.00	12.88	10.26	9.24	–	–	53.12
	Standard Deviation	1.57	1.90	2.01	1.83	2.37	–	–	5.99
	Skewness	4.61	-1.25	-1.06	0.11	1.58	–	–	-0.94
6-Axle Vehicle	Mean	7.14	12.62	12.22	10.30	9.06	9.62	–	60.86
	Mode	7.10	13.08	12.94	9.80	9.30	9.28	–	61.64
	Standard Deviation	0.83	1.24	1.50	1.49	1.16	1.62	–	4.64
	Skewness	0.67	-1.41	-1.25	0.61	-0.46	0.45	–	-5.21
7-Axle Vehicle	Mean	6.91	12.23	11.87	10.18	9.51	6.59	5.75	63.04
	Mode	6.82	13.46	13.16	10.02	9.66	5.38	4.78	61.72

	Standard Deviation	0.76	2.09	2.00	1.78	1.48	1.55	2.38	3.27
	Skewness	0.11	-1.47	-1.15	0.03	-0.71	1.11	0.90	-0.45

Table 4-6: Statistical Properties of vehicles classified in Axles

When fitting the theoretical distributions for different axle configuration vehicles it can be observed that the corresponding probability density function of the gross vehicle mass follow a normal distribution in most of the axle configurations except for group of vehicles of trucks which were classified as five-axle and six-axle trucks in which the distribution was able to follow Weibull distribution which is enumerated in Figure 4-9 and 4-10

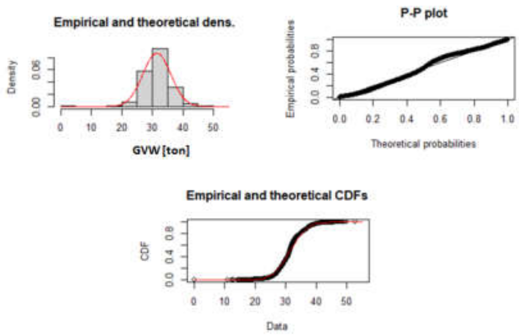


Figure 4-7: Histogram plot and Normal Distribution plot of gross vehicle weights of 3-axle trucks

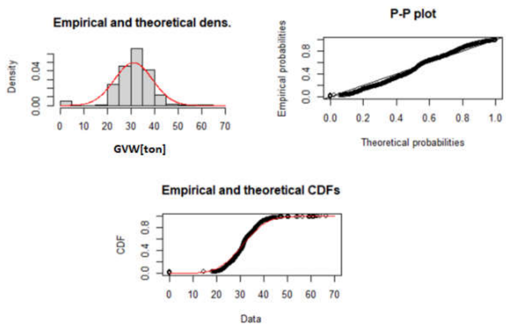


Figure 4-8: Histogram plot and Normal Distribution plot of gross vehicle weights of 4-axle trucks

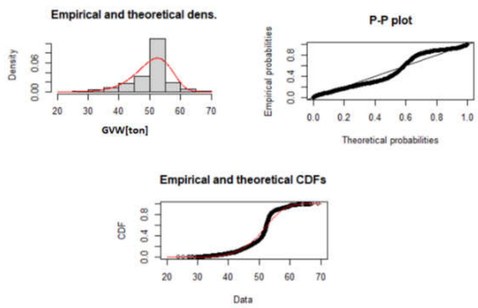


Figure 4-9: Histogram plot and Weibull distribution plot of gross vehicle weights of 5-axle trucks

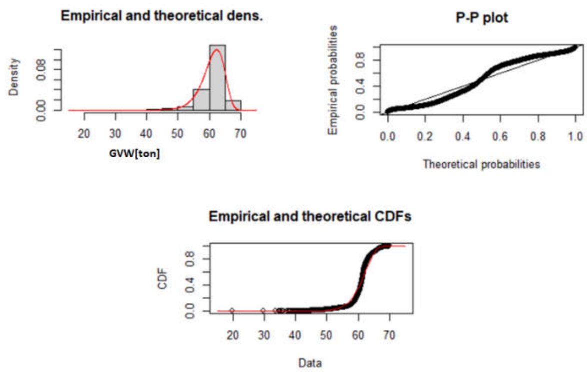


Figure 4-10: Histogram plot and Weibull distribution plot of gross vehicle weights of 6-Axle trucks

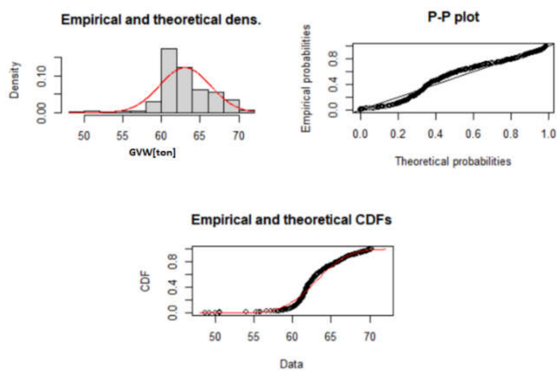


Figure 4-11: Histogram plot and Normal distribution plot of gross vehicle weights of 7-Axle trucks

From Figures 4-7 to 4-11 the gross vehicle weight of the trucks mainly follows two types of distributions as discussed earlier, and this collected data from different weigh stations can be used in the modeling of live loads.

4.4 Traffic Flow Variation

Vehicles recorded in the weighing stations, on the recorded period of two years starting from December 2017 to December 2019, were not identical. As shown in Figure 4-12 where the frequency of the number of vehicles that pass the weighing stations during the whole measurement period is plotted versing the date of passage.

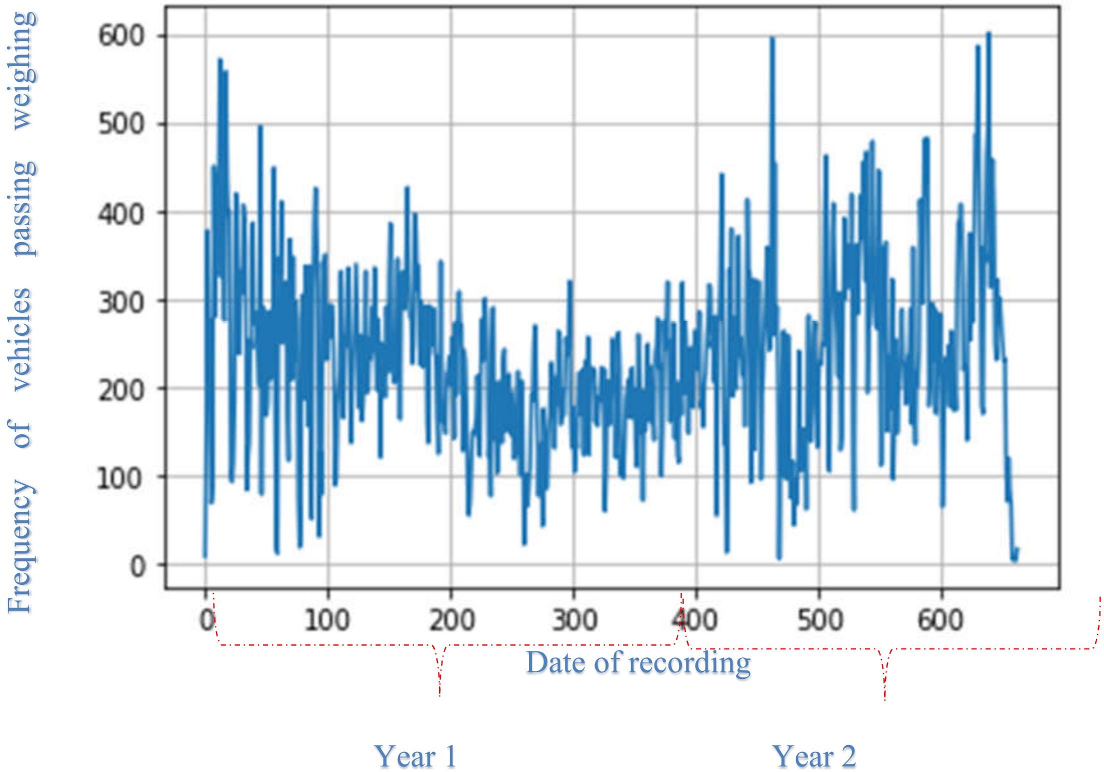


Figure 4-12: Flow of Vehicles In the recorded period

4.5 Field Measurements of Axle Spacing's

An additional important variable in traffic characteristics is the spacing between successive axles. As per the report of the New Brunswick Department of Transportation, 2010 they identify that knowing the value of axle spacing is critical for short and long-span bridges. Therefore, knowing the value of axle spacing is of importance. In this study, since the data found from ERA didn't have axle spacing of trucks it was decided to take a site survey and determine the most prominent heavy vehicle types that are found in Ethiopian bridges. The measurement of truck axle spacing was done in the two sites namely in Modjo and Sululta. The data found from the survey are presented in Table 4-7

Number of Axles of vehicles	Total Length(m)	
	Minimum	Maximum
3- Axle Vehicles	4.50	6.15
4- Axle Vehicles	5.40	13.92
5- Axle Vehicles	12.80	15.85
6- Axle Vehicles	11.54	16.33
7- Axle Vehicles	14.05	15.60

Table 4-7: Maximum and Minimum Length of vehicles from the observation

4.6 OVERLOADING

The current axle load limit regulation is based on a proclamation that was endorsed in September 1990 under the publication of Negarit Gazeta. These restrictions have a strong impact on the construction, maintenance, and highway safety issues. There are eight (8) Basic Articles which are stated under the council of ministers regulation No 11/1990 titled "Council of ministers regulation to amend the vehicle size and weight regulations". The summary is presented in the tables below.

No	Size restriction	Limits
1	Outside width of any vehicle, trailer semitrailer, including load thereon.	≤ 2.5 meters
2	Total Height of any vehicle including load thereon	≤ 4.2 meters
3	The total length of any vehicle	≤ 12 meters
4	Total Length of any combination (Truck Trailer or semitrailer) vehicle	≤ 17 meters
5	Total Length of any combination (Truck with drawbar Trailer) vehicle	≤ 18 meters
6	Combination of Motor Vehicles	≤ 2units

Table 4-8: Restrictions as to the size of the vehicle

No	Weight restriction	Limits
1	The Steering Axle weight	≤ 8 tons
2	Single Axle Dual Wheel	≤ 10 tons
3	Tandem Axle dual Wheel with a distance between axles ≤1.3m	≤ 17 tons
4	Group of two or more axles, with a distance greater than 1.3m	≤ 10 tons

Table 4-9: Restrictions as to the weight of a vehicle

Any vehicle outside the specified restrictions must get a special permit. In practice, no consideration is given in axle spacing in the weighing stations. A simple rule of 8 ton on the front axle and 10 ton on the rear axle is applied as legal limits. Hence a 20 ton on a tandem axle or 30 ton on a tridem axle is considered legal regardless of the axle spacing.

The extent of overloading present on the roads from the data given was analyzed to give insight to the extent in which overloading was occurring. The results are shown in Table 4-10.Illegal overloading of trucks or vehicles accounts on average 53.3 % over the given vehicles were found to be in tress pacing with the axle load limit Regulations (1990). However, it was seen that 20% on average of the heavy vehicles were outside the restrictions speaking in terms of the Gross Vehicle Mass. This implies that abnormal vehicles merit special attention from law enforcement agencies.

Number of axles	Number of vehicles	Percentage of overloaded vehicles In terms of Axle	In terms of GVW
3	15,058	66%	32%
4	1,581	26%	7%
5	1,718	52%	75%
6	15,840	49%	5%
7	433	38%	8%

Table 4-10: Number of observed illegal vehicles

4.7 Discussion

The data which was obtained from Modjo, Sululta, Semera can be described as a representative of Ethiopian Traffic numerous observations were made on the data the Gross vehicle weight distribution, axle configuration, vary significantly the vast majority of trucks consist of 3-Axle and 6-Axle trucks, it was seen that the truck type changes with the vehicle weight.

The study shows that from the collected data which consisted of the routs from Modjo, Semera and Sullulta were of good quality but some trucks were registered with inaccurate weight recordings, hence these vehicles were excluded.

Besides, a comparison of the statistical properties with regards to axle loadings and gross vehicles was made the fitting of theoretical distributions was also done most of the vehicles fitted the normal distribution except for five and six-axle trucks which fitted the Weibull distributions.

In the analysis of the weighing station data, it was observed that the overloading of axles and a set of axles was found to be more critical than the overloading of the whole vehicle. Particularly, event in which overloading is occurring requires attention from Law Enforcement agencies.

5. FINDINGS AND DISCUSSION

In the previous chapter we were able to see how the data was gathered organized, filtered, and the representative theoretical distributions of axle masses. In this chapter using the data, we are going to view the magnitude of the load effects which are produced by heavy vehicles in Ethiopia over the bridge structures. This chapter gives a thorough presentation of a program developed by the author for traffic combination generation and simulation. The program has been written in the python language, and results from the simulation were compared with the corresponding values calculated using the traffic load model from the Ethiopian bridge design Truck/ Tandem loading.

5.1 Organization of the data for simulation

The raw data from weighing stations were organized into spreadsheets based on vehicle number of axels. Then, the axle spacing's which were recorded from the site survey were randomly assigned to their corresponding axle configurations. As it is enumerated in Table 5-1 below.

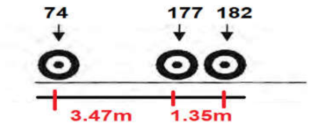
No of Axles	A1	S1	A2	S2	A3	S3	A4	S4	A5	S5	A6	S6	A7
3	75.9	4.7	137.3	1.45	136.5	–	–	–	–	–	–	–	–
4	88.1	1.5	147.9	2.75	91.63	1.4	109.7	–	–	–	–	–	–
5	80.6	3.26	130.7	1.3	119.7	6.9	124.8	1.4	119.1	–	–	–	–
6	77.5	4.2	133.8	1.4	134.2	4.8	116.2	3.8	88.9	1.4	95.9	–	–
7	66.5	1.5	134.9	2.7	134.0	1.4	90.4	4.8	95.7	3.8	59.8	1.4	48.3

No of Axles - vehicle classification per number of axle.
A1-A7 -Weight of Axles given in kN with their corresponding locations
S1-S7 – Inter axle spacing in meter.

Table 5-1: Vehicle Classifications combined with their corresponding inter-axle spacings

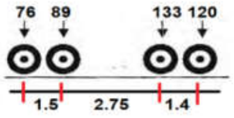
5.2 Sample of Recorded Vehicle Configuration and Classification

Sample Configuration and loading of 3-Axle Truck



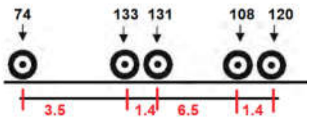
GVW = 433kN

Sample Configuration and loading of 4-Axle Truck



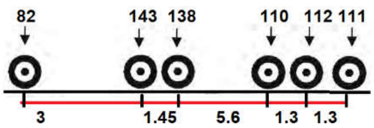
GVW = 418kN

Sample Configuration and loading of 5-Axle Truck



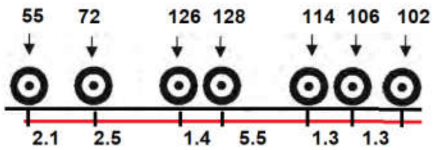
GVW = 566kN

Sample Configuration and loading of 6-Axle Truck



GVW = 696kN

Sample Configuration and loading of 7-Axle Truck



GVW = 703kN

5.3 Headway Distributions

The definition for headway is the time-interval between the arrivals of two consecutive vehicles at a fixed point. Headways are important in the study of bridge loading and Traffic Engineering and are focus of research for years (Haight, 1963, Banks, 2003). As has been stated by numerous authors such as (HRB, 1965, Gazis, 1974, Lieberman and Rathi, 1992) headway distributions are affected by the driver's behavior.

The headway distribution is, as for all the other traffic characteristics, a site-specific variable. In the case of calculation of bridge loadings, inter-truck headway is of interest.

In the Modeling of headway, Harman Davenport(1979) developed a live load model assuming independent truck traffic with randomly varying headway distance for the simulations of Ontario Highway Bridge Design Codes (OHBDC 1979) and also an article written by OBrien and Caprani (2005) mentions that the statistical distributions were fitted to the measured headways HeDS collected from WIM data, and for bridges which are short to medium span bridges free-flowing traffic is of a governing effect, an article written by (Bruls et al 1996) also agrees with this idea.

For moving traffic usually, we usually define headways in units of time rather than distance. This is because the safe traveling distance is a function of driver reaction time. In this thesis, the method developed by Harman Davenport (1979) development of OHBDC and which was later used in the development of the AASHTO HL-93 Live Load models is adopted.

5.4 Influence lines

The influence line for any given point or a section of a structure is a curve or graph which presents the load position for maximum effect and the magnitude of stress functions such as bending moments, shears, and reactions. Application of influence line has been widely used in the simulation stage determine the load effects caused by the passage of the trucks in different span bridges. In this thesis simply supported bridges ranging in total length from 5 m to 50 m are considered and with that three load effects are considered:

1. Bending moment at mid-span of a simply supported bridge.
2. Shear at Left-end support of a simply supported bridge.
3. Shear at right-end support of a simply supported bridge.

Theoretical influence lines used in this thesis are shown below.

1. Bending moment at mid-span of a simply supported bridge

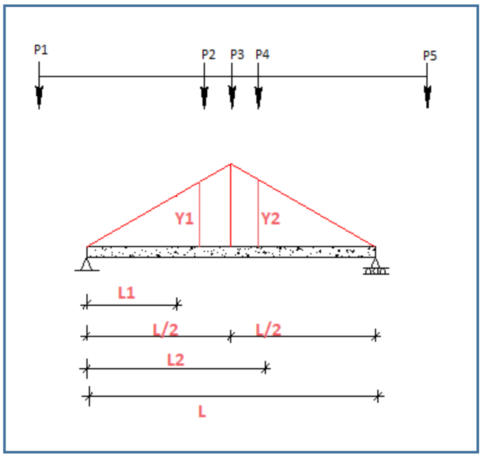


Figure 5-1: Influence Line for Mid- Span Moment

2. Shear at Left-end support of a simply supported bridge.

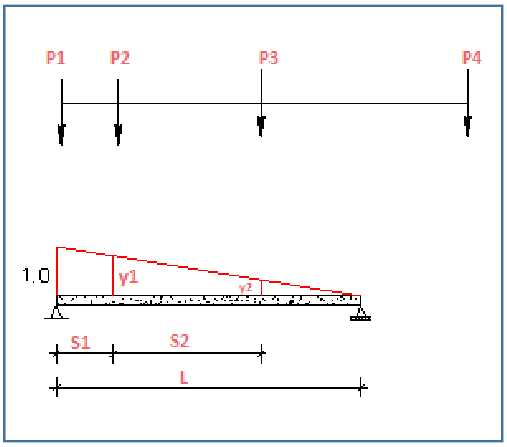


Figure 5-2: Left-hand support Shear

3. Shear at right-end support of a simply supported bridge.

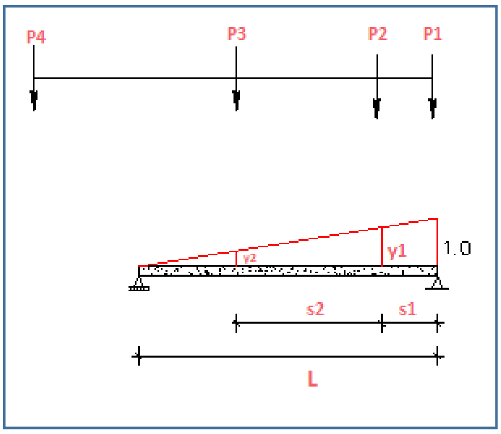


Figure 5-3: Right-hand support Shear

5.5 Development of the Program for the calculation of Load Effects

The recorded vehicle data was a two-year vehicle data and was clustered according to recorded data and was extracted into a separate Excel worksheet to do this an Excel VBA program was created and the extraction of the data into a separate excel worksheet was made. For the analysis of the loading effects of the vehicles, a program was written in python. The traffic file output in the form of Excel is read into the python program developed. Then the program generates and passes the traffic across several bridge lengths and calculates the maximum load effects for each case by the method of influence lines as previously discussed. In this program, it is possible to calculate the load effects using the collected data from the weighing Stations.

The Flow Chart of the python program developed is presented in Figure 5-4.

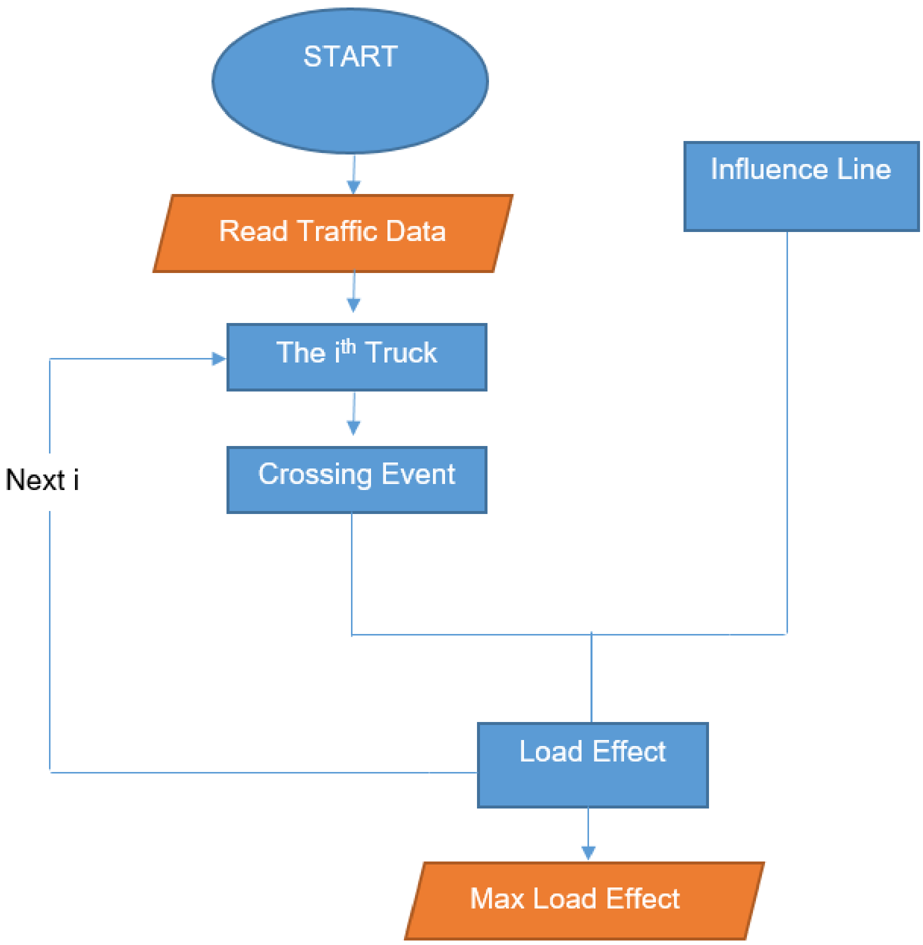


Figure 5-4: Flowchart of the Developed Program

When the number of recorded vehicles is large on a particular day, this can create problems with memory especially memory error will occur an output file stream was created so that this limitation is addressed with regards to RAM usage. In this thesis, 651 days of traffic loadings were simulated using the program.

Around 14 billion traffic loading combinations which were calculated in using this program for two years the maximum value of each load effect, the date of occurrence of the event is kept for further analysis, and the corresponding loading combinations, a large collection of events are stored. The results obtained from the simulation program are shown in the tables below:

Bending Moment (kNm)										
Spans(m)										
	5	10	15	20	25	30	35	40	45	50
Mean	319.2	771.0	1,376.3	2,211.2	3,258.6	4,465.4	5,854.4	7,419.9	9,113.3	11,007.2
standard deviation	39.3	85.5	165.6	287.2	433.6	625.2	838.9	1,089.6	1,388.8	1,728.7
Skewness	0.48	0.7	0.87	0.48	0.11	-0.15	-0.44	-0.68	-0.81	-0.96

Table 5-2 (a): Actual Trucks Bending Moment – Statistical Properties

Shear(kN)										
Spans(m)										
	5	10	15	20	25	30	35	40	45	50
Mean	293.1	363.6	453.3	534.6	617.6	707.1	786.9	863.4	934.8	1,009
standard deviation	34.7	42.8	58.0	68.0	80.7	105.3	120.3	136.1	153.7	170.2
Skewness	0.75	0.98	0.61	0.13	-0.31	-0.31	-0.51	-0.67	-0.73	-0.89

Table 5-2 (b): Actual Trucks Shear Forces – Statistical Properties

The load effects produced by the vehicles which are found from the simulation, on the recorded period of two years starting from December 2017 to December 2019, were not identical. This phenomenon is due to the fact in the variation of loading of the trucks and the mixture of occurrences of trucks this is presented in the time history plots shown in Figure 5-5 and 5-6

Presented below where the load effects: the moments and shears are plotted versing the date of event are plotted for different spans ranging from 5m to 50m.

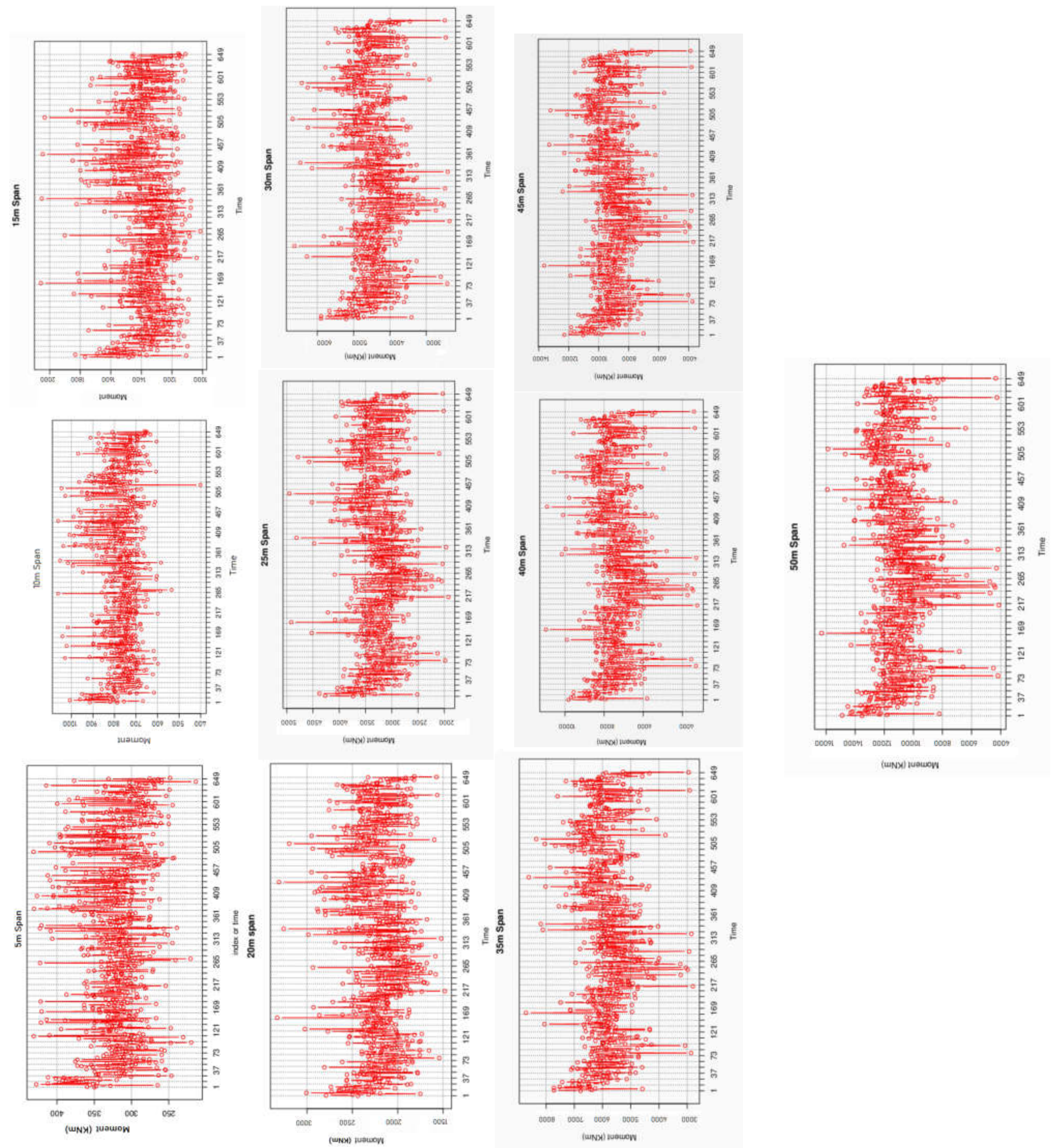


Figure 5-5: Time History Plot for Moment

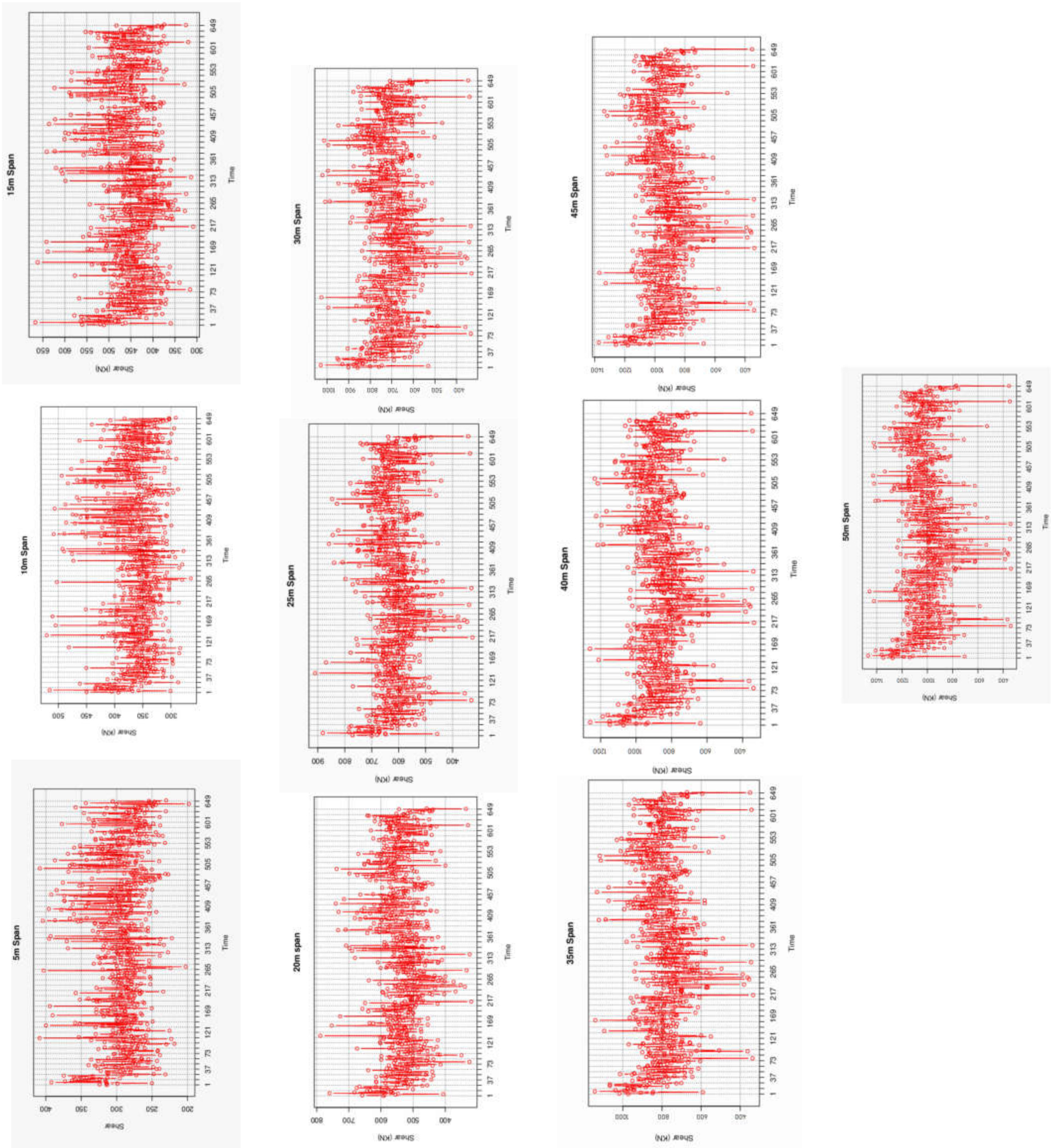
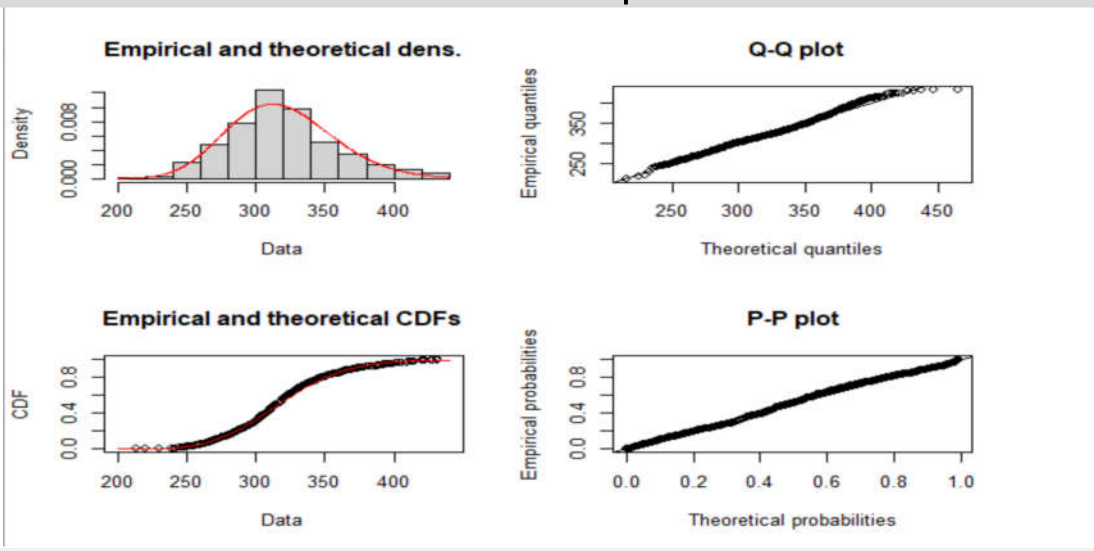


Figure 5-6: Time History Plot for Shear

5.6 Statistical Distributions

The main objective of this study and using the data which was collected from the weighing stations was that to predict the maximum loading effect which a bridge structure will be able to experience by a bridge structure during its design life. As we had seen in the previous time series plots of different spans, we can see that live loading is a random variable that is dependent on time. So a theoretical distribution function could be best fitted to the observed set of events. Then by using these theoretical Distributions we may be able to predict extreme events with a given probability of exceedance an example of curve fitted statistical distribution of load effect for 5m span moment and the CDF is given in Figure 5-7 below. A summary of the results of distributions is presented in Table 5-3 and the corresponding CDF's and Statistical Distribution for moment and shear is presented in the Annex (Annex A and Annex B).

Statistical Distribution for 5m span Moment



Normal Distribution

Figure 5-7: Statistical Distribution for 5m Span Moment

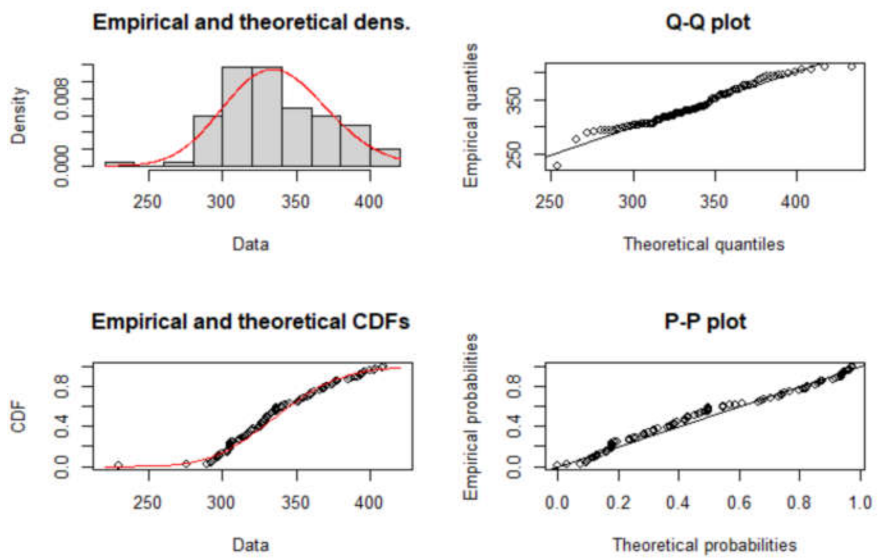
Statistical Distributions for Moment			
Span	**Statistical Distribution**	**Span**	**Statistical Distribution**
5m	Normal	30m	Gama
10m	Log-Normal	35m	Gumbel
15m	Log-Normal	40m	Gumbel
20m	Normal	45m	Gumbel
25m	Gama	50m	Gumbel

Table 5-3: Statistical Distributions for Bending Moment.

The distributions of the Load effects are observed to follow different types of distributions depending on the bridge lengths and load effects encountered, such as the normal, lognormal, Gumbel, Gama, and were observed. This confirms (Carpani 2008; Harmn and Devenport 1979) report which indicates traffic loading effect distribution is not identical.

Statistical distribution of load effect for 5m span moment and the CDF is presented in Figure 5-8.Summary results of the distributions for shear is also given in Table 5-4.

Statistical Distribution for 5m span Shear



Gama Distribution

Figure 5-8: Statistical Distribution for 5m Span Shear.

Statistical Distributions for Moment			
Span	Statistical Distribution	Span	Statistical Distribution
5m	Gama	30m	Normal
10m	Log-Normal	35m	Normal
15m	Gama	40m	Normal
20m	Normal	45m	Normal
25m	Normal	50m	Normal

Table 5-4: Statistical Distributions for Shear.

The main purpose fitting statistical distribution is that to define the type of the parent load effect distributions, it can be observed that the loading effects distributions are different depending on the composition of the loading event.

5.7 Extrapolation using Normal Distribution

The results of the analysis for extrapolated moments and shears are shown in Table 5-5 and 5-6 and plots of the extrapolated moments and shears are presented in Figure5-9 and Figure 5-10 below. The Load Effect for spans is calculated for a return period of 75 years as per AASHTO standards based on the discussions presented on section 2.3.2 and three case scenarios were taken on the selection of the population size. The first is done using the data which was grouped on a monthly basis. Meaning the block size was taken in a time frame of a month, second method we used the Castillo approach which fit the top $2\sqrt{n}$ of distribution of n data. Lastly, the Enright fitting of the top 30% of the data was used and is compared for different spans.

Bending Moment(KNm)			
Span	Block Maxima	Castillo top $2\sqrt{n}$	Enright top 30%
5m	528.05	462.35	479.20
10m	1,351.35	1,176.93	1,219.71
15m	2,539.23	2,514.63	2,439.99
20m	4,032.51	4,145.38	3,947.35
25m	6,193.44	6,349.75	6,075.81
30m	8,451.88	7,806.39	7,861.45
35m	10,848.22	10,040.17	10,063.69
40m	13,443.50	12,560.34	12,503.97
45m	16,432.21	15,585.44	15,427.97
50m	19,442.60	18,664.59	18,510.65

Table 5-5: Extrapolation of Bending Moments using the Normal Distribution

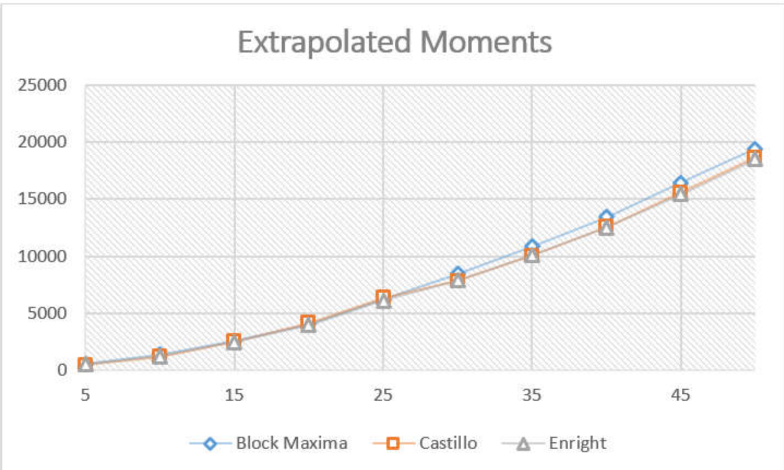


Figure 5-9: Plot of extrapolated Bending Moment using Normal Distribution

As can be seen from the chart above the grouped data on a monthly basis gives a higher result. Whereas the extrapolation of bending moments and shears using the fitting by Castillo and Enright are quite close.

Shear Force(KN)			
Span	Block Maxima	Castillo top $2\sqrt{n}$	Enright top 30%
5m	516.04	498.72	504.25
10m	663.08	624.71	644.69
15m	820.30	787.69	809.06
20m	950.91	919.73	927.03
25m	1,094.55	1,071.24	1,068.61
30m	1,291.23	1,269.15	1,254.85
35m	1,428.15	1,398.38	1,391.06
40m	1,579.95	1,563.11	1,551.58
45m	1,727.24	1,711.10	1,694.91
50m	1,843.33	1,821.75	1,809.34

Table 5-6: Extrapolation of Shear Forces using the Normal Distribution

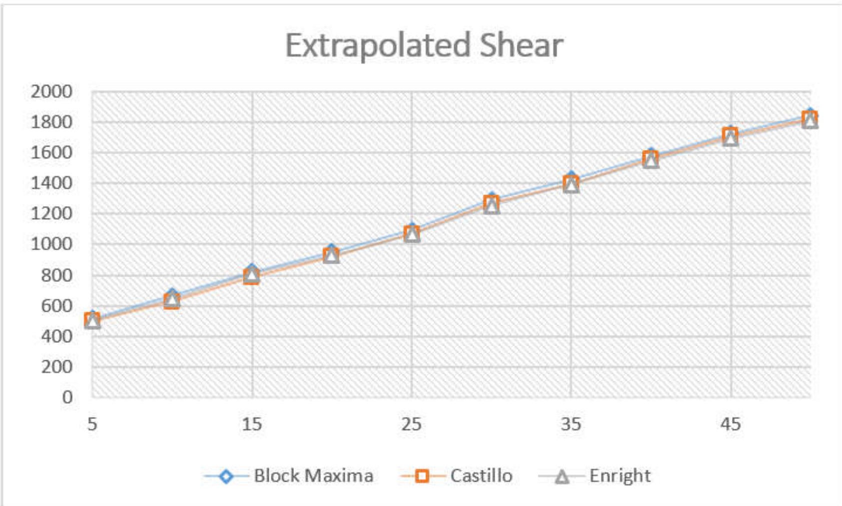


Figure 5-10: Plot of extrapolated Shear Force using Normal Distribution

As can be seen from the chart above extrapolation results from the three populations gave close results. The contrast of extreme events predicted Nowak's method with those found from the extreme value distribution are presented in Section 5.8.2.

5.8 Extrapolation using Chow and Silvakumar methods

As it has been discussed in section 2.3.3 (a) and section 2.3.3 (b) the two methods of extrapolation of loading effects namely the chow and silvakumar method equations have been used. The Load Effect for the spans is calculated for a return period of 75 years based on three case scenarios. One is using the Block Maxima method the data were grouped monthly basis. Meaning the block size was taken in a time frame of a month, second method we used the Castillo approach which fit the top $2\sqrt{n}$ of distribution of n data. Lastly, the Enright fitting of the top 30% of the data was used and is compared for different spans.

The results obtained from the above two methods namely the chow and Sivakumar with variation of population size are presented in Tables 5-7 and 5-8 and Figure 5-11 and Figure 5-12:

Bending Moment(KNm)						
Span	Block Maxima		Castillo top $2\sqrt{N}$		Enright top 30%	
	Chow	Sivakumar	Chow	Sivakumar	Chow	Sivakumar
5m	538.29	540.47	470.49	466.83	491.93	486.21
10m	1,382.96	1,389.69	1,207.36	1,193.69	1,262.92	1,243.52
15m	2,602.09	2,615.49	2,643.86	2,585.83	2,561.02	2,506.67
20m	4,129.40	4,150.05	4,378.44	4,273.78	4,149.15	4,058.54
25m	6,358.77	6,394.00	6,749.89	6,570.21	6,432.16	6,272.14
30m	8,675.87	8,723.61	8,138.03	7,989.11	8,241.54	8,070.86
35m	11,127.73	11,187.29	10,454.55	10,268.47	10,527.68	10,319.32
40m	13,777.95	13,849.22	13,074.73	12,843.74	13,059.52	12,810.05
45m	16,837.30	16,923.63	16,255.27	15,954.49	16,129.57	15,814.51
50m	19,900.63	19,998.24	19,461.62	19,103.71	19,350.47	18,973.35

Table 5-7: Extrapolation of Moment using the Gumbel Distribution

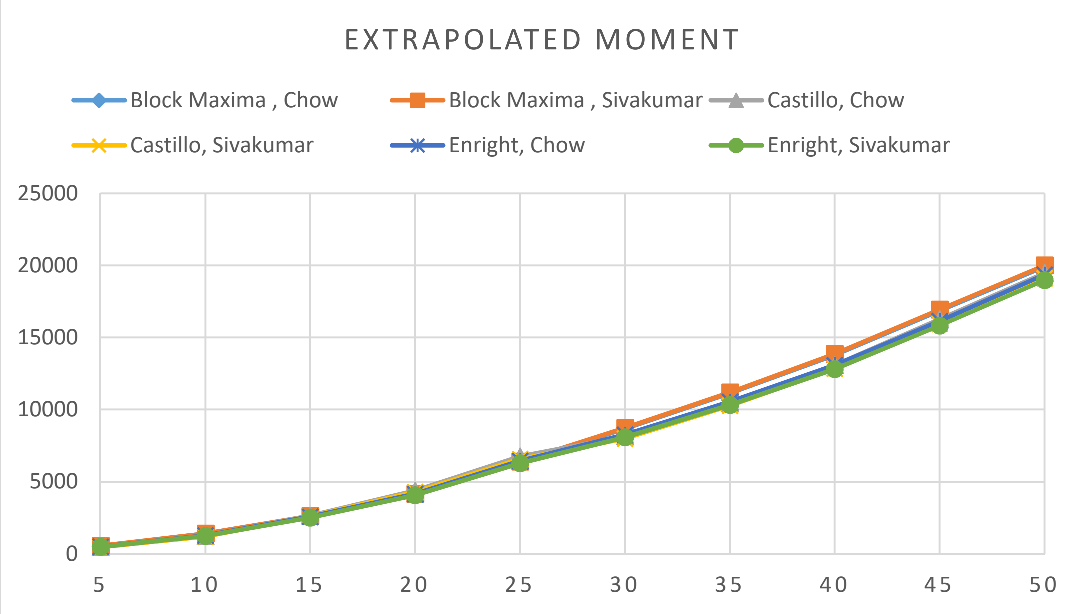


Figure 5-11: Plot of extrapolated Moment using Gumbel Distribution

Shear Force(kN)						
Span	Block Maxima		Castillo top $2\sqrt{N}$		Enright top 30%	
	Chow	Sivakumar	Chow	Sivakumar	Chow	Sivakumar
5m	527.76	530.26	521.6	511.32	529.69	518.27
10m	679.2	682.64	651.93	639.7	678.25	663.18
15m	839.1	843.85	821.6	806.37	851.14	832.25
20m	973.3	977.99	959.99	941.91	973.04	952.38
25m	1,120.61	1,126.17	1,120.68	1,098.48	1,122.32	1,098.20
30m	1,323.12	1,329.92	1,330.10	1,302.73	1,318.54	1,289.94
35m	1,463.06	1,470.50	1,464.09	1,434.58	1,461.28	1,429.75
40m	1,619.57	1,628.01	1,641.03	1,606.04	1,634.03	1,597.01
45m	1,770.95	1,780.26	1,797.89	1,758.92	1,785.94	1,745.06
50m	1,889.11	1,898.87	1,911.89	1,871.41	1,904.64	1,861.85

Table 5-8: Extrapolation of Shear using the Gumbel Distribution

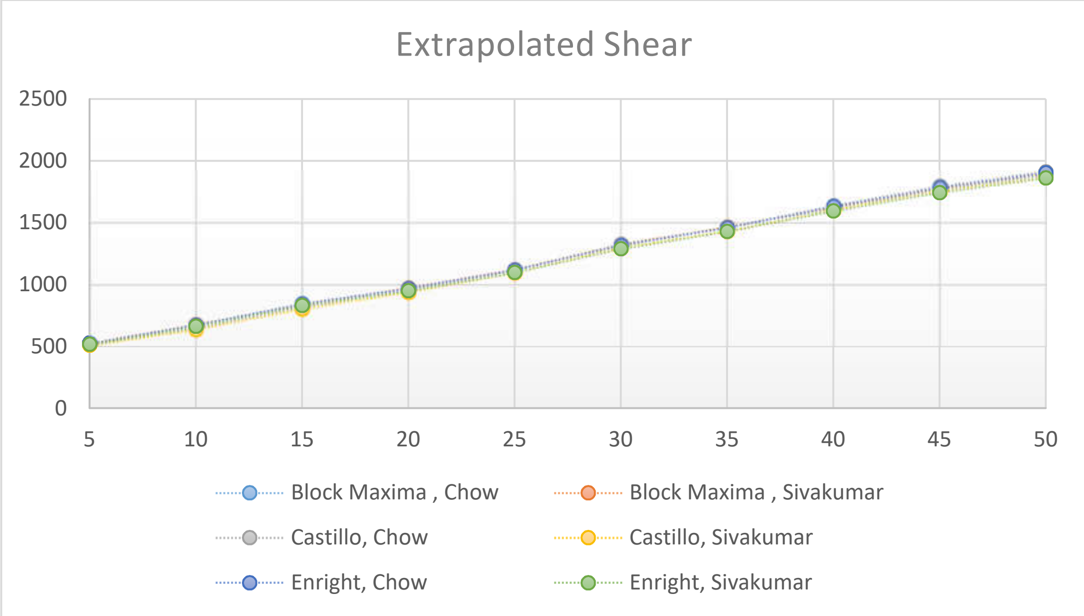


Figure 5-12: Plot of extrapolated Shear using Gumbel Distribution

From the extrapolations, we were able to observe that the load effects obtained using normal distribution were less than the values which were obtained from the Gumbel distribution to the extreme events, and with regards to the data selection method, it was observed that the Block Maxima method gave higher result and this is because when using the Block Maxima method we were not able to absorb the highest values from the total period and due to this fact the standard deviation results were observed to be higher and this gave extreme results in the extrapolation since the standard deviation is multiplied by the frequency factor K_T. The results of the comparison are shown in the histogram presented in Figure 5-13 and 5-14 as well as Tables 5-9 and 5-10. We are able to observe that variation on bending moment and shear Force escalates with the span length.

Bending Moment(KNm)												
	Using Block Maxima				Castillo, Top $2\sqrt{n}$				Enright, Top 30%			
Span	Normal	EV I calculated using the chow equation	EV I calculated using Sivakumar et al	% Difference maximum	Normal	EV I calculated using the chow equation	EV I calculated using Sivakumar et al	% Difference maximum	Normal	EV I calculated using the chow equation	EV I calculated using Sivakumar et al	% Difference maximum
5m	528.05	538.29	540.47	-1.94%	462.35	470.49	466.83	-1.76%	479.2	491.93	486.21	-2.66%
10m	1,351.35	1,382.96	1,389.69	-2.34%	1,176.93	1,207.36	1,193.69	-2.59%	1,219.71	1,262.92	1,243.52	-3.54%
15m	2,539.23	2,602.09	2,615.49	-2.48%	2,514.63	2,643.86	2,585.83	-5.14%	2,439.99	2,561.02	2,506.67	-4.96%
20m	4,032.51	4,129.40	4,150.05	-2.40%	4,145.38	4,378.44	4,273.78	-5.62%	3,947.35	4,149.15	4,058.54	-5.11%
25m	6,193.44	6,358.77	6,394.00	-2.67%	6,349.75	6,749.89	6,570.21	-6.30%	6,075.81	6,432.16	6,272.14	-5.87%
30m	8,451.88	8,675.87	8,723.61	-2.65%	7,806.39	8,138.03	7,989.11	-4.25%	7,861.45	8,241.54	8,070.86	-4.83%
35m	10,848.22	11,127.73	11,187.29	-2.58%	10,040.17	10,454.55	10,268.47	-4.13%	10,063.69	10,527.68	10,319.32	-4.61%
40m	13,443.50	13,777.95	13,849.22	-2.49%	12,560.34	13,074.73	12,843.74	-4.10%	12,503.97	13,059.52	12,810.05	-4.44%
45m	16,432.21	16,837.30	16,923.63	-2.47%	15,585.44	16,255.27	15,954.49	-4.30%	15,427.97	16,129.57	15,814.51	-4.55%
50m	19,442.60	19,900.63	19,998.24	-2.36%	18,664.59	19,461.62	19,103.71	-4.27%	18,510.65	19,350.47	18,973.35	-4.54%

Tables 5-9: Comparison of Normal Distribution and Extreme Value Type I distribution for Bending Moment.

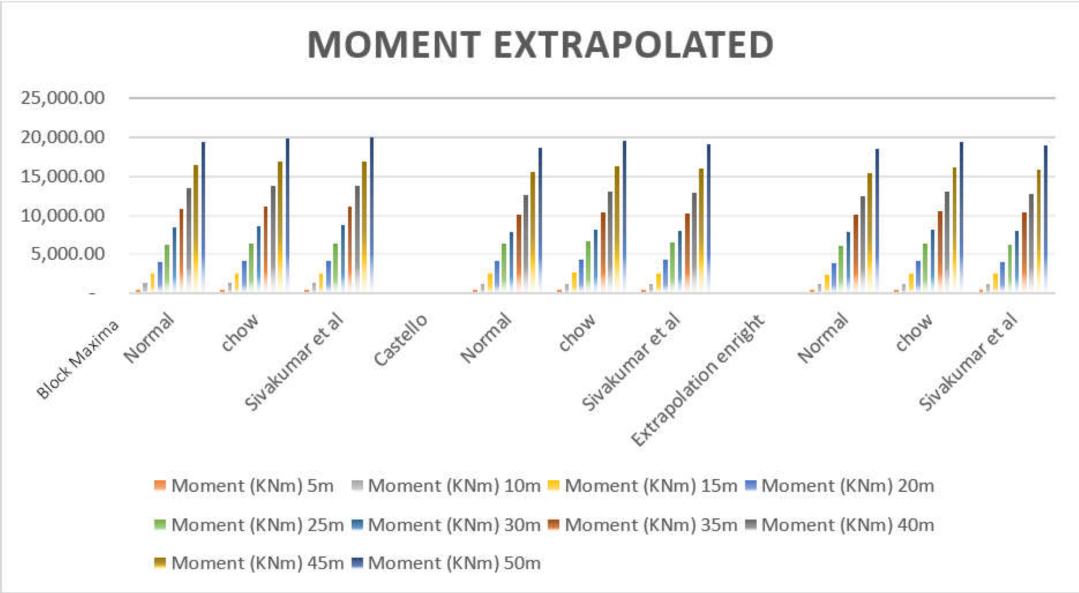


Figure 5-13: Histogram Plot of extrapolated moments using Block Maxima and the POT methods

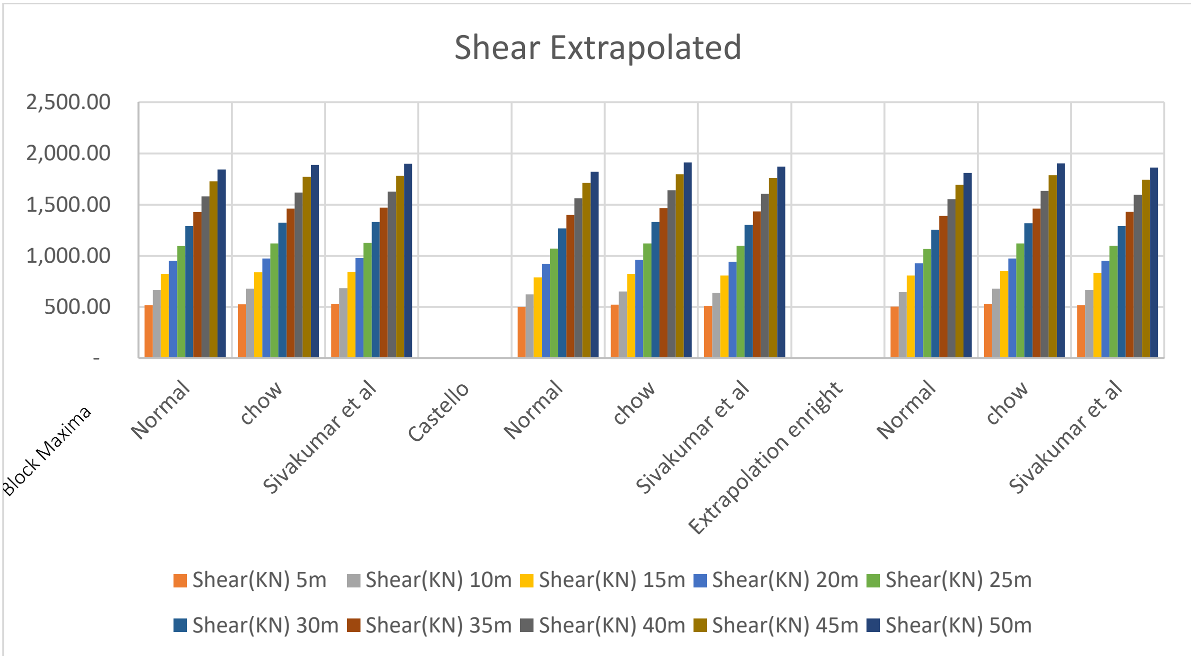


Figure 5-14: Histogram Plot of extrapolated shear using the Block maxima and POT methods.

Span	Shear Force (kN)											
	Using Block Maxima				Castillo, Top $2\sqrt{n}$				Enright, Top 30%			
	Normal	EV I calculated using the chow equation	EV I calculated using Sivakumar et al	% Difference maximum	Normal	EV I calculated using the chow equation	EV I calculated using Sivakumar et al	% Difference maximum	Normal	EV I calculated using the chow equation	EV I calculated using Sivakumar et al	% Difference maximum
5m	516.04	527.76	530.26	-2.27%	498.72	521.6	511.32	-4.59%	504.25	529.69	518.27	-5.05%
10m	663.08	679.2	682.64	-2.43%	624.71	651.93	639.7	-4.36%	644.69	678.25	663.18	-5.21%
15m	820.3	839.71	843.85	-2.37%	787.69	821.6	806.37	-4.30%	809.06	851.14	832.25	-5.20%
20m	950.91	973.23	977.99	-2.35%	919.73	959.99	941.91	-4.38%	927.03	973.04	952.38	-4.96%
25m	1,094.55	1,120.61	1,126.17	-2.38%	1,071.24	1,120.68	1,098.48	-4.62%	1,068.61	1,122.32	1,098.20	-5.03%
30m	1,291.23	1,323.12	1,329.92	-2.47%	1,269.15	1,330.10	1,302.73	-4.80%	1,254.85	1,318.54	1,289.94	-5.08%
35m	1,428.15	1,463.06	1,470.50	-2.44%	1,398.38	1,464.09	1,434.58	-4.70%	1,391.06	1,461.28	1,429.75	-5.05%
40m	1,579.95	1,619.57	1,628.01	-2.51%	1,563.11	1,641.03	1,606.04	-4.98%	1,551.58	1,634.03	1,597.01	-5.31%
45m	1,727.24	1,770.95	1,780.26	-2.53%	1,711.10	1,797.89	1,758.92	-5.07%	1,694.91	1,785.94	1,745.06	-5.37%
50m	1,843.33	1,889.11	1,898.87	-2.48%	1,821.75	1,911.89	1,871.41	-4.95%	1,809.34	1,904.64	1,861.85	-5.27%

Tables 5-10: Normal Distribution vs. Extreme Value Type I distribution Comparisons for Shear Force.

Extrapolation of the Loading Effects gave the opportunity in providing to the sensitivity values of the results to the assumed 75 years return period. A sensitivity analysis is taken in assessing the variation of the population size of the extreme events.

From Extrapolated values a comparative study was done on the results and we were able to observe that the extreme value distributions gave higher results than the normal distributions shows that the variation of bending moment and shear is reliant on the span length.

5.9 Comparison of AASHTO/ERA Loads and Weighing Station Loads

Paramount purpose of this work is established on comparison of the loading effects generated from the collected data from the weighing stations and the loads calculated from the ERA bridge design code. This method helps to evaluate the AASHTO/ERA live load models with the loading effects observed on the site. To make comparisons we used the factored loads versus the actual loads induced. The results and magnitude of differences are presented in Table 5-11 and 5-12 and a graphical comparison is also presented in Figure 5-15 and Figure 5-16

Bending Moment(KNm)				
Span	Actual Data	HL-93 Truck un-factored	HL-93 Truck factored	% Difference Actual: HL-93
5m	486.21	241.10	421.93	15.23%
10m	1,243.52	601.25	1,052.18	18.19%
15m	2,506.67	1,107.58	1,938.27	29.33%
20m	4,058.54	1,714.44	3,000.28	35.27%
25m	6,272.14	2,380.56	4,165.98	50.56%
30m	8,070.86	3,105.14	5,433.99	48.53%
35m	10,319.32	4,225.59	7,394.78	39.55%
40m	12,810.05	5,486.31	9,601.04	33.42%
45m	15,814.51	6,805.20	11,909.10	32.79%
50m	18,973.35	8,182.25	14,318.94	32.51%

Table 5-11: Bending Moment Results

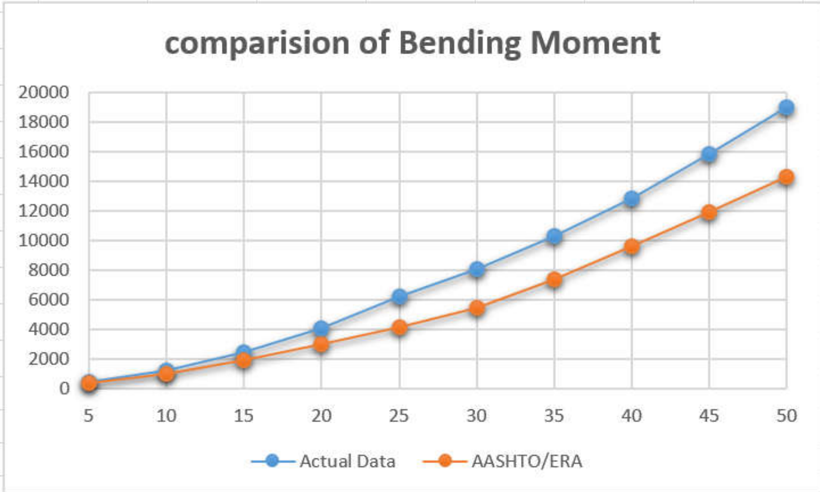


Figure 5-15: Comparison of Bending Moment Actual data Vs. AASHTO/ERA

From Table 5-11 the percentage of difference of the actual data versing the AASHTO/ERA loading we can see that for 20 to 25m span the percentage of overloading increases from 35.27% to 50.56 %(i.e.15% increment in percentage). This occurrence happens due to the fact that, when considering the case of 20m span a 4-Axle truck loading and configuration is governing the moment while for 25m span a 6-Axle truck is the governing case for the moment.

Shear Force(KN)				
Span	Actual Data	HL-93 Truck un-factored	HL-93 Truck factored	% Difference Actual: HL-93
5m	518.27	216.43	378.76	36.83%
10m	663.18	279.76	489.58	35.46%
15m	832.25	333.59	583.78	42.56%
20m	952.38	374.18	654.82	45.44%
25m	1,098.20	431.61	755.32	45.40%
30m	1,289.94	505.89	885.30	45.71%
35m	1,429.75	569.65	996.89	43.42%
40m	1,597.01	623.29	1,090.76	46.41%
45m	1,745.06	682.85	1,194.98	46.03%
50m	1,861.85	752.04	1,316.07	41.47%

Table 5-12: Shear Force Results

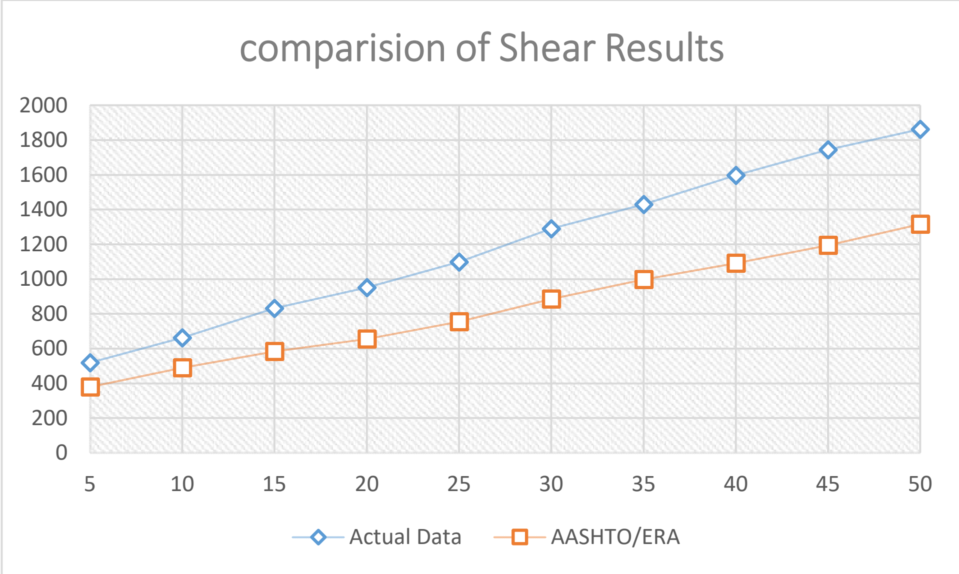


Figure 5-16: Comparison of Shear Force Results from Actual data Vs. AASHTO/ERA Loading

5.10 Ethiopian Bridges Alternative Live Load Models

The above analysis shows that there is a demand for developing a revised live load model that reflects the actual traffic loadings at the ground in the Ethiopian context. Thus, this study tries to develop an alternative live load model that will help to overcome the emerging problem. The development of the live load model involves calibration against target values. The results obtained from the simulation of load effects could be used to calibrate new-live load models based on truck weights recently changed. The targeted values were taken from the extrapolated loading effect results.

Thus, a simplified live load model was developed to reproduce the critical loading effects induced by simulated vehicles. The load model has been validated by comparing the extrapolated values obtained from the simulations with the simulation results of the truck model.

In this work, three alternative live load models are presented:

- The first is method assumes the increase of the load modifier value without changing the AASHTO/ERA truck model. The calculation was done by dividing the actual data with the un-factored truck loading of AASHTO/ERA truck

Span	Shear Force(KN)			Bending Moment(KNm)		
	Actual Data	AASHTO/ERA un-factored	Ratio of actual data and AASHTO/ERA	Actual Data	AASHTO/ERA un-factored	Ratio of actual data and AASHTO/ERA
5m	518.27	216.43	2.39	486.21	241.10	2.02
10m	663.18	279.76	2.37	1,243.52	601.25	2.07
15m	832.25	333.59	2.49	2,506.67	1,107.58	2.26
20m	952.38	374.18	2.55	4,058.54	1,714.44	2.37
25m	1,098.20	431.61	2.54	6,272.14	2,380.56	2.63
30m	1,289.94	505.89	2.55	8,070.86	3,105.14	2.60
35m	1,429.75	569.65	2.51	10,319.32	4,030.04	2.56
40m	1,597.01	623.29	2.56	12,810.05	5,115.98	2.50
45m	1,745.05	682.85	2.56	15,814.51	6,499.77	2.43
50m	1,861.85	752.04	2.48	18,973.35	8,059.49	2.35
Sample Mean			2.50	**Sample Mean**		2.38
Standard Deviation			0.07	**Standard Deviation**		0.21
Lower Limit 95% Confidence			2.46	**Lower Limit 95% confidence**		2.25
Upper Limit 95% Confidence			2.54	**Upper Limit 95%Confidence**		2.51

Table 5-13: Ratio of AASHTO/ERA Truck Model for Moment and Shear.

From these results, we can see that the ratio of actual data versus the AASHTO/ERA loading the lower limits is even higher than the load modifier value of 1.75 given in the ERA Bridge Design Manual. As it can be seen in table the standard deviation of the factor of safety for the moment and shear is small. Therefore, this method proposes the use of a load modifier value of 2.5 to be used for the calculation of Shear and moment.

- The second method proposes the use of AASHTO truck/tandem loading with the same factor of safety but with a new equation for a uniformly distributed load. From the output of the program which was prepared for this study different values of lane load (w) values were obtained and from those results, a regression analysis was done and the selected function was a power function and for spans up to 50m the Equation 5.1 could be used for the determination of UDL.

$$w = 96.8(\frac{1}{L})^{0.24}$$ Equation 5.1

Where w is expressed in $\frac{kN}{m}$, and L is given in meter.

- The third proposed live load model is obtained from the simulation program the loading truck was based on the most frequent loading combination and it can replace the AASHTO truck/tandem loadings. The design truck has the form of a combination between a series of point loads and a uniformly distributed load. The load effect results of the design models found from the simulation software the loadings are initially based on frequently occurring trucks and by using iterative procedure for the inter axle spacing we were able to find the following truck configuration to be optimal result. The results were checked for bending moment and shears of the extrapolated results and the following truck was proposed with the load modifier value of 1.75 as per the ERA Bridge Design Manual.

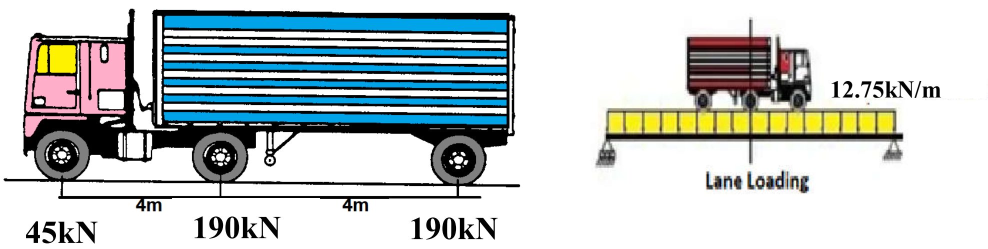


Figure 5-17: Proposed Notational Model consisting of point load and lane load

In sum, static loading effects have been studied through the use of influence lines. In generation and simulation of the traffic, a program was written in python. The program allows the passage of vehicles of any truck configuration using the traffic characteristics derived from the measured data from the weighing stations. For any type of bridge length and loading effects to be considered to be studied. The program is capable of recording the maximum load effect value corresponding to the given crossing effect identified (i.e. the presence of occurrence of more than one truck in a bridge). In addition to this, the maximal or critical loading events are recorded and outputted for the later analysis.

Simulations were performed for all the gathered traffic files typical critical loading events were derived from the collected bridge lengths. Then the loading effects were fitted to their representative distributions and we were able to notice that the loading effect distributions obtained from the simulated data for different spans followed a mixture of several distributions, this could be due to the effects caused by the loading of axles and axle sets.

This study in this chapter shows that in finding the most appropriate means for the extrapolation of the loading effects of heavy vehicles on simply supported beams confirmed that the use of Extreme Value type I (Gumbel) distribution is preferred than the use of the normal distribution.

When applying the Extreme value type I distribution, we were able to see the extrapolated load effects are found to be sensitive to the selected tail size i.e. the population of extreme events. These effects were analyzed using the Block Maxima and the Peak over Threshold methods, and in selecting the threshold values the assumptions by Castillo and Enright were checked and results were presented.

6. CONCLUSION AND RECOMMENDATION

During the project journey efforts were exerted to assess the existing live load model recommended by the ERA Bridge Design Manual. The assessment of the load effects were calculated based on the data obtained from different weighing stations, namely Modjo, Sululta, and Semera. The data from those stations were chosen because of their proximity of having to experience a high volume of heavy vehicles. This efforts helps to collect quality of data, which reduce the uncertainty of traffic loading.

The study focus on analysis and modeling of short to medium span bridges and some of the recommendations based on our research are the following:

- Different live load models like THM7 South African model, The AASHTO bridge live load model, The BS5400, the CAN\CSA-S6-00: the Canadian highway bridge design code, and the Eurocode were explored and the best to our context found to AASHTO with some proposed propositions as shown below.
- The load effects found from this study show an important reference for critically reviewing the live load models given in Ethiopian Bridge Design Manual. It was observed that the loading effects obtained from the actual trucks weighted in the weighing stations gave higher results than the given AASHTO/ERA truck model given in the bridge design manual. Thus, based on the analysis three live load models which reflect the actual traffic loadings are proposed, as follows :
    1. Increasing the value of load modifier without changing the AASHTO/ERA truck model to 2.5 for Shear and moment.
    2. Using of AASHTO truck/tandem loading with the same factor of safety but with a new equation for uniformly distributed load given in Equation 5.1
    3. Using the alternative design truck and lane load given in Figure 5-17
- Analysis on the weighing station data shows that the weight on different axels were different from that of gross vehicle weights, which demands to consider the overloading of axles if any than looking overloading of the whole vehicle.
- In reviewing the Load effects different types of distributions were observed depending on the bridge lengths and load effects encountered. For example: Normal, Lognormal, Gumbel, and Gama distributions were observed to be fitted. Based on these models the Load Effect have been calculated for a return period of 75 years using normal and extreme value distributions.

- During this study process, it was observed that the Gumbel (EV Type I) distribution was found to be the most suitable means of extrapolation of the loading effects induced by heavy vehicles over simply supported bridges. When using the Gumbel distributions we tried to vary the population size of the extremes using several methods proposed by several authors such as the Block Maxima, and in using the POT method we used the assumptions of Castillo and Enright and what we were able to observe is that the extrapolated events are quite sensitive to the population size of the extremes.

6.1 Recommendations

- The finding of the study indicates that there is a significant gap between the AASHTO/ERA model used and the actual truck loadings obtained from the study, which shows the need to revise the axle load weight control systems.
- Weigh-in-Motion systems would help to get more accurate loading data from the locations without any human interference , thus with growing economy in the country it would be worth to think and have the system in place by ERA to enhance the decision making process based on more accurate , dependable and reliable data.
- For health monitoring and bridge management purpose the data obtained from Static weighing stations are found to be very important. Hence, it is recommended that the client ERA should increase the use of weighing stations at all critical locations to get more accurate data, which in return helps to obtained better and accuracy of results.
- This thesis focuses on the short to medium span bridges (span up to 50m) on live load models only, however there is a need to make similar research on long-span bridges that are bridges greater than 50m span in order to have a full picture.

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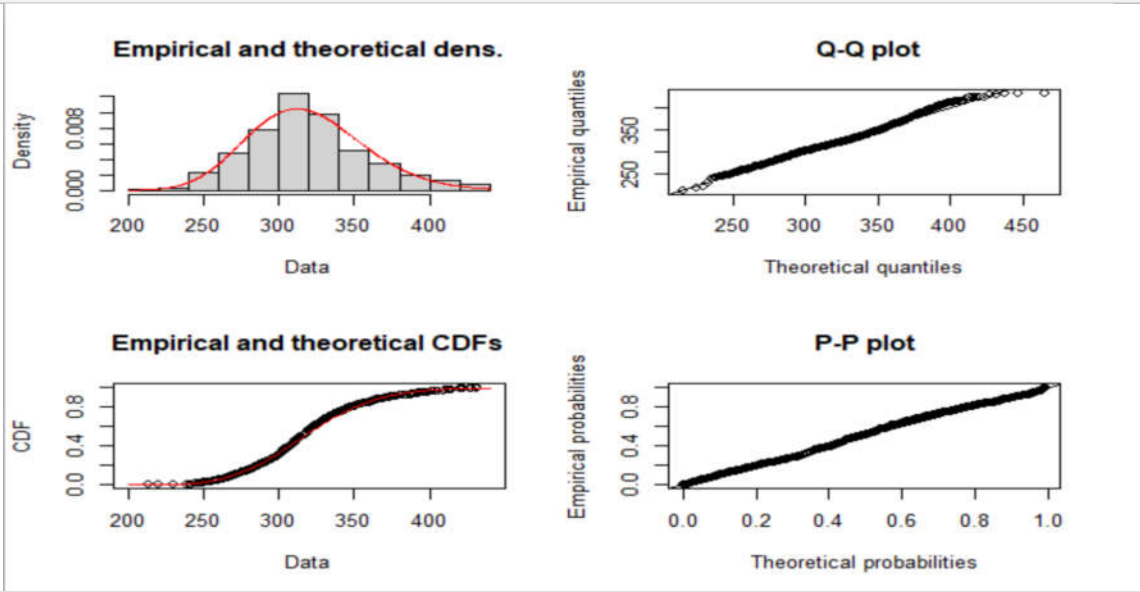
8. ANNEX

8.1 Annex A

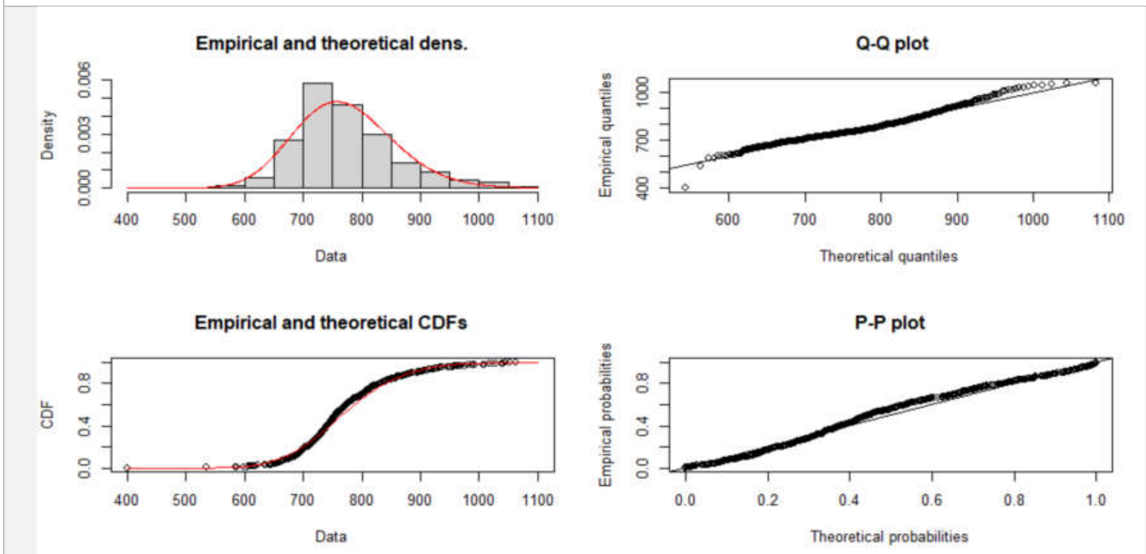
Statistical Distributions of Moments for Different Spans

For 5m span Moment



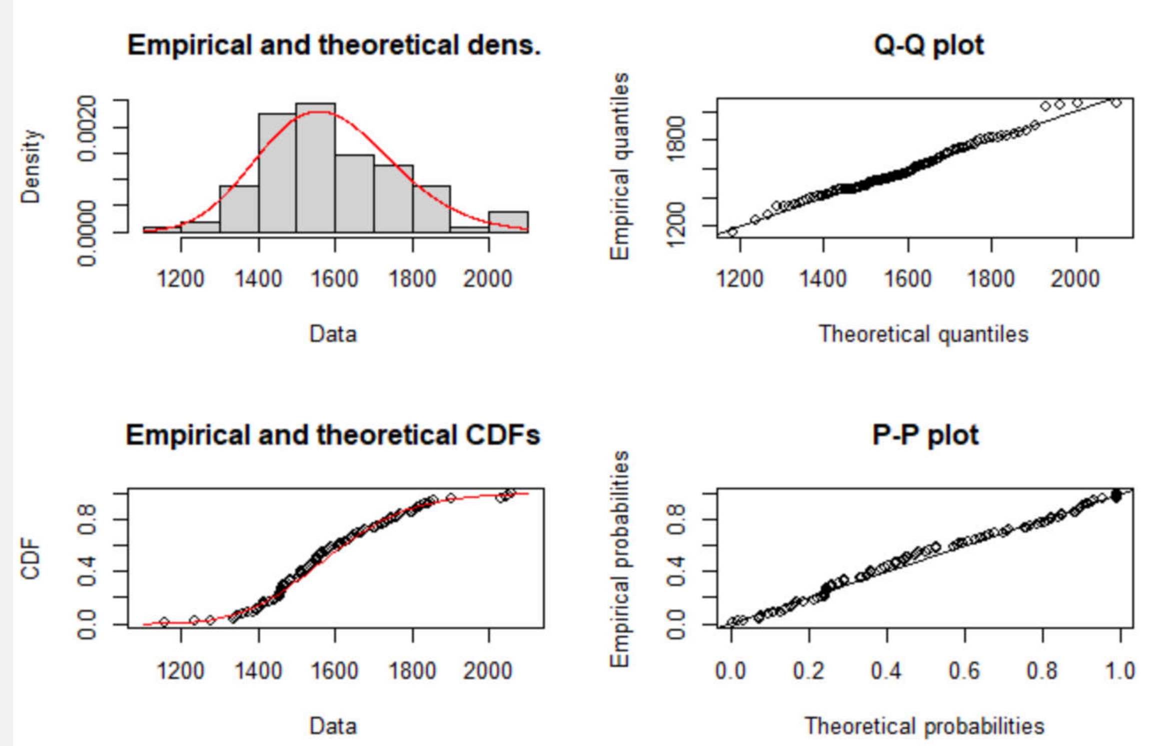
Normal Distribution

For a 10m span Moment



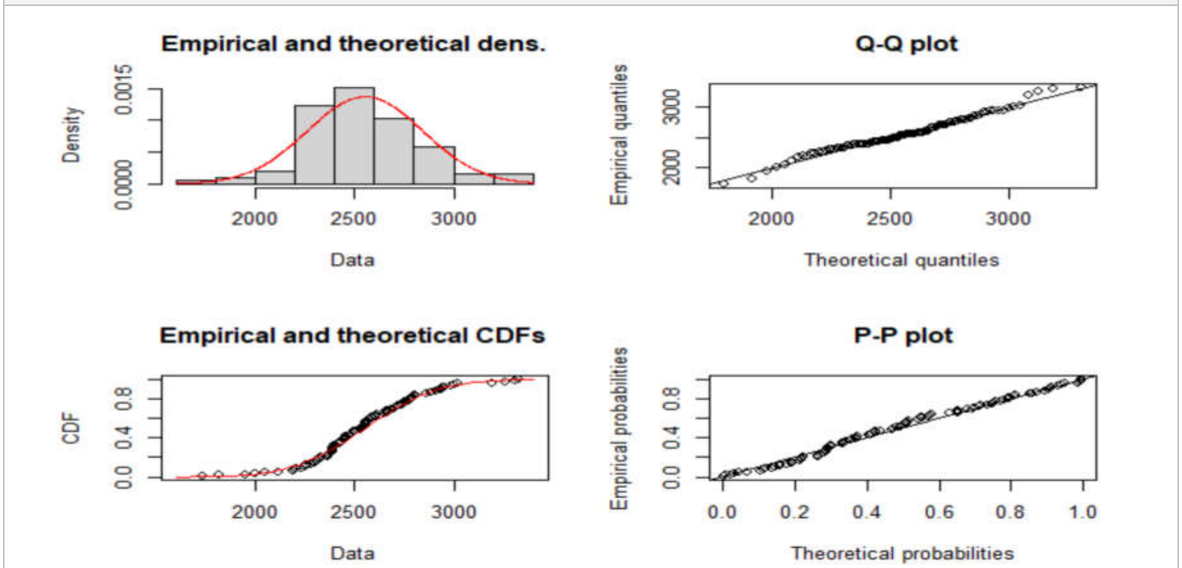
Log-Normal Distribution

For 15m span Moment



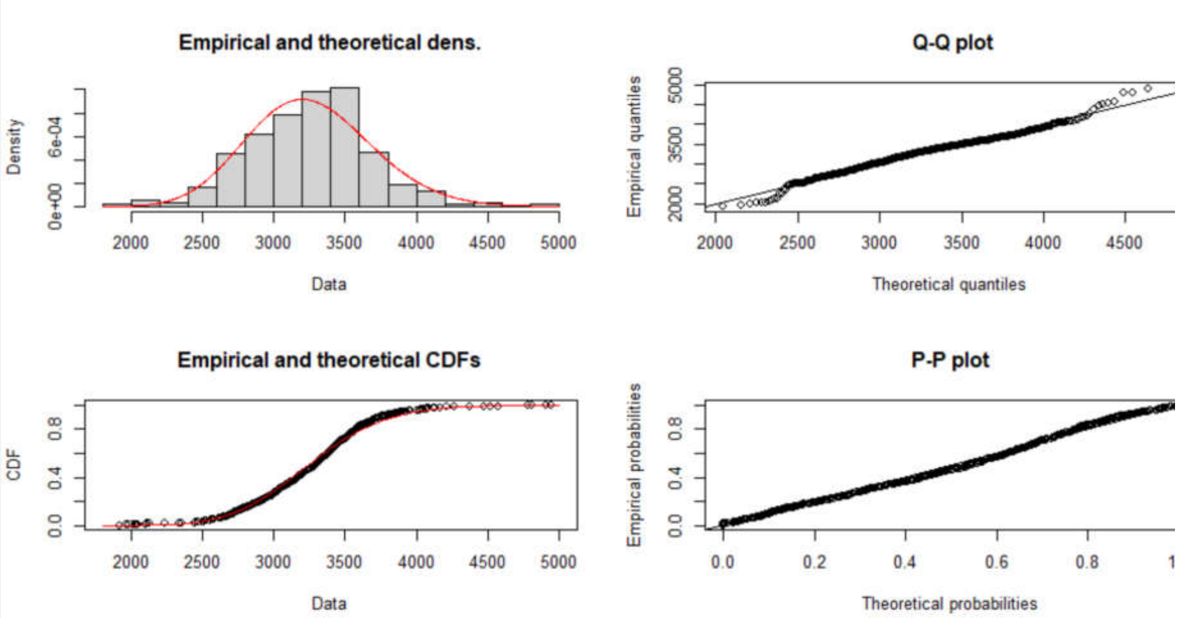
Log-Normal Distribution

For a 20m span Moment



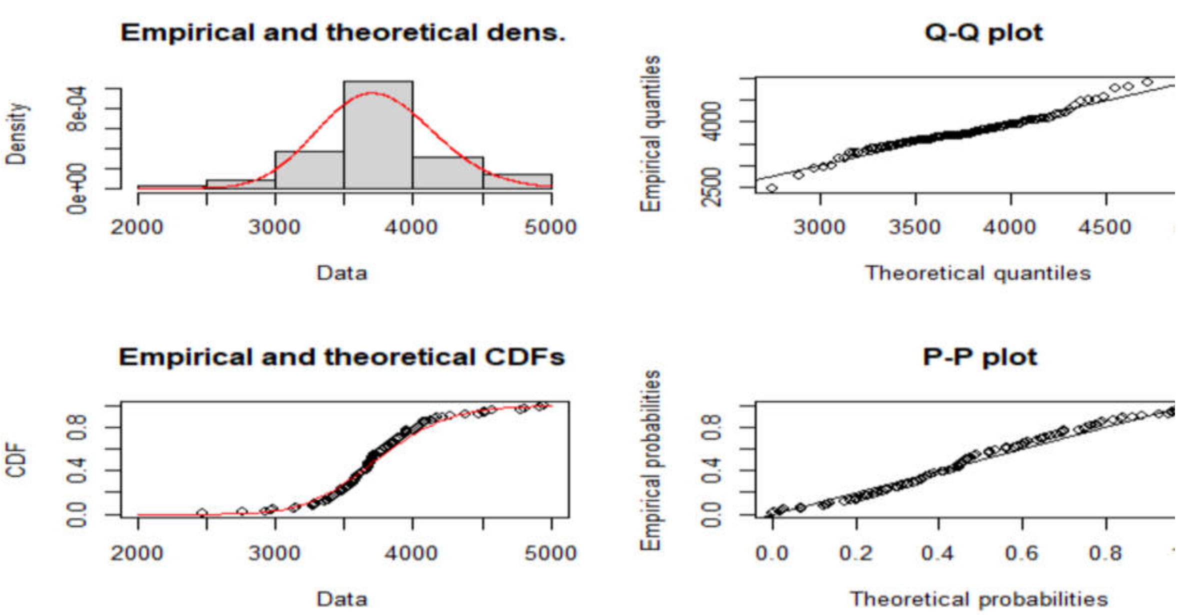
Normal Distribution

For 25m span Moment



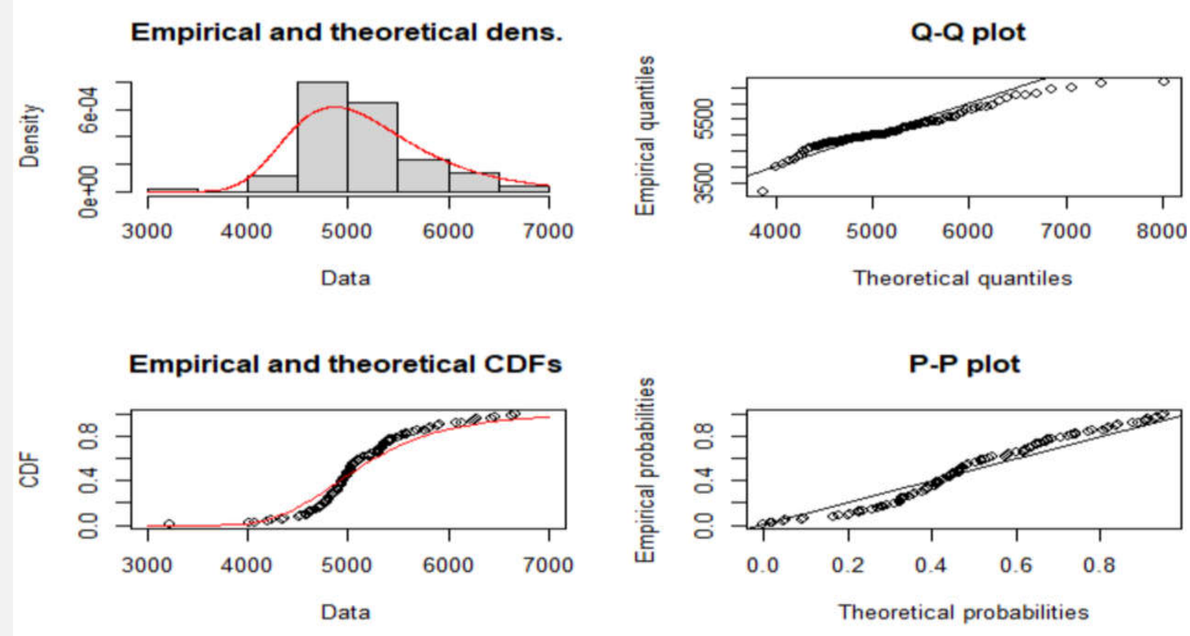
Gama Distribution

For a 30m span Moment



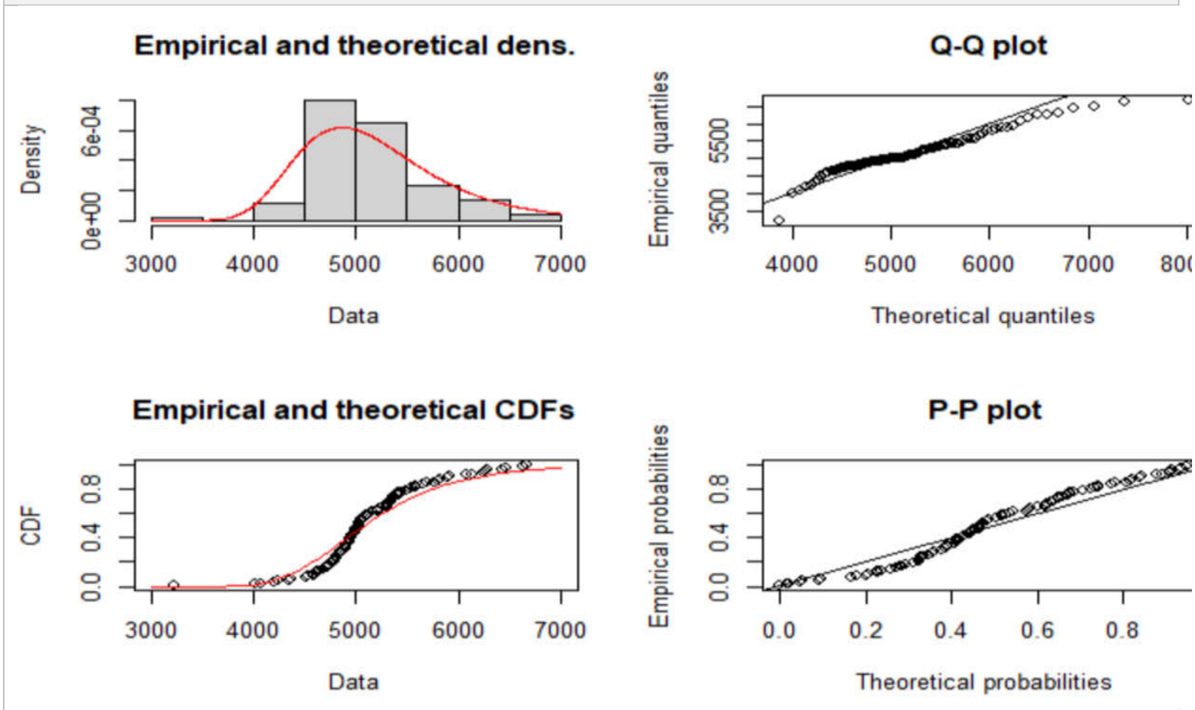
Gama Distribution

For 35m span Moment



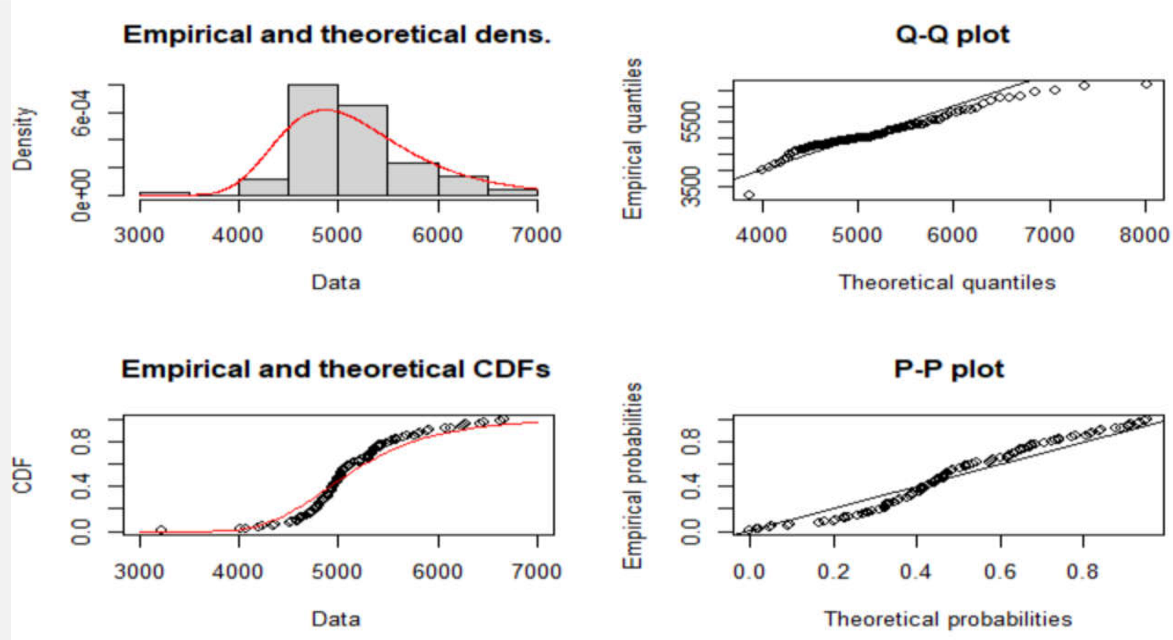
Gumbel Distribution

For 40m span Moment



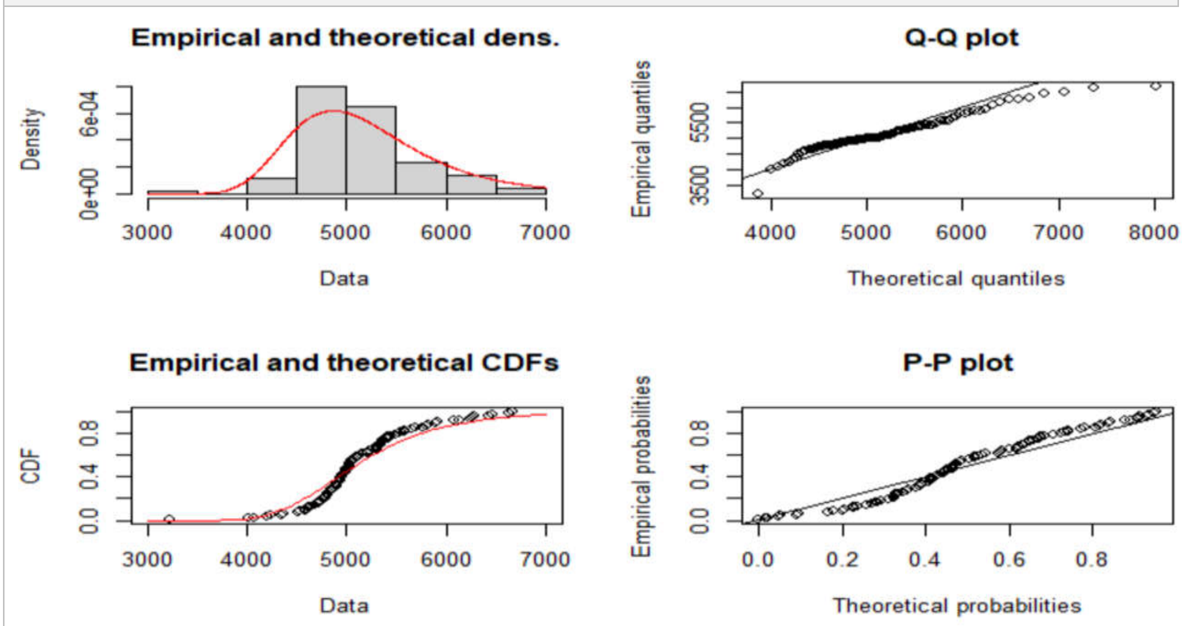
Gumbel Distribution

For a 45m span Moment



Gumbel Distribution

For 50m span Moment

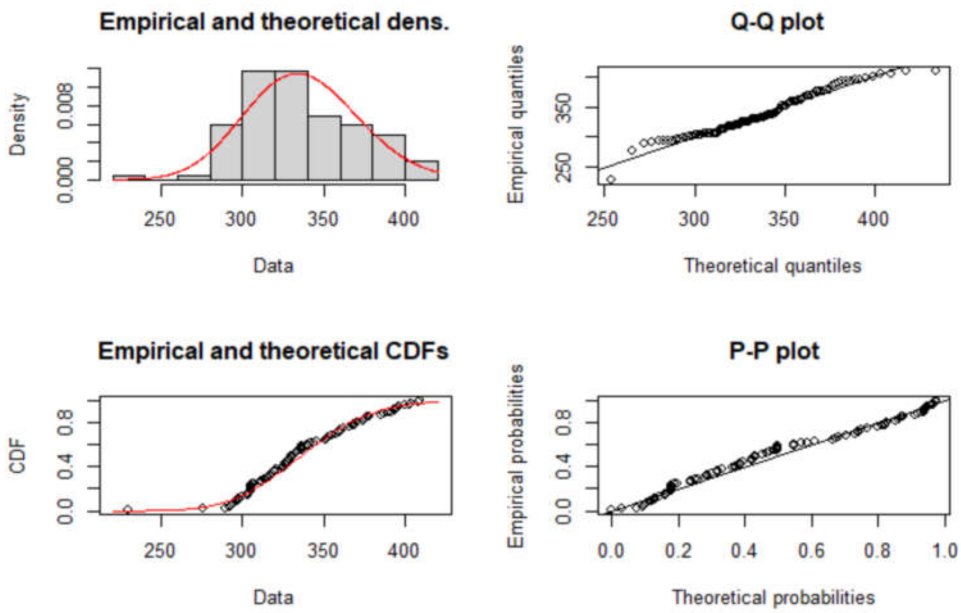


Gumbel Distribution

8.2 Annex B

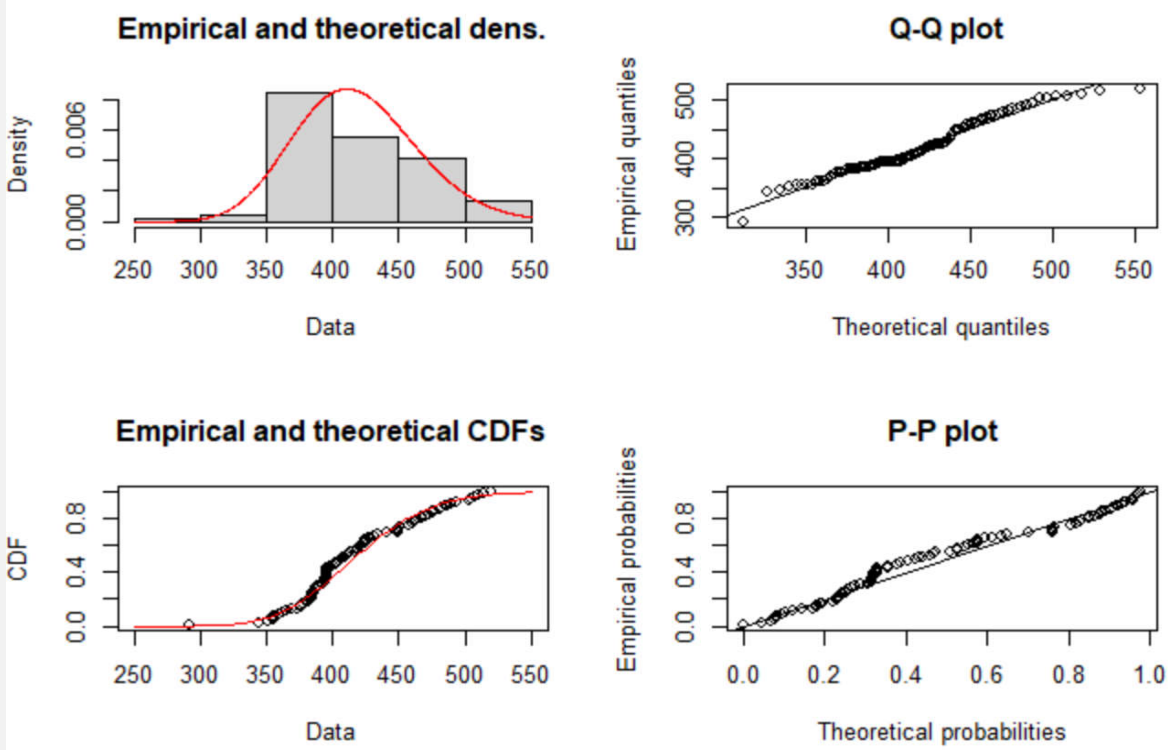
Statistical Distributions of Shear for Different Spans

For 5m span Shear



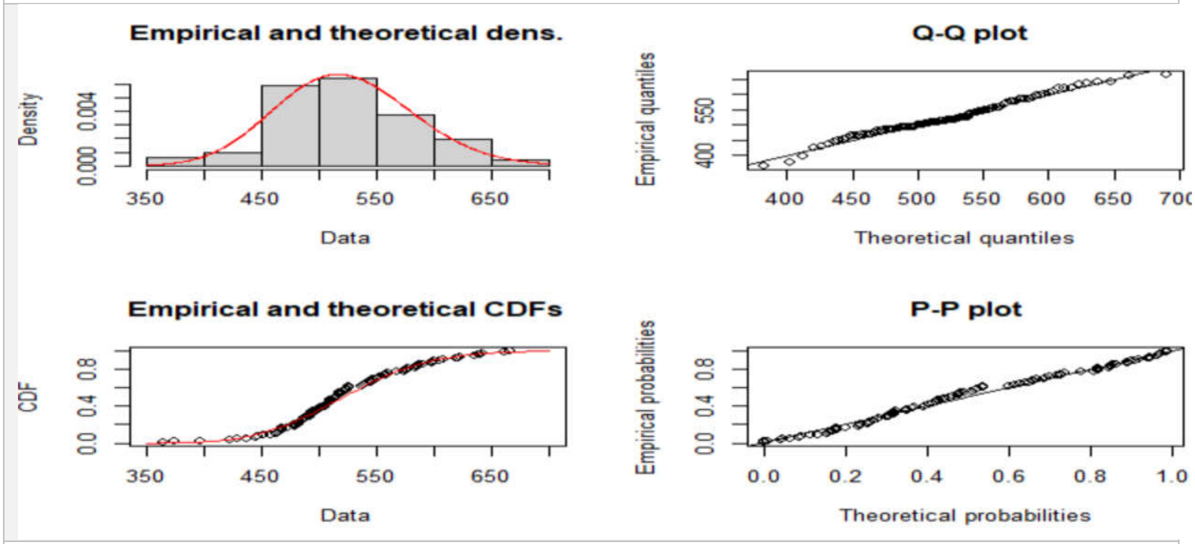
Gama Distribution

For 10m span Shear



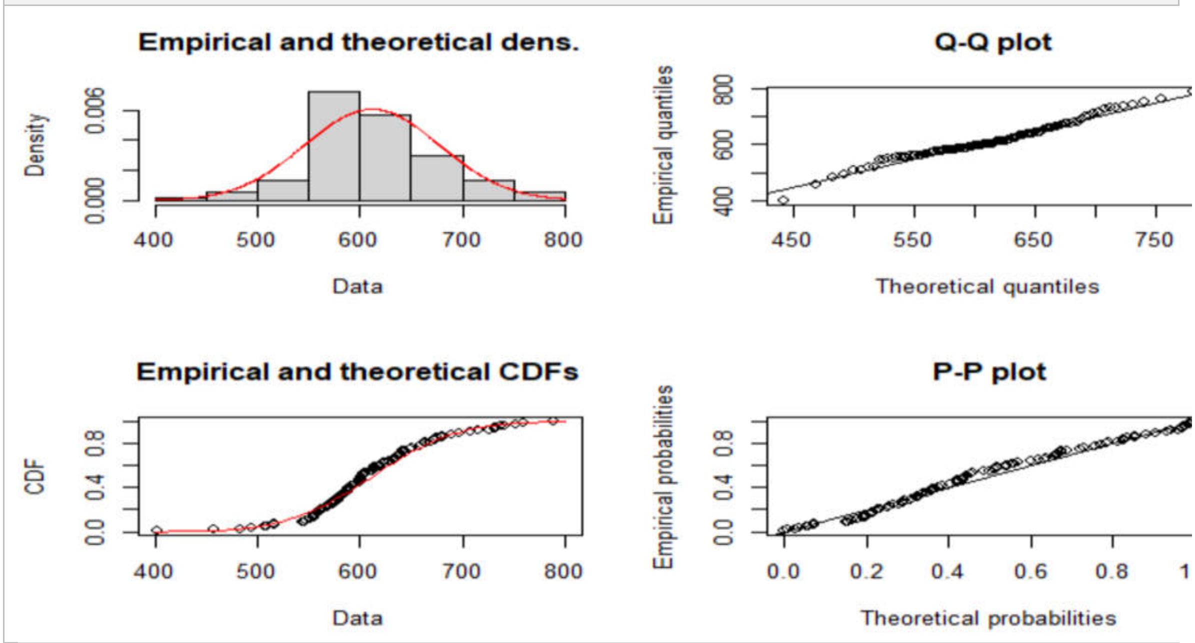
Log-Normal Distribution

For 15m span Shear



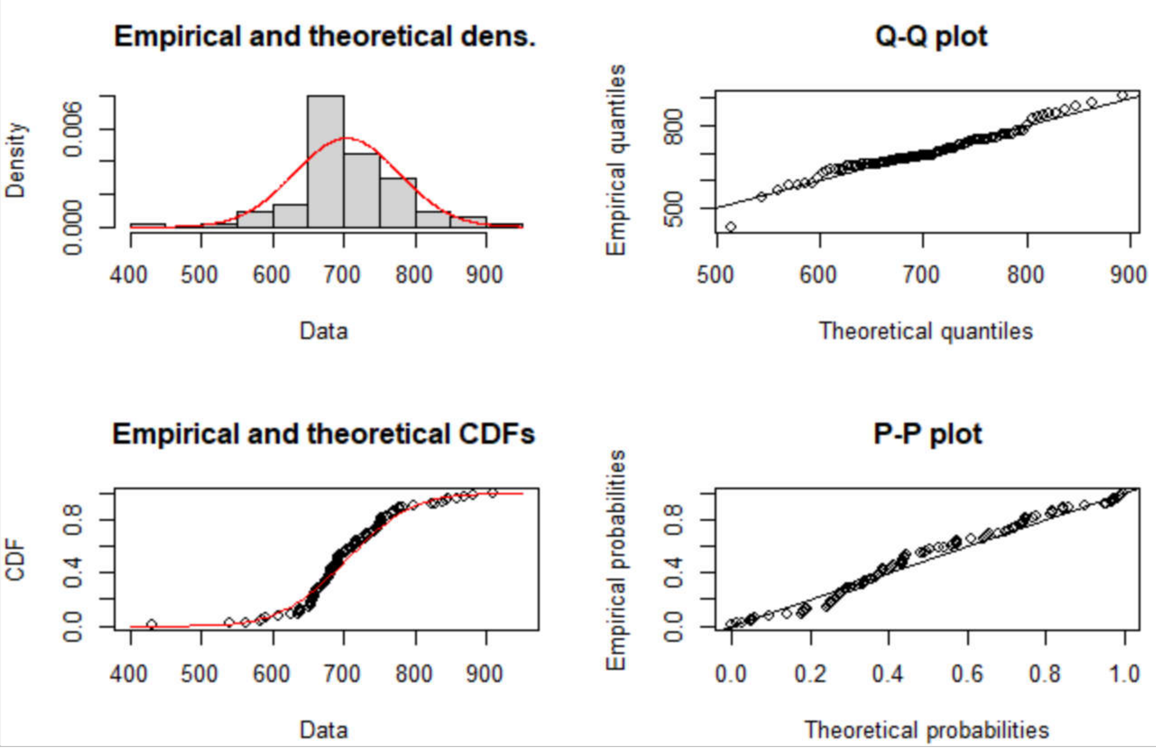
Gama Distribution

For 20m span Shear



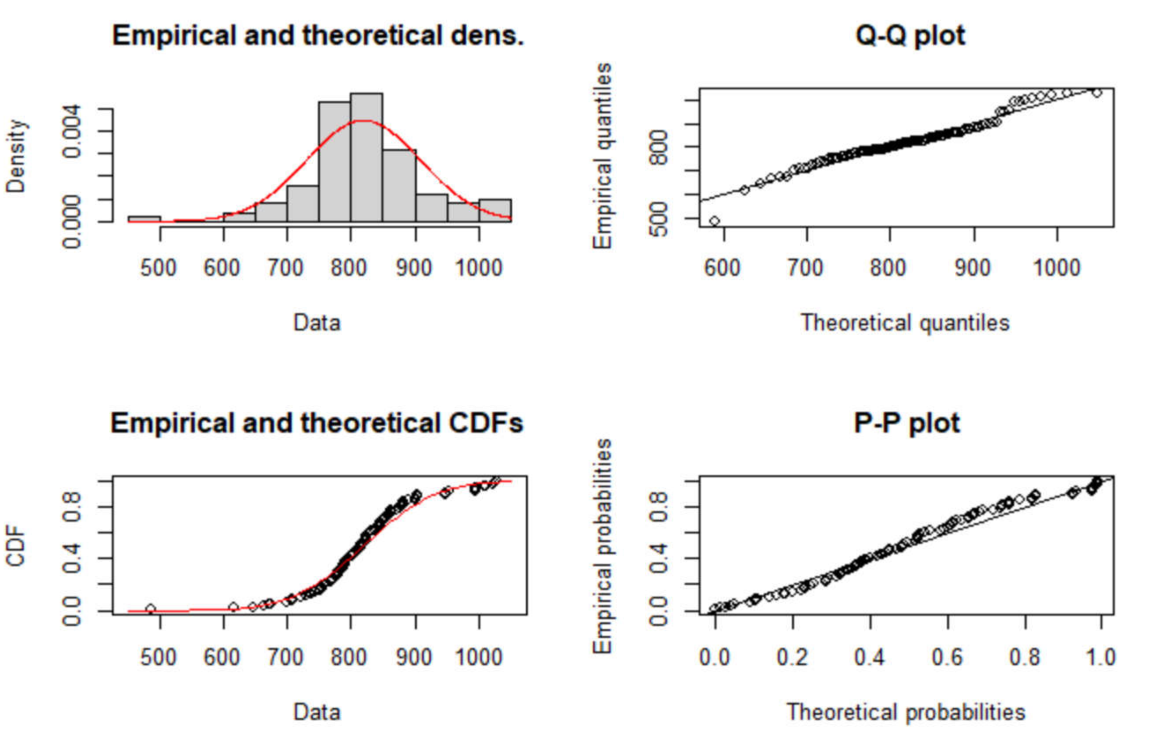
Normal Distribution

For 25m span Shear



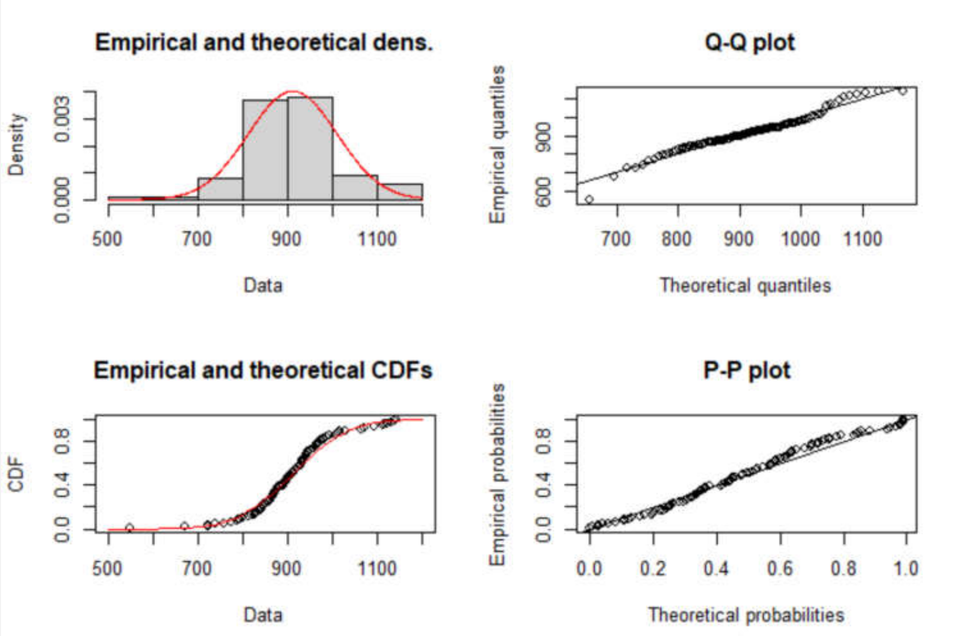
Normal Distribution

For 30m span Shear



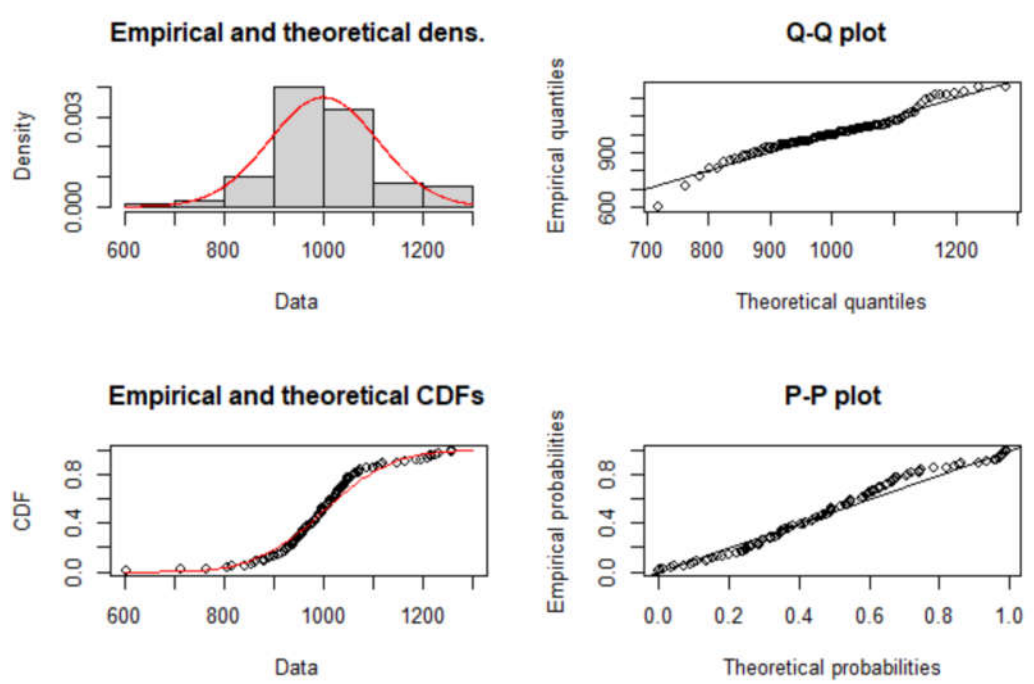
Normal Distribution

For 35m span Shear



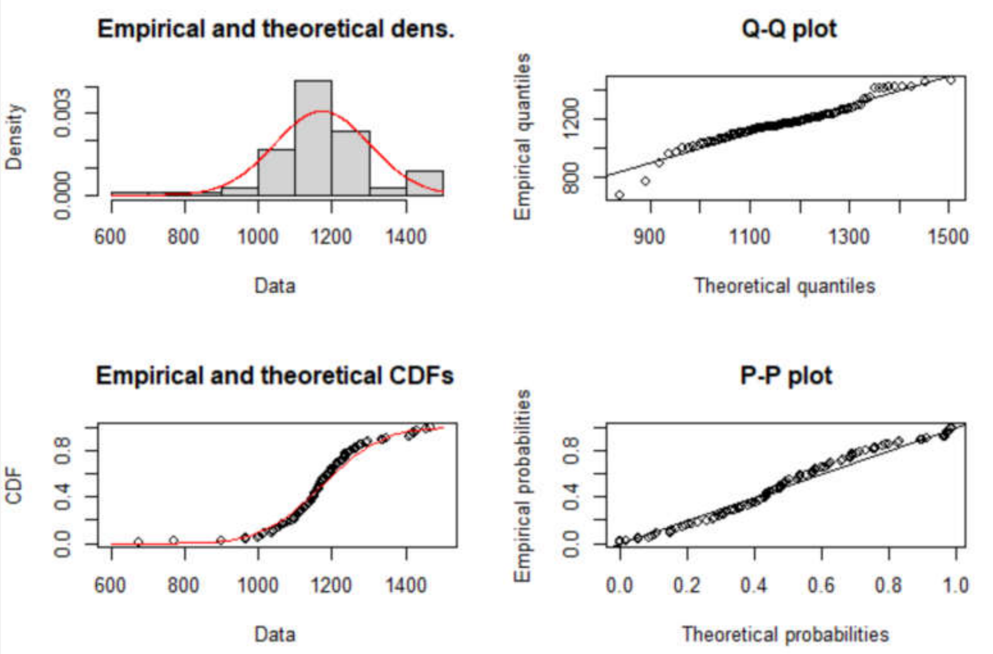
Normal Distribution

For 40m span Shear



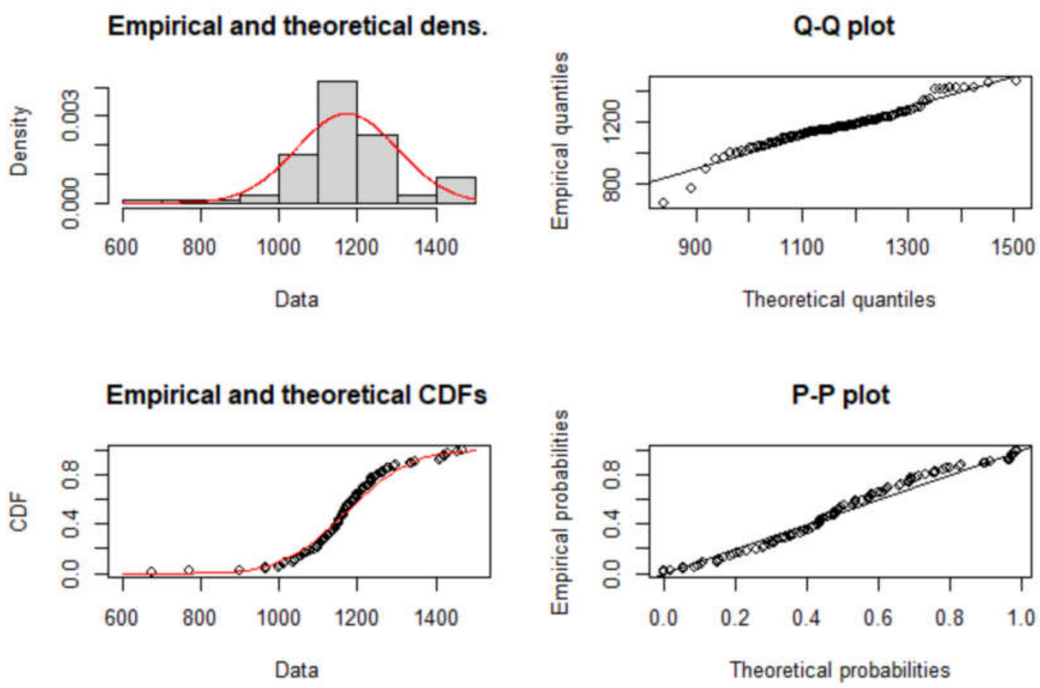
Normal Distribution

For 45m span Shear



Normal Distribution

For 50m span Shear



Normal Distribution