

**REVIEW OF SOUTH AFRICAN LIVE LOAD MODELS FOR TRAFFIC  
LOADING ON BRIDGE AND CULVERT STRUCTURES USING  
WEIGH-IN-MOTION (WIM) DATA**

**WRITTEN BY**

**JOHN ROBERT BEVERIDGE ANDERSON**

**BEng (Hons) PrEng MSAICE**

**A thesis submitted in partial fulfilment of the requirements for the degree of**

**MASTER OF SCIENCE (STRUCTURES)**

**In the**

**FACULTY OF ENGINEERING AND THE BUILT ENVIRONMENT**

**UNIVERSITY OF CAPE TOWN**

**February 2006**

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## ABSTRACT

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This thesis uses the axle weights and axle spacings of heavy vehicles recorded by weigh-in-motion (WIM) sensors to calculate the load effects on single lane, simply supported structures spanning up to 30m. The main objective was to compare the load effects caused by the recorded vehicles with those calculated using TMH7 Part 2 and the alternative live load models proposed in subsequent research. Through the probabilistic analysis of the truck survey data, the thesis predicts the magnitude of extreme events that may occur within a bridge structure's design life. The results reinforce the deficiencies of TMH7 Part 2's NA loading curve to cater for normal traffic conditions on spans of 10m and less. They also highlight the conservative assumptions made in the configuration of vehicle convoys used to simulate serviceability loads in 20m to 30m spans. The findings of the thesis support the need for the rational calibration of the partial factors used in limit state design.

The WIM data was analysed to highlight the extent of overloading. The results provide evidence that the overloading of individual axles and axle sets is prevalent and that overloading has a greater impact on 5m and 10m spans than 30m spans.

Research was carried out into the basis of the bridge live load models in TMH7 Part 2 and those recently developed in Europe, the United States and Canada. The thesis documents the advancement of rationally based live load models derived from actual vehicle data.

Alternative live load models were calibrated against the extreme events predicted by the WIM data. The results independently validate the alternative live load model proposed by the latest research commissioned by the Department of Transport. This live load model takes a similar form to the one proposed in the Eurocode - ENV 1991-3.

## DECLARATION

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I know the meaning of plagiarism and declare that all work in the document, save for that which is properly acknowledged, is my own.

Signed : 

Signed by candidate
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February 2006

John R B Anderson

## ACKNOWLEDGMENT

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I wish to thank the late Professor Rolf Kratz for his mentorship during my professional career and Vela VKE Consulting Engineers for their support in completing this thesis. The guidance and direction provided by my supervisor, Dr Pilate Moyo, is gratefully acknowledged. Finally, I thank my wife's for her support and motivation.

# TABLE OF CONTENTS

	Page No.
<b>1. INTRODUCTION</b> .....	<b>1-1</b>
1.1 BACKGROUND .....	1-1
1.2 OBJECTIVES OF THE STUDY .....	1-2
1.3 SCOPE OF THE STUDY .....	1-3
1.4 METHODOLOGY .....	1-4
1.4.1 Review of Bridge Live Load Models .....	1-4
1.4.2 Analysis of Traffic WIM Data .....	1-4
1.4.3 Critical Assessment of TMH7 Part 2 .....	1-5
1.5 ALTERNATIVE LIVE LOAD MODEL TO TMH7 PART 2 .....	1-6
1.6 CONCLUSIONS AND RECOMMENDATIONS .....	1-6
1.7 REPORT STRUCTURE .....	1-7
<b>2. DEVELOPMENT OF BRIDGE LIVE LOAD MODELS</b> .....	<b>2-1</b>
2.1 INTRODUCTION .....	2-1
2.2 DETERMINISTIC AND PROBABILISTIC DERIVATIONS .....	2-2
2.3 DETERMINISTIC APPROACH .....	2-2
2.4 PROBABILISTIC APPROACH .....	2-4
2.4.1 BD 37/88 .....	2-4
2.4.2 AASHTO LRFD .....	2-7
2.4.3 CANADIAN CODE .....	2-9
2.4.4 EUROPEAN CODE .....	2-12
2.5 CONCLUSIONS .....	2-15
<b>3. ANALYSIS OF WEIGH-IN-MOTION DATA</b> .....	<b>3-1</b>
3.1 INTRODUCTION .....	3-1
3.2 ANALYSIS OF WEIGH-IN MOTION DATA .....	3-1
3.2.1 Actual Vehicles .....	3-2
3.2.2 Legal Vehicles .....	3-3
3.2.3 National Road Traffic Regulations .....	3-4
3.3 STATISTICAL APPROACH .....	3-6
3.3.1 Accuracy of Data .....	3-6
3.3.2 General Statistical Properties of WIM Data .....	3-7
3.4 STATISTICAL DISTRIBUTIONS .....	3-10
3.4.1 Normal Distribution .....	3-11
3.4.2 Extreme Distributions .....	3-13
3.4.3 Confidence limits .....	3-22
3.5 RESULTS .....	3-23
3.6 OVERLOADING .....	3-27
3.7 CONCLUSIONS .....	3-31

<b>4.</b>	<b>CRITICAL REVIEW OF TMH7 PART 2 &amp; SUBSEQUENT RESEARCH .....</b>	<b>4-1</b>
4.1	TMH7 PART 2.....	4-1
4.1.1	Background and Development .....	4-1
4.1.2	NA Loading Curves .....	4-3
4.1.3	Review of Truck Combinations.....	4-5
4.1.4	Comparison of Dynamic to Static Loads.....	4-7
4.1.5	Lateral Bunching .....	4-8
4.1.6	NB Loading .....	4-8
4.2	RR 91/004/01 - PERMISSIBLE HEAVY VEHICLE LOAD RESEARCH.....	4-9
4.2.1	Problem Statement .....	4-9
4.2.2	Development of Live Load Model .....	4-9
4.2.3	Critical Review .....	4-11
4.3	RR 91/004/02 - DEVELOPMENT OF ALTERNATIVE DESIGN LOAD TO TMH7 .....	4-13
4.3.1	Traffic loading .....	4-13
4.3.2	Impact Factor .....	4-14
4.3.3	Test Loading .....	4-15
4.3.4	Assessment & Design Loads .....	4-15
4.3.5	Report Conclusions.....	4-17
4.3.6	Critical Review .....	4-17
4.4	COMPARISON OF DESIGN LOADS VERSUS ACTUAL LOADS .....	4-19
4.4.1	TMH7 versus Actual Traffic Measurements .....	4-20
4.4.2	RR 91/004/02 versus Actual Traffic Measurements .....	4-21
4.5	CONCLUSIONS.....	4-23
<b>5.</b>	<b>ALTERNATIVE LIVE LOAD MODEL TO TMH7 PART 2.....</b>	<b>5-1</b>
5.1	CALCULATION OF LOAD FACTOR .....	5-1
5.2	RESULTS.....	5-3
<b>6.</b>	<b>FINAL CONCLUSIONS AND RECOMMENDATIONS .....</b>	<b>6-1</b>

Appendix A: Vehicle Configurations and Classifications

Appendix B: Statistical Distributions

Appendix C: Liebenberg Combinations

Appendix D: Impact Formula

Appendix E: Alternative Load Model

Appendix F: Visual Basic Programs

## List of Figures

Figure 1.1 - Map of National Route 3 (Source: Wikipedia Encyclopedia).....	1-3
Figure 2.1 - (a) Ministry of Transport Standard Loading Train (1922): (b) BS 153 Unit Loading Train (1923) (Source: O'Connor, C., 2001).....	2-3
Figure 2.2 - Henderson's Vehicle Combination (Source: Henderson, 1954).....	2-3
Figure 2.3 - Revised HA loading curve (Source: Dawe, 2003).....	2-5
Figure 2.4 - Characteristics of H20 Design Truck (Source: LRFD 1994).....	2-7
Figure 2.5 - Extrapolated Moments from Cumulative Distribution Functions. (Source: Nowak, 1991).....	2-9
Figure 2.6 - CL-W Truck (Source: CAN\CSA-S6-00).....	2-10
Figure 2.7 - CL-W Lane Load (Source: CAN\CSA-S6-00).....	2-10
Figure 2.8 - Histogram in $W/B_M$ Space, Ontario 1967 Census Data (Source: O'Connor, C., 1981).....	2-11
Figure 2.9 - ENV 1991-3 Load Model 1 (Source: Dawe, 2003).....	2-13
Figure 3.1 - Histogram of Sample GVM's.....	3-8
Figure 3.2 - Probability Density Function of GVM's – 6 Axle Vehicles.....	3-9
Figure 3.3 - Probability Density Function of Bending Moments – 30m span.....	3-9
Figure 3.4 - Probability Density Function of Bending Moments – 5m span.....	3-9
Figure 3.5 - Distribution of Bending Moments - 30m span.....	3-14
Figure 3.6 - Distribution of Shear Forces - 30m span.....	3-15
Figure 3.7 - Fit of Theoretical Distributions to Plotted Points - 6 Axle Vehicles on 15m spans.....	3-18
Figure 3.8 - Fit of Theoretical Distributions to Plotted Points - 7 Axle Vehicles on 15m spans.....	3-19
Figure 3.9 - Fit of Theoretical Distributions to Plotted Points - 8 Axle Vehicles on 15m spans.....	3-20
Figure 3.10 - Confidence limits for Gumbel Distribution for 7 Axle Vehicle on 30m Span.....	3-23
Figure 3.11 - Cumulative Distribution of Axle Weights.....	3-27
Figure 4.1 - Uniformly Distributed Lane Loads Derived From Bending Moments.....	4-4
Figure 4.2 - Uniformly Distributed Lane Loads Derived From Shear Forces.....	4-5
Figure 4.3 – Plot of Bending Moments Due to Travelling and Stationary Traffic.....	4-7
Figure 4.4 - Configuration of Class 14 Vehicle. (Source: RR 91/004/02, 1995).....	4-14
Figure 4.5 - Live Load Model Proposed in RR 91/004/02 (Source: RR91/004/02).....	4-16
Figure 4.6 - Comparison of Bending Moments.....	4-19
Figure 4.7 - Comparison of Shear Forces.....	4-20
Figure 5.1 - Equivalent Load Models.....	5-2
Figure 5.2 - Bending Moment Load Factors – Models 1 & 2.....	5-3

Figure 5.3 - Bending Moment Load Factors using WIM data– Models 3, 6, 7 & 8..... 5-4

Figure 5.4 - Shear Force Load Factors using WIM data – Models 3, 6, 7 & 8 ..... 5-4

Figure 5.5 - Bending Moment Load Factors using RR 91/004/02 – Models 3, 6, 7 & 8 ..... 5-5

Figure 5.6 – Shear Force Load Factors using RR 91/004/02 – Models 3, 6, 7 & 8..... 5-5

## List of Tables

Table 2.1 - Basic Values of ENV 1991-3 Load Model 1 (Source: ENV 1991-3).....	2-13
Table 3.1 - Number of Recorded Heavy Vehicles .....	3-2
Table 3.2 - Actual Vehicle Bending Moments – Statistical Properties .....	3-3
Table 3.3 - Actual Vehicle Shear Forces – Statistical Properties .....	3-3
Table 3.4 - Legal Vehicle Bending Moments – Statistical Properties .....	3-4
Table 3.5 - Legal Vehicle Shear Forces – Statistical Properties .....	3-4
Table 3.6 - Counts of Axle Mass Distributions.....	3-7
Table 3.7 - Bending Moments Extrapolated using the Normal Distribution.....	3-12
Table 3.8 - Shear Forces Extrapolated using the Normal Distribution .....	3-12
Table 3.9 - Statistical Properties of Recorded Vehicle Classes .....	3-12
Table 3.10 - Sensitivity of Predicted Bending Moments for 6 Axle Vehicles .....	3-16
Table 3.11 - Sensitivity of Predicted Bending Moments for 6 Axle Vehicles to Sample Size .....	3-17
Table 3.12 - Extrapolated Bending Moments .....	3-23
Table 3.13 - Extrapolated Shear Forces.....	3-23
Table 3.14 - Statistical Properties of Axle Weights and GVM .....	3-24
Table 3.15 - Predicted Bending Moment Confidence Limits .....	3-24
Table 3.16 - Predicted Shear Force Confidence Limits.....	3-25
Table 3.17 - Nowak/Gumbel Comparison – Bending Moments .....	3-25
Table 3.18 - Nowak/Gumbel Comparison – Shear Forces.....	3-25
Table 3.19 - Bending Moments for 6 Axle Vehicles with Varying Return Periods.....	3-26
Table 3.20 - Number of Observed Illegal Vehicles .....	3-27
Table 3.21 - Percentage of Overloaded Axles.....	3-28
Table 3.22 - Variance of Load effects derived from Complete Set of Events and Extreme Set of Events .....	3-29
Table 3.23 - Overloading Results using Normal Distribution – Bending Moments.....	3-29
Table 3.24 - Overloading Results using Normal Distribution – Shear Forces .....	3-30
Table 4.1 - Following Probability.....	4-6
Table 4.2 - Impact Allowance in TMH7.....	4-7
Table 4.3 - Impacts Allowances.....	4-14
Table 4.4 - Design Load Values .....	4-16
Table 4.5 - Comparison of Bending Moments, RR 91/004/02 versus TMH7 .....	4-17

Table 4.6 - Comparison of Shear Forces, RR 91/004/02 versus TMH7.....	4-17
Table 4.7 - Bending Moments Results .....	4-19
Table 4.8 - Shear Force Results.....	4-20
Table 4.9 - Bending Moment Comparison, WIM data v TMH7 .....	4-20
Table 4.10 - Shear Force Comparison, WIM data v TMH7.....	4-21
Table 4.11 - Bending Moment Comparison, WIM data v RR 91/004/02 .....	4-22
Table 4.12 - Shear Force Comparison, WIM data v RR 91/004/02.....	4-22
Table 5.1 - Calibration of Model Bending Moments to WIM Data .....	5-7
Table 5.2 - Calibration of Model Shear Forces to WIM data .....	5-8
Table 5.3 - Calibration of Model Bending Moments to RR 91/004/02 .....	5-9
Table 5.4 - Calibration of Model Shear Forces to RR 91/004/02.....	5-10

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## 1. INTRODUCTION

### 1.1 BACKGROUND

The live load model used to simulate traffic loading on bridge structures in South Africa is specified in the “Code of Practice for the Design of Highway Bridges and Culverts in South Africa”, TMH7 Part 2. The code provides bridge design engineers with an exact methodology for quantifying live loads. Rigorous requirements for applying the live load model are set (Liebenberg, 1974) with the aim of simulating the most onerous global and local load effects.

TMH7 was first published in 1981 and the specified live load model was taken from research work carried out by Liebenberg (1974). Although revisions and errata for TMH7 Part 1 and 2 were issued in 1988, shortcomings associated with the live load model for normal traffic conditions on narrow and short span bridges were identified (Ullman, 1988). Under normal traffic loading, the live load model was found to underestimate the bending moments for spans between 4m and 9m. Shear forces were found to be underestimated on spans less than 23m. TMH7 Part 2 caters for this shortcoming by specifying that the live load model for abnormal loads be applied to all highway bridges.

Since 1988, the Department of Transport (DOT) has received requests from various bodies, including the Road Freight Association, to increase the legal axle mass limits for trucks. In response, the DOT commissioned the following reports to consider the possible amendments to the National Road Traffic Regulations.

- (i) Report RR 91/004/01, “The effect of an Increase in the Permissible Heavy Vehicle Loads on Bridges”, June 1994.
- (ii) Report RR 91/004/02, “The effect of an Increase in the Permissible Heavy Vehicle Loads on Bridges – Assessment Code”, December 1995.

The main objective of the reports was to compare the load effects caused by vehicles complying with the specified legal limits to those calculated using TMH7 Part 2. The effect of the increased permissible axle and vehicle loads on road bridges was also considered within the reports.

As a result of the abovementioned research, in 1996, the DOT increased the legal permissible axle loads and amended the bridge formula (National Road Traffic Regulations, 1996). However, the main conclusion of the reports was that the live load model in TMH7 Part 2 underestimated the load effects in short span structures. The results of the research also demonstrated the significant variance in the load effects caused by different overloading assumptions. Overloading allowances were derived from vehicle statistics collected in Switzerland (Bez, 1989) and from the limited data available in South Africa at the time. In conclusion, RR 91/004/02 called for the verification of overloading ratios based on the analysis of traffic survey data collected on South African roads.

In drafting TMH7 Part 2, Liebenberg (1978) judged that a probabilistic analysis of extreme truck events was not viable due to a lack of statistical information. Possible future trends in vehicle configurations and weights were judged to add a level of uncertainty that would invalidate the original assumptions made in the formulation of the design loads. The loading formulas were therefore developed using a credibility approach where engineering judgement was used to determine probable combinations and arrangement of heavy vehicles.

In contrast, the reports RR 91/004/01 & 02 employed a probabilistic approach in developing a live load model. The approach entailed the use of a Monte Carlo simulation to generate random vehicle streams as proposed by researchers Bez (1991) and Moses and Verma (1987).

FitzGerald (1998) highlighted the level of dissatisfaction among South African bridge engineers with the provisions relating to the application of traffic loading within TMH7 Part 2. Specifically, Ullman (1988) stated that the application of the specified live load model was cumbersome and that there was room for its simplification. Given the deficiencies of TMH7 Part 2 and the findings of FitzGerald (1998) and Ullman (1988) there is an urgent need to update the live load model contained within the code. The availability of adequate traffic survey data removes the constraints listed by Liebenberg (1978) and adds impetus to the required update.

## **1.2 OBJECTIVES OF THE STUDY**

The advent of toll roads in South Africa has facilitated the collection of traffic survey information through the use of weigh-in-motion (WIM) sensors. Concessionaires are required to continuously collect vehicle data including the axle weights and axle spacings of individual trucks.

The WIM data provides information on heavy vehicles travelling on South African roads that was not available when reports RR 91/004/01 & 02 were drafted. In those reports, virtual simulations were used to model heavy vehicle configurations, masses and occurrences. Therefore, utilising the available WIM data, the objectives of the study are:

- (i) To verify the magnitude of the load effects caused by heavy vehicles on bridge structures in South Africa as set out in RR 91/004/01 & 02;
- (ii) To verify the assessment/design load derived in RR 91/004/02;
- (iii) To confirm the extent of the deficiencies in TMH7 Part 2 in regard to short term spans; and
- (iv) To quantify the extent of overloading present on the National Route 3.

The following further objectives are set with the aim of contributing to the development of an alternative live load model. These objectives are taken from the review of research work referenced in RR 91/004/01 & 02:

- (i) The identification of parameters that describe heavy vehicles;
- (ii) The review of the use of the Gumbel distribution to extrapolate extreme load effects; and
- (iii) The calibration of the design loads extracted from the probabilistic analysis of traffic survey data.

During the research period, no references were found describing the derivation of the TMH7 Part 2's NA loading curve for simulating normal traffic conditions. Similarly, no reference for its increase by 6kN in 1988 was found. Liebenberg's (1974) assumed vehicle combinations were, however, referenced in Ullman (1987). A further objective of the study was therefore to create a concise reference setting out the basis of the NA loading curve. It was considered that this reference was necessary in the future revision of TMH7 Part 2.

### 1.3 SCOPE OF THE STUDY

The scope of the thesis includes the analysis of WIM data collected at Heidelberg, on the National Route 3 (N3) in February 2005. As shown in Figure 1.1, the N3 connects Durban and its port with South Africa's commercial hub, Johannesburg. The route was chosen because of the high volumes of heavy vehicles it experiences. Heidelberg is situated on the N3 approximately 50km south of Johannesburg. For the month considered, 106,917 heavy vehicles were recorded by the WIM sensors.



Figure 1.1 - Map of National Route 3 (Source: Wikipedia Encyclopedia)

The load effects caused by the heavy vehicles on single span structures were calculated for the purpose of verifying the load effects generated in RR 91/004/01 & 02. Spans ranging from 5m to 30m were considered.

## **1.4 METHODOLOGY**

### **1.4.1 Review of Bridge Live Load Models**

The thesis reviews the methods of formulating live load models that simulate traffic loading on bridges structures. This review was done for the purpose of critically reviewing TMH7 Part 2. The approaches identified were:

- (i) The deterministic method, using engineering judgement to deal with the unknowns associated with the random nature of traffic loading; and
- (ii) The probabilistic method, deriving and calibrating a live load model from actual traffic data.

The reviewed codes include the American Load Resistance Factor Design (LRFD), "Bridge Design Specifications" (1994) and the CAN\CSA-S6-00, "Canadian Highway Design Code" (2000). These codes are proponents of the rationally based probabilistic derivation of load models and partial design factors. The Eurocode, ENV1991-3:2000, "Basis of design and action on structures – Part 3: Traffic loads on bridges", was also reviewed as the current forerunner in the probabilistic approach. TMH7's close relatives, the British code of practice BS5400, "Steel, Concrete and Composite Bridges", Part 2: "Specification of loads" (1978) and the Department Standard BD 37/01, "Loads for Highway Bridges" (2001) are included as examples of codes that have developed from both deterministic and probabilistic approaches.

### **1.4.2 Analysis of Traffic WIM Data**

#### **(a) Processing of WIM Data**

The traffic data collected from the WIM survey was utilised to create two separate vehicle populations. The first population consisted of the vehicles with the recorded axle masses and axle configurations. This population was known as the "actual" vehicles. In the second population, the recorded axle configurations were assigned with the maximum permissible axle masses in terms of the National Road Traffic Regulations (1999). Depending on the number of axles, and their configuration, the loads were apportioned to produce the maximum load effects. This population was known as the "legal" set of vehicles. Its purpose was to simulate the maximum legal load effects that could be generated by an individual vehicle, thus creating a benchmark to measure the impact of overloading.

The maximum load effects caused by the vehicles from both population sets were calculated for simply supported spans ranging from 5m to 30m. A Visual Basic (VB) program was written for this purpose. The program also ranked the results and calculated the statistical properties of the data.

Population sets were further subdivided into subsets that grouped vehicles according to their total number of axles. As spans of 30m and less were considered, the actions of a single heavy vehicle were judged to be critical (Nowak, 1991). Single heavy vehicles that could legally obtain a Gross Vehicle Mass (GVM) of 500kN were therefore determined to be of interest. As a result, vehicles containing 6 axles and more were studied.

#### **(b) Statistical Properties**

The statistical properties of the recorded axle weights and calculated load effects of 30,000 heavy vehicles were extracted. This data provides an insight into the nature and distribution of the heavy vehicles travelling on the N3.

#### **(c) Statistical Distributions**

As live loading due to traffic is a random time dependent variable, a probability distribution function may be fitted to the observed events. This theoretical distribution can be used to predict extreme events with a given non-exceedence probability. The study analyses the load effects of both the “actual” and “legal” vehicle populations and fits the appropriate statistical distribution to the results. The maximum load effects occurring within a 120 year period, for spans ranging between 5m and 30m, are extrapolated from the theoretical distribution.

The study considers two separate approaches for extrapolating extreme events. The first of these was developed by Nowak (1991) in the calibration of the LRFD (1994) and assumes that a normal distribution best fits the load effects derived from a surveyed population of overloaded trucks. The second method, used in RR 91/004/01 & 02 (1994, 1995), involves the application of an extreme distribution to a set of extreme events. The study assesses the most appropriate extreme distribution in describing the characteristic properties of the extreme events. A comparison of the results generated by both methods is given.

### **1.4.3 Critical Assessment of TMH7 Part 2**

#### **(a) Background and Development of TMH7 Part 2**

A literature search was carried out to investigate the basis of TMH7 and its development since its introduction in 1981. As literature to the derivation of the TMH7's NA loading curves was not located, the thesis attempts to replicate the loading curves using Liebenberg's (1974) vehicle combinations and those formulated by Henderson (1954). The combinations were used to calculate the maximum bending moments and shear forces in simply supported spans ranging from 10m to 900m. A VB program was written to calculate the dynamic and static load effects of the vehicles in combination with an assumed lane load. An equivalent lane load was then derived to simulate the maximum bending moments in each span increment and the results plotted to obtain the loading curve.

**(b) Critical Assessment**

The load effects generated from the probabilistic analysis of the WIM data are compared against those calculated using TMH7 Part 2 and a critical assessment is given. A comparison between the WIM data's results and those derived in RR 91/004/01 & 02 is also done.

**(c) Overloading**

The extent of overloading was quantified by comparing the predicted 28 day event of the "actual" vehicle population set against the "legal" vehicle population set. This approach uses the statistical properties of the data sets rather than individual results. Due to the inherent inaccuracies associated with the WIM data, the comparison of individual maximum results will not provide conclusive results. Cumulative distributions of vehicle weights are, however, plotted to indicate the percentage of overloaded vehicles travelling on the N3 in a given month.

**1.5 ALTERNATIVE LIVE LOAD MODEL TO TMH7 PART 2**

Alternative live load models that simulate the load effects calculated from the WIM data are reviewed. A live load model that may supersede the NA loading curve in TMH7 Part 2 is recommended.

**1.6 CONCLUSIONS AND RECOMMENDATIONS**

The conclusion of the thesis provides recommendations for the revision of the live load model in TMH7 Part 2 using the probabilistic analysis of traffic data. Additional research required to calibrate an alternative load model, as proposed in RR 91/004/02, is detailed. The option of adopting ENV 1991-3, by drafting a National Application Document is discussed.

## 1.7 REPORT STRUCTURE

Chapter 1 provides the background of the development of bridge live load models in South Africa. It describes the development of the relevant codes of practice, TMH7 Part 2, and references the subsequent research work carried out in RR 91/004/01 & 02. The objective of verifying the referenced live load models using the collected WIM data is described. The further objectives of quantifying the extent of overloading on the National Route 3 and the review of the statistical distributions associated with the extrapolation of extreme traffic events are also described. A summary of methods used in achieving these objectives is provided.

Chapter 2 is concerned with the different approaches used in deriving bridge live load models. The chapter documents the deterministic approach, where engineering judgement is used to deal with the unknowns associated with the random nature of traffic loading. The probabilistic approach involving the analysis of actual traffic data to derive and calibrate live load models is also described. The differing methods developed in Canada, the United States, the United Kingdom and Europe are reviewed in detail.

Chapter 3 deals with the analysis of the WIM data. The methods adopted in sorting and analysing the WIM data are described in detail. The statistical properties of a sample of 30,000 vehicles are also given to provide insight into the nature of all heavy vehicles on the National Route 3. For single lane simply supported spans, ranging from 5m to 30m, the load effects caused by the WIM data are calculated. In extrapolating extreme events from these results, the chapter investigates the use of alternative statistical distributions and return periods. The prevalence of overloading is also quantified through the analysis of the load effects generated by the WIM data.

Chapter 4 provides a critical assessment of the live load models in TMH7 Part 2 and RR 91/004/01 & 02. The background and development of these live load models is reviewed in detail. A quantitative assessment of each live load model is given using the load effects calculated from the WIM data (Chapter 3). Comment on the methods used in TMH7 Part 2 and RR 91/004/01 & 02 is also given with reference to the codes of practice reviewed in Chapter 2.

Chapter 5 investigates the development of an alternative live load model to the one specified in TMH7 Part 2. Various models are considered and their ability to simulate the load effects generated by the WIM data is quantified. From these results, recommendations for an alternative live load model are given.

Chapter 6 presents recommendations for the future development of the live load model in TMH7 Part 2 in light of the development of the probabilistic techniques described in Chapter 2 and the results of the analysis of the WIM data (Chapter 3).

## 2. DEVELOPMENT OF BRIDGE LIVE LOAD MODELS

### 2.1 INTRODUCTION

Although heavy vehicles have changed substantially since the development of the first live loading curves in 1931 (Ministry of Transport), the basic form of the live load models used by design engineers has remained relatively unchanged. This is because traffic loading may be simulated with reasonable accuracy by the use of a uniformly distributed load and point loads (Buckland, 1978). The historical development of live load models has therefore concentrated on the quantification and calibration of these applied loads.

The gross vehicle mass (GVM) and the axle spacings of heavy vehicles vary from country to country, depending on the legal requirements. As a result, different live load models have developed, for example, in the United States, Canada and the United Kingdom. The live load model in TMH7 Part 2 is also unique and a product of South Africa's road traffic regulations in 1974. Although the aforementioned live load models vary in form and magnitude, common methods were applied in their derivation. For the purpose of critically reviewing TMH7's live load model, the methods used to derive the live load models in the following codes of practices were researched:

- (i) BS 153: 1958, "Specification for Steel Girder Bridges", British Standards Institute;
- (ii) BS5400: 1978 "Steel, Concrete and Composite Bridges", Part 2: "Specification of loads", British Standards Institute;
- (iii) Department Standard BD 37/88 & 01, "Loads for Highway Bridges", British Department of Transport (1988, 2001);
- (iv) American Association of State Highway Transportation Officials (AASHTO), Load Resistance Factor Design (LRFD), "Bridge Design Specifications" (1994);
- (v) CAN\CSA-S6-00, "Canadian Highway Bridge Design Code" (2000) ; and
- (vi) Eurocode, ENV 1991-3:2000, "Basis of design and action on structures – Part 3: Traffic loads on bridges".

Since the last revision of TMH7 Parts 1 and 2 in 1988, significant developments in bridge engineering have taken place. These include:

- (i) The development of limit state design principles and the use of rationally based partial safety factors that are derived from the probabilistic analysis of the given variable (ENV 1991-3);
- (ii) The use of WIM sensors that have allowed the collection of a huge amount of traffic survey data; and

- (iii) The advent of modern computers and the increased ability of engineers to process and analyse large amounts of data.

There is no doubt that the updating of THM7 Part 2 requires investigation when considering the above facts. TMH7's traffic loading is a nominal load derived from deterministic methods rather than a characteristic load derived from the statistical analysis of traffic survey data. The partial factors used in TMH7 were calculated using engineering judgement taking into account the intention of the ultimate limit state. Probabilistic theory was not used to determine acceptable probabilities of achieving a particular limit state (Dawe, 2003). The following section therefore summarises the derivation of bridge live load models in Europe and North America to provide recommendations for the revision of the TMH7 Part 2.

## **2.2 DETERMINISTIC AND PROBABILISTIC DERIVATIONS**

The probabilistic analysis of actual traffic data was used to derive the bridge live load models in all of the reviewed codes of practice issued since 1988. The traffic data included information on the volumes and composition of traffic flows, the frequency of traffic jams and the actual weights and spacing of vehicles axles. Statistical methods were used to calculate characteristic loads and to calibrate the partial safety factors used in limit state design. This method provided a more scientific approach that researched the actual events rather than creating idealised events. Data was collected by either conducting traffic surveys or by the use of weigh-in-motion sensors.

The deterministic method used in TMH7 and BS 5400, uses engineering judgement to deal with the unknowns associated with the random nature of traffic loading. Idealised combinations of vehicles representing an extreme event are used to derive the live load models. Historically these combinations were chosen using engineering judgement. More recently computer programs were used to find the most onerous combinations of fully loaded legal vehicles. The partial factors applied to the extreme events are also based on engineering judgment rather than a rational approach. Allowances for overloading and dynamic loads are incorporated by factoring the vehicle axle weights.

## **2.3 DETERMINISTIC APPROACH**

The deterministic approach was developed in the United Kingdom (Henderson, 1954) and has formed the basis of the live load models used in BS 153 and later on in BS 5400. A review of these codes is pertinent as the THM7's live load model is largely based on the methods developed by Henderson (1954) to derive BS5400's live load model.

The first 'modern' loading model derived in the United Kingdom consisted of a 22.9m (75 feet) long loading train that consisted of a tractor and four trailers. The tractor contained a major axle of 219kN with the following trailers having a series of 100kN axle loads. The standard Ministry of Transport Loading curve issued in 1931 was largely based on this design vehicle. Details of the design vehicles are shown in Figure 2.1.

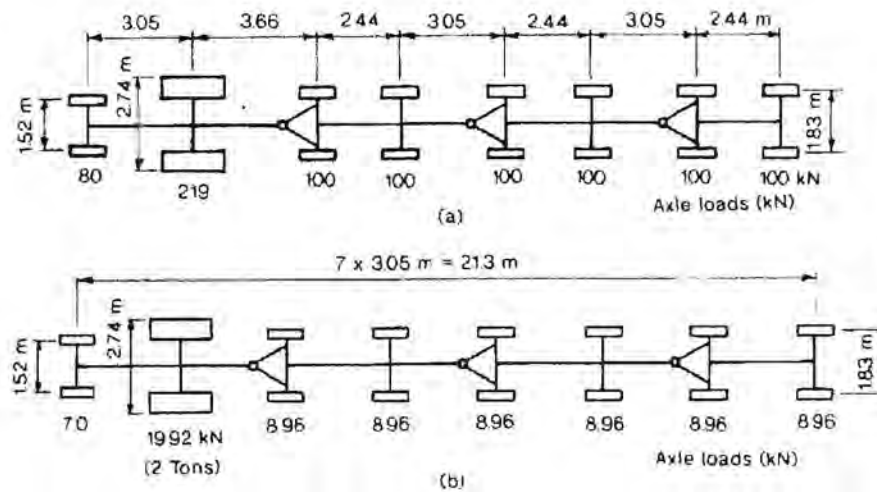


Figure 2.1 - (a) Ministry of Transport Standard Loading Train (1922); (b) BS 153 Unit Loading Train (1923) (Source: O'Connor, C., 2001)

In constructing the loading curve, the 100kN axles were assumed to act over a 3.05m by 3.05m area giving a uniformly distributed load of 10.7kPa. This value was rounded down to 10.5kPa and assumed to act from 3.05m to 22.9m, the length of the loading train. The difference between the major 219kN and the 100kN axles i.e. 119kN was then applied as a knife-edge load across the design lane. An impact factor of 1.5 was used for spans less than 22.9m reducing to 1.15 for spans at 122m and to zero for spans greater than 762m (O'Connor, C., 2001).

The concept of normal and abnormal traffic was introduced in the revision of BS153 in 1958. The normal load was based on a so-called "credibility" approach, which used judgement to determine the most onerous combination and arrangement of trucks complying with the legal axle weights. A design truck with four axles spaced at 1.22m, 3.05m and 1.22m was considered. For a loaded length of 22.9m three trucks with a total weight of 219kN each were used, as shown in Figure 2.2. In longer spans five trucks interspersed with lighter vehicles were utilised. Equivalent loads were then derived from these combinations for various loaded lengths and factored up to take into account impact loads. For a loaded length of 22.9m and a notional lane width of 3.05m, a uniformly distributed load of 10.5kPa resulted. This load was identical to that derived in the previous MOT loading. However, for longer spans the specified uniformly distributed load was much smaller. For example, at a span of 152m, BS153 specified a distributed load of 4.8kPa in comparison to the MOT distributed load of 6.7kPa (O'Connor, C., 2001). The new KEL of 120kN was similar to the MOT loading.

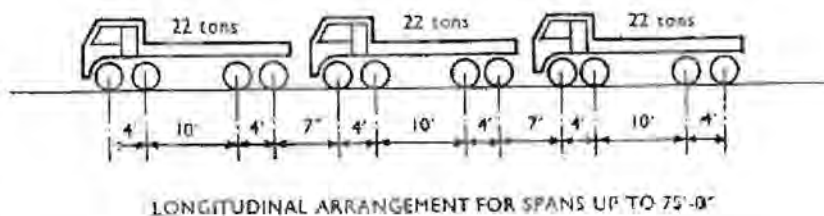


Figure 2.2 - Henderson's Vehicle Combination (Source: Henderson, 1954)

The United Kingdom's first limit state code was introduced in 1978 in the form of BS5400, issued in ten parts. As with BS153, traffic loading was classified as normal loading (HA loading) or abnormal loading (HB loading). The HA loading did not differ significantly from the BS153 loading other than at longer spans it did not permit the distributed load to fall below 9.0kN/m. This increase was judged necessary because of the decrease of the dead load partial safety factor from 1.4 to 1.2 (Dawe, 2003). Again, the HA loading was based on a credibility approach. In BS 5400, HA loading was derived from the load effects of various combinations and arrangements of 235kN vehicles interspersed with additional 98kN and 49kN vehicles for longer lengths (Henderson, 1954). A 25% impact allowance on one axle group was also included. The configuration of the HB vehicle was revised to allow for the increased lengths of abnormal loads travelling on British roads. A configuration that is essentially that of the current TMH7 Part 2 NB live load model was adopted.

The methods used in developing the live load model in BS 5400, issued in 1978, are similar to those used to develop the live load model for normal traffic conditions (NA loading) in the current revision of TMH7 Part 2. Although BD 37/88 has since superseded BS 5400, TMH7 Part 2's loading model has remained unchanged. It was recognised in the United Kingdom that the live load model for normal traffic conditions should accurately simulate actual traffic events. The deterministic method of using a small number of design vehicles was not considered to accurately simulate these events (Dawe, 2003). The probabilistic analysis of traffic survey data was therefore used in drafting BD 37/88. Randomly generated streams of legal vehicles were also developed using Monte Carlo simulations. Although these techniques were used in RR 91/004/01 & 02, South African bridge engineers still use a live load model based on the legal vehicles of 1974. There is an urgent need to translate the research work carried out in RR 91/004/01 & 02 into a revised loading model in TMH7 Part 2.

## **2.4 PROBABILISTIC APPROACH**

The basis of probabilistic analysis is fitting a mathematical distribution to the random nature of traffic loading. In the following section, the various methods are reviewed that were developed in Canada, the United States, the United Kingdom and Europe.

### **2.4.1 BD 37/88**

In the United Kingdom, a full review of traffic loading on both short and long span bridge began in 1980 (Dawe, 2003). The British Department of Transport considered that a live load model was required that was based on limit state principles and represented the actual vehicles travelling on the highways. In addition, the use of 30 units HB loading to derive the load effects on short spans for normal traffic conditions was considered illogical. The aim was therefore to revise HA loading to simulate normal traffic conditions for both short and long spans.

The outcome of the review was the issue of BD 37/88 in 1988, with revised HA loading curves for normal traffic conditions. The issue of BD 37/88 was seen as an interim measure during a long-term review of BS 5400 taking cognisance of the development of the Eurocodes. The method of applying a lane load and a KEL was continued. In comparison to BS5400, BD37/88 resulted in an increase of

10% in the applied HA distributed load for loaded lengths between 25m and 60m and as much as 56% for loaded lengths of 150m (O'Connor, C., 2001). As shown in Figure 2.3, for spans of less than 30m, the distributed load for normal traffic conditions was increased substantially. The application of 30 units of the HB loading in conjunction with HA loading was no longer required. The revised HA loading curves therefore provided a single live load model for normal conditions. BS 37/88 also increased the units of HB loading to be carried by structures on the various classifications of roads.

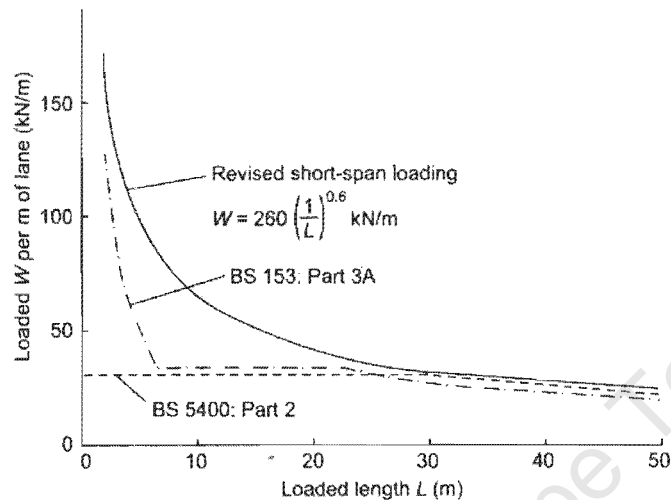


Figure 2.3 - Revised HA loading curve (Source: Dawe, 2003)

The HA loading curve, published in BD 37/88, was based on two separate live load models derived for short and long span bridges. Both deterministic and probabilistic methods were used.

In 1982, BS 5400's HA loading curve was revised using research work carried out by Flint & Neill (1986) on long span bridges (Dawe, 2003). These revisions were carried through to BD 37/88. The research work was a milestone because actual traffic data was used rather than a combination of idealised vehicles. In formulating the loading curves, the collected traffic data was extrapolated using statistical distributions to calculate the characteristic load effects that might occur in 120 years. The characteristics of traffic jams were modelled using random sequences of vehicles. Each type of vehicle was chosen relative to its recorded average proportion.

BD 37/88's HA loading curves for short spans was derived from extreme combinations of legal vehicles. The configuration of the vehicle convoy causing the most extreme load effects was identified using a computer programme. All vehicles in a convoy were assumed to be laden to the legal limits. Allowances for overloading, impact and lateral bunching were included by separately factoring the legal axle loads. An impact factor of 1.8 was applied to a single axle. The overloading factor was set at 1.4 for spans up to 10m, reducing linearly to unity at 60m span. In the case of lateral bunching, a factor of 1.46 was applied for slow moving traffic on spans up to 20m reducing to unity at 40m span. The approach used was deterministic, using computer technology to find the most onerous combination rather than engineering judgement.

The loading model in BD37/88 is derived from the basic assumption that the most extreme traffic loading can reasonably be expected to occur in the 120 year design life of a bridge (Dawe, 2003). In design terms the extreme event was taken as 1.5 x the nominal loading. Work carried out in calibrating the partial factors (Flint and Neill, 1980) showed that the value of the partial factor was relatively insensitive to the return period assumed. For this reason, it was considered that the HA design load could be based on the most extreme traffic load even though it had a very low probability of occurring in practice. When considering the design load against actual traffic survey data it was shown that the ultimate design loading would occur approximately once in 200,000 years and the nominal un-factored load would occur once in 120 years.

In conclusion, BD 37/88's live load models were derived from both the probabilistic analysis of actual traffic data and the deterministic analysis of convoys of legal trucks crossing short spans. However, the method used in deriving the short spans load effects is not proposed in the revision of TMH7 Part 2. The use of a convoy of fully laden, bumper to bumper vehicles leads to the finding that multiple vehicle loads are critical for spans 25-40m. This finding is in contradiction with the findings of Nowak (1991), that the effects of a single vehicle are dominant for spans up to 40m.

## 2.4.2 AASHTO LRFD

Bridge design in the United States is currently carried out in accordance with the probabilistic limit state code of practice, the AASHTO Load Resistance Factor Design (LRFD) “Bridge Design Specifications” (1994). This specification has replaced the allowable stress code of practice, the AASHTO Standard Specification for Highway Design. The impetus for the review came from the inconsistencies in the AASHTO Standard Specification, which resulted from its many revisions, and the advent of limit state codes of practice such as the Ontario Highway Bridge Design Code (1979).

Traffic loading in the LRFD is simulated by the use of a design truck and a design lane load of 9.3kN/m. The design truck is known as the H20 truck and has its origins in the first issues of the AASHTO specification prior to 1931. In 1931 a uniformly distributed lane load was introduced for use in conjunction with a combination of point loads. Although state legal axle limits and bridge formulas were in place, many States drafted exclusions into their regulatory policies. These exclusions allowed vehicles in excess of the prescribed legal limits to operate on the roads. Increasingly, it was recognised that the loading model for normal traffic conditions did not bear a uniform relationship to many vehicles that were present on the roads.

The State Bridge Engineers (National Highways Institute, 1995) decided that a live load model representative of the legal vehicles permitted on the highways was needed. A population of probable legal trucks was therefore created; their load effects on bridges structures were then calculated. The results showed that the existing H20 load model was significantly underestimating the load effects caused by legal vehicles on the highways. A series of alternative load models, including the H20 truck in combination with a lane load, were therefore proposed. The legal vehicles' maximum force effect envelopes were compared with those simulated by the proposed load models. The combination of the H20 truck and of a lane load of 9.3kN/m was found to produce the nearest fit. Figure 2.4 shows the characteristics of the design H20 truck.

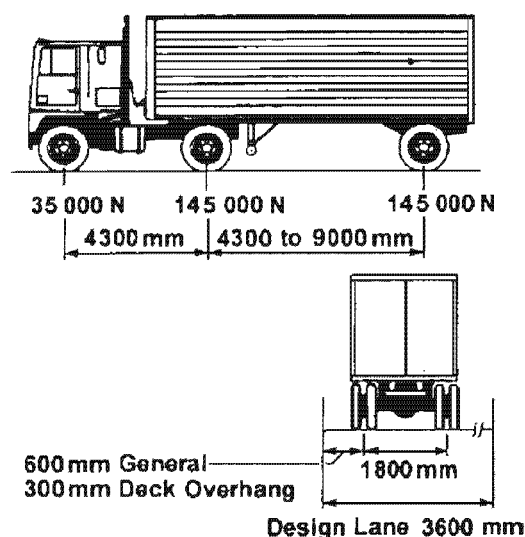


Figure 2.4 - Characteristics of H20 Design Truck (Source: LRFD 1994)

The limit state partial factors used with the design load were derived (Nowak, 1995) from the probabilistic analysis of actual truck survey data collected by the Ontario Ministry of Transport in 1975. About 10,000 trucks that appeared to be heavily loaded were measured and included within the survey data base. For simple spans from 9.0m to 60m, the maximum bending moments and shears were then calculated. The resulting cumulative distribution functions (CDF) of these load effects were then plotted on normal probability paper. The vertical scale,  $z$ , is a product of the inverse standard normal distribution function.

$$z = \Phi^{-1}[F(M)] \quad (2.1)$$

Where:

$M$  = Moment

$F(M)$  = CDF of the moment  $M$

$\Phi^{-1}$  = inverse standard normal distribution function.

Using the plotted distributions, the maximum truck moments and shears were extrapolated for each span. It was assumed that the survey data gave a population set representative of two weeks of heavy traffic on the Interstate. It was therefore concluded that for a 75 year time period, the population of trucks would be about 2,000 ( $52/2 \times 75$ ) times larger than in the survey. The corresponding value of inverse normal distribution value,  $z$ , was then calculated for the occurrence probability of the 1 in 20,000,000 ( $2,000 \times 10,000$ ) truck. The extrapolation of the cumulative distribution functions is shown in Figure 2.5. Using this method, the maximum truck event in the design life of the structure can be predicted. The ratio of the load effects of the extreme event against the design loads were then used to derive the partial factors used in ultimate limit state design.

$$\text{Partial factor of safety} = \text{Extreme load effect} / \text{Design load effect} \quad (2.2)$$

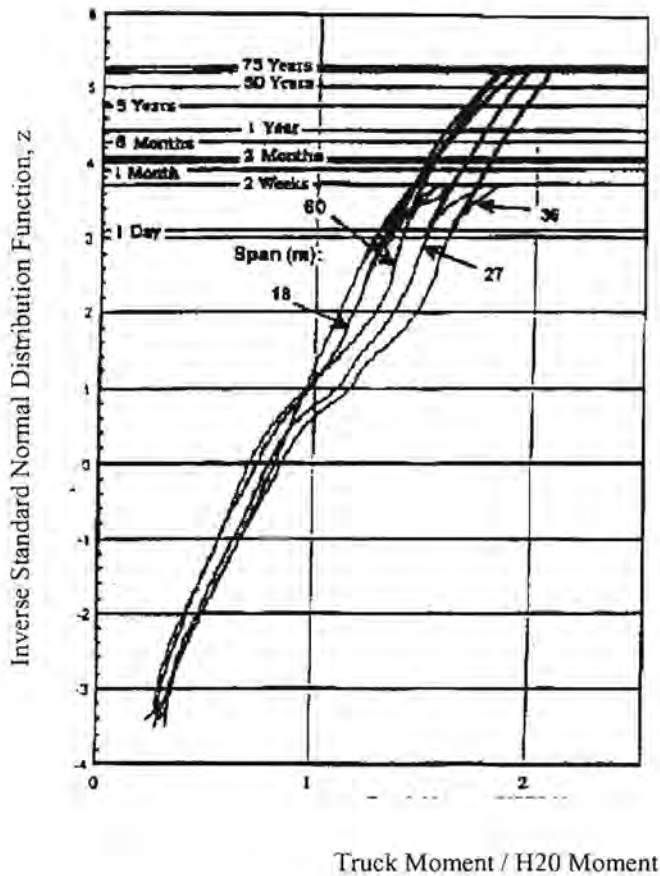


Figure 2.5 - Extrapolated Moments from Cumulative Distribution Functions. (Source: Nowak, 1991)

In summary, the LRDF uses a uniformly distributed load and series of knife edge loads to simulate the load effects of a set of legal vehicles. Using the probabilistic analysis of actual traffic survey data, the load model was calibrated so that the factored ultimate limit state design load represented the 1 in 75 year event. This approach differs significantly from BD 37/88 where the ultimate limit state design load represents a 1 in 200,000 year event.

### 2.4.3 CANADIAN CODE

In 1979 the Ontario Highway Bridge Design Code (OHBC) was published, becoming the first limit state code of practice for bridge design. The code was a forerunner to the LRFD and developed the probabilistic analysis of truck survey data to derive a live load model. The OHBC was superseded by the CANCSA-S6-00, "Canadian Highway Bridge Design Code" (2000).

CSA-S6-00's live load is formulated to represent actual vehicle loads and has a direct relationship to the legal loads permitted on Canadian highways. A design truck, the CL-625, is therefore used to simulate the effects of heavy vehicles. A lane loading, CL-W, is used to represent loading from lighter traffic. The magnitude and arrangement of these loads is shown in Figure 2.6 and 2.7.

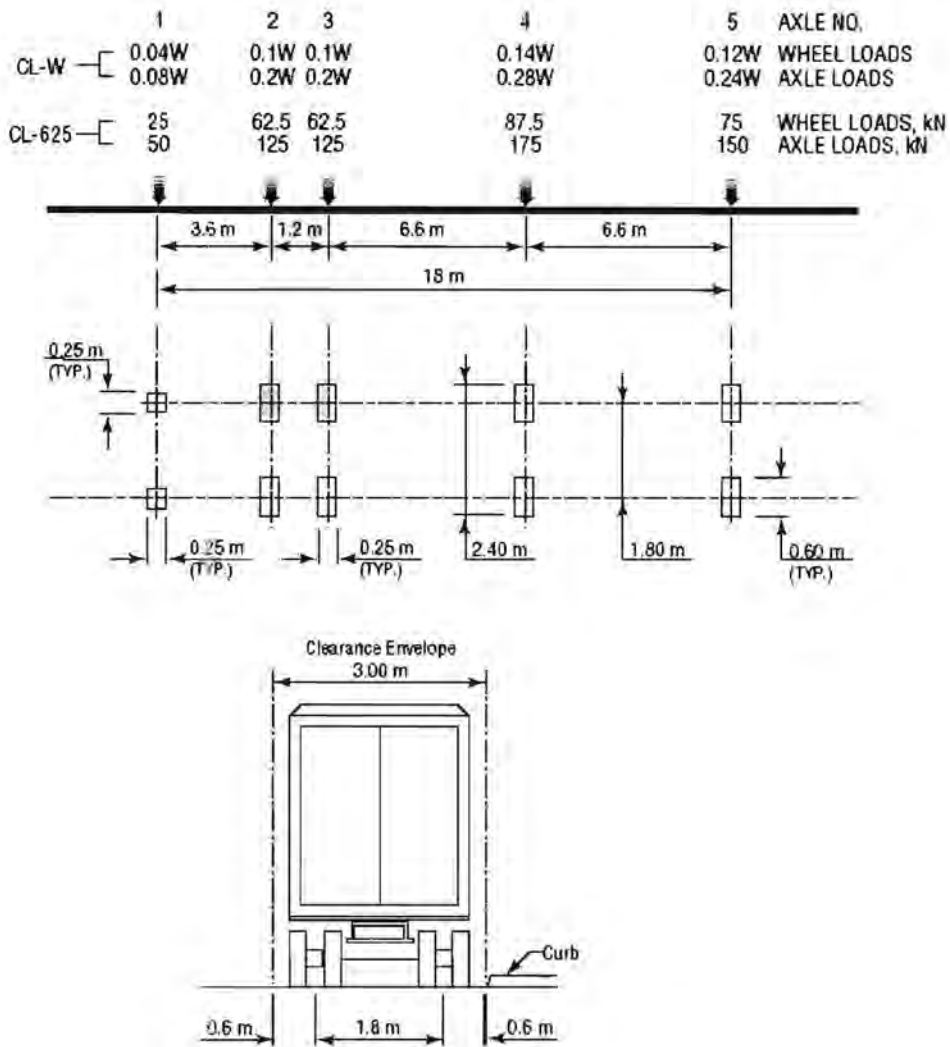


Figure 2.6 - CL-W Truck (Source: CAN/CSA-S6-00)

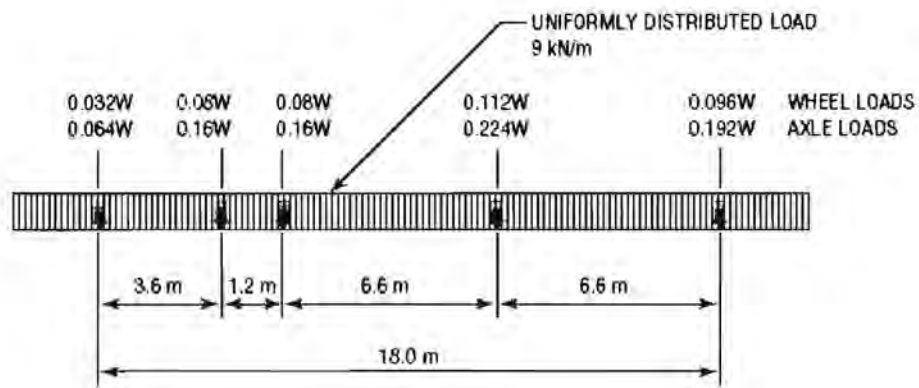


Figure 2.7 - CL-W Lane Load (Source: CAN/CSA-S6-00)

At the time the OHBDC was drafted, there were a plethora of heavy vehicles operating on Canada's provincial roads. Engineers who were drafting the code were interested in the critical vehicles causing the most onerous load effects on a bridge structure. A means assessing the common dimensional properties of these critical vehicles was considered necessary in deriving an equivalent load that

modelled the maximum load effects. The concept of the Ontario Equivalent Base Length was therefore developed as a means of assigning two dimensional properties to a truck (O'Connor, C., 1981). These properties were then used to derive an equivalent design vehicle from a surveyed population of trucks. The two properties assigned were those of the total weight of the vehicle,  $W$ , and the vehicle's equivalent base length.

The equivalent base length,  $B_m$ , was defined as:

“An imaginary finite length on which the total length of a given set of sequential set of concentrated loads is uniformly distributed such that this uniformly distributed load would cause load effects in a supporting structure not deviating unreasonably from those caused by the sequence themselves.”

A set of values,  $W$  and  $B_m$ , were found from the “set of concentrated loads” in the surveyed population. The set of values included both complete vehicles and subsets of adjacent loads. In analysing the properties of a set, a histogram of  $W$  against  $B_m$  was created, as shown in Figure 2.8 (O'Connor, C., 1981). A curve was then fitted to points some distance above the upper bound of the survey. This curve was called the Ontario Bridge Formula. Subsequent vehicles surveys were then undertaken to establish and confirm a virtual upper bound of vehicles creating a curve know as the Maximum Observed Load (MOL). This curve was then used to select a design vehicle whose  $W$  and  $B_m$  values followed its signature.

The above method demonstrated a means of assigning properties to vehicles that could be used to derive an equivalent design vehicle. It was accepted, however, that it was not possible to describe accurately the full range of variables associated with a complex truck by two properties alone. The value of the concept was recognised as the ability to define a vehicle in  $W, B_m$  space for the purpose of observing those properties that should be incorporated into a design load model.

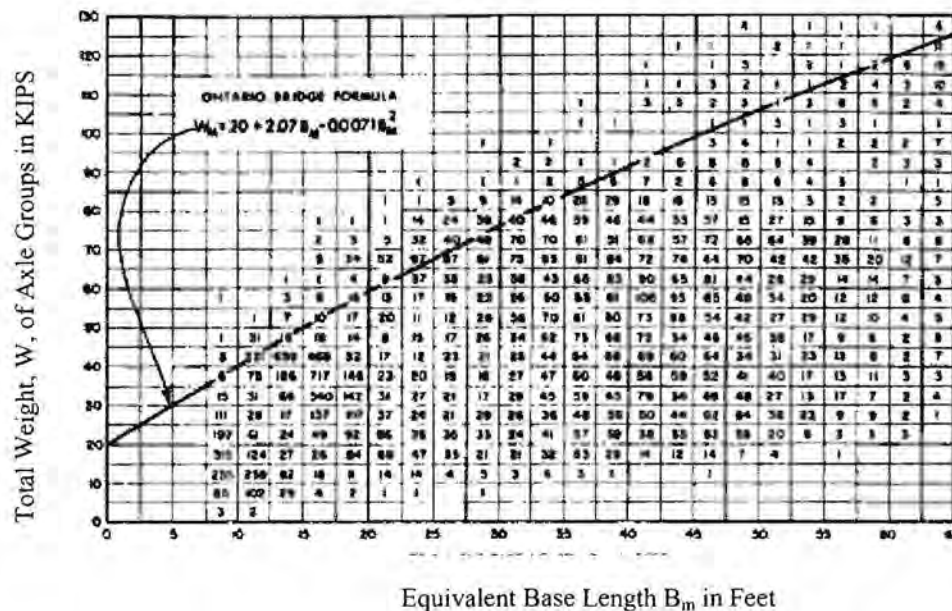


Figure 2.8 - Histogram in  $W/B_m$  Space, Ontario 1967 Census Data (Source: O'Connor, C., 1981)

The CL truck is based on a set of regulations for interprovincial transportation that is signed by all Canadian provinces. It is essentially a legal truck with axles weights and spacing that meet the Ontario Bridge Formula and whose properties follow the signature of the MOL curve.

The lane loading, CL-W is based on the traffic loading for long span bridges recommended by the American Society for Civil Engineers Committee on Loads and Forces on Bridges (Buckland 1978). These recommendations are derived from the survey of trucks crossing the Second Narrows Crossing in the Greater Vancouver area. For the purpose of the study, trucks were weighed and the overall bumper to bumper length measured.

The CL-625 truck was used for the calibration of load factors, load combinations and resistance factors. The methodology used in calculating these factors was similar to that used in calibrating the LRFD. However, in calibrating the CL-625 truck, a Gumbel distribution was used to extrapolate the loading from a set of independent truck samples. Loadings associated with a 50-year return period were calculated. From the ratio of extreme loads and design loads, the bias coefficients and standard deviations were found and the partial live load factors calculated.

As described, the use of a rational probabilistic method to calibrate the chosen live load model is similar to that of the LRFD. However, CSA-S6-00's takes the rational method a step further in using the Ontario Equivalent Base Length to derive a design vehicle that produces the most extreme load effects caused by normal traffic conditions. The approach used in the calibration of the load factors, load combinations and resistance factors is considered a more representative approach than that used in BD 37/88.

#### **2.4.4 EUROPEAN CODE**

The need for a common loading code in Europe is a practical necessity given the volumes of cross border traffic. Since 1975 and the Treaty of Rome, the European Community has embarked on a programme to harmonise technical specifications. The resulting specification for the design loads on bridges is ENV 1991-3:2000, "Basis of design and action on structures – Part 3: Traffic loads on bridge".

The interesting aspect of the ENV 1991-3 is that it has to cover all eventualities and idiosyncrasies of traffic loading originating from each of its member states. Parallels may be drawn within Southern Africa where significant cross border trade takes place by means of the road networks. It should be noted that it is expected that each member of the European Union will qualify the code for its local circumstances. This qualification will ensure that existing levels of safety are maintained. In the United Kingdom, a National Application Document (NAD) for ENV 1991-3 was published in 2000, setting out adjustment factors for the design loads and the partial factors.

The ENV 1991-3 code specifies two load models for normal traffic loading. The first consists of a uniformly distributed load in conjunction with a double axle or tandem set of point loads as shown in Figure 2.9. These loads are applied to the notional lanes named Lane 1, Lane 2 and so on. Lane 1 is classified as the lane in which the applied loads will produce the most unfavourable effects and Lane 2

the second most unfavourable effects. The magnitude of the applied loads is reduced from Lane 1 to Lane 2, as shown in Table 2.1. A second load model is intended to simulate the dynamic effects of traffic loading on short structural elements and consists of a single 400kN axle. The effects of dynamic amplification are included within the specified applied loads.

A further load model exists to cater for abnormal loads. This load model is only applied to structures on specific routes designated for abnormal loads.

Location	Tandem System	UDL system
	Axle loads $Q_{ik}$ (kN)	$q_{ik}$ ( $\text{kN/m}^2$ )
Lane number 1	300	9
Lane number 2	200	2.5
Lane number 3	100	2.5
Other lanes	0	2.5
Remaining area	0	2.5

Table 2.1 - Basic Values of ENV 1991-3 Load Model 1 (Source: ENV 1991-3)

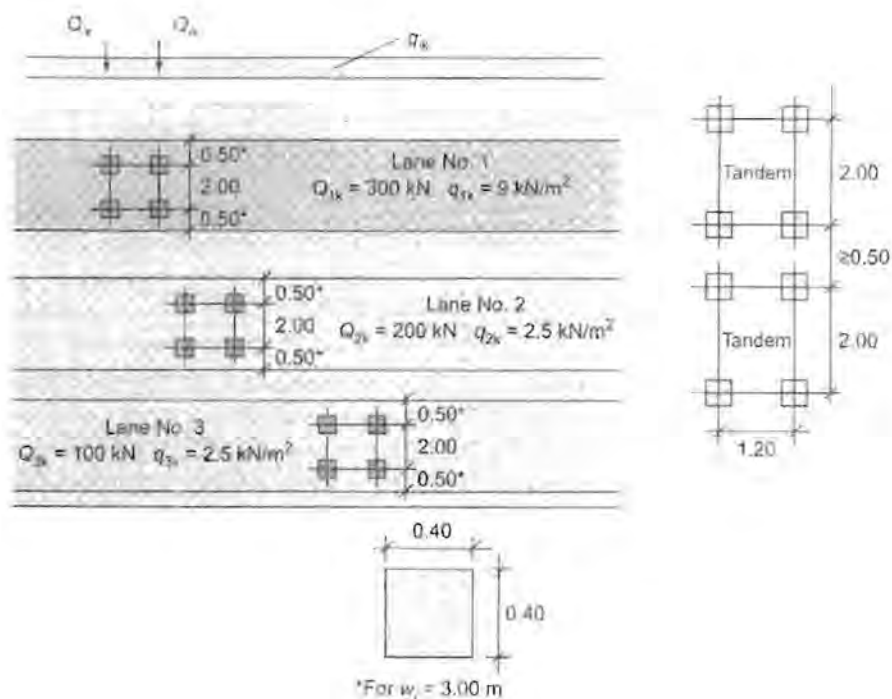


Figure 2.9 - ENV 1991-3 Load Model 1 (Source: Dawe, 2003)

The derivation of the normal load models 1 and 2 is taken directly from traffic data recorded on the A6 dual carriageway in France. Due to the number of international vehicles using this route it was judged to provide a representative sample.

Initially the load effects generated by the actual traffic loads were analysed and extrapolated to correspond to a probability of exceedence of 5 % in 50 years; this represents a return period of 1000 years. This extrapolation allowed the determination of target values for the extreme load that were used

to calibrate the live load model. The approach adopted involved the extrapolation of the following three traffic parameters:

- (i) Axle and Gross vehicle masses;
- (ii) Extreme total loads on the span; and
- (iii) Extreme load effects.

In the case of axle weights and GVM the data was judged to fit a normal distribution and ultimate limit single, double and triple axle loads were predicted for the extrapolation of the sample data. These values were important for establishing the design loading for shorter spans.

The prediction of the extreme total loads on the bridge involved five separate statistical approaches that were then compared. These approaches involved the use of differing distributions (Gaussian, Poisson and extremal distributions) for varying return periods. Dawe (2003) stated that “by and large there was reasonable agreement between the approaches when comparing the maximum total loads for different spans and different return periods”. The following three traffic situations were considered in the review of the total loads:

- (i) Free flowing;
- (ii) Congested traffic including cars; and
- (iii) Congested traffic with only trucks.

Monte Carlo simulations were used to generate random traffic streams from a sample of selected vehicles. As expected, the congested traffic with only trucks produced the most onerous total loads.

In predicting the extreme loads effects, similar extrapolation techniques as described above were used. For a 1000 year return period, reasonable correlation was found for the predicted equivalent uniformly distributed load. In conclusion, it was found that the different methods of extrapolation produced similar results. This meant that theoretically any of the methods developed could be used.

The development of ENV 1991-3 has involved the most extensive use of probabilistic methods in deriving the live load models, which are specified within the code of practice. Of particular relevance is the sensitivity analysis of the results using different extrapolation parameters and techniques. This analysis effectively provides a level of confidence in the methods used. A great strength of the ENV 1991-3 is that it may be calibrated by each member state. This calibration is based on the probabilistic analysis of the state’s traffic characteristics. Through the publication of a NAD, a loading model appropriate to each country is easily derived. This calibration would not be possible if the live load model was derived by a deterministic method.

## 2.5 CONCLUSIONS

The review of the listed codes of practice highlights the extent of research and development undertaken in recent years in the field of bridge live loading. In each case, deterministic methods of deriving live load models have been replaced by probabilistic methods. Deterministic methods were developed because of a lack of statistical data and the complexity of the variables associated with traffic movements. WIM sensors and traffic surveys have now provided a wealth of traffic data that has removed this constraint.

The 'modern' philosophy developed in BD37/88 and ENV 1991-3 was to derive live load models that accurately simulate actual traffic conditions. As a result, rationally based probabilistic methods that study actual traffic survey data were used. In comparison, the deterministic methods only models an extreme event, using a small number of virtual vehicles derived from engineering judgement. This leads to conservative results (O' Connor, A., 2005).

South Africa has yet to progress to a bridge live load model that is developed using probabilistic methods. Although research work, culminating in the Reports RR 91/004/01 & 02, was carried between 1994-1995 in South Africa, TMH7 Parts 1 and 2 remains unaltered since 1998. Its closest relative, BS5400, was superseded by BD 37/88 in 1988. The advent of the ENV 1991-3 in Europe further dates the deterministic derivations of THM7 Part 2's live load models.

The review of BD 37/88 also highlights a number of developments in the practice of deriving live load models that have yet to be adopted in South Africa. These developments include the derivation of loading curves that do not require the use of abnormal load models in short spans and the concept of lateral bunching. It is recommended that both developments be researched in the future revision of TMH7 Part 2.

The great advantage of basing live load models, and their calibration, on the probabilistic analysis of traffic survey data, is that site-specific load models may be easily derived. In addition, as the properties of traffic change for technical and economic reasons, it is relatively simple to update the live load model.

Of the codes reviewed, ENV 1991-3 provides the most recent and extensive use of probabilistic methods to derive a live load model. For this reason, the approach used in ENV 1991-3 provides an excellent reference for the updating of the live load model contained with TMH7. As in the case of the European Community's member states, a NAD based on the probabilistic analysis of local truck survey data may be developed in South Africa and other southern African countries.

In the chapters that follow, the probabilistic analysis of WIM data collected in South Africa is used to provide a critical assessment of the loading model contained within TMH7 Part 2. Methods of developing and calibrating an alternative load model are also investigated.

### **3. ANALYSIS OF WEIGH-IN-MOTION DATA**

#### **3.1 INTRODUCTION**

The use of weigh-in-motion (WIM) sensors to collect traffic survey information on South African Toll Roads has taken place on the National Route 3 at Heidelberg since 2000. This data provides an insight into the complex random nature of traffic. The National Route 3 was chosen because of the large volumes of heavy vehicles that regularly travel between Johannesburg and Durban. In the development of the live load models in TMH7 Part 2 (1981), and the reports that subsequently reviewed it (RR 91/004/01 & 02, 1994 & 1995), traffic survey information was not directly used. It is considered that the data now available provides the opportunity to research the actual load effects caused by heavy vehicles on bridge structures. This research can be used to verify the earlier assumptions of the deterministic approach of TMH7 Part 2 and the Monte Carlo simulation undertaken in Reports RR 91/004/01 & 02.

It is documented that the most onerous load effects in spans up to 40m are caused by a single heavy vehicle (Nowak, 1991). Given that over 90% of bridges in South Africa (RR 91/004/01, 1994) have a span of less than 40m, the research of the heavy vehicles properties is fundamental. In review of the WIM data, the following objectives were set:

- (i) To verify the magnitude of the load effects caused by heavy vehicles on bridge structures in South Africa as set out in RR 91/004/01 & 02;
- (ii) To verify the assessment load derived in RR 91/004/02;
- (iii) To confirm the extent of the deficiencies in TMH7 Part 2 with regard to short spans; and
- (iv) To quantify the extent of overloading present on the National Route 3.

#### **3.2 ANALYSIS OF WEIGH-IN MOTION DATA**

The population of heavy vehicles reviewed in the following section was recorded by WIM sensors on the National Route 3 in February 2005. Although WIM sensors collect a range of data, only the recorded vehicle's axle weights and axle spacings were used in this study.

In February 2005, the WIM sensors recorded 106,917 heavy vehicles. In order to manage and process this amount of data it was necessary to sort the vehicles into separate subsets. Two means of classifying vehicles were considered. The first was the Van Wyk & Louw Vehicle Classifications (1991) used in Reports RR 91/004/01 & 02, as shown in Appendix A. The second was simply classifying the vehicles in terms of their total number of axles.

In review of the data it was identified that not all vehicles complied with the Van Wyk & Louw Vehicle Classifications (1991). Therefore, to ensure that a critical vehicle was not excluded, the classification in terms of the total number of axles was adopted.

The analysis of the WIM data took two forms. Firstly, the WIM data was analysed to calculate the load effects associated with the recorded “actual” heavy vehicles moving across a range of simply supported spans. In the second stage, a set of so-called “legal” vehicles was created by increasing or decreasing the axle masses of the actual trucks to the maximum values permitted by the National Road Traffic Regulations (1999). Depending on the number of axles, and their configuration, the loads were apportioned to produce the maximum load effects. The load effects caused by these “legal” vehicles crossing the set range of simply supported spans were then calculated.

The purpose of the legal vehicles was to simulate the maximum legal load effects that could be generated by the individual vehicles. In comparing the distribution of the load effects of both the “actual” and the “legal” vehicles, the extent of overloading for various spans could be quantified.

When analysing the “actual” load effects against the synthesised “legal” loads effects only static conditions were considered. This approach was considered valid given that the purpose of the study was to quantify the variance between the two sets of vehicles.

### 3.2.1 Actual Vehicles

The raw data from the WIM was exported into a spreadsheet and the vehicles grouped in terms of the total number of axles. As spans of 30m and less were considered, the actions of a single heavy vehicle are known to be critical (Nowak, 1991). Single heavy vehicles that could obtain, or come close to obtaining, the maximum legal Gross Vehicle Mass (GVM) of 560kN (National Road Traffic Regulations, 1999) were therefore determined to be of interest. As a result, only vehicles containing 6 axles and more were studied.

In the analysis of the WIM data, the details of over 200, 9-axle vehicles were extracted from the vehicle population set. The majority of these vehicles contained four axle axle-sets. Given the number of axles in the axle-set, these vehicles were considered as abnormal loads and outside the scope of the study. Any 6, 7 or 8 axle vehicle containing a four axle axle-set was also considered as an abnormal load and extracted from the vehicle population set. The remaining 6, 7 or 8 axle vehicles were judged to represent normal traffic. The total number of vehicles used in extrapolating the load effects is shown in Table 3.1.

	6 Axle Vehicles	7 Axle Vehicles	8 Axle Vehicles	Total
No. of Recorded Vehicles	24,901	34,951	2,587	62,079

Table 3.1 - Number of Recorded Heavy Vehicles

In the analysis of the vehicles, a VB program was written to calculate the maximum bending moments and shears caused by each vehicle moving across a simply supported span. Spans ranging from 5m to 30m were considered, as per RR 91/004/01 & 02. From these results, the statistical distribution of the bending moments and shears forces for each span was found. The results obtained are shown in Table 3.2 and 3.3. The VB programs written are listed in Appendix F.

		Bending Moments (kNm)				
		Spans (m)				
		5	10	15	20	30
6 Axle Veh.	Mean	104.0	280.1	473.4	750.7	1431.9
	Std Dev.	33.4	88.7	145.8	242.6	461.8
7 Axle Veh.	Mean	108.4	283.8	518.1	847.4	1704.6
	Std Dev.	33.0	82.9	154.3	257.6	552.7
8 Axle Veh.	Mean	104.5	284.4	519.6	841.3	1698.3
	Std Dev.	34.5	91.5	170.1	279.4	586.5

Table 3.2 - Actual Vehicle Bending Moments – Statistical Properties

		Shear Forces (kN)				
		Spans (m)				
		5	10	15	20	30
6 Axle Veh.	Mean	95.2	122.4	150.0	178.2	211.5
	Std Dev.	30.9	37.0	49.1	58.3	68.4
7 Axle Veh.	Mean	98.9	131.2	163.9	197.4	248.4
	Std Dev.	29.5	38.5	50.1	61.9	77.7
8 Axle Veh.	Mean	95.6	129.0	162.5	194.1	244.3
	Std Dev.	32.4	41.8	54.6	65.4	81.5

Table 3.3 - Actual Vehicle Shear Forces – Statistical Properties

### 3.2.2 Legal Vehicles

The intent of the “legal” set of vehicles was to generate a set of vehicles fully laden to the limits allowed by the National Road Traffic Regulations (1999). Previous research (RR 91/004/01 & 02, 1995) used a garage of 28 legal vehicles that adhered to the Van Wyk & Louw Classifications (1991). It is proposed that the creation of a set of 62,079 “legal” vehicles, whose axle spacing and configurations are truly representative, develops this approach further. It is evident that the significant size of the population set provides a more representative sample than the smaller hand picked garage of vehicles.

A VB program was written to first assign the maximum allowable axle mass to the vehicles. The gross vehicle mass was then checked against the maximum allowable weight of 56 tonnes. Compliance with the bridge formula was also checked. If the vehicles did not comply with the GVM or the bridge formula, the axle masses were reduced till compliance was achieved. In reducing the axle masses, the maximum permissible mass of the central tandem or tridem axle set was retained; the balance of the vehicle mass was then proportionally allocated to the remaining axles. This method was aimed at producing the maximum load effects from each legal vehicle. For the span lengths considered, it was important the critical axle sets were loaded to their legal limits (O’Connor, C., 1981). If this was not done the maximum “legal” load effects would be underestimated and the overloading factor overestimated.

In order to correctly assign the axle masses it was necessary to identify the various configurations of vehicles that were present on the N3. The recorded vehicle configurations with the assigned maximum axle masses are shown in Appendix A.

As in the case of the “actual” vehicles, the statistical distribution of the bending moments and shears forces caused by the “legal” vehicles was calculated. The results obtained are shown in Tables 3.4 and 3.5. The results represent the legal maximum load effects and as a result have a relatively small standard deviation.

		Bending Moments (kNm)				
		Spans (m)				
		5	10	15	20	30
6 Axle Veh.	Mean	167.7	456.1	764.4	1247.2	2424.2
	Std Dev.	6.5	21.4	25.2	48.1	62.0
7 Axle Veh.	Mean	161.2	399.5	759.5	1270.7	2633.9
	Std Dev.	5.5	18.3	44.7	51.2	63.6
8 Axle Veh.	Mean	171.7	460.7	850.3	1372.6	2730.1
	Std Dev.	8.5	38.5	70.5	74.8	74.2

Table 3.4 - Legal Vehicle Bending Moments – Statistical Properties

		Shear Forces (kN)				
		Spans (m)				
		5	10	15	20	30
6 Axle Veh.	Mean	152.8	195.3	251.9	305.5	364.3
	Std Dev.	9.7	7.6	11.7	12.4	13.0
7 Axle Veh.	Mean	143.1	194.0	250.1	295.9	376.4
	Std Dev.	9.5	12.0	9.6	10.1	10.7
8 Axle Veh.	Mean	155.5	211.1	265.4	303.0	383.0
	Std Dev.	12.1	16.7	15.2	18.1	33.5

Table 3.5 - Legal Vehicle Shear Forces – Statistical Properties

### 3.2.3 National Road Traffic Regulations

The National Road Traffic Regulations (1999) limit the GVM and individual axle masses of heavy vehicles on South African roads. The set of “legal” vehicles created complies with the following regulations:

- (i) The axle massload of an axle fitted with two or three wheels, that is a steering axle, shall not exceed 7,700 kilograms;
- (ii) The axle massload of an axle fitted with two or three wheels, that is not a steering axle, shall not exceed 8,000 kilograms;
- (iii) The axle massload of an axle fitted with four wheels shall not exceed 9,000 kilograms;
- (iv) The axle massload of an axle unit that consists of two axles, each of which are fitted with two or three wheels, that is not a steering axle shall, not exceed 16,000 kilograms;
- (v) The axle massload of an axle unit that consists of three or more axles, each of which are fitted with two or three wheels, that is not a steering axle, shall not exceed 24,000 kilograms; and

- (vi) The bridge formula states that total axle massload of any group of axles of a vehicle shall not exceed a mass determined by multiplying the dimension of such group by 2,100 and adding 18,000.

University of Cape Town

### 3.3 STATISTICAL APPROACH

#### 3.3.1 Accuracy of Data

Weigh-in-motion sensors estimate static axle loads from the measurement of dynamic tire loads. Obtaining accurate results requires the careful calibration of the WIM sensors, considering the type and condition of the calibration truck, the driver's performance and the condition of the road surface in the vicinity of the sensor. Between the calibration periods there are many variables that can lead to inaccurate results. These include the behaviour of drivers on the road, the eccentricity of loading and environmental factors such as temperature and wind. A further factor is the magnitude of the axle spacing threshold. If the threshold value is exceeded, the programme records a separate vehicle. Therefore, if the headway distance between two vehicles is less than the axle threshold, the programme will record two vehicles as a single vehicle.

The WIM system that is used to collect the data on the National Route 3 is set to meet the *International Standard Specification for Highway Weigh-In-Motion (WIM) Systems with User Requirements and Test Methods*, E1318-02. This specification requires that the error in the estimated static wheel load should not exceed 25%. Given the number of variables mentioned, achieving this target requires the daily monitoring of the recorded WIM data.

It is recognised that the magnitude of the recorded axles loads used in this study may be 25% more or less than the actual vehicles axle loads present on the road. The possibility for erroneous results that do not represent actual vehicles on the road is also noted. Given the level of uncertainty, individual results are not used to derive definitive conclusions regarding the extreme loading produced by heavy vehicles. The statistical properties of a sample of vehicles are rather used to extrapolate extreme load effects. In using this approach, a single erroneous result will not significantly skew the overall results.

It is pertinent to note that the calibration of the live load model in ENV 1991-3 was in part carried out using WIM data (O'Connor et al., 2001). In that instance the accuracy of the WIM data was set at 5% of the static values as required by the *European specification on weigh-in-motion of road vehicles* (COST 323 1997).

In processing the WIM data, erroneous vehicles were removed from the analysis. These vehicles were identified by either low axle mass readings or by axle spacings that could not logically represent an actual vehicle.

### 3.3.2 General Statistical Properties of WIM Data

The following section provides an overview of the general statistical properties of heavy traffic vehicles travelling on the National Route 3. For this purpose, all heavy vehicles, including 2 to 5 axle vehicles were considered. The first 30,000 vehicles logged during February 2001 were analysed. The sample set was limited to 30,000 vehicles for the purpose of analysing the data in Excel.

The count of the vehicles axle masses, shown in Table 3.6, reveals a significant number of axle masses above the legal limits (90kN) for all axles other than the steering axle. This may be considered indicative rather than representative, given the possible errors in the estimated static axle loads. The results showed that overloading to be particularly prevalent on axles 2 and 3. Whether this trend is due to the overloading of 2 and 3 axle vehicles requires further research.

<b>Max</b>	94.0	130.0	150.0	146.0	118.0	120.0	122.0	38.3
<b>Mean</b>	44.2	51.9	50.7	47.0	48.9	47.5	48.2	19.6
<b>Std Dev</b>	10.0	20.7	20.8	21.6	20.7	21.3	20.7	87.0
<b>Tonnes</b>	<b>Axle 1</b>	<b>Axle 2</b>	<b>Axle 3</b>	<b>Axle 4</b>	<b>Axle 5</b>	<b>Axle 6</b>	<b>Axle 7</b>	<b>Axle 8</b>
5	0	22	79	122	66	85	65	32
10	0	150	231	525	214	465	146	27
15	0	502	431	1192	676	1007	513	46
20	272	1267	1062	1535	1339	1190	831	88
25	1476	2026	1734	1464	1522	1083	679	47
30	1776	2242	1948	1260	1247	899	485	54
35	2051	2146	1654	1128	1119	805	422	48
40	3662	1796	1360	1119	1068	824	464	63
45	5976	1722	1342	1183	1061	943	535	49
50	6436	1769	1304	1301	1233	1135	633	73
55	4961	1775	1454	1476	1489	1375	864	72
60	2460	2291	1850	1803	1788	1672	1195	67
65	794	2747	2328	2117	2026	1805	1200	45
70	144	3033	2590	2039	2046	1658	1080	36
75	24	2694	2107	1528	1571	1218	705	13
80	4	1976	1376	928	933	716	388	9
85	2	1019	719	394	423	275	151	4
90	0	487	305	164	170	127	51	1
95	1	213	112	58	47	47	18	0
100	0	80	43	31	23	19	6	0
105	0	43	14	8	5	5	3	0
110	0	17	8	4	2	2	2	0
115	0	11	9	2	0	3	0	0
120	0	3	3	0	2	2	0	0
125	0	6	1	1	0	0	1	0
More	0	2	1	1	0	0	0	0
<b>Sum</b>	30039	30039	24065	21383	20070	17360	10437	774

Table 3.6 - Counts of Axle Mass Distributions

A histogram of the gross vehicle masses, Figure 3.1, shows a twin peaked bimodal distribution of GVMs from 10 tonnes through to 50 tonnes. This form of distribution is typical for gross vehicle weights (Harman and Davenport, 1979; O'Connor et al., 2001). The first mode contains the partially loaded 4 to 7 axle vehicles and the fully loaded 2 and 3 axle vehicles. The second mode involves the fully loaded 5 to 7 axle vehicles. Knowledge of the frequency of GVM's is off use in Monte Carlo simulations that employ vehicles that reflect actual traffic flows (O'Connor et al., 2001).

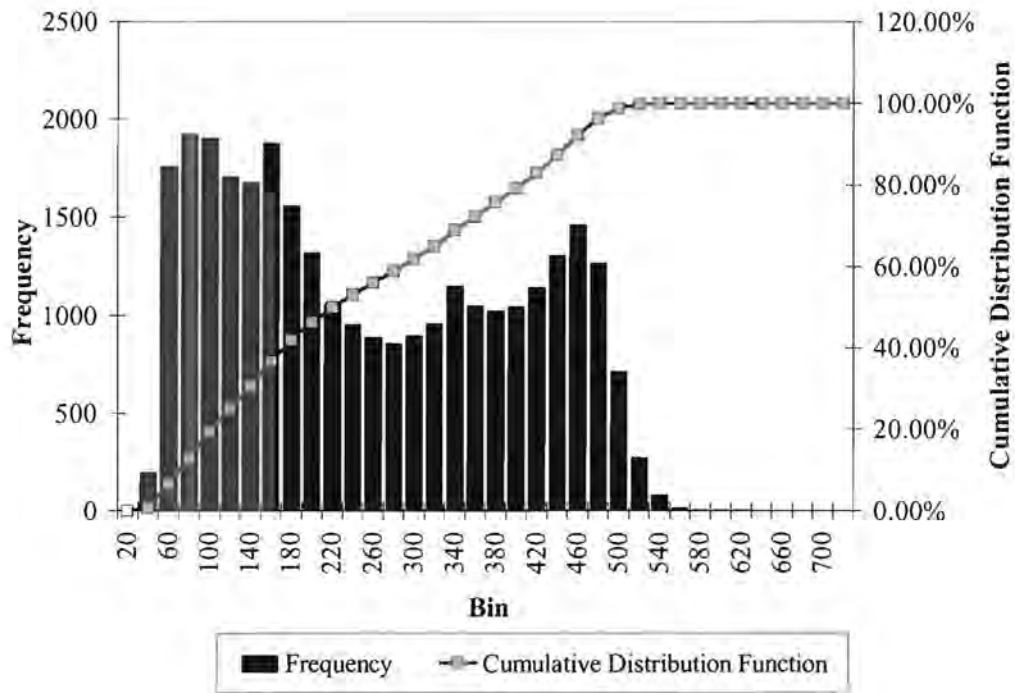


Figure 3.1 – Histogram of Sample GVM's

The probability density function of the gross vehicle mass was observed to follow a normal distribution as shown in Figure 3.2. As shown in Figures 3.3 and 3.4, the maximum bending moments caused by vehicles crossing a 5m and 30m span also follows a normal distribution. This confirms Nowak's (1991) approach and the distributions found in derivation of the live load model in ENV 1991-3 (Dawe, 2003).

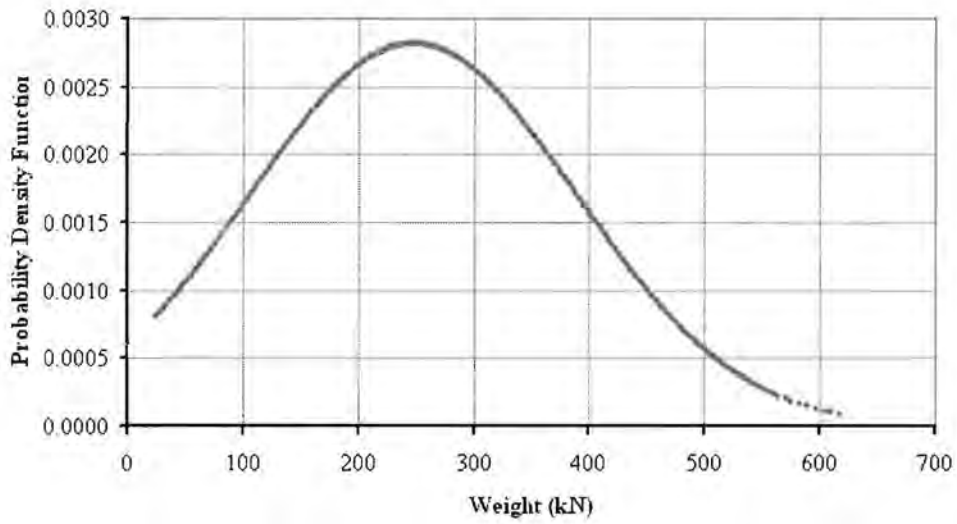


Figure 3.2 - Probability Density Function of GVM's – 6 Axle Vehicles

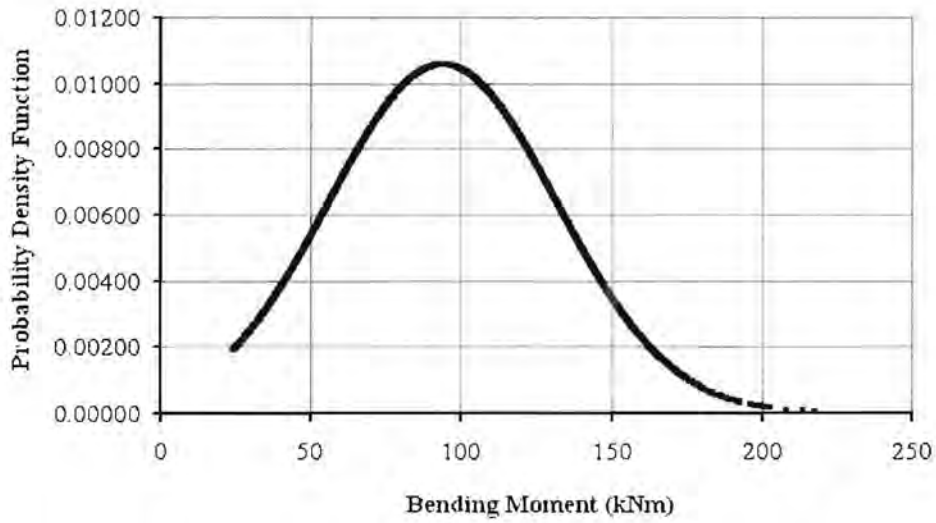


Figure 3.3 - Probability Density Function of Bending Moments – 30m span

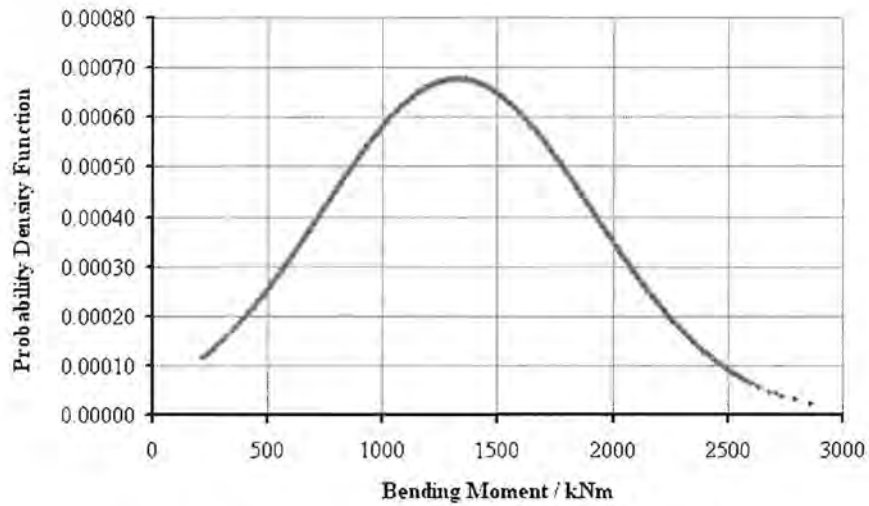


Figure 3.4 - Probability Density Function of Bending Moments – 5m span

### 3.4 STATISTICAL DISTRIBUTIONS

Using the WIM data, the main objective of the study was to predict the extreme bending moment and shear forces that would be experienced by a bridge structure during its design life. Given that live loading due to traffic is a random time dependent variable, a probability distribution function may be fitted to the observed events. This theoretical distribution may then be used to predict extreme events with a given probability of exceedance. Similarly, for a given time period, the exceedance probability may be specified and the extreme event predicted.

In review of current research, two distinct approaches for extrapolating extreme events were identified. The first of these was developed by Nowak (1991) in the calibration of the LRFD (1994). The method assumed that a normal distribution best fitted the load effects derived from a surveyed population of trucks identified as being overloaded. The second method, used in RR 91/004/01 & 02 (1994 & 1995), involved the use of the Gumbel extreme distribution, to extrapolate a set of extreme events obtained from a Monte Carlo simulation. A similar approach was in the calibration of the live load model in ENV 1991-3 (O'Connor et al., 2001).

The present study undertakes to review both approaches for the purposes of comparison. In the case of the application of an extreme distribution the study further aims to;

- (i) Assess which extreme distribution best describes the characteristic properties of the extreme events; and
- (ii) Investigate the sensitivity of the analysis in relation to the size of the extreme event population.

In RR 91/004/01, the load effects of the traffic streams were extrapolated to represent the total number of traffic streams expected in a 50 year return period. This approach was amended in RR 91/004/02, where the load effects were extrapolated to a level that had a 5% probability of being exceeded within a 120 year design life. The resulting return period of 2976 years is more onerous than the 120 year and 1000 year return periods associated with the live load models in BD 37/01 and ENV 1991-3 respectively.

For the purposes of the study, it was considered that characteristic loads are those appropriate to a return period of 120 years as per the recommendations of BD 37/01 (2001). This assumption was considered valid given that the extrapolated results are relatively insensitive to variation in the return periods (Dawe, 2003).

### 3.4.1 Normal Distribution

Nowak's (1991) method of using a normal distribution to extrapolate the load effects from a set of heavy trucks was carried out using the recorded 6, 7 and 8 axle vehicles. Each class of vehicle was considered separately, with the bending moments and shears forces being calculated for 5m, 10m, 15m, 20m and 30m spans. The mean and standard deviation of the load effects was calculated for each span and for each class.

As in case of Nowak (1991), the number of vehicles was assumed as representative of the survey period. For example, the 24,901 recorded 6 axle vehicles were taken as characteristic of a one month period. For the proposed 120 year return period, the total population of 6 axle vehicles occurring was assumed to be 1440 (120 x 12) times larger. This gave a total number of 6 axle vehicles,  $N$ , of 35,857,440. The probability level corresponding to the maximum truck event was then calculated as  $1/N$ .

In calculating the standard normal distribution value,  $z$ , the intermediate variable,  $w$ , was first calculated and inputted into the formula estimating  $z$  (Chow, 1988).

$$w = \left[ \ln \left( \frac{1}{P^2} \right) \right]^{1/2} \quad (3.1)$$

$$z = w - \frac{2.515517 + 0.80853w + 0.010328w^2}{1 + 1.432788w + 0.189269w^2 + 0.001308w^3} \quad (3.2)$$

Where:

- $P$  = exceedence probability
- $w$  = intermediate variable
- $z$  = standard normal distribution value.

Given that a normal distribution was assumed, the frequency factor,  $K_T$ , was equal to  $z$ . The magnitude of an event at given time,  $T$ , was therefore calculated using the formula:

$$x_T = \bar{x} + K_T s \quad (3.3)$$

Where:

- $x_T$  = Event at time  $T$
- $\bar{x}$  = Mean of events
- $K_T$  = Frequency factor

s = Standard deviation of events

The results of the analysis are shown in Table 3.7 and 3.8. For spans of less than 15m, the results are consistent for each class of vehicle. For the 20m and 30m spans the predicted load effects of the 6 axle vehicles are lower than those of the 7 and 8 axle vehicles. These load effects are lower because for 20m and 30m spans the effects of a complete vehicle are dominant. The average GVM of a 6 axle vehicle is less than its 7 or 8 axle counterparts, as shown in Table 3.9. The extrapolated load effects are therefore lower. In the case of the 5m, 10m and 15m span, individual axles and axle sets are the dominant. Overloaded axle sets were observed in each of the vehicle classes and the predicted load effects are therefore similar.

Span /m	Bending Moments (kNm)			Max
	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.	
5	285	290	278	290
10	762	739	743	762
15	1266	1365	1372	1372
20	2069	2262	2242	2262
30	3940	4740	4638	4740

Table 3.7 - Bending Moments Extrapolated using the Normal Distribution

Span /m	Shear Forces (kN)			Max
	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.	
5	263	261	258	263
10	323	343	339	343
15	417	439	436	439
20	495	537	522	537
30	583	675	653	675

Table 3.8 - Shear Forces Extrapolated using the Normal Distribution

Vehicle Class	Maximum GVM (kN)	Average GVM (kN)	Standard Deviation	Skewness
6 Axle	533	281	91	-0.21
7 Axle	653	367	118	-0.73
8 Axle	584	363	122	-0.56

Table 3.9 - Statistical Properties of Recorded Vehicle Classes

In conclusion, Nowak's method is intended for the review of a complete set of heavy vehicles rather than subsets of that data. However, the load effects of each class were observed to follow a normal distribution and the maximum extrapolated values (7 & 8 axle vehicles) are considered representative of the total surveyed population. Due to the large sample set used, the 95% confidence limits gave a very small range for the true event magnitude about the extrapolated 120 year event. For 6 and 7 axle vehicles, a variance of 0.4% about the predicted event was calculated. In the case of 8 axle vehicles a variance of 1.2% was found. A comparison of the extreme events predicted using Nowak's method with those obtained using the extreme distribution is given in Section 3.5.

### 3.4.2 Extreme Distributions

For independent events, such as traffic loading, it is often the case that the distribution of maximum extreme events is relatively insensitive to the distribution of the common events. The method developed in RR 91/004/01 & 02 of extracting a set of extreme events from the set of common events was therefore undertaken. In this instance, extreme events were selected from the visual inspection of the load effect distribution graphs. The extreme distribution best describing the characteristics of those events was then used to extrapolate the results.

The practice of plotting the magnitude of an event against a linearised exceedence probability was used to visually identify the distribution of the load effects for each class of vehicle, for each span. Having sorted and ranked the load effects, the exceedence probability of the  $m^{\text{th}}$  largest value  $x_m$  was calculated using Cunnane's formula (Cunnane, 1978).

$$P(X > x_m) = \frac{m - 0.4}{n + 0.2} \quad (3.4)$$

Where:

$P(x)$  = Exceedence probability of event  $x$

$n$  = Total number of values

$m$  = Rank of value in a list ordered by descending value

Cunnane (1978) derived the formula from the study of plotting positions using the criteria of unbiasedness and minimum variance. An unbiased plotting method is one that will cause the average of the plotted points of each value of,  $m$ , to fall on the theoretical distribution line.

For the purpose of plotting the distribution graph of the load effects, the reduced variate,  $y$ , of the exceedence probability was calculated using the formula below (Chow, 1988). The distribution graph of the bending moments and shear forces associated with a 6, 7 and 8 axle vehicle on a 30m span is shown in Figures 3.5 and 3.6. The complete set of the distribution graphs for all spans is contained with Appendix B.

$$y_T = -\ln \left[ \ln \left( \frac{T}{T-1} \right) \right] \quad (3.5)$$

Where:

$T$  = Return Period, where  $T = \frac{1}{P}$

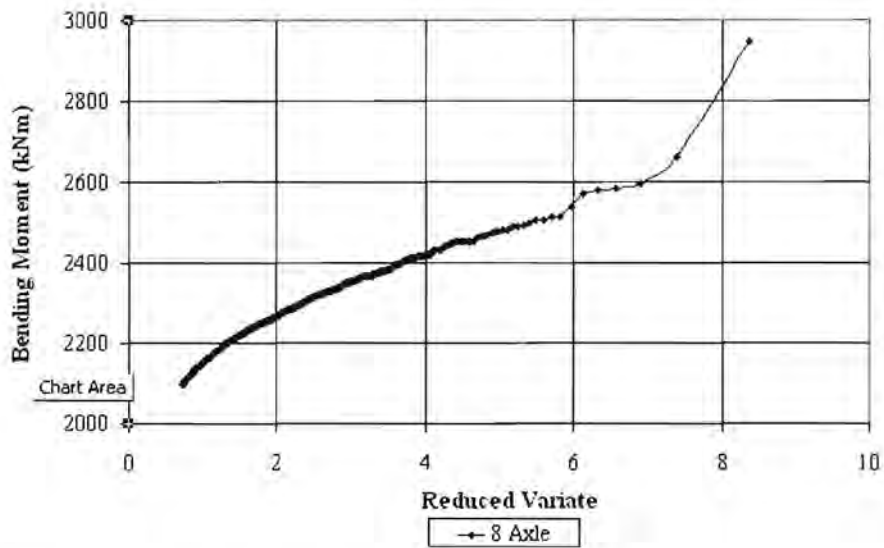
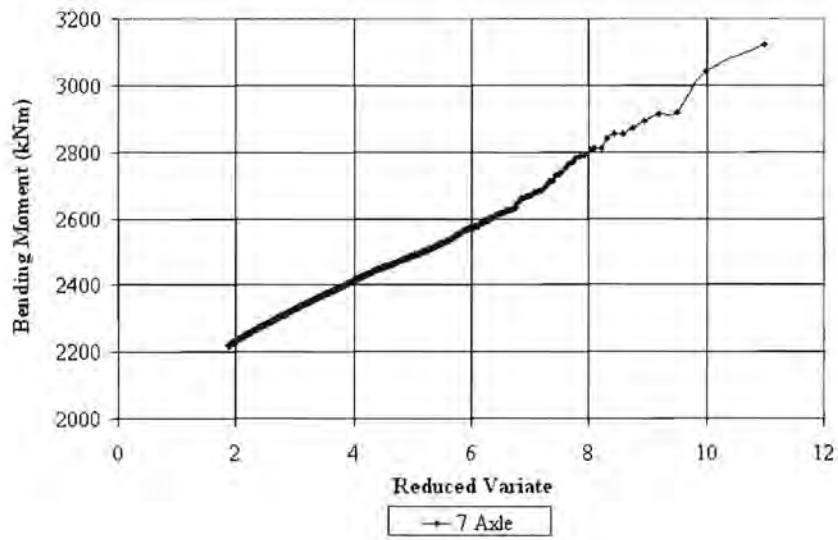
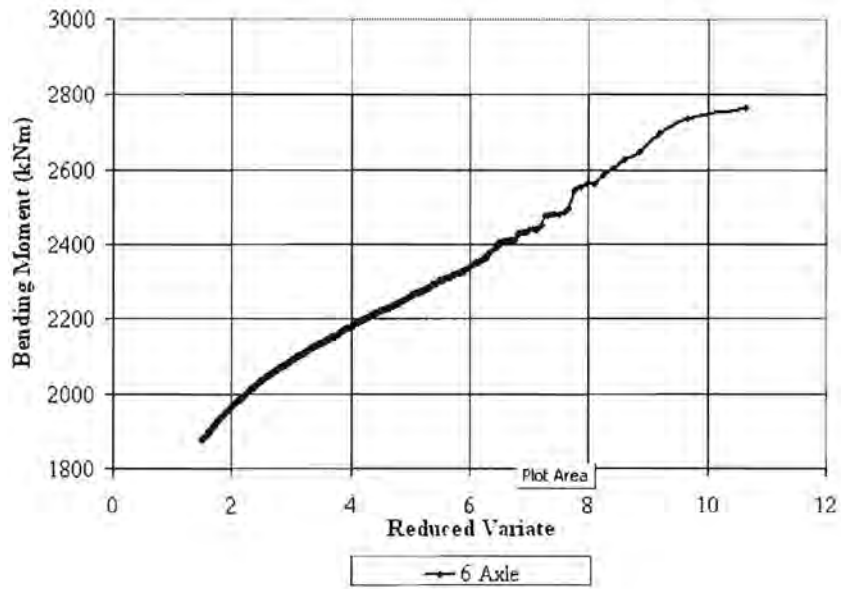


Figure 3.5 - Distribution of Bending Moments - 30m span

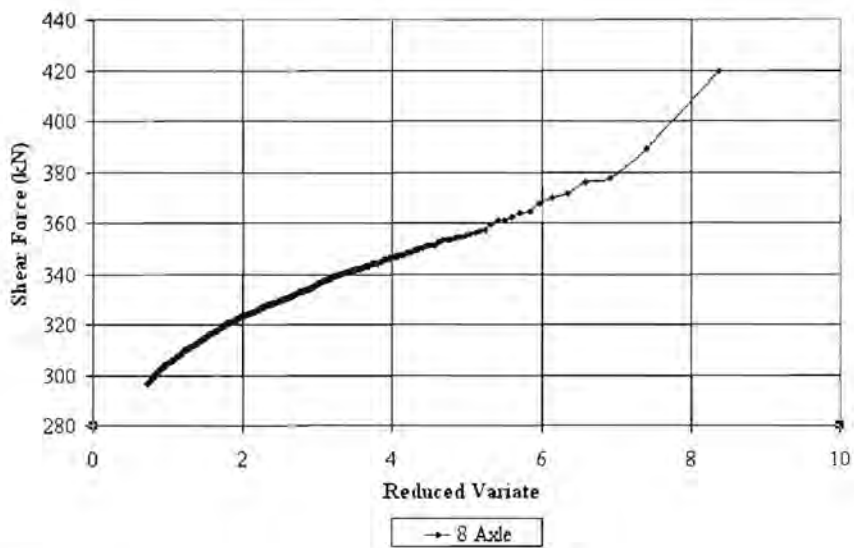
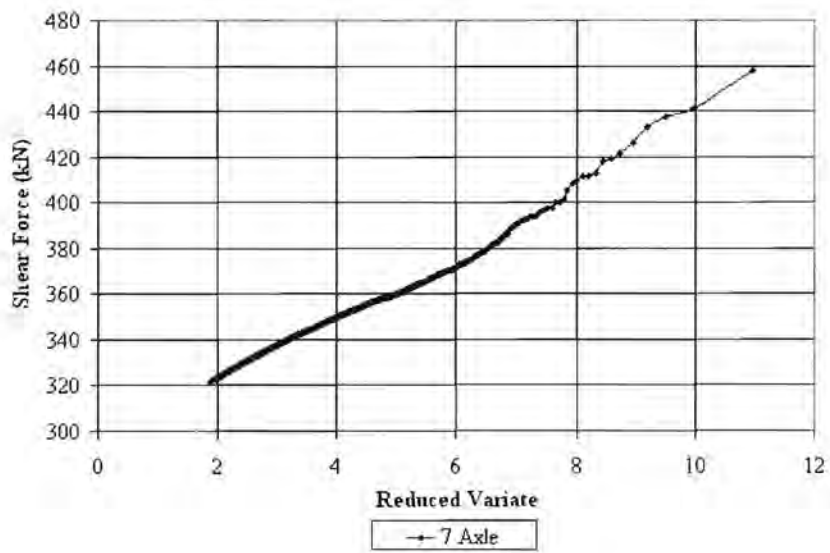
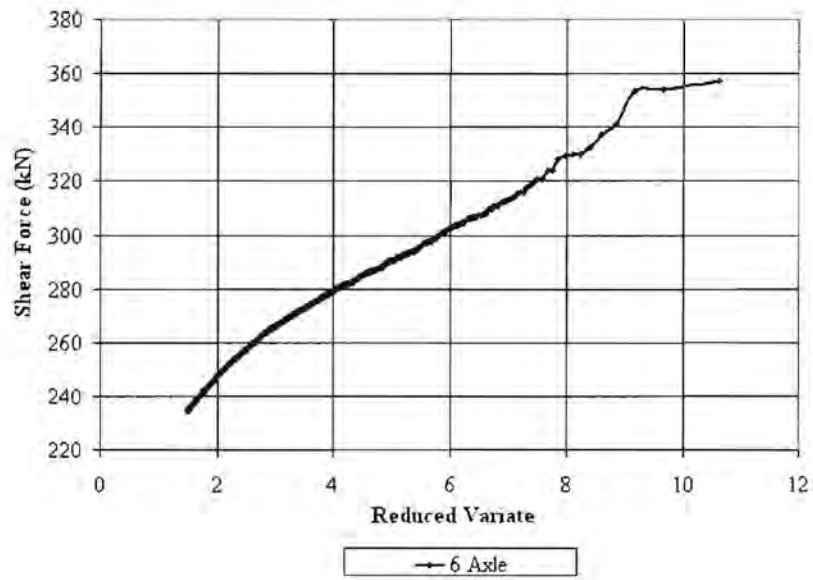


Figure 3.6 - Distribution of Shear Forces - 30m span

From the plotted graphs it was possible to identify the cases where the maximum events deviated from the regression of the general population. The distribution graph for each span for each vehicle class was reviewed and the set of extreme events extracted. A sensitivity analysis was undertaken to assess the impact of varying the population size of the extreme events. This was done by calculating the mean, standard deviation and skewness of the extreme events and using the Gumbel distribution to extrapolate a 1 in 120 year event. The predicted 1 in 120 year events for the various population sizes were then compared. The results of this sensitivity analysis, for the extreme bending moments caused by 6 Axle vehicles, are shown in Table 3.10. Where a straight line was easily regressed, it was confirmed that the distribution was relatively insensitive to the population size. In the case where distribution of the extreme events deviated substantially from the general distribution, the distribution was relatively sensitive to the size of the population assumed. In these cases, the final decision on the population size was done by the visual inspection of the distribution graph.

	No. of Extreme Events	Mean	Standard Deviation	Periods in 120 Years	Reduced Variate, yt	Frequency Factor, Kt	Event magnitude, xt, (kNm)
5m Spans	22	202.4	9.9	31680	7.51	7.63	278
	28	200.0	9.9	40320	7.27	7.82	277
	42	196.1	9.8	60480	6.87	8.13	276
10m Spans	22	544.8	26.7	31680	7.51	7.63	749
	28	537.9	27.1	40320	7.27	7.82	750
	42	525.4	28.5	60480	6.87	8.13	757
15m Spans	22	893.7	37.6	31680	7.51	7.63	1181
	28	883.5	38.7	40320	7.27	7.82	1186
	42	865.9	40.5	60480	6.87	8.13	1195
20m Spans	22	1372.8	58.3	31680	7.51	7.63	1818
	28	1357.2	59.7	40320	7.27	7.82	1824
	42	1331.3	61.1	60480	6.87	8.13	1828
30m Spans	22	2546.8	96.9	31680	7.51	7.63	3285
	28	2522.4	97.9	40320	7.27	7.82	3288
	42	2483.1	97.4	60480	6.87	8.13	3275

Table 3.10 - Sensitivity of Predicted Bending Moments for 6 Axle Vehicles

In RR 91/004/02, the extreme set of results was assumed to be the top 15% of the sample set. A sensitivity analysis compared this approach to the graphical regression of each distribution. The results, shown in Table 3.11, demonstrate that the method used in RR 91/004/02 gives results up to 22% lower. For this reason, the extreme events are defined from the regression of each distribution plot rather than from an assumed percentage of the total sample set.

Bending Moments (kNm)			
Span (m)	Graphically Regressed Sample	15% of Survey Population Sample	% Difference Graphic: 15%
5	301	238	-21%
10	812	631	-22%
15	1364	1129	-17%
20	2097	1756	-16%
30	3631	3380	-7%

Table 3.11 - Sensitivity of Predicted Bending Moments for 6 Axle Vehicles to Sample Size

Having isolated the set of extreme events, the statistical properties of the events were used to fit an appropriate theoretical distribution. As for the general population set, the extreme events were sorted, ranked and an exceedence probability was calculated. The graph of the magnitude of the event versus the reduced variate was then plotted. This graph was used to plot fit the various theoretical distributions to the distribution of the plotted points.

Given the nature of the data, the extreme distributions were considered. It has been shown that the distributions of extreme events converge to one of three forms of extreme value (EV) distributions. EV1 and EV2 are also known as the Gumbel and Frechet distributions respectfully. If a variable,  $x$ , is described by the EV3 distribution, then  $-x$  is said to have a Weibull distribution. Given the positive skewness of the data, the Wiebull distribution was not considered further. In addition to the extreme distributions, the Normal and the Log Normal distributions were considered for the purposes of comparison. The distribution of the load effects for 6, 7 and 8 axle vehicles on a 15m span, together with the various theoretical distributions, are shown in Figures 3.7 to 3.9. A complete set of the distribution graphs for each vehicle class and span is included within Appendix B.

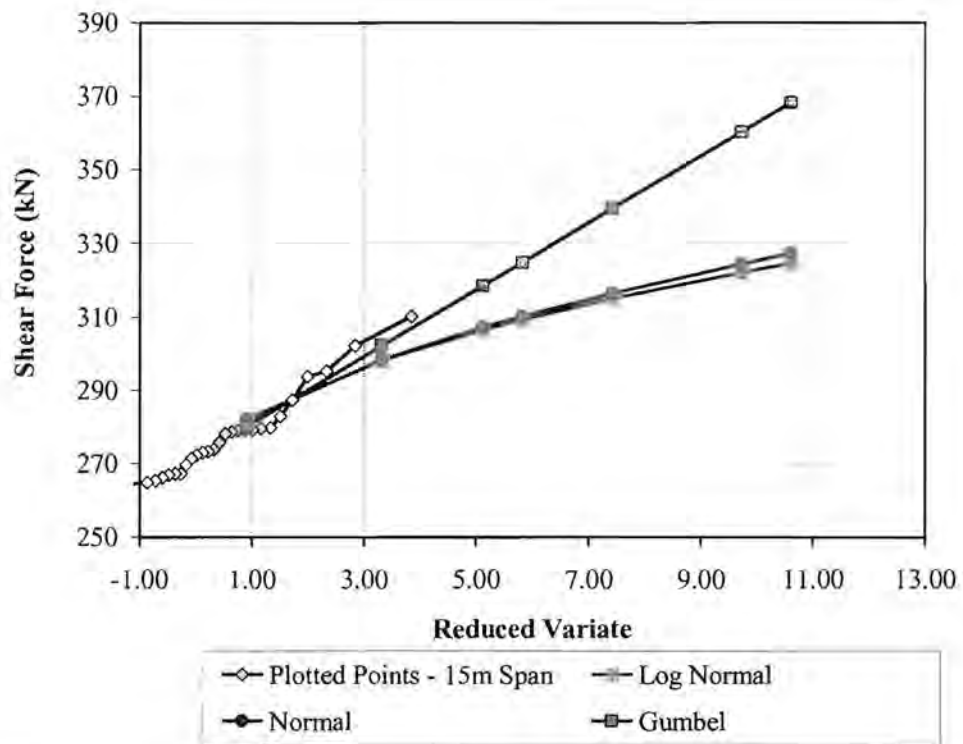
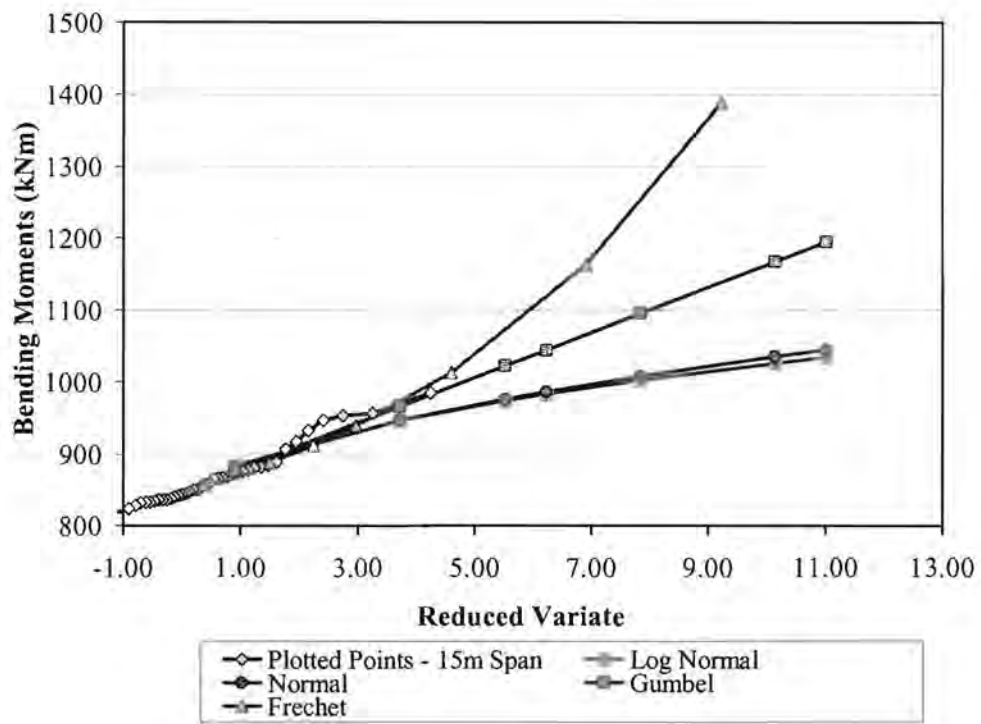


Figure 3.7 - Fit of Theoretical Distributions to Plotted Points - 6 Axle Vehicles on 15m spans

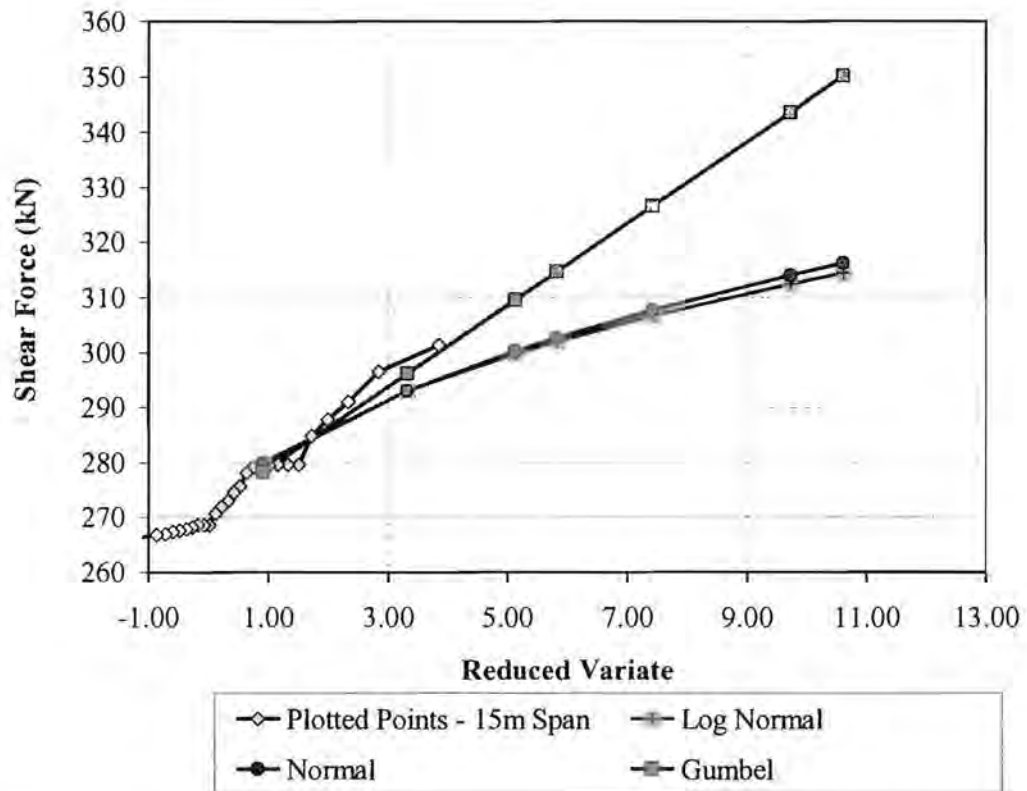
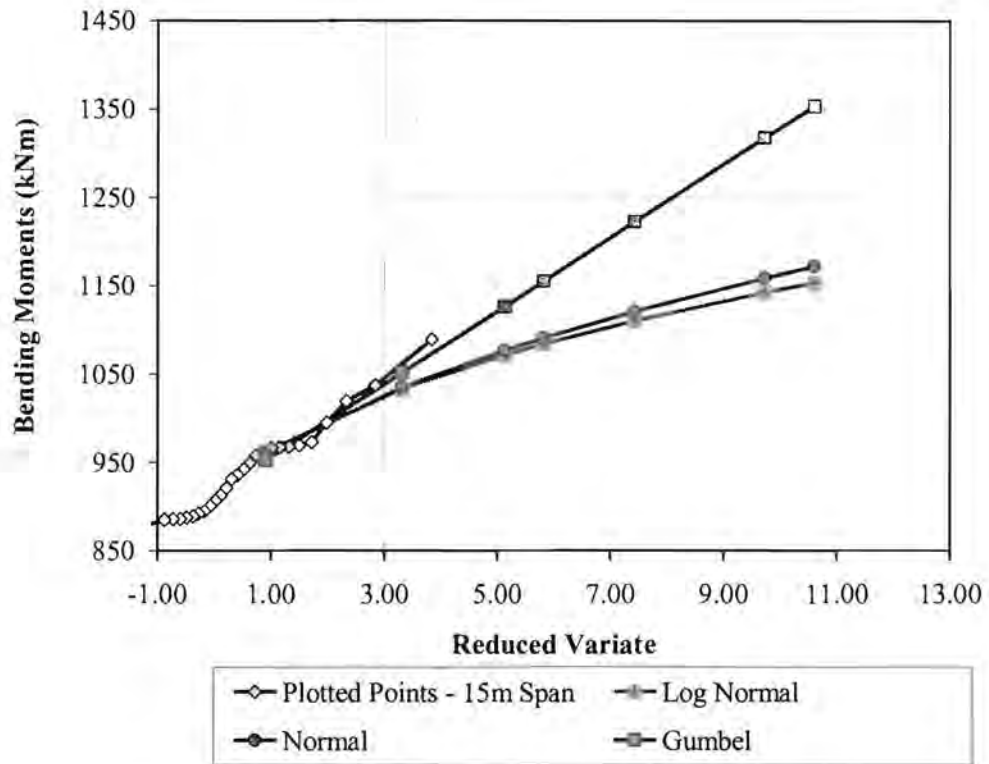


Figure 3.8 - Fit of Theoretical Distributions to Plotted Points - 7 Axle Vehicles on 15m spans

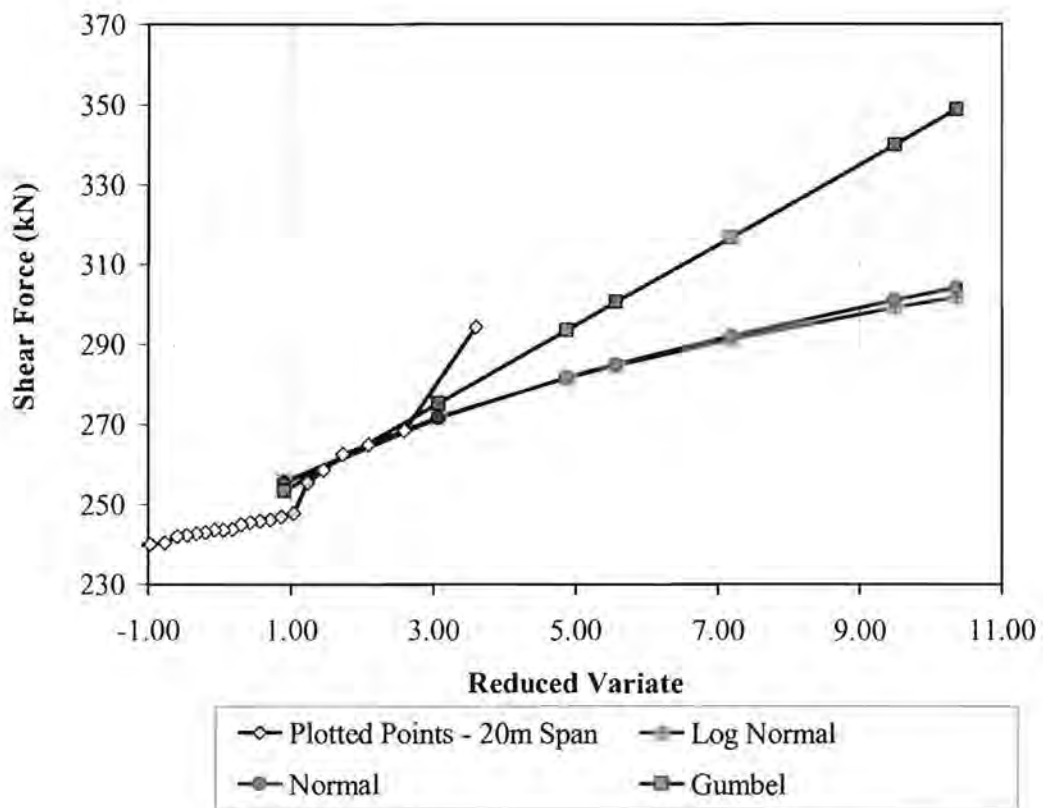
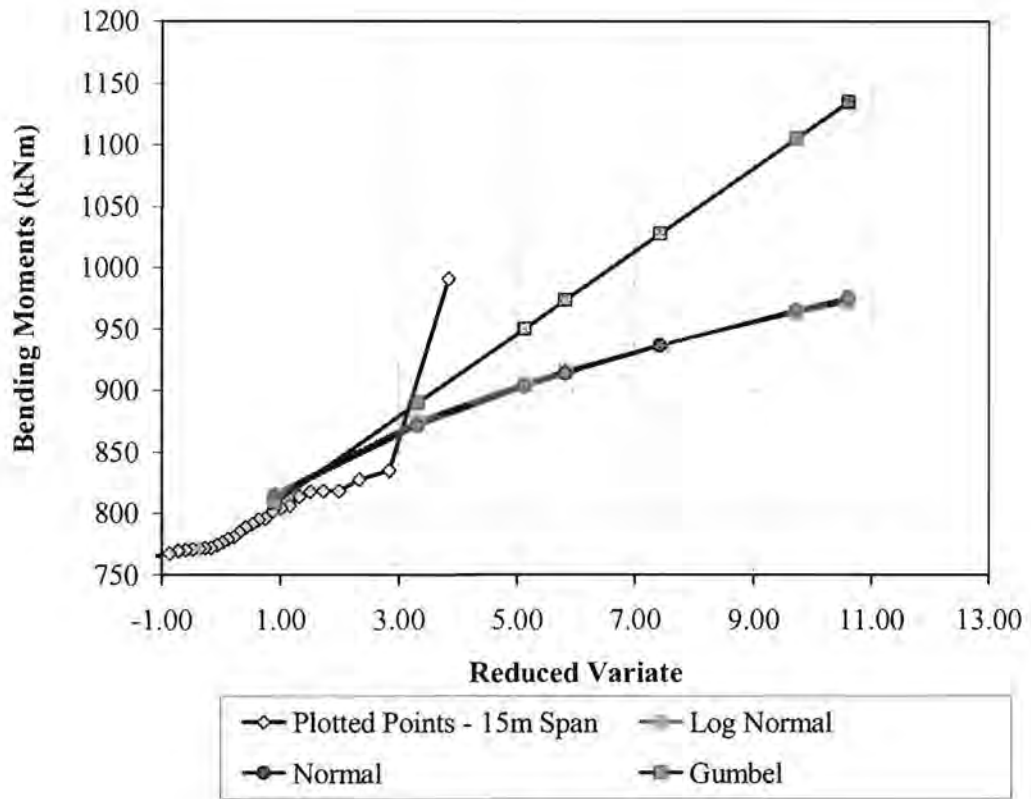


Figure 3.9 - Fit of Theoretical Distributions to Plotted Points - 8 Axle Vehicles on 15m spans

In plotting the distributions, the frequency factor,  $K_T$ , was calculated for the Normal and Gumbel distributions. In the case of the Normal distribution, the frequency factor was taken as,  $z$ , the standard normal distribution variable. In the case of the Gumbel distribution, the following formula was used (Chow, 1988):

$$K_T = -\frac{\sqrt{6}}{\pi} \left\{ 0.5772 + \ln \left[ \ln \left( \frac{T}{T-1} \right) \right] \right\} \quad (3.6)$$

Where  $T$  = Return Period

In considering the effective return period, it was necessary to note the population size of the extreme events and the fact that survey data represented a single month of traffic flow. Each event was set to represent an independent period within the month. Where 28 extreme events were identified, 28 effective periods were judged to have occurred within the month. In considering a return period of 120 years, 40320 ( $28 \times 12 \times 120$ ) effective periods were considered to have occurred. In the case where only 16 extreme events were found, for a return period of 120 years, 23040 ( $16 \times 12 \times 120$ ) effective periods were considered to have occurred.

In review of the distribution graphs, the plotted points did not extend sufficiently to allow a conclusive comparison with the various distributions. Although a significant number of vehicles were analysed, a limited number of extreme events were identified. Typically these events had a return period of 1 day. It is apparent that extreme events representing a larger time period are required to conclusively identify the distribution of events up to a return period of 120 years.

In review of the Normal and the Log Normal distributions, it was noted that for the shorter spans, a number of the plotted events were within 5% of the extrapolated 120-year event. A variance of 5% between the 28 day events and the 120 year events was judged unacceptable and the use of the Normal and the Log Normal distributions was discounted.

Considering the extreme distributions, the data's positive skewness points to the use of the Frechet and the Gumbel distributions. In the majority of cases, the Frechet Distribution predicted events far in exceedence of those of the Gumbel distribution, as shown in Figure 3.7.

In the design of bridge structures, a high degree of confidence is required given the human and economic cost of a structures' failure. Although the Frechet Distribution provided the most conservative predictions, the magnitude of the events was considered beyond that expected from traffic live loading. The Gumbel distribution was therefore chosen as providing a distribution that will adequately cater for extreme events and potential outliers.

### 3.4.3 Confidence limits

In review of the events predicted by the chosen statistical distribution, the true event magnitude may lie within a range about the extrapolated value. To quantify this range, confidence limits were found using the standard error of estimate. The size of confidence limits are dependent on the confidence level,  $\beta$ , and associated with the confidence level is a significance level,  $\alpha$ , given by,

$$\alpha = \left( \frac{1 - \beta}{2} \right) \quad (3.7)$$

For example, for a confidence level of 95%, the significance level is 2.5%  $((1-0.95)/2)$ .

For a sample of size,  $n$ , and standard deviation,  $s$ , the below equations were used to calculate the standard error of estimate,  $s_e$ , for the Normal and Gumbel distributions.

$$\text{Normal} \quad : \quad s_e = \left( \frac{2 + z^2}{n} \right)^{1/2} s \quad (3.8)$$

$$\text{Gumbel} \quad : \quad s_e = \left[ \frac{1}{n} (1 + 1.1396K_T + 1.1000K_T^2) \right]^{1/2} s \quad (3.9)$$

The confidence limits were calculated for a confidence level of 95%. For an event,  $x_T$ , the confidence limits were taken as  $x_T \pm s_e z_\alpha$ . For a confidence level of 95% the standard normal variable,  $z$ , is 1.96.

An example of the plotted confidence limits is shown in Figure 3.10. In the case of the 7 axle vehicles, the confidence limits for the extrapolated 1 in 120 year bending moments are  $\pm 10\%$ . This error, compounded with the inherent inaccuracies of the WIM data, is significant. The means of reducing it include the use of a larger set of extreme events.

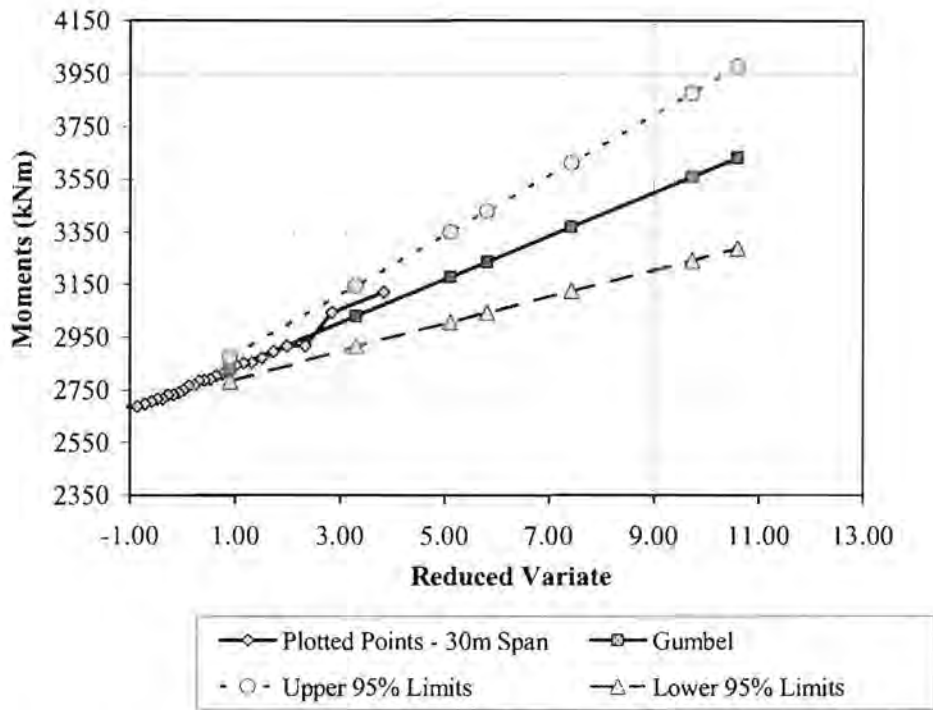


Figure 3.10 - Confidence limits for Gumbel Distribution for 7 Axle Vehicle on 30m Span

### 3.5 RESULTS

Using the Gumbel distribution, the load effects of the “actual” vehicles were extrapolated to a 1 in 120 year event. The results of that extrapolation are shown in Table 3.12 & 3.13. The results are characteristic load effects that represent serviceability loads using limit state principles as discussed in Section 3.4. In combination with an impact factor, the load effects generated from actual traffic data may be compared to those calculated by TMH7 Part 2 and the design load derived by Reports RR 91/004/01 & 02.

Span (m)	Bending Moments (kNm)			Max.
	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.	
5	276	301	231	301
10	757	812	624	812
15	1195	1364	1135	1364
20	1828	2097	1780	2097
30	3275	3631	3289	3631

Table 3.12 - Extrapolated Bending Moments

Span (m)	Shear Forces (kN)			Max.
	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.	
5	297	233	201	297
10	320	314	279	320
15	369	350	349	369
20	431	433	392	433
30	489	542	481	542

Table 3.13 - Extrapolated Shear Forces

The results show that the 7 axle vehicles produced the highest predicted bending moments. For the spans of 15m and greater this was to be expected as:

- (i) The average GVM of 7 axle vehicles is greater than the 6 axle vehicle, as shown in Table 3.10; and
- (ii) In complying with the bridge formula and the GVM restrictions, the 7 axle vehicle can achieve higher axle masses in closer proximity than its 8 axle counterpart.

The 7 axle vehicles cause larger bending moments on the shorter spans because their axles and axle sets are statistically heavier than those of the 6 and 8 axle vehicles. Table 3.14 that details the statistical properties of the surveyed vehicles supports this statement. No specific trends in the shear force results were observed between the vehicle classes; this is because the shear load effects are not as sensitive to the vehicle's axle configurations.

		Weights (kN)								
		Axle 1	Axle 2	Axle 3	Axle 4	Axle 5	Axle 6	Axle 7	Axle 8	GVM
6 Axle Vehicles	Mean	46.7	52.9	51.8	42.3	42.9	44.1			280.6
	Std Dev	6.8	19.6	19.9	19.9	18.8	18.9			91.3
	Skewness	0.06	-0.14	-0.09	0.12	0.01	0.03			-0.21
7 Axle Vehicles	Mean	48.8	55.7	55.5	52.9	54.5	50.1	49.0		366.5
	Std Dev	6.7	20.1	19.1	21.5	20.4	22.4	20.5		117.6
	Skewness	-0.16	-0.59	-0.50	-0.55	-0.50	-0.45	-0.36		-0.73
8 Axle Vehicles	Mean	47.0	50.4	52.6	45.5	45.6	44.6	39.1	38.4	363.2
	Std Dev	6.7	20.0	19.8	19.7	18.7	19.7	18.9	18.6	121.6
	Skewness	-0.20	-0.34	-0.28	-0.24	-0.25	-0.15	-0.06	-0.03	-0.56

Table 3.14 - Statistical Properties of Axle Weights and GVM

In review of the extrapolated events, it is important to recognise that the true event magnitude probably sits within a range about the predicted events. Using 95% confidence limits, this range was calculated in each case. The ranges of  $\pm 11\%$ , shown in Tables 3.15 and 3.16, are significant when considering the possible error in the WIM data. The results reinforce the need for a larger population of extreme events.

Span (m)	$\pm$ % of 95% Confidence Limits About the Predicted Bending Moment Event		
	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.
5	9.8%	14.7%	11.7%
10	10.4%	14.1%	11.9%
15	9.3%	12.8%	12.4%
20	9.2%	12.8%	11.9%
30	8.2%	9.5%	9.7%

Table 3.15 - Predicted Bending Moment Confidence Limits

Span (m)	± % of 95% Confidence Limits About the Predicted Shear Force Event		
	6 Axle	7 Axle	8 Axle
	Veh.	Veh.	Veh.
5	15.1%	13.0%	10.4%
10	12.0%	11.0%	12.7%
15	10.3%	8.8%	13.3%
20	10.3%	9.5%	11.6%
30	9.4%	10.3%	11.3%

Table 3.16 - Predicted Shear Force Confidence Limits

It was observed that the load effects generated, using Nowak's method, were generally higher than those calculated by applying the Gumbel distribution to the set of extreme events. The results of the comparison are shown in Tables 3.17 and 3.18. It is observed that the variance in the bending moment effects increases with the span. The results suggest that the distribution of axle and axle set weights differs from the distribution of the GVM. Given that the deviation of the extreme events from the common events, Nowak's method of applying a single distribution to the total data set is not supported.

Span (m)	Bending Moment (kNm)		
	Nowak	Gumbel	% Difference Nowak:Gumbel
5	290	301	-4%
10	762	812	-6%
15	1372	1364	1%
20	2262	2097	8%
30	4740	3631	31%

Table 3.17 - Nowak/Gumbel Comparison – Bending Moments

Span (m)	Shear Forces (kN)		
	Nowak	Gumbel	% Difference Nowak:Gumbel
5	263	297	-11%
10	343	320	7%
15	439	369	19%
20	537	433	24%
30	675	542	25%

Table 3.18 - Nowak/Gumbel Comparison – Shear Forces

The extrapolation of the WIM also provided the opportunity to quantify the sensitivity of the results to the assumed return period. The sensitivity of the predicted events to the return periods assumed in BD37/01 (120 years), ENV 1991-3 (1000 years) and RR 91/004/02 is shown in Table 3.19. The 10% variance between the assumed 120 year period and the 2976 period associated with RR 91/004/02 is significant when comparing the two set of results.

	Predicted Bending Moments (kNm)									
	Spans									
	5m		10m		15m		20m		30m	
120 year event	276		757		1195		1829		3275	
1000 year event	292	(+5.9%)	804	(+6.2%)	1262	(+5.6%)	1930	(+5.5%)	3436	(+4.9%)
2976 year event	301	(+8.9%)	828	(+9.4%)	1297	(+8.5%)	1982	(+8.4%)	3519	(+7.4%)

Table 3.19 - Bending Moments for 6 Axle Vehicles with Varying Return Periods.

### 3.6 OVERLOADING

The overloading of vehicles was accounted for in TMH7 Part 2 and RR 91/004/01 & 02 through the use of an overloading factor applied to the GVM and axle sets. In RR 91/004/01 & 02, the factor was derived from measurements taken in Switzerland by Bez (1989) because of a lack of data in South Africa. The final recommendation of RR 91/004/01 was, however, that the extent of overloading on South African roads be verified using traffic survey data.

In assessing the prevalence of overloading, the GVM of each vehicle was reviewed in terms of the maximum limit of 560kN and the bridge formula. The results of this review are shown in Table 3.20. Over 99% of the 6, 7 and 8 axle vehicles were found to be in compliance with the National Road Traffic Regulations (1999). However, only 40% of abnormal vehicles met the restrictions in terms of the GVM. The study indicates that the abnormal vehicles merit special attention from the law enforcement agencies. The cumulative distribution of vehicle GVM, shown in Figure 3.11, graphically indicates the percentage of 9 axle vehicles that are overloaded.

Vehicle Class	Total No. of Vehicles	No. of Illegal Vehicles	% of Illegal Vehicles
6 Axle	24901	19	0.08%
7 Axle	34951	43	0.12%
8 Axle	2587	15	0.57%
9 Axle	45	27	60.00%

Table 3.20 - Number of Observed Illegal Vehicles

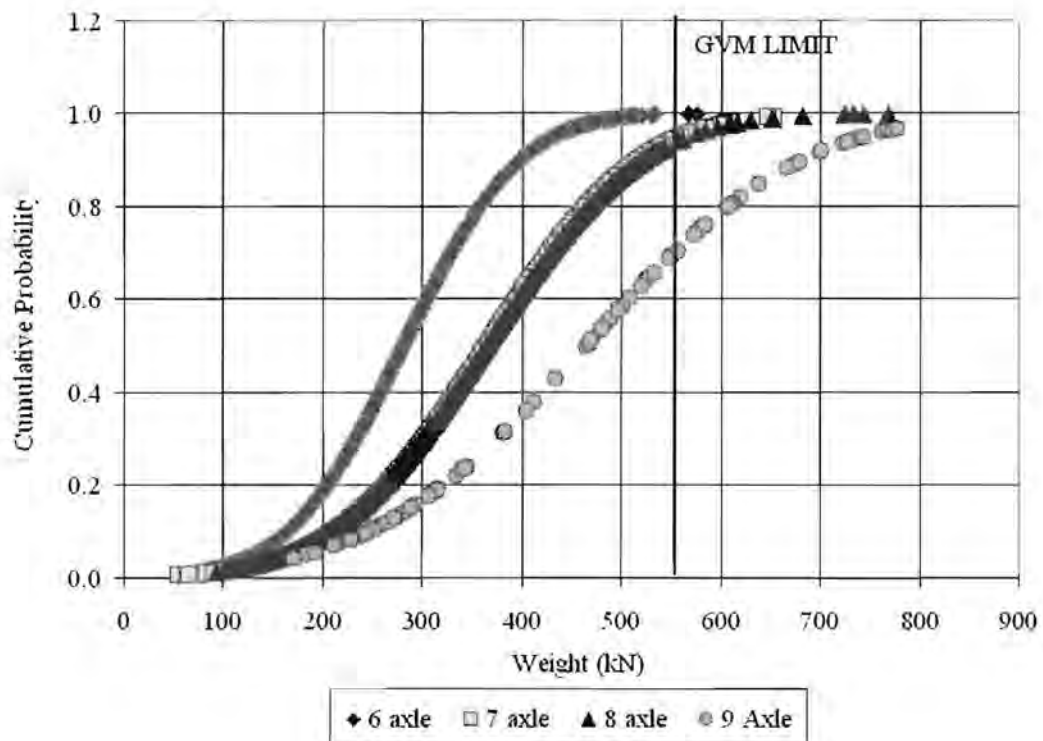


Figure 3.11 - Cumulative Distribution of Axle Weights

The impact of individual axles and axle sets on short spans bridges are well documented (Ullman, 1988). A review of the extent of overloading associated with individual axles was therefore undertaken. The allowable axle mass was set at the permissible limits stipulated in the National Road Traffic Regulations (1999) and those allowed by the bridge formula in terms of the vehicle's length and axle spacings. The findings of this exercise are shown in Table 3.21.

It was observed that a maximum of 2.5% of axles in 6 axle vehicles were overloaded. The second and third axles of 7 and 8 axle vehicles were seen to be prevalent to overloading. In particular, the third axle of the 8 axle vehicles was overloaded in 31% of the recorded events. The greatest prevalence in axle overloading was observed in the 9 axle vehicles. Over 34% of the axles were observed to be overloaded. Due to inaccuracies of the WIM data these results are indicative rather than representative.

Vehicle Class	Percentage of Axles Overloaded								
	Axle No.								
	1	2	3	4	5	6	7	8	9
6 Axle	0.0%	1.9%	2.0%	2.1%	1.2%	2.5%	-	-	-
7 Axle	0.3%	7.7%	6.9%	0.9%	0.9%	5.6%	3.7%	-	-
8 Axle	8.3%	28.0%	31.0%	1.3%	0.9%	4.4%	6.7%	7.0%	-
9 Axle	13.3%	46.7%	42.2%	40.0%	35.6%	44.4%	44.4%	44.4%	37.8%

Table 3.21 - Percentage of Overloaded Axles

It is important to recognise that overloading is a time dependent variable. The quantification of an overloading factor is therefore dependent on the considered return period. In the case of RR 91/004/01, the overloading factor was applied to the vehicle mass prior to the extrapolation of the associated load effects. This approach is considered valid; however, it is not followed in this study.

In the calculation of the overloading factor cognisance of the design approach is required. The current limit state design codes are based on the use of partial factors that limit the exceedence probability of an event for a given time period. The objective of this thesis is to calibrate a load model based on the probabilistic analysis of the collected traffic survey data. Given that the traffic survey data is a product of overloading, the need for a separate overloading factor was judged unnecessary. However, the set objective of quantifying the prevalence of overloading within a specific period on the National Route 3 was retained.

For the purpose of quantifying the increase in load effects, an overloading factor is defined as the percentage increase caused by the "actual" vehicles in comparison to those of the "legal" vehicles. As stated previously, the percentage error associated with the WIM results prevents the use of individual results to draw definitive conclusions. The statistical properties of a set of results are used rather to predict specific events. In the case of overloading, a 1 in 28 day event was used. The results therefore predict the maximum overloading event likely within the 28 day period.

In calculating the 1 in 28 day event, two statistical approaches were used. The first applied a normal distribution to the complete population set of "legal" vehicles. The second applied a normal distribution to a set to extreme "legal" vehicles identified from the distribution graphs, as in the case of

the “actual” vehicles. These results were then compared with the 1 in 28 day events predicted using the “actual” vehicles. For the “actual” vehicles, a normal distribution was also used as a means of comparing similar statistical distributions.

The normal distribution of the extreme population set was considered the more indicative set of results. The complete set of events did not always fit a single regression line. In the case of 8 axle vehicles, the plotted points demonstrated a bimodal distribution. The variance between the load effects calculated using the complete set of events and the extreme set of events is shown in Table 3.22. In 80% of the cases, the results vary by less than 10%. Although the calculated overloading factors results vary for the two approaches, similar trends develop in both cases.

Span (m)	Bending Moments (kNm)			Shear Forces (kN)		
	% Difference			% Difference		
	Total Pop:	Extreme Pop		Total Pop:	Extreme Pop	
	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.
5	-2%	-6%	3%	5%	-1%	7%
10	6%	-13%	14%	-9%	-4%	6%
15	-11%	-11%	11%	1%	-6%	5%
20	-4%	-7%	6%	5%	-3%	8%
30	-4%	-1%	3%	5%	2%	21%

Table 3.22 - Variance of Load effects derived from Complete Set of Events and Extreme Set of Events

The results of the analysis, shown in Tables 3.23 and 3.24, demonstrate the impact of overloading on short span structures. The obvious trend is that overloading is prevalent in 6 and 7 axle vehicles but not in 8 axle vehicles. In review of the bending moment effects, overloading has a significant impact on spans of 15m and less. This finding indicates that the use of a blanket overloading factor is not appropriate. The results support the conclusion that the overloading of individual axle sets is more prevalent than the overloading of a complete vehicle. This finding is consistent with work carried out in the drafting of BD 37/88 (Dawe, 2003), where a 1.4 overloading factor was applied for spans up to 10m and then reduced linearly to unity at 60m spans.

Span (m)	Bending Moments (kNm)						% Difference Actual: Legal		
	Legal Vehicles			Actual Vehicles					
	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.
5	197	195	196	216	220	181	10%	13%	-8%
10	510	547	520	582	600	486	14%	10%	-7%
15	968	1054	986	946	1034	875	-2%	-2%	-11%
20	1504	1582	1537	1452	1599	1388	-3%	1%	-10%
30	2767	2917	2899	2676	2944	2697	-3%	1%	-7%

Table 3.23 - Overloading Results using Normal Distribution – Bending Moments

Span (m)	Shear Forces (kN)								
	Legal Vehicles			Actual Vehicles			% Difference Actual: Legal		
	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.	6 Axle Veh.	7 Axle Veh.	8 Axle Veh.
5	182	183	184	210	197	164	15%	8%	-11%
10	246	251	252	249	250	220	1%	0%	-13%
15	295	308	302	298	293	272	1%	-5%	-10%
20	337	347	337	349	358	316	4%	3%	-6%
30	398	411	410	405	440	391	2%	7%	-5%

Table 3.24 - Overloading Results using Normal Distribution – Shear Forces

The overloading factor, with respect to the predicted 28 day shear forces, varied with a similar trend to those associated with the bending moments. However, the results showed that overloading caused an increase in the shear effects of the 7 Axle vehicles for spans of up to 30m. For longer spans, the shear force effect is not as sensitive to the location of critical axle sets as the bending moment effect. Overloaded axles will therefore still contribute significantly to the total shear forces on spans of 30m. The results, therefore, highlight the prevalence of the overloading of 6 and 7 axle vehicles.

As stated, the load effects on short spans are dominated by the action of individual axle and axle sets. Overloaded axles on 2 and 3 axle trucks, will therefore impact on calculated results for 5m and 10m spans. It is therefore recommended that future studies include a review of all heavy vehicles, regardless of the vehicle's number of axles.

### 3.7 CONCLUSIONS

In conclusion, the probabilistic analysis of the truck survey data produced load effects that can be compared with those calculated from TMH7 Part 2 and RR 91/004/02. In addition, the creation of a “legal” set of vehicles allowed the quantification of the load effects due to overloading.

In regard to the probabilistic analysis of the WIM data, the study confirmed the use of the Gumbel distribution (RR91/004/01, 1994) as the most appropriate means for extrapolating the load effects of heavy vehicles on simply supported spans. Nowak's (1991) application of a normal distribution to the complete set of events was not favoured, as the distribution of the extreme load effects was observed to deviate from the distribution of the common load effects. The use of the normal distribution produced load effects up to 31% higher than those calculated by applying the Gumbel distribution to the extreme set of events.

In applying the Gumbel distribution, it was shown that the extrapolated load effects are sensitive to the population size of the extreme events. For each vehicle class and span, it was necessary to visually identify the population size of the extreme events from the distribution graphs. RR 91/004/01's assumption that the top 15% of load effects from a population set are extreme events is therefore not supported. In addition, a sample of extreme events representing a larger time period is required. This will reduce the confidence limits of the predicted events.

The probabilistic analysis of the WIM data was shown to be relatively insensitive to the return period selected. However, the 2976 year return period used by RR 91/004/02, is conservative when compared with ENV 1991-3 and BD 37/01. For the serviceability limit state a maximum return period of 1000 years, as per ENV 1991-3, is recommended.

The potential inaccuracy of the WIM data ( $\pm 25\%$ ) raises a question over the validity of the results. This question should be answered, in future research, by quantifying the impact of the potential error on the predicted load effects. ENV 1991-3 was calibrated (O'Connor, 2005) using WIM data with a maximum error of 5%. Similar standards are required in South Africa if WIM data is to be used in the calibration of bridge live load models.

The number of overloaded vehicles recorded on the National Route 3 was found to be low. Their occurrence, however, raises concerns for the serviceability and ultimate limit states of bridge structures. In particular, the extent of the overloading associated with the abnormal vehicles requires the attention of law enforcement agencies. The results of the probabilistic analysis of the WIM data show that the overloading of individual axles, rather than of the overloading of complete vehicles, is prevalent. A comprehensive survey of heavy vehicles, using weighbridges, is necessary to accurately quantify overloading on South African roads. These survey results may be used to calibrate the partial load factors used with the chosen live load model. Using this approach, there is no longer the need to calculate overloading factors.

## **4. CRITICAL REVIEW OF TMH7 PART 2 & SUBSEQUENT RESEARCH**

The following chapter undertakes a critical review of the live loading model contained with TMH7 Part 2 using the load effects generated from the probabilistic analysis of the WIM data. Also reviewed are the load effects used to derive the alternative load model proposed in the Department of Transport reports:

- (i) Report RR 91/004/01, “The effect of an Increase in the Permissible Heavy Vehicle Loads on Bridges”, June 1994.
- (ii) Report RR 91/004/02, “The effect of an Increase in the Permissible Heavy Vehicle Loads on Bridges – Assessment Code”, December 1995.

In order to undertake a meaningful comparison of the various load effects, a detailed appraisal of the methods used by Liebenberg (1974) and RR 91/004/01 & 02 is undertaken.

The following chapter also reviews the assumptions used by Liebenberg’s deterministic methods against the statistical information provided by the WIM data. The probabilistic methods used in RR 91/004/01 & 02 are reviewed with reference to the latest research (O’Connor et al., 2001) used in drafting ENV 1991-3. In both instances, the logic used to calibrate the live load model for limit state design is examined.

### **4.1 TMH7 PART 2**

#### **4.1.1 Background and Development**

In 1974, the then National Transport Commission recognised the need to create uniform standards for bridge design across South Africa that incorporated the latest theory and practice. Subsequently, the “Code of Practice of the Design of Highway Bridges and Culverts”, TMH7, was issued in 1981 with a number of errata and revisions being issued in 1988. Although the code was largely based on the provisions of the BS5400, “Steel, Concrete and Composite Bridges”, Part 2: “Specification of loads” issued in 1978, TMH7 Part 2 (1981) differed significantly in regard to the application of live loads due to traffic.

The development of the live load models contained within TMH7 is largely based on research work carried out by Liebenberg (1974). In turn, the basis of his research was taken from the loading formula developed by Henderson (1954) for the inclusion within BS153 (1954) and the subsequent issue of BS 5400 (1978). Henderson (1954) developed a “credibility” approach where engineering judgement was used to determine most onerous probable combinations and arrangements of heavy vehicles. Liebenberg (1978) favoured this approach versus the probabilistic analysis of truck survey data. The lack of available statistical data and the complexity of the variables associated with traffic movements meant the “credibility” approach was deemed the only feasible method. The combinations of vehicles chosen by Liebenberg (1974) and Henderson (1954) are shown in Appendix C.

TMH7 divides live loading due to traffic into the three categories of normal (NA) loading, abnormal (NB) loading and superloads (NC) loading. NC loading represents multi-wheeled trailer combinations with controlled hydraulic suspension and steering. For the purposes of this thesis only the live load models associated with normal (NA) loading are considered.

The original form NA loading was based on the existing legal loads in South African in 1972. Two traffic states were considered. One of these cases was bumper-to-bumper traffic that modelled the static load conditions and thus contained no allowance for impact loading. The other case took account of moving traffic, at set following distances, with allowances for impact based on the Swiss Impact formula (1970).

$$\phi = 5 \left( \frac{100 + L}{10 + L} \right) \quad \text{Swiss Impact Formula (1970)} \quad (4.1)$$

Where:

$\phi$  = Impact factor

$L$  = Equivalent span length

From the analysis of these two states a loading curve was derived that specified a uniformly distributed lane load as a function of the loaded length. This lane load was applied as two line loads at a set spacing within a notional design lane. The lane load was applied in conjunction with a single knife-edge load (KEL) to ensure that the maximum bending moments and shear forces were produced. Although a set of KELs are required to model both the bending moments and shears, a single KEL (Henderson, 1954) was chosen for simplicity. Although this approach correctly estimates the shear forces, it overestimates the bending moments. Multi presence is taken as a function of the loaded length as is the presence of critical axle loads.

The application of the uniformly distributed load (UDL) to obtain the maximum load effects is somewhat complex. In order to achieve the maximum load effects, TMH7 Part 2 requires that:

- (i) The transverse position of the lane loads within the notional lanes is varied to derive the maximum effects on the structural element under consideration;
- (ii) The intensity of the UDL in the longitudinal direction is varied on separate parts of an influence line to produce the most onerous effects; and
- (iii) A correction factor,  $k$ , be used to cover the case where the partial loading of the base of any portion of an influence line creates the most onerous effects.

The amount of computation required to correctly apply NA loading has caused dissatisfaction with South African bridge engineers (Fitzgerald, 1998), when compared with the simpler loading models in BD 37/01, LRFD and ENV 1991-3.

They were deemed necessary by Liebenberg (1978), who stated:

“At first consideration, the above requirements may appear to greatly increase the complexity of analysis if the maximum effects are to be calculated. These refinements cannot, however, be entirely neglected as total discrepancies exceeding 50% can occur.”

The following sections review the components of TMH7 Part 2 in greater detail with the purpose of commenting on the assumptions made in comparison to the latest research and development.

#### 4.1.2 NA Loading Curves

The loading curve for Type NA Loading in TMH7 Part 2, is used by design engineers for the quantification of the uniformly distributed loads that model normal traffic conditions on bridge and culvert structures. A study of the methodology used in constructing this curve was undertaken to comment on the assumptions made with reference to the characteristics of heavy vehicles recorded by the WIM sensors.

As stated, Liebenberg (1978) used a credibility approach in determining the most onerous configuration and arrangement of various types of heavy vehicles to model live load effects. In developing these configurations, stationary, bumper-to bumper conditions and moving traffic conditions were considered. In the case of moving traffic, an allowance for the dynamic effects was included in the quantification of the load effects. The axle loads of the chosen vehicles were the pre 1972 South African legal loading increased by 20%. Direct reference to the derivation of the loading curve from these combinations was not found during the literature search. However, the vehicle combinations assumed by Liebenberg (1974) and Henderson (1954) were referenced in Ullman (1987) and are shown in Appendix C.

The following insight into the development of the loading curve was taken for Ullman (1987,1988), referencing Liebenberg's earlier work.

- **Short Span (< 40m):** The combinations used by Liebenberg (1974) included a convoy of five heavily loaded vehicles weighing up to 228kN. These were preceded and followed by a combination of light vehicles represented by a line load of 6.0kN/m. In the case of travelling vehicles, a spacing of 4.5m was assumed between vehicles and allowances were made for impact using the Swiss Impact Formula (1970). To allow for the eventuality of overloading, a 20% surcharge was added to all axle weights or, alternatively, a 40% surcharge to a single axle group.
- **Long Spans (> 40m):** In the case of long spans, identical vehicle combinations were considered, with stationary traffic condition being dominant. No allowance for impact loading was therefore made. The blanket 20% surcharge to account for overloading was considered excessive for the number of vehicles associated with longer spans. Instead, a 10% surcharge was applied to allow for the future possible increase in legal axle limits.

In 1988, the NA loading curve was revised to increase the uniformly distributed load by an additional 6kN/m for all loaded lengths. No reference was found on the rationale for this increase. It is proposed, however, that the deficiencies in TMH7 Part 2 in both the short and long span cases motivated this revision. In particular, the specified lane load of 6kN/m was low in comparison to the 9kN/m recommended by comprehensive studies that Buckland (1978) carried out in the United States. A further factor for the increase was the inclusion of a 9kN/m lane load for long span structures in BS 5400. This was an increase over the 5.8kN/m lane load in BS 153. The increase allowed for the adoption of lower partial factors (1.2 in lieu of 1.4) for dead loads. Previously, the dead load partial factor provided an increased factor of safety against an underestimation of the live load. A similar reduction in the dead load partial factor also occurred in South Africa.

Using the original vehicle combinations, the loading curves were replicated using a VB computer program (Appendix F). The load effects from the Liebenberg's vehicle combinations and a lane loading were calculated for static and dynamic conditions for spans ranging from 10m to 900m. From the calculated force effects, an equivalent UDL was calculated using both the calculated bending moment and shear load effects. In the case of the long spans, a lane load was assumed to precede and to follow the vehicle combination. The sensitivity of varying the assumed lane load was also reviewed. The results of this study are shown in Figures 4.1 and 4.2.

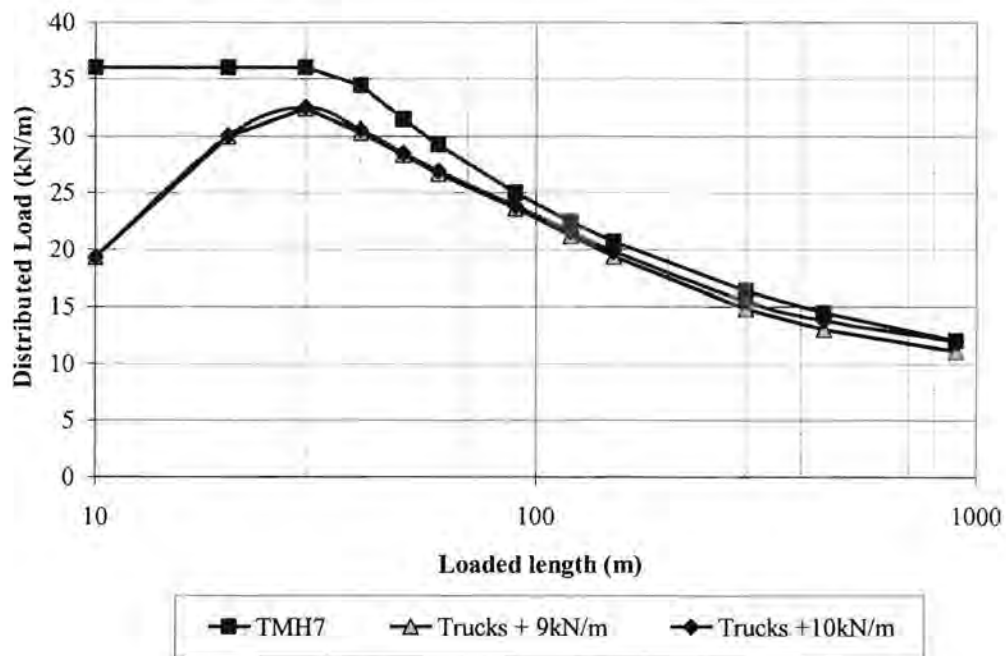


Figure 4.1 - Uniformly Distributed Lane Loads Derived From Bending Moments

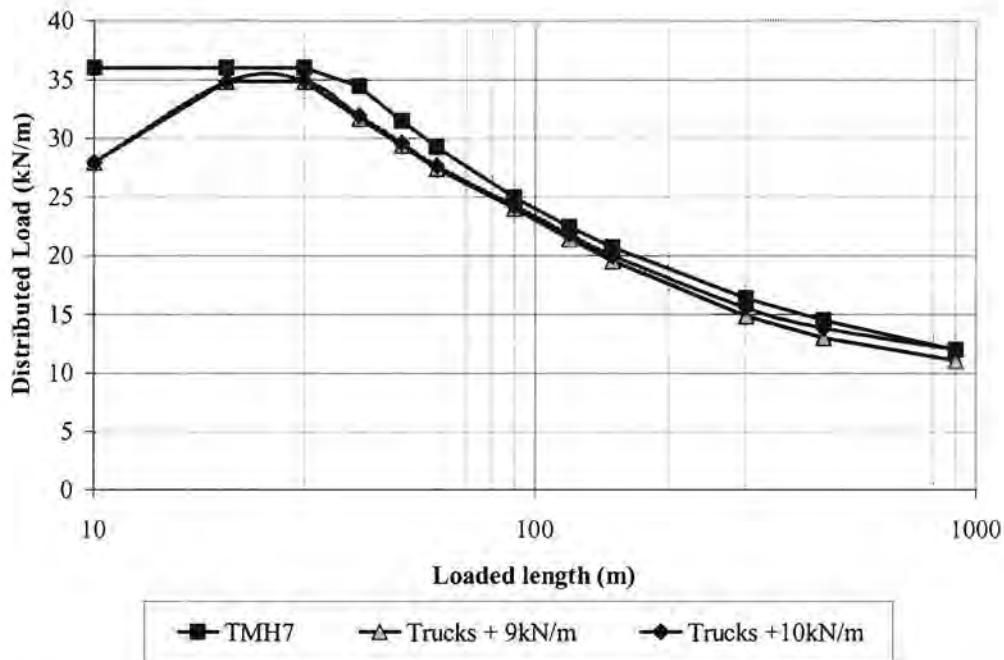


Figure 4.2 - Uniformly Distributed Lane Loads Derived From Shear Forces

The figures confirm Liebenberg's (1978) statement that the shear forces dictate the form of the loading curve. In order to simulate both bending moments and shears accurately, at least two different knife edge loads would be required. Liebenberg therefore took the approach of fitting the loading curve to the shear forces while overestimating the bending moments. This approach greatly simplifies the loading model. The figures also confirm that the increase of the uniformly distributed load in 1988 by 6kN/m effectively provided for an increased lane loading of 10kN/m. This value is comparable to the lane loads introduced into BS 5400 (1978) and the findings of Buckland (1978).

#### 4.1.3 Review of Truck Combinations

Liebenberg's (1974) combination of vehicles was selected to represent an extreme event. The possibility of human manipulation in creating convoys of heavily loaded vehicles was taken into account in selecting these combinations. In review, the static truck combinations J1 and J2 are found to be the most onerous events other than for very short spans. These combinations contained five heavily loaded short axle vehicles. Liebenberg (1978) stated that one of the most important requirements of a live load specification was that it should be a reasonable simulation of characteristic traffic loading based on a non-zero but sufficiently low probability of occurrence during the useful lifetime of the bridge. The question arises how the probability of occurrence may be calculated when the occurrence and sequence of the vehicles is selected using engineering judgement.

Current design codes are based on a limit state approach. In the case of the LRFD and CSA-S06-00, the live loads are factored with a partial factor based on a reliability index. This index is derived from the statistical evaluation of the probability of an event being exceeded within a given time frame. The approach uses of the statistical characteristics of an event to quantify a rational partial factor that

provides the required level of serviceability. It is considered that the use of the credibility approach does not support the rationally based calculation of partial factors.

In developing a consistent approach to limit state design of bridge structures, it is proposed that the statistical characteristics of traffic loading in South Africa require investigation. Through this investigation, the development of a live load model that is calibrated to the required serviceability and ultimate limits of a bridge structure can be derived.

The collected WIM data provides the opportunity to assess the probability of occurrence of the truck combinations selected by Liebenberg (1974). To calculate the probability of a specific convoy of vehicles occurring, the probability of one type of vehicle being followed by another was calculated. The results of this calculation are shown in Table 4.1. For example, there is a 21.6% probability of a 2 axle vehicle being followed by another 2 axle vehicle. The probability of a 3 axle vehicle being followed by another 3 axle vehicle is 10.6%.

Vehicle Type	Following Probability							
	2-Axle	3-Axle	4-Axle	5-Axle	6-Axle	7-Axle	8-Axle	9-Axle
2-Axle	21.6%	20.4%	19.5%	19.0%	18.3%	18.6%	18.4%	19.6%
3-Axle	9.7%	10.6%	9.4%	8.6%	8.2%	8.2%	8.2%	16.5%
4-Axle	4.8%	4.3%	5.1%	4.3%	4.3%	4.2%	4.5%	8.2%
5-Axle	8.6%	8.6%	8.5%	10.1%	9.1%	9.0%	8.7%	10.3%
6-Axle	22.3%	22.0%	22.1%	23.3%	25.1%	23.1%	22.8%	12.4%
7-Axle	30.7%	31.4%	32.8%	32.1%	32.6%	34.3%	32.8%	25.8%
8-Axle	2.3%	2.5%	2.5%	2.4%	2.3%	2.5%	4.5%	4.1%
9-Axle	0.1%	0.1%	0.0%	0.1%	0.1%	0.1%	0.2%	3.1%
Total	100%	100%	100%	100%	100%	100%	100%	100%

Table 4.1 - Following Probability

The following vehicle combinations assumed by Liebenberg were reviewed:

- Combination J1 – 2 axle vehicles:** Liebenberg assumed a GVM of five co-existent adjacent vehicles of 197kN with axle spacing of 2.4m. From the survey of 20,086 2 axle vehicles, it was found that 0.0023% of vehicles have a GVM of 197kN or greater. It was also observed that only 0.02% of vehicles have an axle spacing of 2.4m or less. From the sequence analysis on the WIM data, it was calculated that there is a 0.05% chance of five 2 axle vehicle occurring in sequence. The probability that each of these vehicles would have the GVM and axle spacing assumed by Liebenberg represents a 1 in  $3.8 \times 10^{39}$  year event. Detailed workings of this calculation are provided in Appendix C.
- Combination J2 – 3 axle vehicles:** From the survey of 9,000 3 axle vehicles, it was found that 2.54% of vehicles have a GVM of 228kN and more. It was also observed that only 4.9% of vehicles have internal axle spacing between axle sets of 2.8m or less. The probability of five 3 axle vehicles occurring in sequence was calculated as 0.0016% in a given month. Liebenberg's assumption of five co-existent adjacent vehicles each with a GVM of 228kN and an internal axle spacing between axle sets of 2.8m was calculated as a  $1.9 \times 10^{14}$  year event. Detailed workings of this calculation are provided in Appendix C.

The statistical analysis of the WIM data highlights the conservative assumptions made by Liebenberg in formulating the truck combinations used to derive the design loading in TMH7 Part 2. The load effects calculated by these combinations are further factored to give ultimate limit state effects. In order to prevent engineers designing for serviceability limits that will not occur within the design life of a structure, there is the need for a rational assessment of South African heavy vehicles.

#### 4.1.4 Comparison of Dynamic to Static Loads

Dynamic load effects result from the heavy vehicle travelling over irregularities on the surface of the bridge deck. The magnitude of the effect is dependent of the magnitude of the irregularities, the natural frequency of the bridge as well as the suspension of the heavy vehicle.

TMH7 Part 2 uses the Swiss Impact formula specified in the SIA Specification 160 (1970). For Liebenberg's combinations, the dynamic load effects exceed those of stationary bumper-to-bumper traffic for spans below 11.0m (Ullman, 1988). As shown in Figure 4.3 this fact was verified by calculating the load effects of Liebenberg's vehicle combinations for both static and dynamic states. A Visual Basic program was written for this purpose. The magnitude of the impact factor for various spans is shown in Table 4.2.

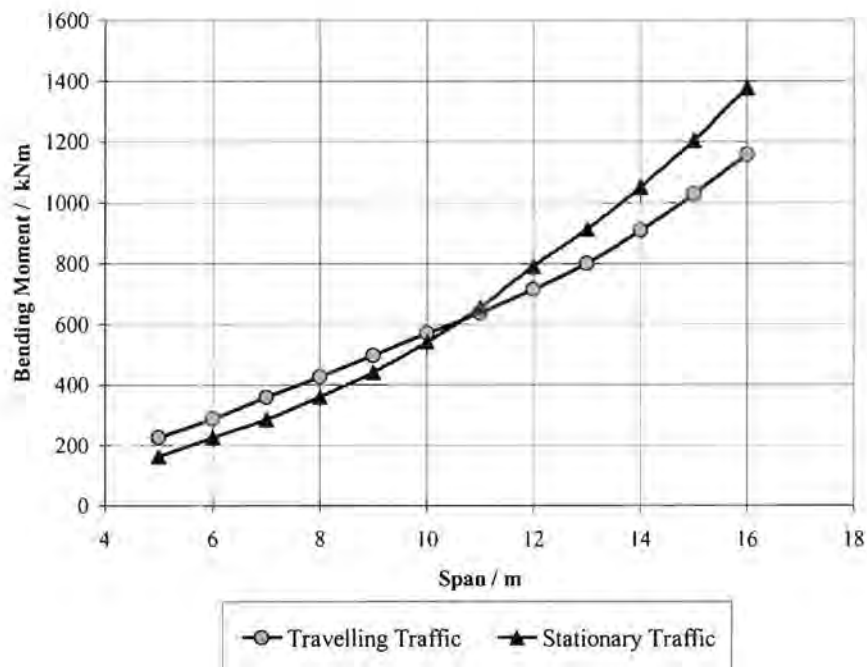


Figure 4.3 – Plot of Bending Moments Due to Travelling and Stationary Traffic

Span (m)	Impact Allowance
5	35%
10	28%
15	23%
20	20%
30	16%

Table 4.2 - Impact Allowance in TMH7

The advent of 7 and 8 axle vehicles with a GVM of up to 560kN has led to a single heavy vehicle causing the most onerous load effects in spans below 40m (Nowak, 1991; O'Connor, A., 2005). Given this fact, dynamic loading becomes the dominant load case. Considering the relevancy of TMH7 Part 2, the study of the dynamic effects of single heavy vehicles moving across spans up to 40m is particularly relevant.

The impact allowance made within TMH7 was based on the Swiss Impact formula from SIA 160 (1970). This impact formula has since been superseded in the SIA 160 (1989) and significant research work has been carried out in the field of dynamic loading on bridge structures (Hwang and Nowak, 1989). These developments were incorporated into the review carried out in RR 91/004/01 & 02.

#### **4.1.5 Lateral Bunching**

The concept of lateral bunching accounts for the event where adjacent lanes of traffic are squeezed together laterally. For example, three lanes of traffic may be squeezed into two lanes to pass a broken down vehicle. In the United Kingdom, the magnitude of HA loading included with BD 37/01 (2001) was increased to take into account the effects of lateral bunching. A lateral bunching factor of 1.4 was applied to spans up to 20m and then reduced linearly to unity at 40m.

TMH7 Part 2 makes no allowance for lateral bunching. This is also true for the LRFD and CSA-S06-00. Further consideration of this effect is merited, for during the 120 year design life of a bridge in the South African metropolitan centres, there is a reasonable probability of lateral bunching occurring. The issue requiring further research is whether this event is concurrent with the maximum load effects.

#### **4.1.6 NB Loading**

The impact on short spans of individual heavy axles, which is caused by rogue overloading, was recognised by Liebenberg (1978) in TMH7 Part 2. It was therefore specified that 24 units of NB loading be applied to all highway bridges. This approach is not consistent with the latest codes of practice. In the case of the BD 37/88, the HA loading curve was revised for the purpose of catering for heavy point loads on short span structures. In the case of the LRFD, CSA-S06 and ENV 1991-3, the live load model contains a design axle group that adequately simulates the load effects that develop on the shorter spans. It is proposed that any revision of TMH7 Part 2 includes a single live load model that adequately simulates normal traffic loading.

## **4.2 RR 91/004/01 - PERMISSIBLE HEAVY VEHICLE LOAD RESEARCH**

In the 1990's, various bodies approached the Department of Transport with requests to increase the legal load limits for heavy vehicles contained within the then Road Traffic Act.

As a result of those requests, the Report RR 91/004/01 "The Effect of an Increase in the Permissible Heavy Vehicle Loads on Road Bridges", June 1994, was commissioned by the Department of Transport. The aims of the study were stated as:

- (i) To evaluate the present legal limit in relation to past and present bridge design codes and international practice; and
- (ii) To quantify the effect of increased permissible loads on road bridges.

The report is the most recently published research on the effect of heavy vehicles on South African bridge structures and represents an important body of knowledge. The methodology adopted in considering the load effects of heavy vehicles on bridge structures draws on a wide body of current international research. Also incorporated was research work carried out into TMH7's shortcomings by Ullman (1988) and Oosthezien et al. (1991).

The report provides a valuable reference for further research into bridge live loads in South Africa. Cognisance was therefore taken of the subjects highlighted in the report as requiring further research.

### **4.2.1 Problem Statement**

The report begins with a problem statement that sets the optimum use of South Africa's transport infrastructure against the safety of its roads and bridges. It describes the current situation in the country where overloading is commonplace and law enforcement is judged inadequate. The report tasks itself with developing a new set of truck mass restrictions that meet the following goals:

- (i) Fair balance between increased massloads (resulting in increased revenue for the operator) and additional costs associated with the strengthening or replacement of bridges;
- (ii) Ease of understanding for the truck owner and driver; and
- (iii) Ease of enforcement.

In developing the mass restrictions, RR91/004/01 considered 17 different variations to the regulated axle mass limitations. These variations included increases to the legal axle weights, amendments to the bridge formula, and the impact of disregarding the bridge formula all together.

### **4.2.2 Development of Live Load Model**

In developing the live load model, 10 vehicles were chosen to represent the most common classes of heavy vehicle found on South African roads. The likelihood of occurrence of each of these classes was taken from surveys carried out by Van Wyk & Louw (1991).

The following variables were considered:

- (i) **Overloading:** In assigning the vehicle mass to each of the vehicles, overloading ratios were applied as derived from studies in Switzerland and Germany (Bez, 1989). The statistical information associated with South African Vehicles was judged insufficient to derive a locally applicable factor.
- (ii) **Vehicle Spacing:** Vehicle spacing for stationary and jam-packed conditions were derived from various sources from Switzerland (Bez, 1989) and survey data collected in South Africa.

**Impact:** The dynamic interaction of a vehicle moving at speed and a bridge deck of a given surface profile and natural frequency is known to create more onerous effects than those of a stationary vehicle. An important part of the report is the calculation of the impact factor, as shown below, using the recommendations of research work carried out in Switzerland (SIA 160, 1989). This research work supersedes the Swiss Impact formula (SIA 160, 1970) used in TMH7 Part 2. A detailed review of the Swiss Impact formula (1989) is provided in Appendix D.

$$I_f = I_b f_m f_s \cdot (1 + \varepsilon) \quad \text{Swiss Impact Formula (1989)} \quad (4.2)$$

Where

- $I_f$  = the final impact factor
- $I_b$  = the impact factor for the bridge
- $f_m$  = the reduction factor for the vehicles mass
- $f_s$  = the speed reduction factor
- $\varepsilon$  = the coefficient of variation.

The report adopts a probabilistic approach by using the Monte Carlo simulation technique to randomly generate 5,000 different traffic streams. A garage of 10 vehicles was used to generate random stationary and moving traffic conditions. All vehicles were assumed to be loaded to the permissible limits with an overloading ratio applied in line with the measured field distributions. The load effects of these 5,000 vehicle combinations were then calculated for various spans of simply supported one, two and three span continuous structures.

To simulate the load effects on the bridge structure over its design life, the results from the 5,000 vehicle streams were extrapolated to a total of 1.8 million traffic streams. In this regard, research work carried out by Grouni and Nowak (1984) and Moses and Verma (1987) was referenced, which postulated the use of a 50-year return period for moving loads. The number of critical static occurrences was taken at 10% of the total vehicle streams within the 50-year period. The extreme events were assumed to follow a Gumbel distribution.

The above process was repeated using the 17 alternative truck mass limitation criteria for a single span bridge. The load effects were then compared to those of TMH7 Part 2.

#### 4.2.3 Critical Review

The following comments are made following the review of Report RR 91/004/01:

- (i) **Axle Loads:** The full permissible load was applied to each of the 10 vehicles in the simulation. As stated this load was then multiplied by an overloading ratio derived from the distribution of observed axle loads. The comment is made that the overloading of individual axles is more prevalent than the overloading of complete vehicles (Section 3.6). The use of a convoy of vehicles, loaded to the legal limit and beyond, is considered conservative. It is proposed that an overloading factor that decreases as the span increases is more appropriate (Section 3.6; Dawe, 2003). As stated in the RR 91/004/01, additional research is required in quantifying the extent of overloading and its impact.
- (ii) **Monte Carlo Simulation:** The scope of RR 91/004/01 was set to consider spans of up to 30m. It is generally accepted that for spans of up to 30m, a single heavy vehicle causes the most onerous load effects (Nowak, 1991; O'Connor, A., 2005). The Monte Carlo simulation assumed that the maximum effects were caused by a convoy of fully laden, overloaded trucks. This event does not happen in reality. Cognisance of this fact was taken in the Monte Carlo simulation used in the calibration of ENV 1991-3, where the simulation vehicles were representative of recorded vehicle configurations and weights (O'Connor et al, 2001).
- (iii) **Extrapolation of Load effects:** The statistical approach used in extrapolating the load effects is similar to that used in the calibration of the CSA-S06-00 (2000). However, the extrapolated load effects represented ultimate limit state events in CSA-S06-00 (2000) as opposed to nominal load effects in the RR 91/004/01. The extrapolation of the load effects using the Gumbel distribution is considered valid (Section 3.4.2). However, the definition of extreme events as the upper 15% of the sample set is not always valid. It is considered that the distribution of the extreme load effects is sensitive to span and class of vehicle (Section 3.4.2).
- (iv) **Design Rational:** The live load model derived in BD 37/88 followed a similar methodology carried out on RR 91/004/01. In the case of BD 37/88, the results of the simulation of fully loaded vehicles were judged to represent ultimate limit state events. The nominal loads were then calculated by dividing the extreme load events by 1.5. In RR 91/004/01, the extrapolated results of the Monte Carlo simulation are taken as nominal results and multiplied by a partial factor of 1.5 to derive ultimate limit state load effects. This approach is considered overly conservative and not based on rational limit state design principles.

In conclusion, the report adopts a probabilistic approach in the derivation of load effects, using a Monte Carlo simulation to generate random traffic streams and a statistical distribution to predict extreme events. This approach significantly differs from the deterministic approach adopted in the derivation of the live load model within TMH7 Part 2. However, a further step is the use of truck survey data to assign the simulation vehicles with a distribution of truck axle weights and GVM's. This step will supersede the assumption used in the RR 91/004/01 that all vehicles are fully laden and overloaded. It will also allow the derivation of partial limit state factors based on the probabilistic analysis of the truck survey data, as in the case of ENV 1991-3 (Connor et al., 2001).

University of Cape Town

### 4.3 RR 91/004/02 - DEVELOPMENT OF ALTERNATIVE DESIGN LOAD TO TMH7

As a result of the work carried out in Report RR 91/004/01, the Department of Transport recommended an increase in the allowable axle masses and the amendment of the bridge formula. Given the shortcoming in TMH7 Part 2 (RR 91/004/01, 1994) there was a requirement to derive assessment loads that would accurately model the load effects associated with the new traffic loads. Report RR 91/004/02 "The Effect of an Increase in the Permissible Heavy Vehicle loads on Road Bridges – Assessment Code" was therefore commissioned and published in December 1995. The primary objectives stated within the executive summary of the report were:

- (i) Following the recommendations of the task group concerning the increases in permissible axle masses, to develop an assessment load that is simple in format and is easy to apply. This load should accurately predict the expected increased load effect produced by traffic;
- (ii) To substantiate theoretical work with full-scale load tests; and
- (iii) To develop a code procedure for the evaluation process.

The development of traffic streams consisting of vehicles complying with the proposed new regulations was undertaken using the same methodology as in RR 91/004/01. In this case, a set of 55 different vehicles falling within 24 vehicle classifications were selected. The extent of overloading was again quantified by Bez (1989). The need to confirm the extent of overloading on South African roads was again emphasised.

An interesting component of the research was the attempt to correlate the measurement of deflections and stress in three bridges in the field with those predicted by the proposed theoretical live load model. The results of this research are in summarised in Section 4.3.3.

#### 4.3.1 Traffic loading

RR 91/004/02 comments that the bridge formula complicates law enforcement. The case of the technical overloading of common classes of vehicles is highlighted. The following example is given in the report.

"In the case of a typical class 14 vehicle, with an inter axle distance between the second and last axle of 14.74m, the total load of 49.4 tonnes is 4.5 tonnes more than the 48.95 tonnes allowed by the bridge formula."



**KLAS 14**

Figure 4.4 - Configuration of Class 14 Vehicle. (Source: RR 91/004/02, 1995)

A revision of the bridge formula that allows the most common classes of vehicles to be loaded to the sum of the permissible axle loads was therefore proposed. It was postulated that the bridge formula be changed to  $16+3.0L$  for  $L < 13.3$  and 56 ton for  $13.4 > L < 22.2\text{m}$ . This change would eliminate the technical overloading of the most popular classes of vehicles; this proposal was then used in assigning the axle masses to the vehicles used in generating the vehicle streams. The revision of the bridge formula is merited as it simplifies law enforcement that may effectively combat overloading.

### 4.3.2 Impact Factor

The impact factor was calculated in accordance with the method set out in Appendix D. However, in calculating the impact factor, a reduced vehicle mass reduction factor,  $f_m$ , is used in RR 91/004/02. For a vehicle of mass  $T$  tons, the following reduction factors for the vehicles mass,  $f_m$ , were used:

$$\text{For } T < 16\text{t} \quad f_m = 1.0$$

$$\text{For } T > 50\text{t} \quad f_m = 0.42$$

This amendment significantly reduces the impact factor applied to the load effects on spans greater than 5m, as shown in Table 4.3.

Span (m)	Impact Allowance		
	TMH7	RR91/004/ 01	RR91/004/ 02
5	35%	37%	36%
10	28%	26%	18%
15	23%	17%	12%
20	20%	13%	9%
30	16%	9%	6%

Table 4.3 - Impacts Allowances

### 4.3.3 Test Loading

The methodology adopted in the testing of the three bridges is not included within this document. The findings of the testing are, however, summarised below:

- (i) The correlation between measured and calculated strain was considered acceptable;
- (ii) The results confirmed that the present design practice is realistic with respect to load effects under serviceability conditions;
- (iii) The correlation between the calculated crack widths and the spacing measured is poor. Fewer but larger cracks occur, which exceed the code limits; and
- (iv) Some reserve strengths and stiffnesses are present in each bridge. The amount of reserves was considered dependent on the restraint at the supports, the actual constitutive behaviour of the material and the global response of the bridge.

### 4.3.4 Assessment & Design Loads

An important differentiation in the report is in the definition of an assessment load and a design load. The assessment load is defined as the load that results in load effects equivalent to those produced by the full range of heavy vehicles under the present legislation. The design load is then considered equal to the assessment load, plus a contingency of 10%.

From the previous studies undertaken, the report proposes that neither TMH7 Part 2 nor BD 37/88 provides suitable design loads for South African conditions in relation to their value and format. An assessment load was therefore derived from the maximum load effects produced by the generated vehicle streams. Using the methodology adopted in the RR 91/004/01, the load effects were extrapolated to a characteristic value with a 5% chance of being exceeded in 120 years.

The assessment load derived by the report is shown in Figure 4.5 and was formulated to match the predicted bending moments and shears. The use of a double axle concentrated load model in conjunction with a uniformly distributed load (UDL) bears close resemblance to the live load model of ENV 1991-3.



Span (m)	RR 91/ 004/02	TMH7 / NA	% Difference RR:TMH7
5	415	293	42%
10	949	810	17%
15	1510	1553	-3%
20	2442	2520	-3%
30	5397	5130	5%

Table 4.5 - Comparison of Bending Moments, RR 91/004/02 versus TMH7

The shear forces calculated from the report's design load are consistently higher than TMH7. This difference is in part due to the increase in permissible axle masses and GVM since the 1978. As expected, the increase in axle masses impacts most significantly on the short spans.

Span (m)	RR 91/ 004/02	TMH7 / NA	% Difference RR:TMH7
5	378	234	62%
10	413	324	27%
15	506	414	22%
20	576	504	14%
30	739	684	8%

Table 4.6 - Comparison of Shear Forces, RR 91/004/02 versus TMH7

#### 4.3.5 Report Conclusions

In conclusion, RR 91/004/02 states that short span bridges need to be assessed individually to ensure their continued safety and serviceability under the increased permissible axle loads. It was recommended that the assessment and design load derived within the report be adopted. In addition, it was concluded that the present design load provisions in TMH7 Part 2 require adjustment to eliminate substantial deficiencies in the short span range. With regard to overloading, it was again concluded that the assumptions made within the report need to be verified through traffic surveys.

#### 4.3.6 Critical Review

The following comments are made following the review of Report RR 91/004/02:

- i.) As in the case of RR 91/004/01, the use of a Monte Carlo simulation using convoys of fully laden vehicles to simulate nominal load effects for spans up to 30m is considered conservative.
- ii.) The form of the assessment and design load is valid as is the process involved in quantifying the uniformly distributed load and point loads to replicate the maximum actual load effects.
- iii.) As stated in the report, a detailed review of the impact of overloading is required from traffic surveys.
- iv.) The impact factor applied is less onerous than the factor used in deriving the NA loading curves in TMH7 Part 2.

v.) The design approach uses a characteristic load with a 5% probability of being exceeded in 120 years. This gives a return period of 1 in 2976 years. This return period is somewhat higher than the 1 in 1000 years assumed in ENV 1991-3 and the 1 in 120 years used in BD 37/88 (2000). The key observation is that RR 91/004/02 extrapolates events that are extreme in their own right. Using the same method, BD 37/88 considered the extrapolated events to represent an ultimate limit state. In RR 91/004/02 a partial factor was further applied to the extrapolated characteristic values to give an ultimate limit state event. This factor was 1.5 as in the case of TMH7 Part 2. The factor is based on engineering judgment rather than a rational approach. It is therefore proposed that the logic used in RR 91/004/02 is extremely conservative. In the case of ENV 1991-3, the Monte Carlo simulation used simulation vehicles with a range of GVM's. The resulting events were therefore representative of normal traffic conditions rather than extreme conditions. This approach is recommended in future simulations.

In the following sections, the extreme load effects predicted in RR 91/004/02 are compared with those extrapolated from the WIM data.

#### 4.4 COMPARISON OF DESIGN LOADS VERSUS ACTUAL LOADS

The main objective of the study was to compare the load effects generated by the WIM data with those calculated using the live load models contained within TMH7 Part 2 and RR 91/004/01 & 02. This approach allows the assessment of theoretically derived live load models with the load effects of actual trucks. For the purpose of the comparison, the static load effects extrapolated from the WIM data using the Gumbel distribution were factored by the impact factors used in RR 91/004/02. The magnitude of the load effects calculated from each source are shown in Table 4.7 and 4.8. A graphical comparison is also given in Figures 4.6 and 4.7.

Span (m)	Bending Moments (kNm)		
	WIM data	RR 91/004/02	TMH7 Part 2
5	409	415	293
10	957	949	810
15	1527	1510	1553
20	2284	2442	2520
30	3847	5397	5130

Table 4.7 - Bending Moments Results

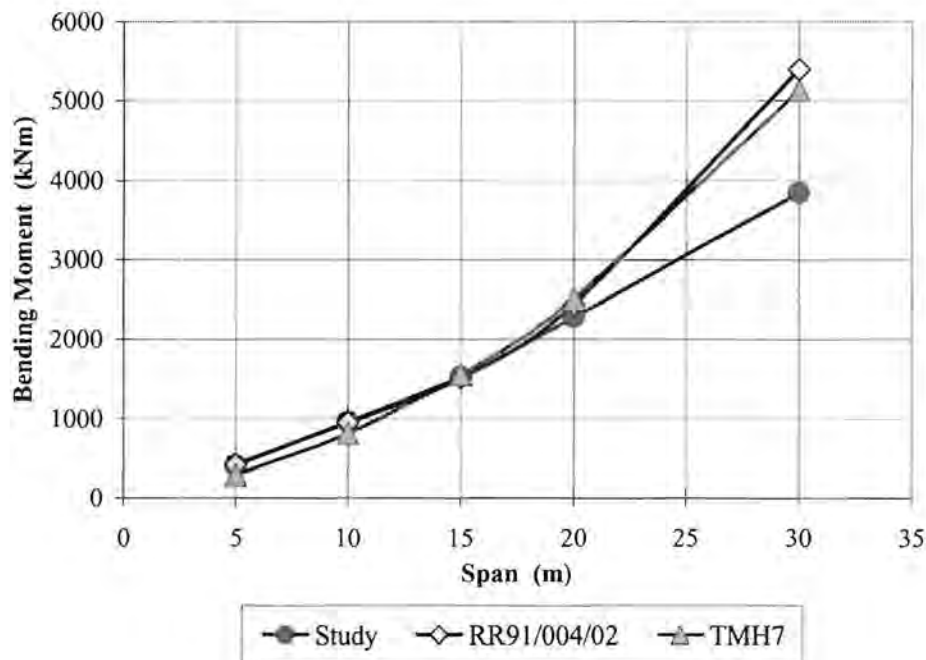


Figure 4.6 - Comparison of Bending Moments

Span (m)	Shear Forces (kN)		
	WIM data	RR 91/004/02	TMH7 Part 2
5	297	378	234
10	320	413	324
15	369	506	414
20	433	576	504
30	542	739	684

Table 4.8 - Shear Force Results

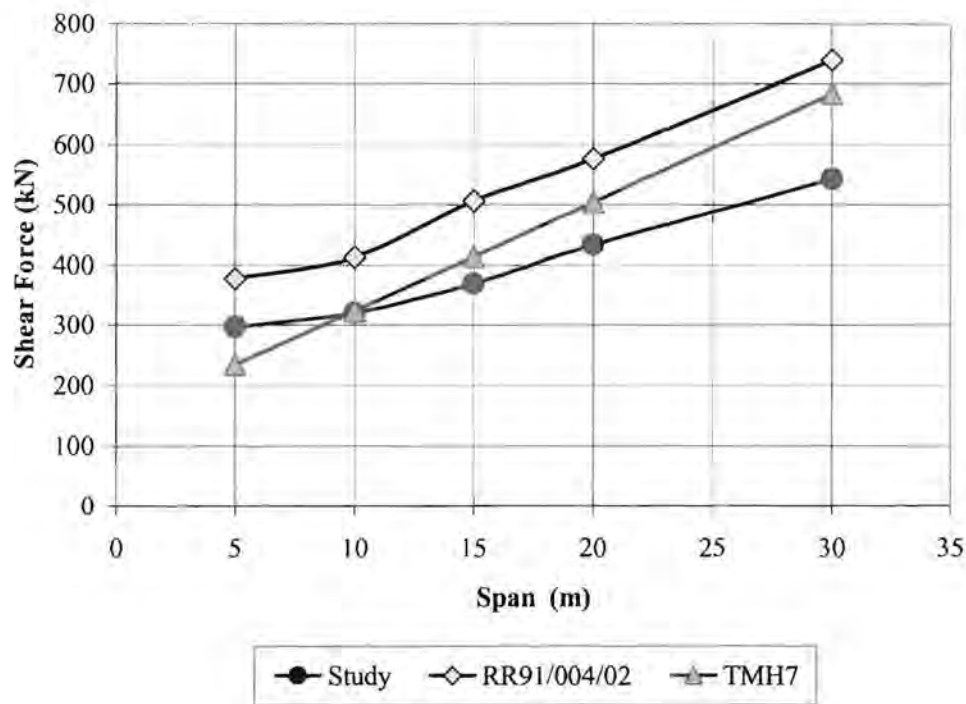


Figure 4.7 - Comparison of Shear Forces

#### 4.4.1 TMH7 versus Actual Traffic Measurements

It was observed that TMH7's bending moment effects are less than the study's effects for 5m and 10m spans. TMH7's NA loading is known to be deficient in catering for normal traffic conditions over short spans (Ullman, 1988). This is confirmed by the results of the study, shown in Table 4.9. In drafting the code, it was intended that 24 units of NB loading be applied to all structures to cover this shortcoming. A normal design loading that covers all spans is considered more logical.

Span (m)	Bending Moment (kNm)		
	WIM data	TMH7 Part 2	% Difference WIM: TMH7
5	409	293	39%
10	957	810	18%
15	1527	1553	-2%
20	2284	2520	-9%
30	3847	5130	-25%

Table 4.9 - Bending Moment Comparison, WIM data v TMH7

The bending moment effects calculated on a 15m span are similar for the WIM data and TMH7 Part 2. This similarity is because a combination of axle groups are the critical load event for 15m spans. The results indicated that Liebenberg's (1974) vehicle axle groupings replicate those in the longer, single heavy vehicle on South African roads today.

For spans of 20m and 30m, TMH7's loads increase significantly above those found by the WIM data. TMH7's loadings for the 20m and 30m spans are derived from the static combinations of bumper-to-bumper fully laden vehicles. These vehicles are taken to be overloaded by 10%. This approach differs to that taken by Nowak (1991) and O'Connor (2005), who judge a single dynamic vehicle loading as dominant for spans up to 30m. Nowak's approach is based on the fact that in practice the return period associated with a convoy of fully laden overloaded trucks is extremely high (Section 4.1.3). Although BD 37/88 is based on a similar approach to TMH7, the results are judged to represent the ultimate limit state. Serviceability limit state effects are then calculated by dividing the ultimate limit state results by 1.5. In TMH7 Part 2, the loads effects derived from the overloaded convoy are further factored by a partial factor of 1.5. This fact explains the difference between the WIM data's results and TMH7's. Although there are factors such as lateral bunching and multi-lane loading that are not covered in this study, the results expose the extremely conservative logic used by TMH7 Part 2 in deriving the static nominal load effects on bridge structures.

The results of the comparison of the shear forces generated in the WIM data and those of TMH7 Part 2 are shown in Table 4.10. Only in the case of the 5m spans does the WIM data's predicted shear force exceed that of TMH7 Part 2. This factor is due to the action of overloaded tridem and tandem axles on the shorter spans. Liebenberg's (1974) vehicle combinations do not adequately cater for such an event. For spans of 10m and greater, Liebenberg's (1974) J1-combination causes shear forces in excess of those predicted by the WIM data. This combination includes bumper to bumper 2 axle vehicles with a rear axle weight of 115kN. This assumption is extremely conservative, especially for 30m spans (Section 4.1.3).

Span (m)	Shear Force (kN)		% Difference WIM: TMH7
	WIM data	TMH7 Part 2	
5	297	234	27%
10	320	324	-1%
15	369	414	-11%
20	433	504	-14%
30	542	684	-21%

Table 4.10 - Shear Force Comparison, WIM data v TMH7

#### 4.4.2 RR 91/004/02 versus Actual Traffic Measurements

The WIM data and RR 91/004/02 show close correlation of the bending moments for 5m, 10m, and 15m spans. These are the spans for which a single vehicle is dominant in both approaches. The comparison of the results is shown in Table 4.11. Since RR 91/004/02 used a garage of legal vehicles with a overloading factor this correlation is expected. The report's bending moments increase above

those of the study for the 20m and 30m spans. This is because the report considers a convoy of fully laden vehicles the most critical case, as per TMH7 Part 2.

Span (m)	Bending Moment (kNm)		
	WIM data	RR 91/004/02	% Difference WIM: TMH7
5	409	415	-2%
10	957	949	1%
15	1527	1510	1%
20	2284	2442	-6%
30	3847	5397	-29%

Table 4.11 - Bending Moment Comparison, WIM data v RR 91/004/02

The WIM data's results for the shear load effects are consistently lower than those predicted by RR 91/004/02, as shown in Table 4.12. This difference is because the report assumes all axles are fully loaded with an applied additional overloading factor. The results of the study indicate that the overloading of axles and axle sets is more prevalent than the blanket overloading of a complete vehicle. No reference was found for the overloading ratio's applied in RR 91/004/02 and whether or not it varied with the span. From the findings in Section 3.6 it is proposed that a single overloading ratio applied to all vehicles is conservative in the case of longer spans.

Span (m)	Shear Force (kN)		
	WIM data	RR 91/004/02	% Difference Study: TMH7
5	297	378	-21%
10	320	413	-23%
15	369	506	-27%
20	433	576	-25%
30	542	739	-27%

Table 4.12 - Shear Force Comparison, WIM data v RR 91/004/02

## 4.5 CONCLUSIONS

The load effects generated from the probabilistic analysis of the WIM data provide important reference for the critical review of the live load models in TMH7 Part 2 and RR 91/004/02. They provide a benchmark for the deterministic and probabilistic methods used to simulate actual traffic loadings.

In review of TMH7 Part 2, the statistical analysis of the WIM data showed the deterministically derived vehicle combinations to be conservative in comparison with the recorded traffic flows. Load effects calculated using the WIM data substantiated this finding. For 30m spans, TMH7 Part 2's load effects were 25% higher than those calculated using the actual vehicle data. TMH7 Part 2's deficiency in catering for the load effects on short spans was also confirmed. It is proposed that the deterministically derived vehicle combinations do not adequately cater for overloaded tridem and tandem axle sets. A further point of concern is TMH7 Part 2's use of the Swiss impact formula from SIA 160 (1970) when SIA 160 (1989) has significantly changed the form of the impact formula. In conclusion, it is proposed that the deterministic methods do not adequately simulate the load effects caused by actual vehicles on the roads.

The load effects calculated from the WIM data allow the critical review of the probabilistic methods used in RR 91/004/02. The results demonstrate that the use of a Monte Carlo simulation using fully laden overloaded vehicles is conservative for spans greater than 20m. It is recommended that future simulations be based on a garage of vehicles whose axle weights and GVMs are distributed as in the case of normal traffic conditions. The above method was used in the drafting of ENV 1991-3 (O'Connor et al., 2001). The availability of WIM data from South African Toll roads now provides sufficient data on which to base such an approach.

The analysis of the WIM data indicates that the overloading of axles and axle sets is more prevalent than the overloading of a complete vehicle. As in the case of BD 37/01, the use of an overloading factor that decreases as the span increases is proposed.

The un-factored design loads in RR90/004/01 and TMH7 Part 2 are formulated from the extrapolation of events that are extreme in themselves. This methodology is excessively conservative in comparison to modern codes of practice such as ENV 1991-3 and BD 37/01. It is recommended that the characteristic load events be derived from normal traffic conditions occurring over a rationally-based return period.

In conclusion, the load effects calculated from the WIM data reveal the conservative assumptions associated with the deterministic methods used to derive the live load model in TMH7 Part 2. In the case of the probabilistic methods used in RR 91/004/01 & 02, they highlight the need to base simulations on data derived from actual traffic surveys.

## 5. ALTERNATIVE LIVE LOAD MODEL TO TMH7 PART 2

### 5.1 CALCULATION OF LOAD FACTOR

The development of any traffic live load model requires its calibration against target values. In TMH7 Part 2, the target values were taken from the deterministic review of truck combinations. In ENV 1991-3, the probabilistic analysis of actual truck survey data was used to calculate the target values.

The WIM results provided an opportunity to review the live load model proposed in RR 91/004/02 to replace the NA loading curves in TMH7 Part 2. In carrying out this review, the live load models shown in Figure 5.1 were calibrated against the 1 in 120 year load effects calculated from the WIM data.

The proposed live load models take the form of a uniformly distributed load in combinations with a series of point loads (Buckland, 1978). For each of the 8 live load models considered, lane loads of 9kN/m, 18kN/m and 27kN/m were applied. Equivalent load models 1 and 2 represent the characteristics of actual 6 and 7 axle vehicles respectively. Using the equivalent base length method developed by O'Connor (1981), the  $W/b_m$  values of the 6 and 7 axle vehicles causing the most onerous load effects were identified (Appendix E). The equivalent load models 1 and 2 were created to replicate these properties.

The remainder of live load models take the form of a series of two or three axle sets in combination with a uniformly distributed lane load. The points loads are not chosen to represent any specific vehicle. ENV 1991-3's live load model and the proposed design load from RR 91/004/02 were considered. Variations to these load models were also included for the purposes of comparison.

A VB program was written to calculate the bending moments and shear associated with each of the design models. Spans of 5m to 30m were considered.

The method developed by Nowak (1995) in calibrating the LRFD is used to calculate the preliminary load factors. A full calibration of the partial factors by considering the data's reliability index was not carried out as the thesis does not include a review of the ultimate limit state targets values.

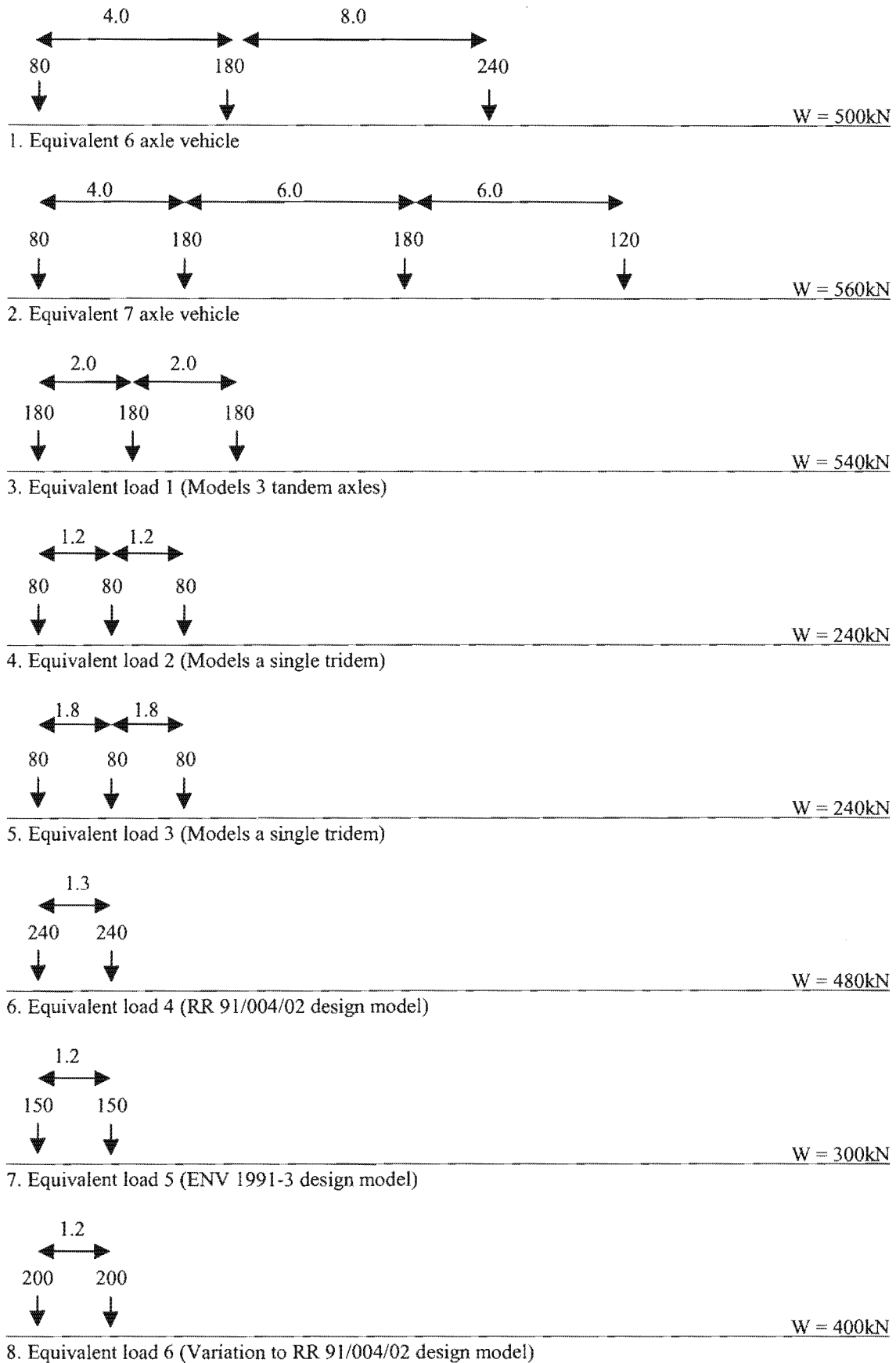


Figure 5.1 - Equivalent Load Models

Nowak's (1995) method involves the use of a bias factor calculated as the ratio of the target values against the load effects from the live load model. The bias factors for the range of spans were found. By calculating the mean, standard deviation and coefficient of variation of the bias factor for a particular load effect, the load factor was calculated using the formula below.

$$\gamma = \lambda(1 + kV)$$

$\gamma$  = load factor

$\lambda$  = bias factor

$V$  = bias factor coefficient of variation, calculated by dividing the mean by the standard deviation.

$k$  = constant,  $k=2$  (Nowak, 1995)

## 5.2 RESULTS

The equivalent vehicle models 1 and 2, applied in conjunction with various distributed lane loads, did not produce consistent load factors. As shown in Figure 5.2, for a lane load of 27kN/m, load factors ranged from 2.1 at 5.0m spans to 0.9 at 30m spans.

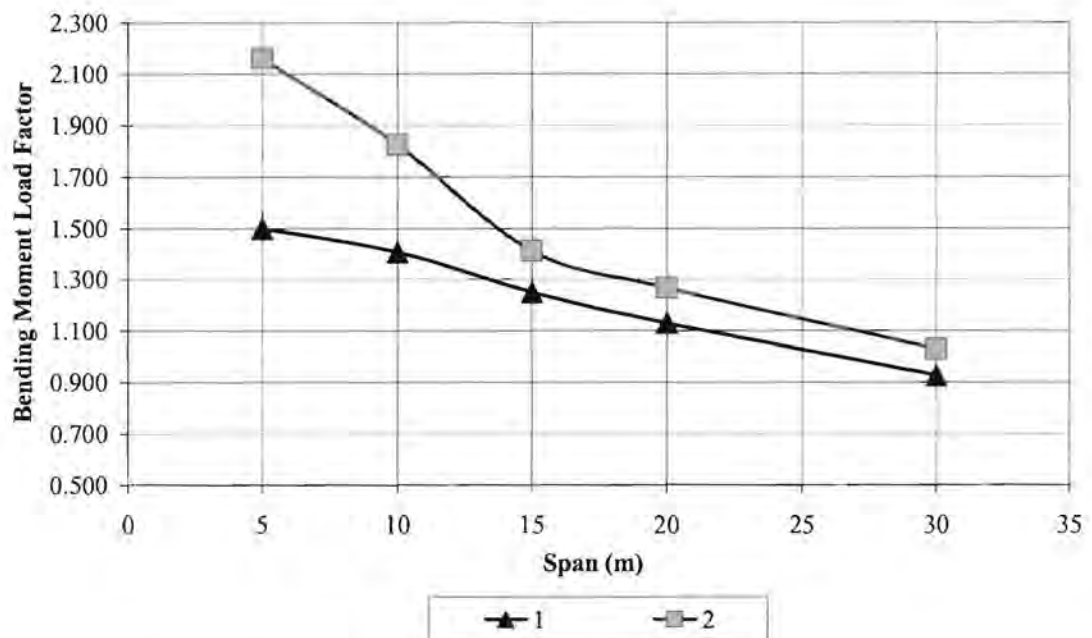


Figure 5.2 - Bending Moment Load Factors – Models 1 & 2

The results indicated that the simple model, containing a set of two or three axle sets spaced at less than 2.0m, produced more consistent load factors across the spans. A comparison of the load factors calculated for these models is shown in Figure 5.3 and 5.4. In the case of model 8, with a lane loading of 9.0kN/m, the bending moment load factor varied by 6.5% and the shear force load factor by 24.7%. This model and lane load was found to replicate the WIM target values with the least variance. The adoption of a load factor of 1.1 accurately modelled the maximum shear forces whilst it overestimated the maximum bending moments by 4.8%.

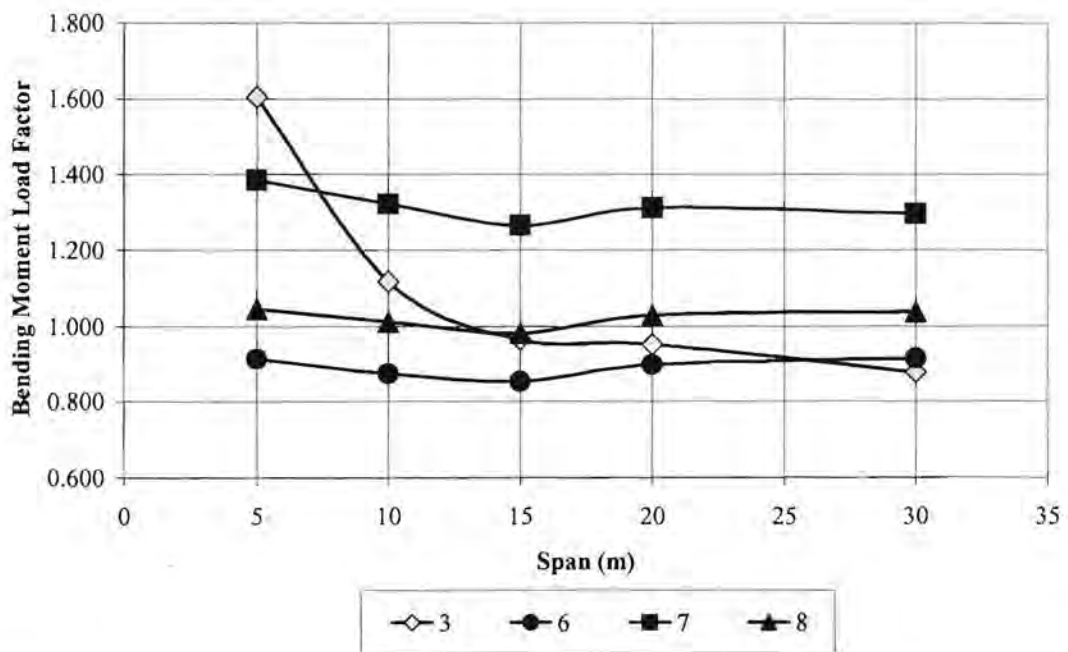


Figure 5.3 - Bending Moment Load Factors using WIM data– Models 3, 6, 7 & 8

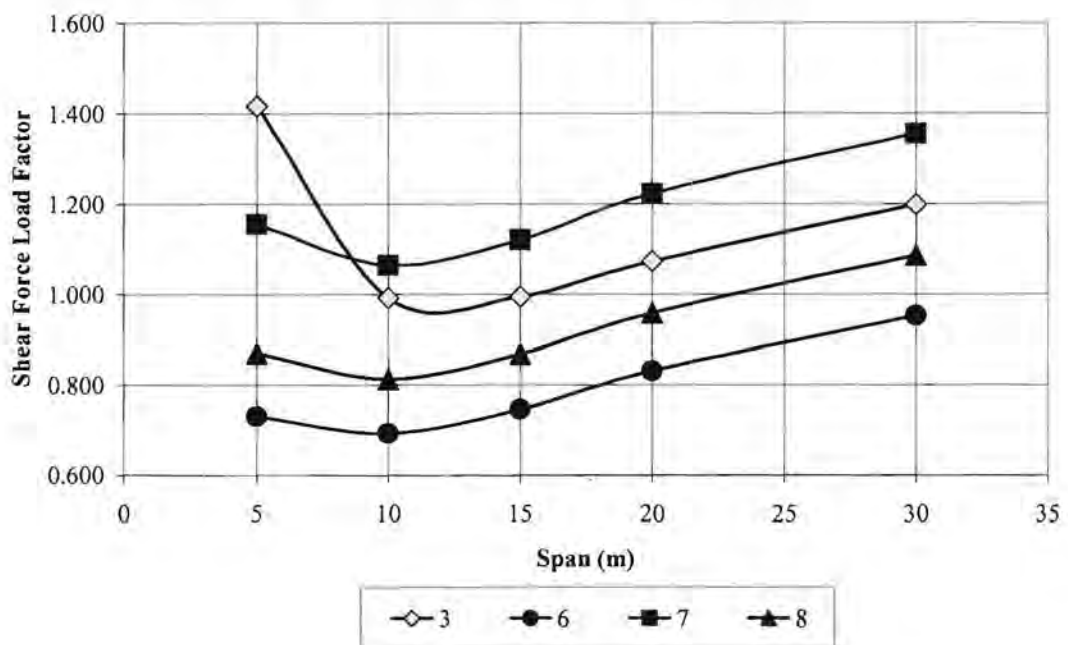


Figure 5.4 - Shear Force Load Factors using WIM data – Models 3, 6, 7 & 8

The calibration process carried out in RR 91/004/02 was replicated using the VB programs written to undertake the calibration of the WIM data. The results, shown in Figures 5.5 and 5.6, confirm the findings of the report that load model 6 in conjunction with a 18kN/m lane load gives a close calibration to the target values.

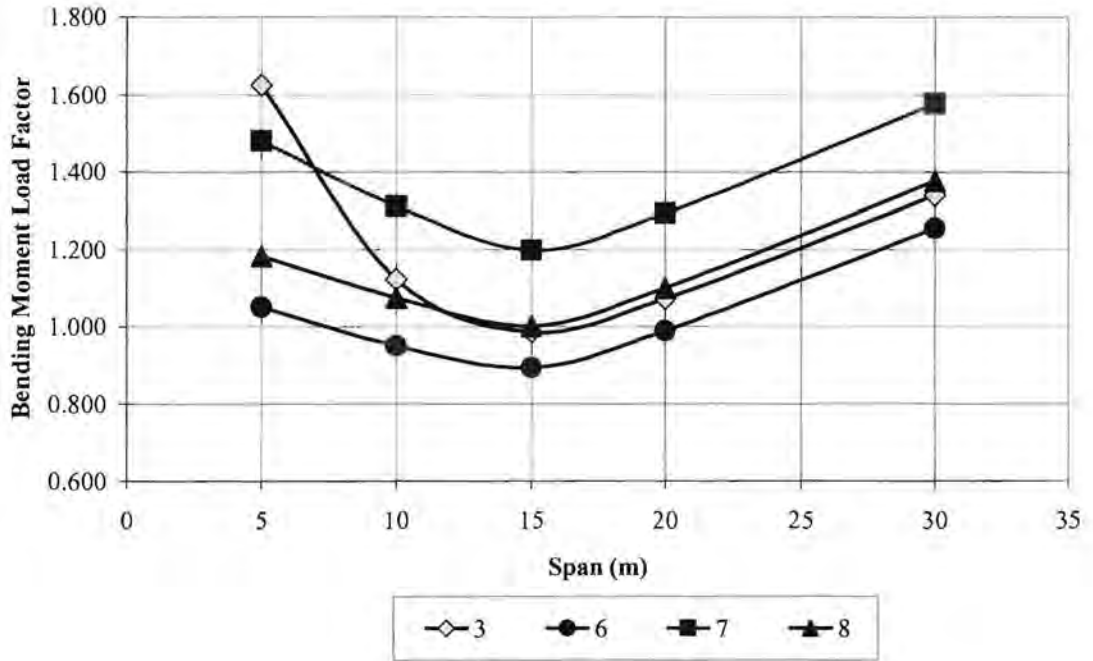


Figure 5.5 - Bending Moment Load Factors using RR 91/004/02 – Models 3, 6, 7 & 8

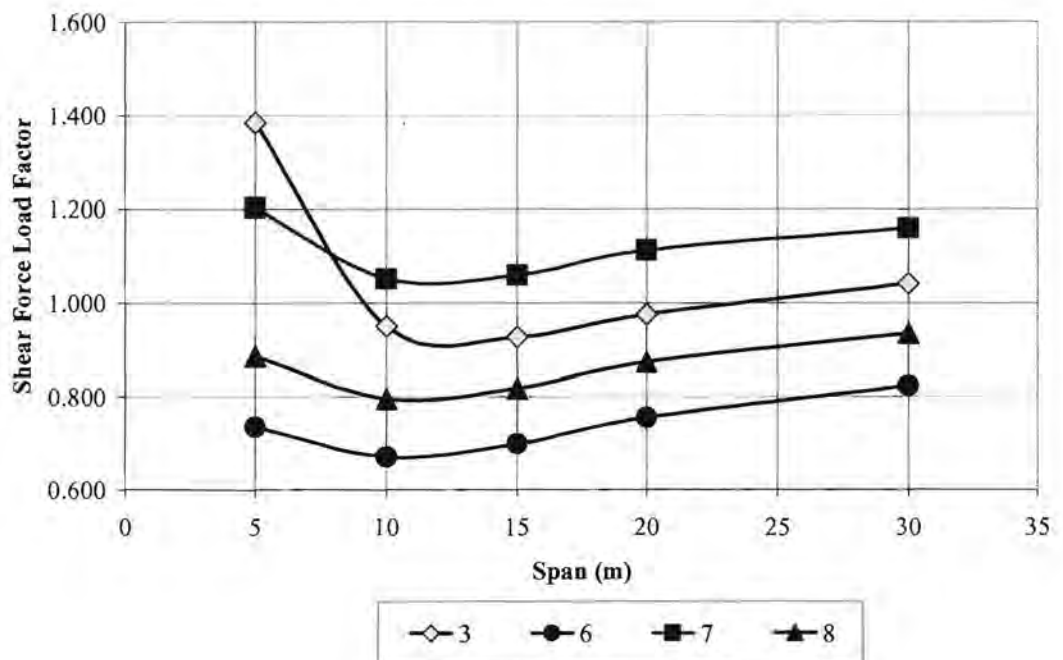


Figure 5.6 – Shear Force Load Factors using RR 91/004/02 – Models 3, 6, 7 & 8

Using Nowak's (1995) method, the bending moment load factor varies by 40% and the shear force load factor by 29%. The adoption of a load factor of 1.2 accurately models both the maximum shear forces and the maximum bending moments. A full set of the results of the calibration process is shown in Table 5.1 to 5.4.

In conclusion, the results clearly indicate that a load model containing two closely spaced point loads in combination with a uniformly distributed load, accurately simulates the load effects of South African heavy vehicles. This research work therefore supports the use of the load model proposed in RR 91/004/02; it also shows that ENV 1991-3's load model may be used in South Africa. The probabilistic analysis of the WIM data demonstrates the means by which ENV 1991-3's load model may be calibrated to South African conditions. Rather than revising TMH7 Part 2, a viable alternative is the adoption of ENV 1991-3 and the drafting of a National Application Document (NAD). The significant research and development carried out in developing the limit state principles and live load model in ENV 1991-3 may be utilised in South Africa with limited expenditure.

## Bending Moments

Lane Load 9 kN/m

Model	Mean	SD	V	Load Factor $\gamma$				
1	1.24	0.12	0.10	1.54	1.62	1.55	1.45	1.25
2	1.33	0.29	0.22	2.40	2.25	1.79	1.66	1.41
3	0.91	0.18	0.20	1.71	1.23	1.11	1.14	1.14
4	1.56	0.17	0.11	2.23	1.93	1.77	1.80	1.73
5	1.72	0.37	0.22	3.39	2.47	2.19	2.19	2.08
6	0.84	0.02	0.03	0.91	0.87	0.85	0.90	0.91
7	1.23	0.04	0.03	1.39	1.32	1.27	1.31	1.29
8	0.97	0.02	0.02	1.05	1.01	0.98	1.03	1.04

Lane Load 18 kN/m

Model	Mean	SD	V	Load Factor $\gamma$				
1	1.05	0.15	0.14	1.53	1.51	1.38	1.26	1.06
2	1.12	0.29	0.26	2.28	2.02	1.58	1.44	1.19
3	0.80	0.19	0.23	1.66	1.17	1.03	1.04	0.99
4	1.26	0.23	0.18	2.23	1.83	1.60	1.57	1.41
5	1.38	0.39	0.28	3.18	2.24	1.90	1.84	1.64
6	0.74	0.04	0.06	0.91	0.84	0.80	0.81	0.79
7	1.03	0.10	0.10	1.43	1.30	1.19	1.19	1.11
8	0.84	0.06	0.07	1.06	0.99	0.92	0.93	0.89

Lane Load 27 kN/m

Model	Mean	SD	V	Load Factor $\gamma$				
1	0.91	0.17	0.18	1.50	1.41	1.25	1.13	0.93
2	0.97	0.29	0.29	2.16	1.82	1.41	1.27	1.03
3	0.72	0.19	0.27	1.60	1.12	0.96	0.95	0.88
4	1.07	0.25	0.24	2.16	1.70	1.44	1.38	1.20
5	1.16	0.38	0.33	2.94	2.03	1.67	1.58	1.36
6	0.67	0.07	0.10	0.93	0.83	0.77	0.77	0.72
7	0.90	0.14	0.15	1.44	1.26	1.11	1.09	0.97
8	0.75	0.09	0.12	1.08	0.97	0.88	0.87	0.80

Best Fit

Table 5.1 - Calibration of Model Bending Moments to WIM Data

## Shear Forces

Lane Load		9 kN/m						
Model	Mean	SD	V	Load Factor $\gamma$				
1	1.02	0.08	0.08	1.35	1.27	1.13	1.13	1.18
2	1.14	0.23	0.20	2.21	1.64	1.44	1.45	1.45
3	0.81	0.13	0.16	1.42	0.99	0.99	1.07	1.20
4	1.44	0.14	0.10	2.00	1.59	1.60	1.71	1.83
5	1.53	0.24	0.16	2.80	2.00	1.96	2.06	2.19
6	0.75	0.10	0.13	0.73	0.69	0.75	0.83	0.95
7	1.11	0.10	0.09	1.16	1.06	1.12	1.22	1.36
8	0.88	0.10	0.12	0.87	0.81	0.87	0.96	1.09

Lane Load		18 kN/m						
Model	Mean	SD	V	Load Factor $\gamma$				
1	0.87	0.11	0.13	1.34	1.19	1.03	1.02	1.02
2	0.97	0.24	0.25	2.10	1.50	1.29	1.27	1.23
3	0.72	0.13	0.18	1.39	0.95	0.93	0.97	1.04
4	1.16	0.17	0.14	1.99	1.51	1.45	1.48	1.49
5	1.23	0.26	0.21	2.66	1.82	1.71	1.73	1.73
6	0.66	0.05	0.08	0.73	0.67	0.70	0.76	0.82
7	0.93	0.05	0.06	1.20	1.05	1.06	1.11	1.16
8	0.76	0.05	0.07	0.89	0.79	0.82	0.87	0.93

Lane Load		27 kN/m						
Model	Mean	SD	V	Load Factor $\gamma$				
1	0.77	0.13	0.17	1.32	1.12	0.96	0.93	0.91
2	0.85	0.24	0.28	1.99	1.38	1.17	1.13	1.07
3	0.64	0.13	0.21	1.35	0.91	0.87	0.89	0.92
4	0.98	0.19	0.19	1.94	1.41	1.31	1.31	1.27
5	1.04	0.27	0.26	2.49	1.65	1.50	1.49	1.43
6	0.59	0.03	0.06	0.75	0.67	0.68	0.71	0.75
7	0.80	0.07	0.09	1.22	1.02	0.99	1.02	1.02
8	0.67	0.04	0.06	0.91	0.78	0.78	0.82	0.84

 Best Fit

Table 5.2 - Calibration of Model Shear Forces to WIM data

## Bending Moments

Lane Load		9 kN/m						
Model	Mean	SD	V	Load Factor $\gamma$				
1	1.34	0.08	0.06	1.46	1.50	1.44	1.44	1.64
2	1.42	0.20	0.14	2.18	2.00	1.59	1.59	1.77
3	0.99	0.20	0.20	1.74	1.22	1.10	1.23	1.61
4	1.69	0.23	0.14	2.37	2.00	1.83	2.01	2.54
5	1.86	0.36	0.19	3.33	2.37	2.09	2.27	2.82
6	0.92	0.17	0.18	1.19	1.12	1.09	1.24	1.65
7	1.35	0.21	0.15	1.72	1.61	1.53	1.72	2.23
8	1.07	0.19	0.17	1.36	1.29	1.25	1.41	1.87

Lane Load		18 kN/m						
Model	Mean	SD	V	Load Factor $\gamma$				
1	1.13	0.07	0.06	1.35	1.30	1.19	1.18	1.30
2	1.19	0.22	0.18	2.09	1.80	1.41	1.38	1.50
3	0.87	0.18	0.21	1.63	1.12	0.99	1.07	1.34
4	1.36	0.20	0.14	2.13	1.71	1.49	1.58	1.87
5	1.48	0.34	0.23	3.02	2.08	1.76	1.84	2.15
6	0.81	0.11	0.14	1.05	0.95	0.89	0.99	1.26
7	1.12	0.13	0.11	1.48	1.31	1.20	1.29	1.58
8	0.92	0.12	0.13	1.18	1.07	1.00	1.10	1.38

Lane Load		27 kN/m						
Model	Mean	SD	V	Load Factor $\gamma$				
1	0.98	0.09	0.10	1.33	1.22	1.08	1.05	1.14
2	1.03	0.23	0.22	1.99	1.64	1.27	1.23	1.31
3	0.78	0.17	0.23	1.54	1.05	0.90	0.96	1.17
4	1.15	0.21	0.18	2.02	1.56	1.32	1.36	1.55
5	1.24	0.33	0.27	2.78	1.87	1.54	1.57	1.77
6	0.72	0.08	0.12	0.96	0.85	0.77	0.84	1.03
7	0.97	0.11	0.12	1.38	1.17	1.04	1.09	1.28
8	0.81	0.09	0.11	1.08	0.95	0.86	0.92	1.11

 Best Fit

Table 5.3 - Calibration of Model Bending Moments to RR 91/004/02

## Shear Forces

### Lane Load 9 kN/m

Model	Mean	SD	V	Load Factor $\gamma$				
1	1.35	0.07	0.05	2.02	1.93	1.81	1.77	1.89
2	1.51	0.26	0.17	2.50	1.88	1.75	1.71	1.75
3	1.08	0.15	0.14	1.79	1.27	1.36	1.43	1.63
4	1.90	0.17	0.09	2.85	2.31	2.46	2.56	2.80
5	2.03	0.27	0.14	3.25	2.35	2.45	2.52	2.72
6	0.99	0.15	0.15	1.18	1.14	1.30	1.42	1.65
7	1.47	0.17	0.12	1.38	1.29	1.44	1.53	1.73
8	1.16	0.16	0.14	1.05	1.00	1.13	1.22	1.41

### Lane Load 18 kN/m

Model	Mean	SD	V	Load Factor $\gamma$				
1	1.15	0.11	0.10	1.88	1.70	1.57	1.50	1.54
2	1.28	0.27	0.21	2.27	1.64	1.50	1.44	1.42
3	0.95	0.15	0.16	1.75	1.22	1.26	1.30	1.41
4	1.54	0.18	0.12	2.36	1.82	1.85	1.85	1.90
5	1.63	0.30	0.18	2.65	1.84	1.83	1.81	1.84
6	0.87	0.09	0.10	1.04	0.97	1.07	1.13	1.25
7	1.23	0.07	0.06	1.27	1.13	1.21	1.24	1.31
8	1.01	0.08	0.08	0.99	0.90	0.99	1.03	1.12

### Lane Load 27 kN/m

Model	Mean	SD	V	Load Factor $\gamma$				
1	1.01	0.14	0.13	1.78	1.54	1.40	1.32	1.31
2	1.12	0.28	0.25	2.16	1.52	1.37	1.29	1.25
3	0.85	0.15	0.18	1.72	1.17	1.19	1.20	1.26
4	1.30	0.21	0.16	2.06	1.52	1.50	1.46	1.44
5	1.37	0.31	0.23	2.36	1.59	1.54	1.48	1.46
6	0.78	0.05	0.06	0.97	0.87	0.94	0.96	1.03
7	1.07	0.07	0.06	1.18	1.00	1.04	1.04	1.06
8	0.89	0.05	0.05	0.94	0.82	0.87	0.89	0.93

 Best Fit

Table 5.4 - Calibration of Model Shear Forces to RR 91/004/02

## 6. FINAL CONCLUSIONS AND RECOMMENDATIONS

This thesis was concerned with the review of the live load models specified in TMH7 Part 2 (1988) and the alternative live models proposed in the Department of Transport Reports RR 91/004/01 & 02 (1994, 1995). The review was carried out using the load effects calculated from WIM data recorded on the National Route 3 at Heidelberg in February 2005. This route was chosen because of the high volumes of heavy vehicles it experiences. In support of the review, the methods of deriving bridge live load models were researched. The methods used are:

- (i) The deterministic approach that uses engineering judgement to deal with the unknowns associated with the random nature of traffic loading. This method was used by Liebenberg (1974, 1978) in the drafting of TMH7 Part 2.
- (ii) The probabilistic approach that fits a mathematical distribution to recorded traffic events. This method was used in drafting Eurocode, ENV 1991-3:2000, "Basis of design and action on structures – Part 3: Traffic loads on bridge".

The basis of the live models in following bridge design codes was investigated:

- (i) BS 153: 1958, "Specification for Steel Girder Bridges", British Standards Institute;
- (ii) BS5400: 1978 "Steel, Concrete and Composite Bridges", Part 2: "Specification of loads", British Standards Institute;
- (iii) Department Standard BD 37/88 & 01, "Loads for Highway Bridges", British Department of Transport (1988, 2001);
- (iv) American Association of State Highway Transportation Officials (AASHTO), Load Resistance Factor Design (LRFD), "Bridge Design Specifications" (1994);
- (v) CAN/CSA-S6-00, "Canadian Highway Bridge Design Code" (2000) ; and
- (vi) Eurocode, ENV 1991-3:2000, "Basis of design and action on structures – Part 3: Traffic loads on bridges".

In each case reviewed, the deterministic methods of deriving live load models were replaced by probabilistic methods. Deterministic methods historically developed because of a lack of statistical data and the complexity of the variables associated with traffic movements. WIM sensors and traffic surveys have now provided a wealth of traffic data and have effectively removed this constraint. In review of TMH7 Part 2, the statistical analysis of the WIM data showed the deterministically derived vehicle combinations to be conservative in comparison with the recorded traffic flows. Load effects calculated using the WIM data substantiated this finding. It was therefore concluded that the deterministic methods do not adequately simulate the load effects caused by actual vehicles on the roads.

South Africa has yet to progress to a bridge live load model developed using probabilistic methods. Although research work, culminating in the Reports RR 91/004/01 & 02, was carried out between 1994 and 1995 in South Africa, TMH7 Parts 1 and 2 have remained unaltered since 1988. Its closest relative, BS5400, was superseded by BD 37/88 in 1988. The advent of the ENV 1991-3 in Europe further dates the deterministic derivations of THM7 Part 2's live load model.

The review of BD 37/01 also highlights a number of developments in the practice of deriving live load models that are yet to be adopted in South Africa. These developments include the derivation of loading curves that do not require the use of abnormal load models in short spans and the concept of lateral bunching. It is recommended that both developments be researched in the future revision of TMH7 Part 2.

Of the codes reviewed, ENV 1991-3 provides the most recent and extensive use of probabilistic methods to derive a live load model. For this reason, its approach provides an excellent reference for updating the live load model contained within TMH7 Part 2. As in the case of the European Community's member states, a National Application Document (NAD) based on the probabilistic analysis of local truck survey data may be developed in South Africa and other South African countries. The great advantage of basing live load models and their calibration on the probabilistic analysis of traffic survey data is that site-specific load models may be easily derived. In addition, as the properties of traffic change for technical and economic reasons, it is relatively simple to update the live load model.

In regard to the probabilistic analysis of the WIM data, the study confirmed the use of the Gumbel distribution (RR91/004/01, 1994) as the most appropriate means for extrapolating the load effects of heavy vehicles on simply supported spans. In applying the Gumbel distribution, it was shown that the extrapolated load effects are sensitive to the population size of the extreme events. It was, however, concluded that a sample period longer than one month was required to narrow the confidence limits of the predicted events.

The probabilistic analysis of the WIM data was shown to be relatively insensitive to the return period selected. However, the 2976 year return period used by RR 91/004/02, is conservative when compared with ENV 1991-3 and BD37/01. For the serviceability limit state a maximum return period of 1000 years, as per ENV 1991-3, is recommended.

The potential inaccuracy of the WIM data ( $\pm 25\%$ ) raised a question over the validity of the results. ENV 1991-3 was calibrated (O'Connor et al., 2001) using WIM data with a maximum error of 5%. Similar standards are required in South Africa if WIM data is to be used in the calibration of future bridge live load models.

The analysis of the WIM data indicates that the overloading of axles and axle sets is more prevalent than the overloading of a complete vehicle. As in the case of BD37/88, the use of an overloading factor that decreases as the span increases is therefore proposed.

The load effects generated from the probabilistic analysis of the WIM data provided an important reference for the critical review of the live load models in TMH7 Part 2 and RR 91/004/02. The following conclusions were reached.

- i.) The deficiency of TMH7 Part 2's NA loading to cater for normal traffic load effects on short spans was proven. The use of NB loading to derive load effects due to normal traffic is not coherent with a rationally based live load model.
- ii.) The use of a Monte Carlo simulation to generate target values that are required in formulating a design load is considered a valid approach (ENV 1991-3). The assumption used in RR 91/004/01 & 02 that all vehicles are fully loaded is considered to be conservative. It is recommended that future simulations be based on a garage of vehicles whose axle weights and GVM's are distributed as in the case of normal traffic conditions. This method was used in the drafting of ENV 1991-3 (O'Connor et al., 2001). The widespread use of WIM sensors on South African Toll roads now provides sufficient data on which to base such an approach.
- iii.) The un-factored design loads in RR 91/004/01 and TMH7 Part 2 are formulated from the extrapolation of events that are extreme in themselves. This methodology is excessively conservative in comparison to modern codes of practice such as ENV 1991-3 and BD 37/01. It is recommended that characteristic load events be derived from normal traffic conditions occurring over a rationally-based return period.
- iv.) The form of the live load model proposed in RR91/004/02 was verified by the findings of the thesis. A method of calibrating this live load model is demonstrated in the thesis. The form of the proposed live load model is almost identical to that used in ENV 1991-3. The adoption of ENV 1991-3 and the issue of a South African NAD is therefore a viable alternative to TMH7 Part 2.

The thesis considers alternative live load models to TMH7 Part 2's NA loading curve. A load model containing two closely spaced point loads in combination with a constant uniformly distributed load was found to accurately simulate the load effects of South African heavy vehicles. This finding supports the use of the load model proposed in RR 91/004/02; it also shows that ENV 1991-3's load model may be used in South Africa. The probabilistic analysis of the WIM data demonstrates the means by which ENV 1991-3's load model may be calibrated to South African conditions. As opposed to the future revision of TMH7, a viable alternative is the adoption of ENV 1991-3 and the drafting of a National Application Document (NAD). The significant research and development carried out in developing the limit state principles and live load model in ENV 1991-3 may then be utilised in South Africa with limited expenditure.

The derivation of a complete load model is a task beyond the scope of this thesis. There are many factors such as dynamic effects, lateral bunching and multi-lane loading that must be considered in formulating a live load model. The aim of the study was to review current international practice with the aim of critically reviewing the live load models in TMH7 Part 2 and RR 91/004/02. This review

has highlighted the fact that further development of live loading in South Africa must be based on the rational assessment of the traffic events on the roads. The use of the WIM data to derive characteristic load effects demonstrates one way of doing this.

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**APPENDIX A**

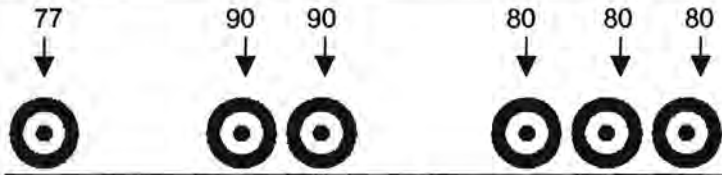
University of Cape Town

**Appendix A:  
Vehicle Configurations & Classifications**

University of Cape Town

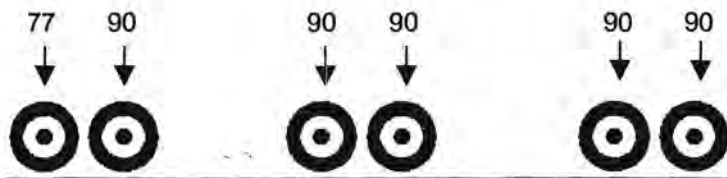
# Recorded 6 Axle Vehicle Configurations

## Configuration 1



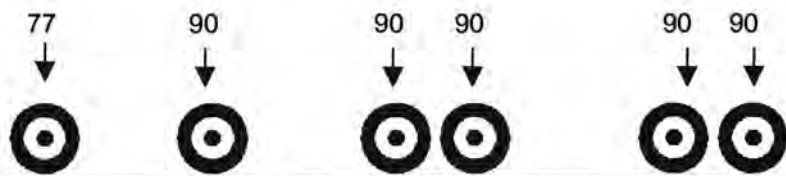
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## Configuration 2



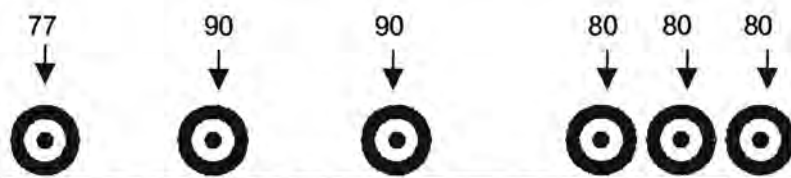
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## Configuration 3



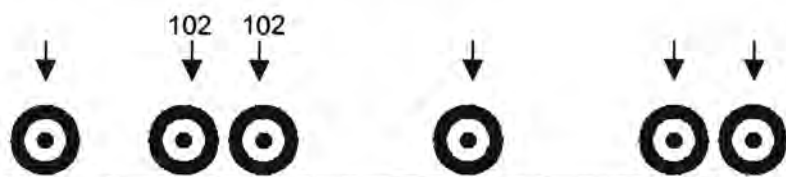
Max W = 527kN

## Configuration 4



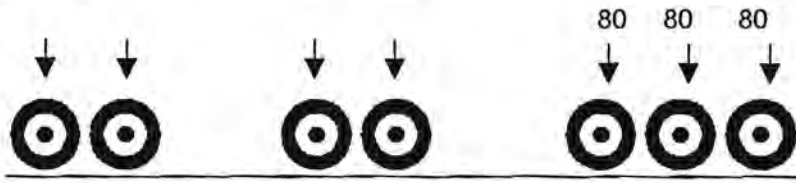
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## Configuration 5



Max W = 560kN

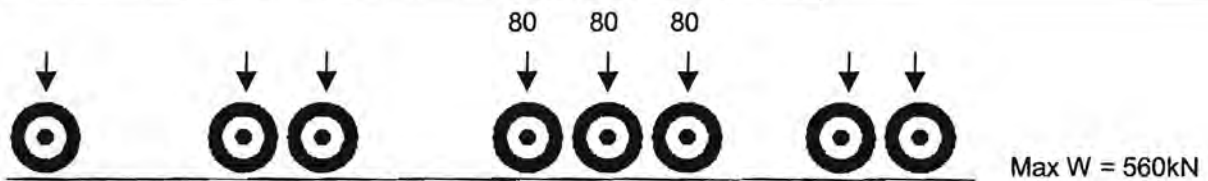
Configuration 7



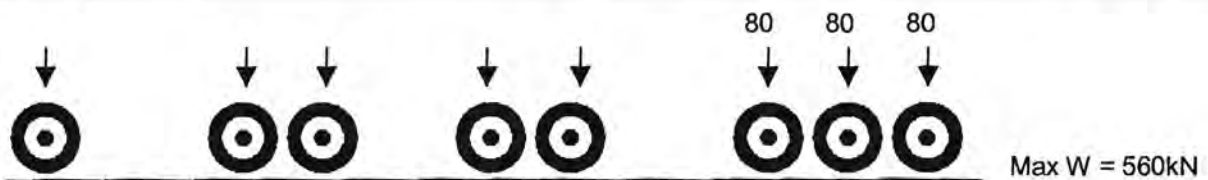
Max W = 560kN

# Recorded 8 Axle Vehicle Configurations

## Configuration 1



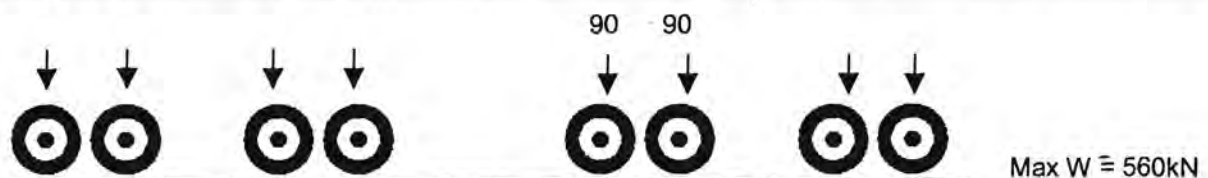
## Configuration 2



## Configuration 3

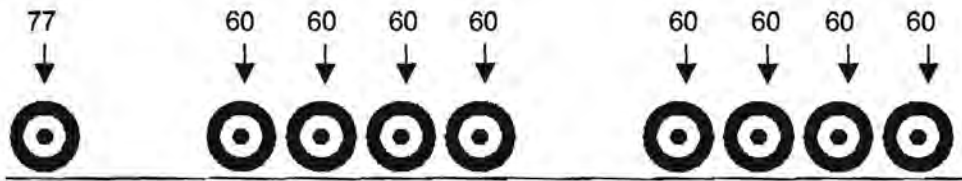


## Configuration 4



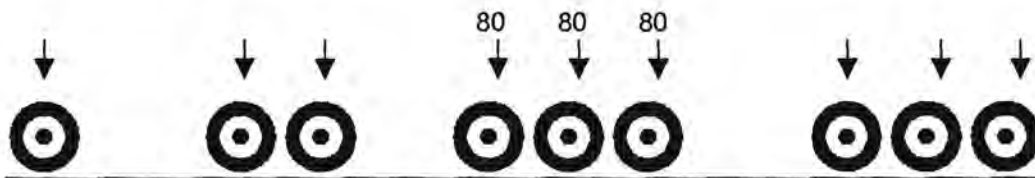
## Recorded 9 Axle Vehicle Configurations

### Configuration 1



Max W = 557kN

### Configuration 2



Max W = 560kN

### Configuration 3

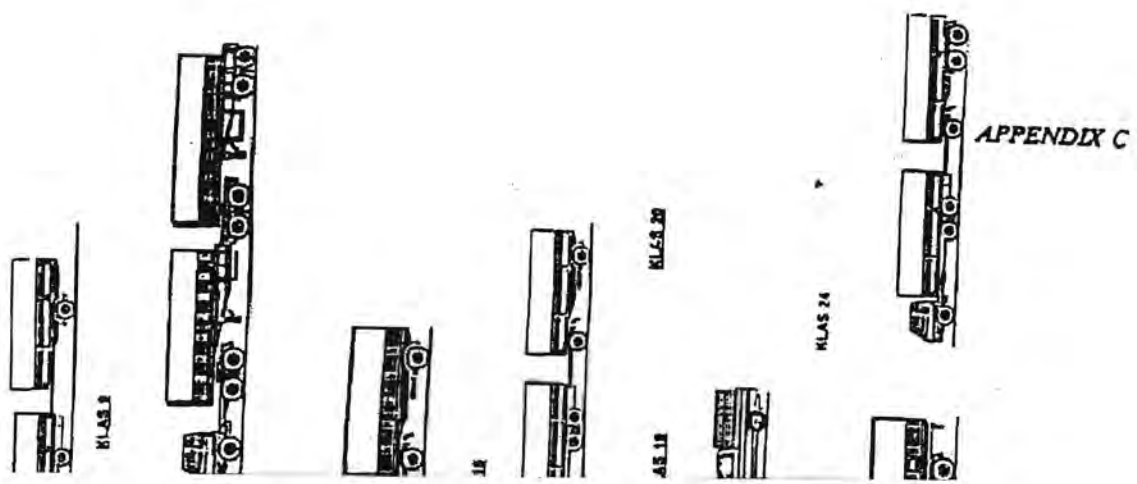


Max W = 557kN

### Configuration 4

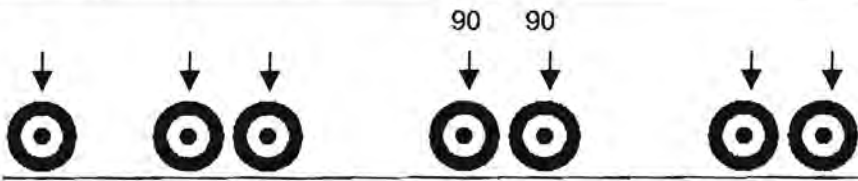


Max W = 557kN



# Recorded 7 Axle Vehicle Configurations

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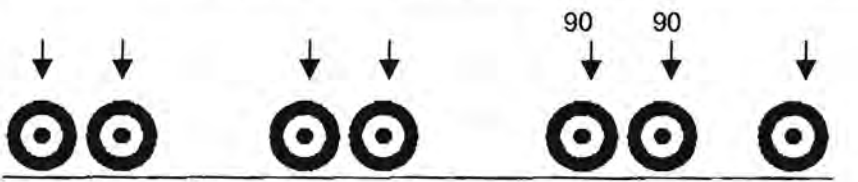
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## Configuration 2



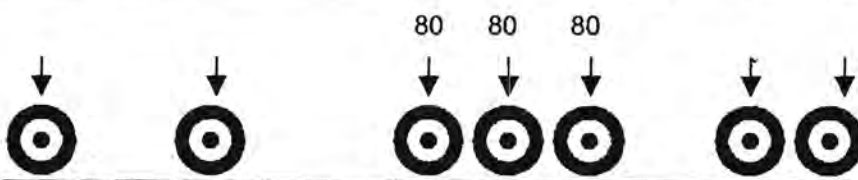
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## Configuration 3



Max W = 560kN

## Configuration 4



Max W = 560kN

## Configuration 5



**Appendix B:  
Statistical Distributions**

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## Frequency Distribution of Bending Moments of Actual Vehicles

### 6 Axle Vehicles

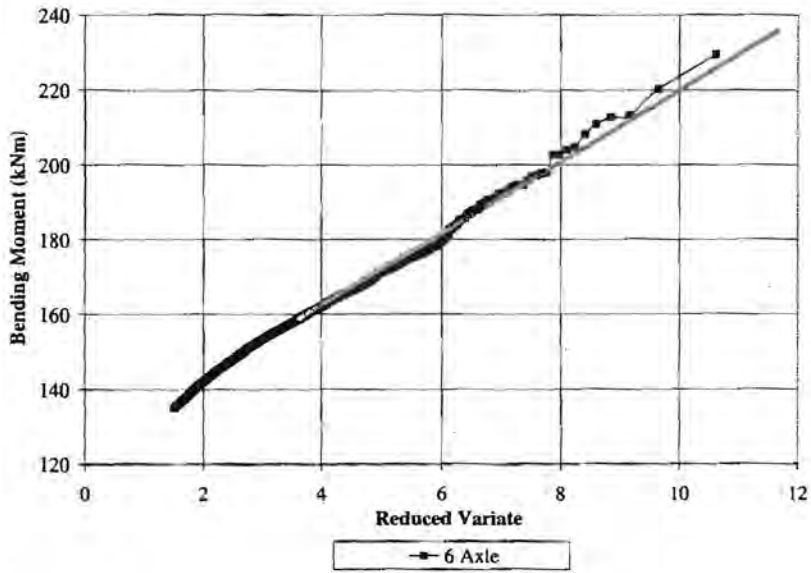


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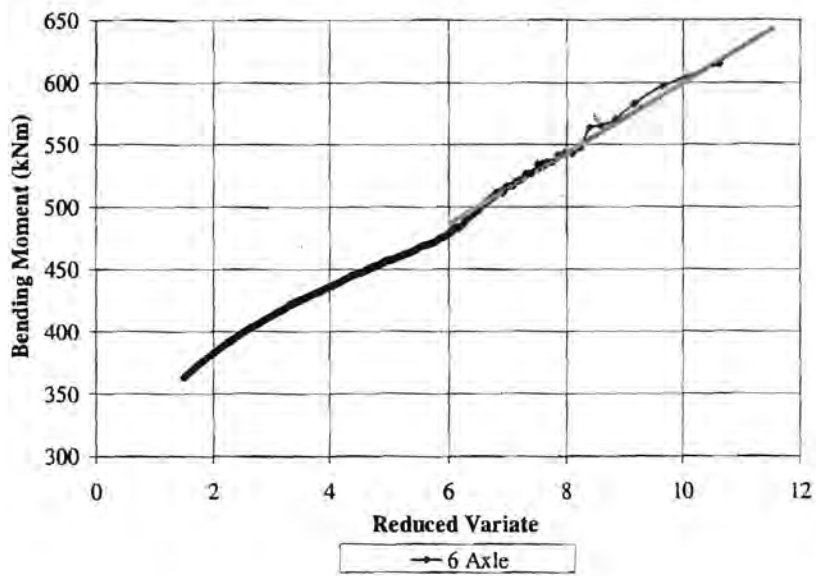


Figure B2: 10m span

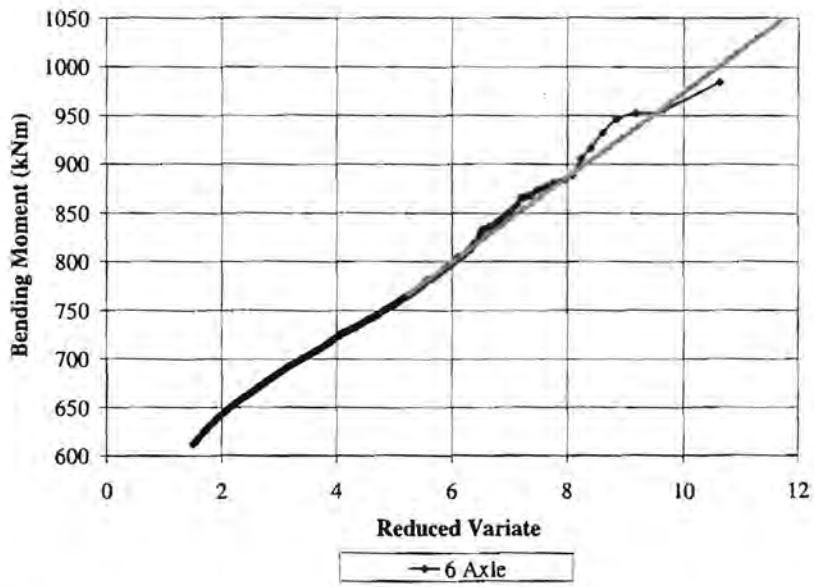


Figure B3: 15m span

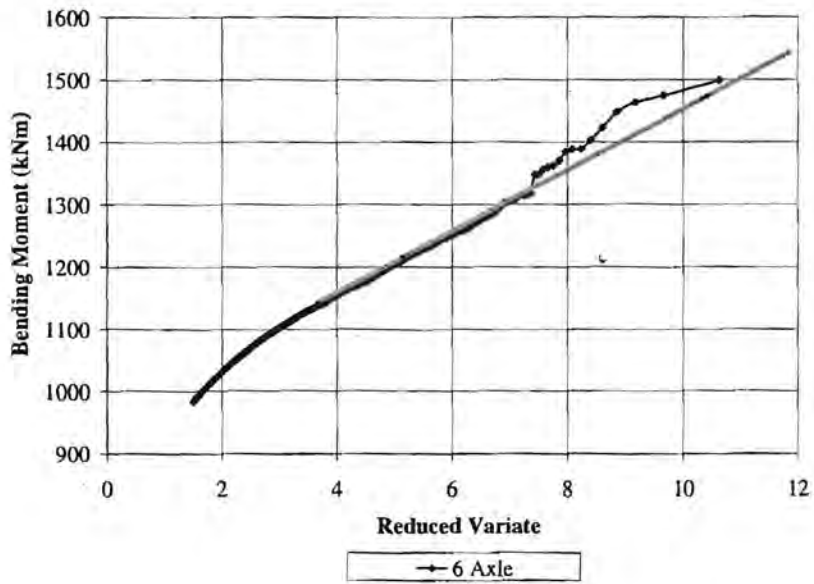


Figure B4: 20m span

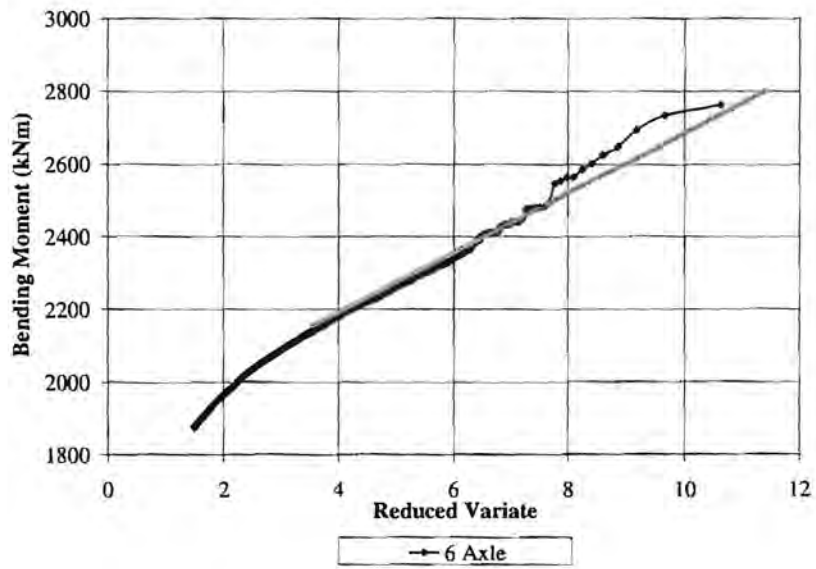


Figure B5: 30m span

### Frequency Distribution of Shear Forces of Actual Vehicles 6 Axle Vehicles

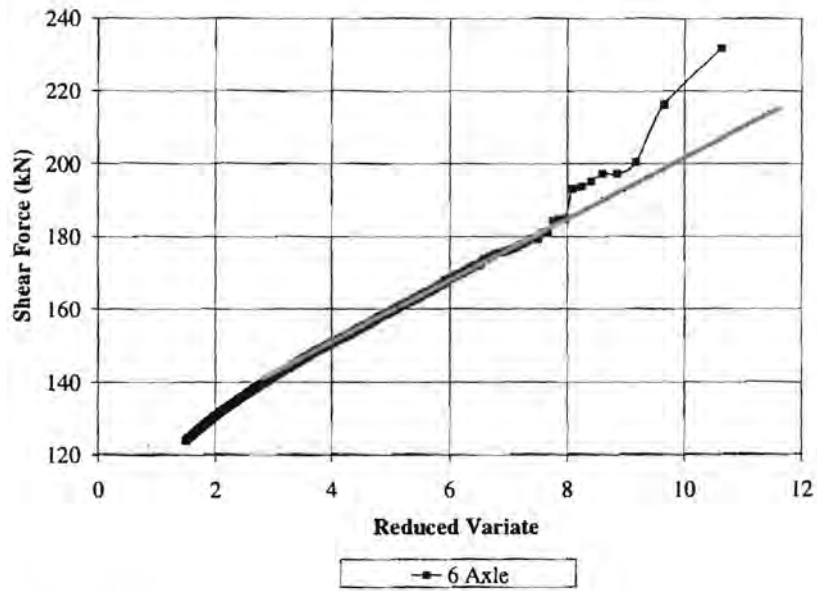


Figure B6: 5m span

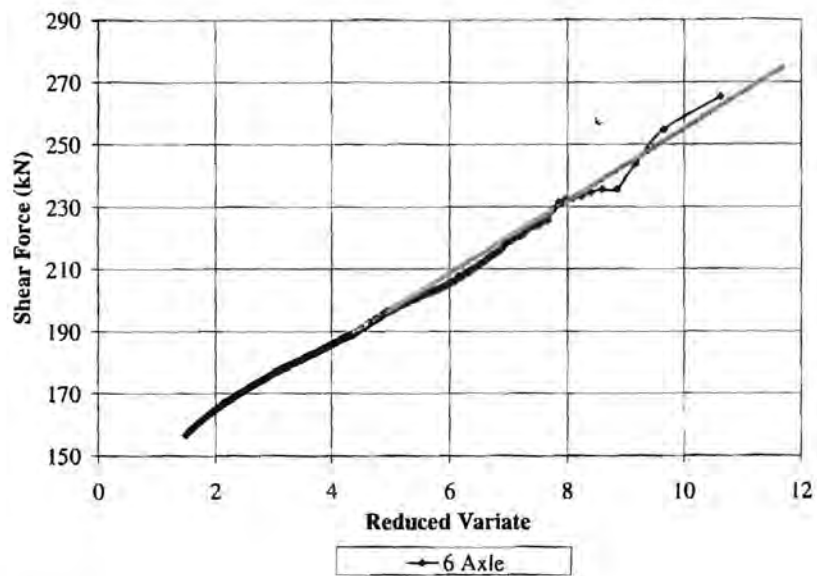


Figure B7: 10m span

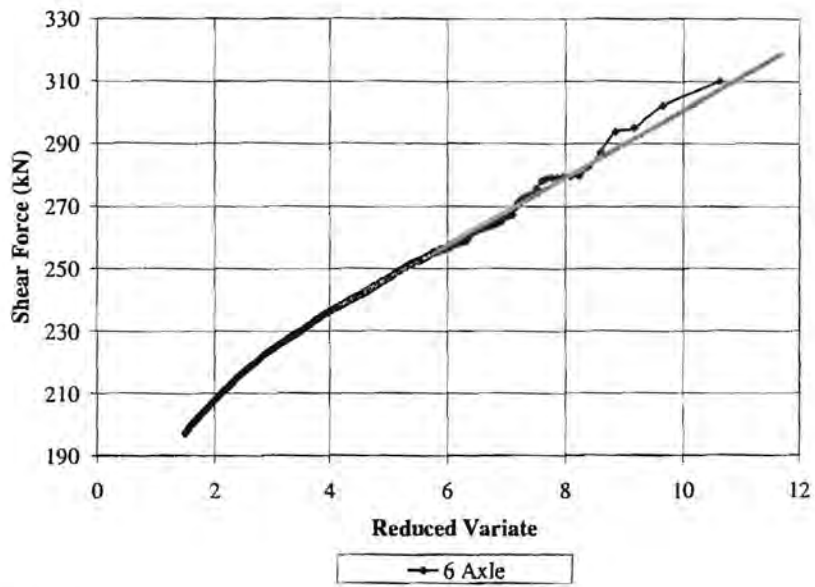


Figure B8: 15m span

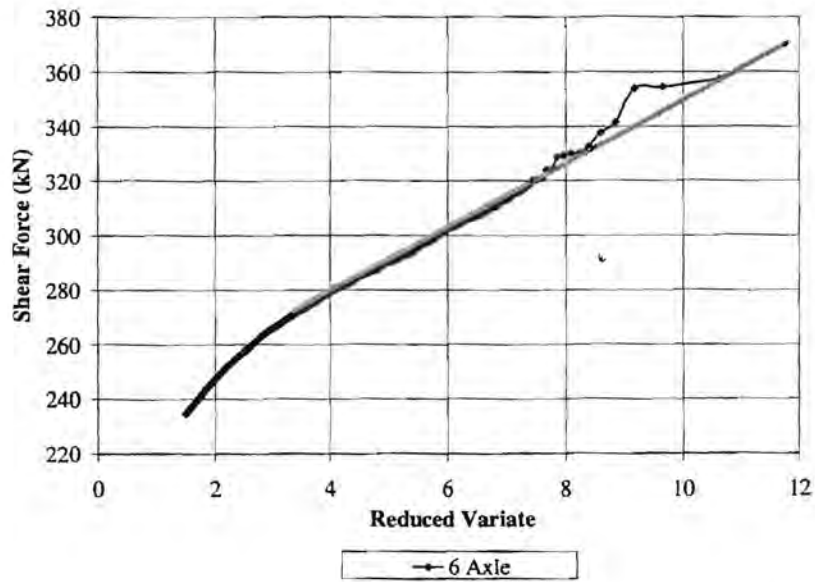


Figure B9: 20m span

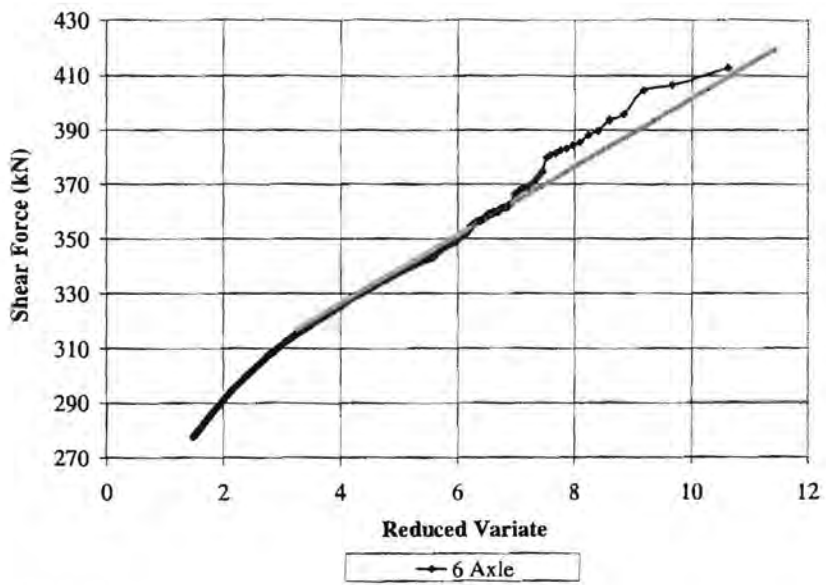


Figure B10: 30m span

## Frequency Distribution of Bending Moments of Legal Vehicles

### 6 Axle Vehicles

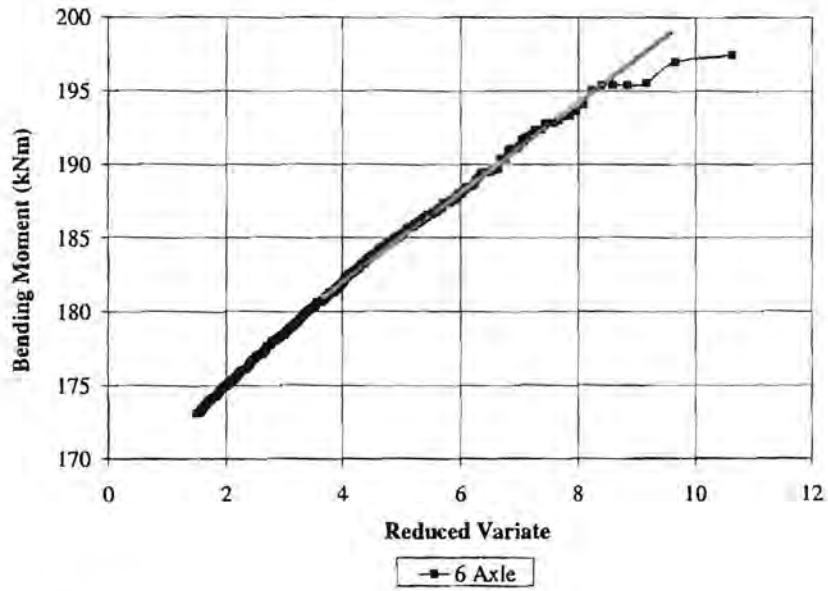


Figure B11: 5m span

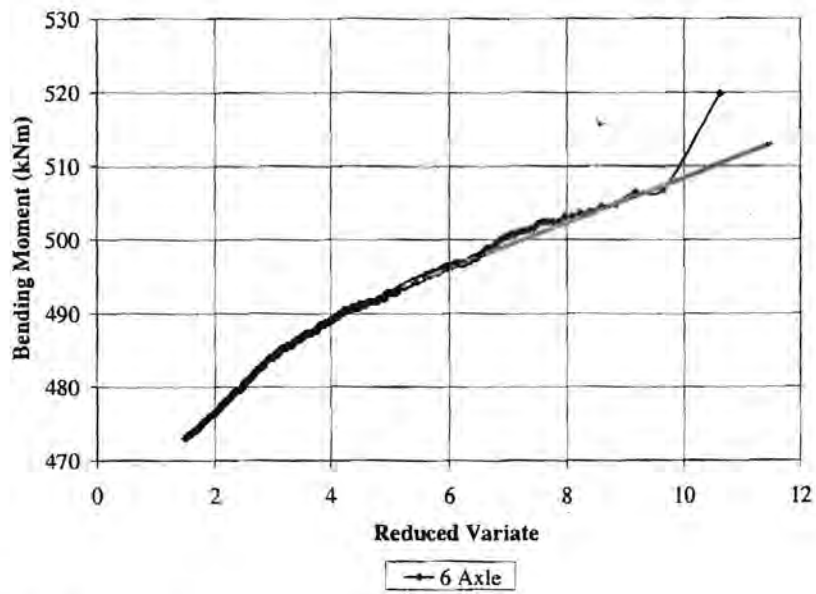


Figure B12: 10m span

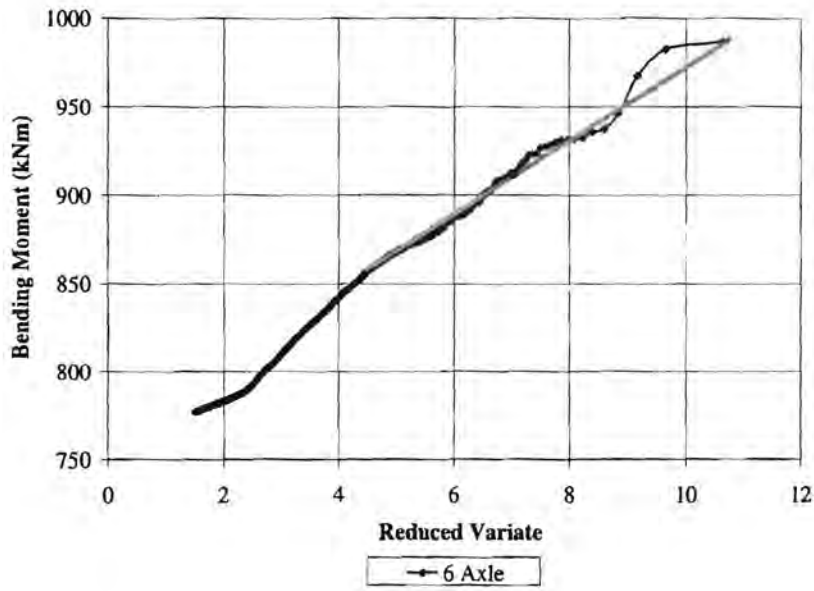


Figure B13: 15m span

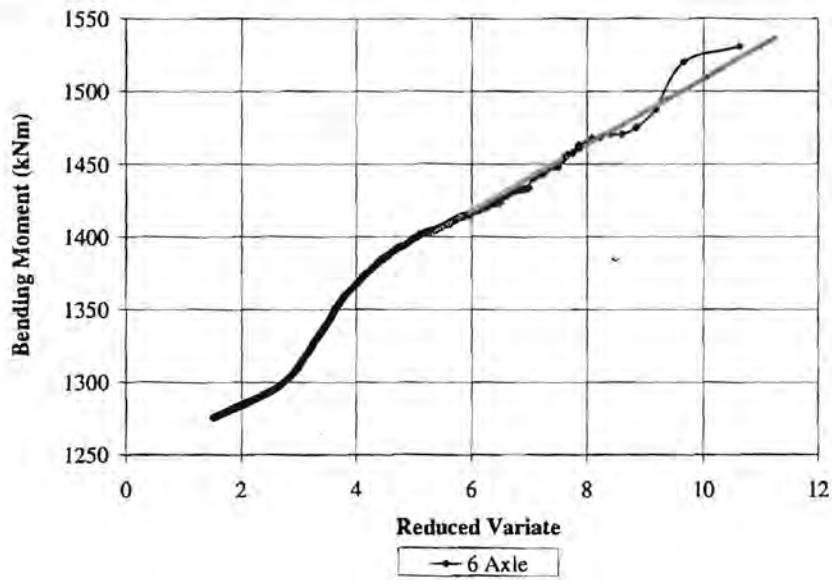


Figure B14: 20m span

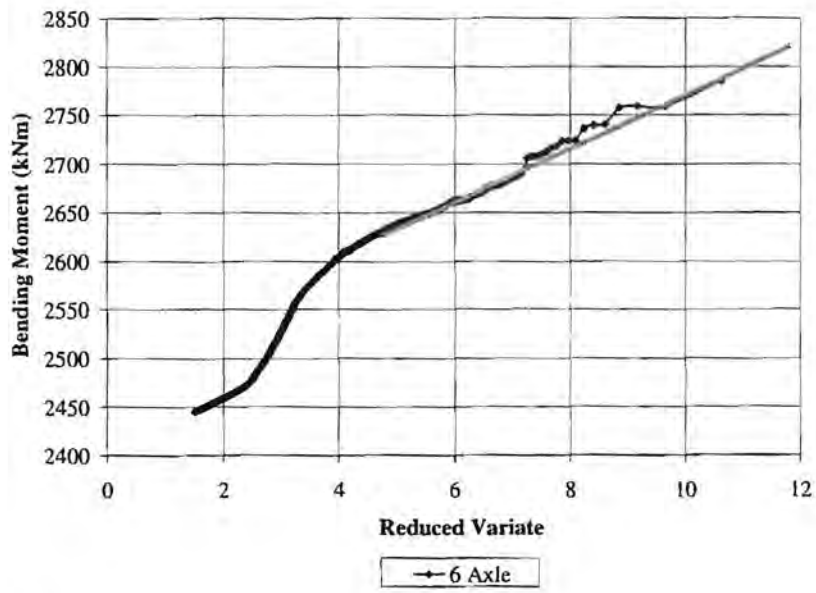


Figure B15: 30m span

## Frequency Distribution of Shear Forces of Legal Vehicles

### 6 Axle Vehicles

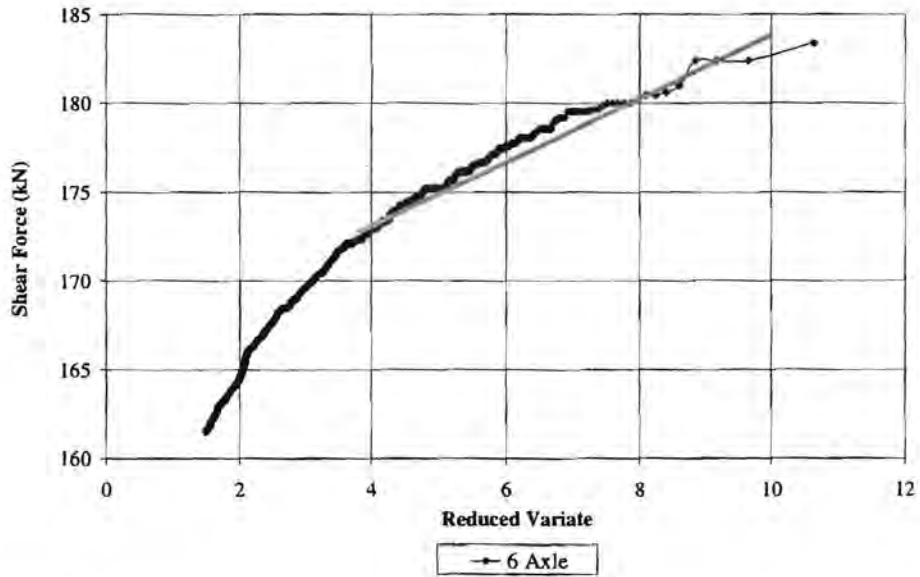


Figure B16: 5m span

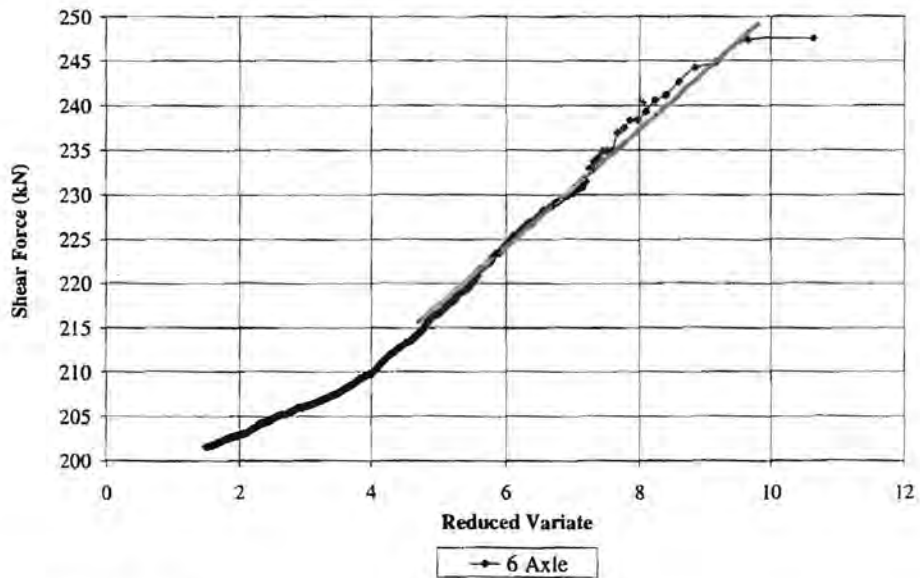


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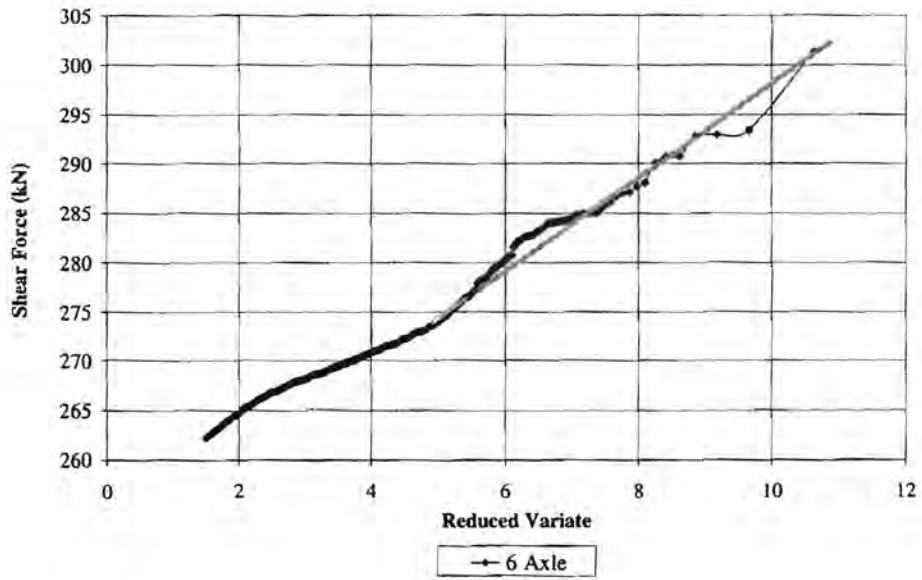


Figure B18: 15m span

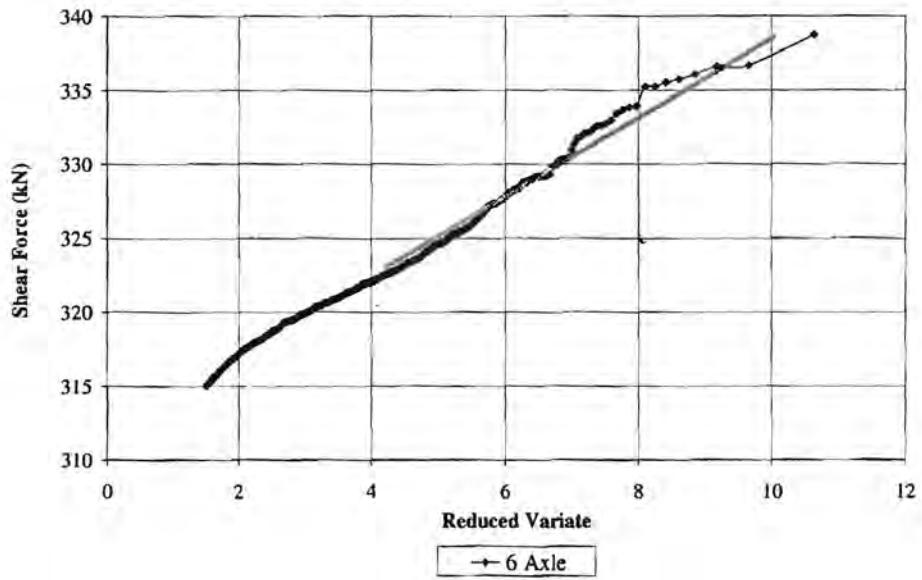


Figure B19: 20m span

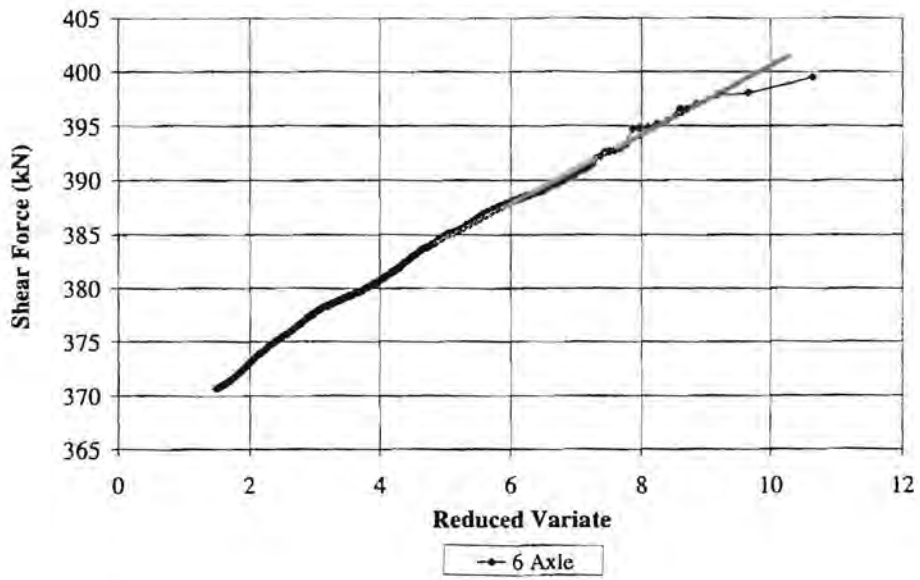


Figure B20: 30m span

## Frequency Distribution of Bending Moments of Actual Vehicles

### 7 Axle Vehicles

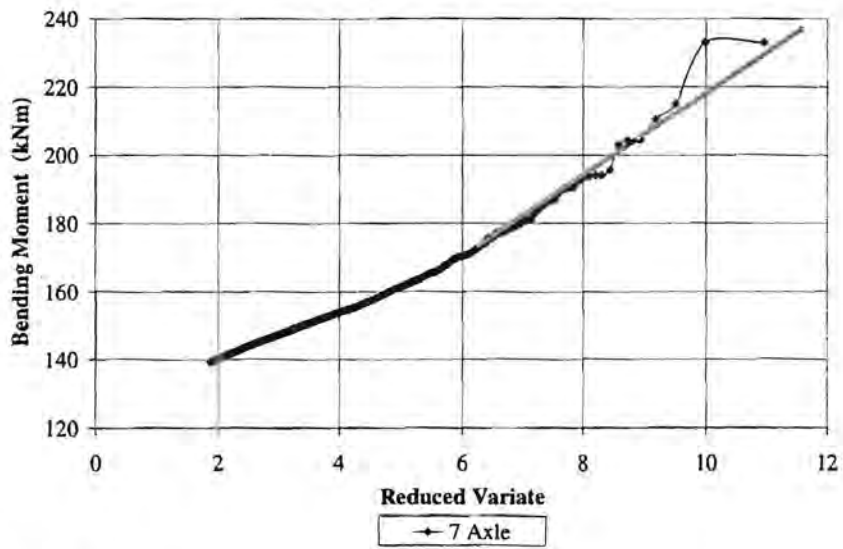


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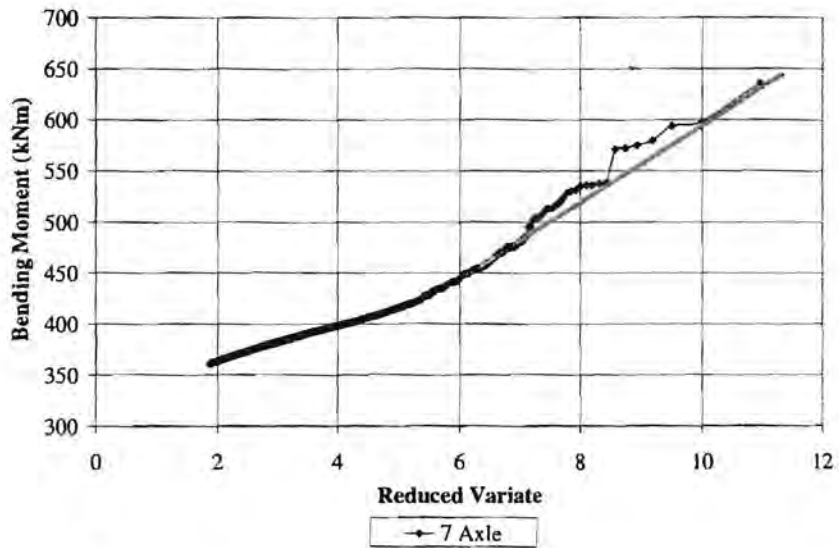


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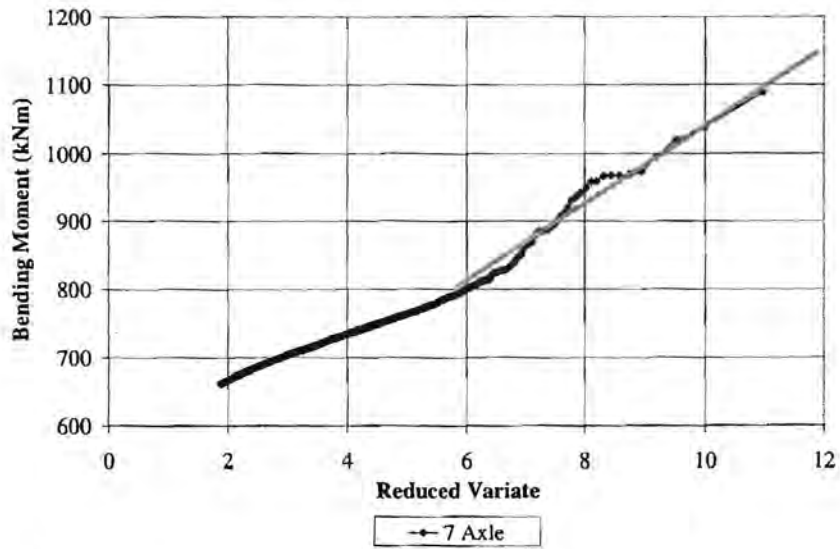


Figure B23: 15m span

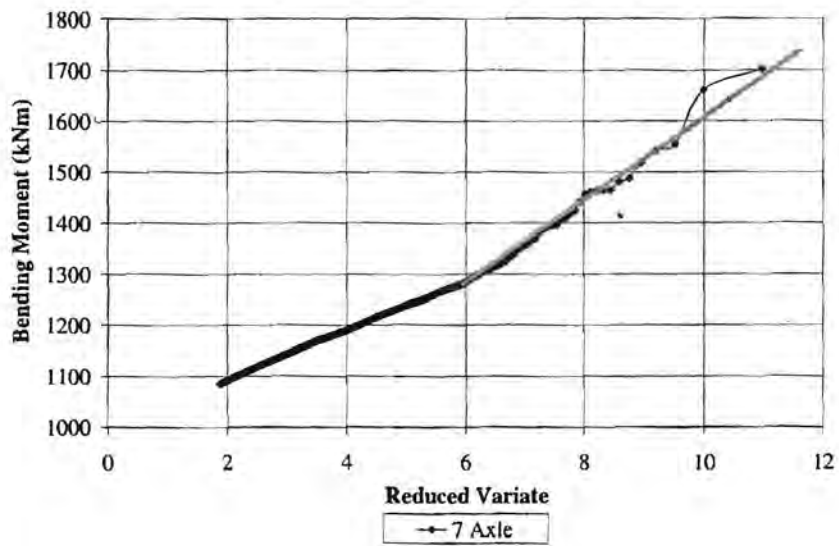


Figure B24: 20m span

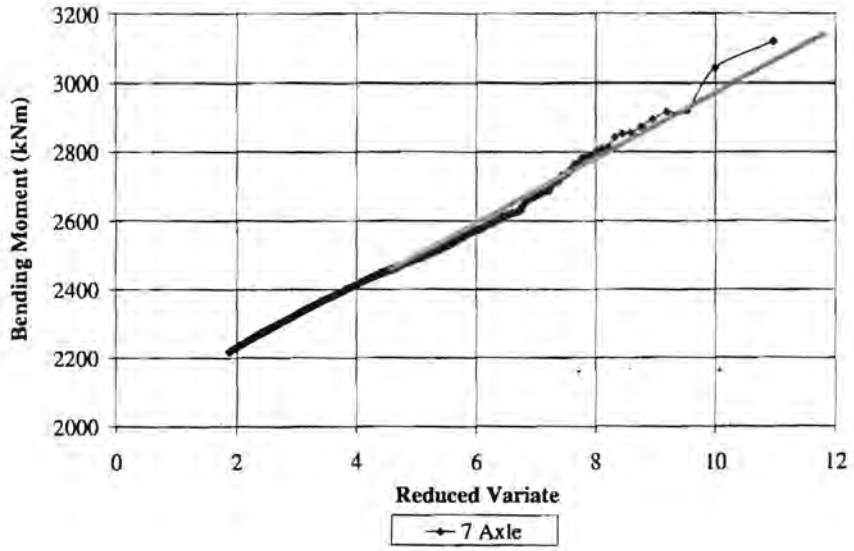


Figure B25: 30m span

## Frequency Distribution of Shear Forces of Actual Vehicles 7 Axle Vehicles

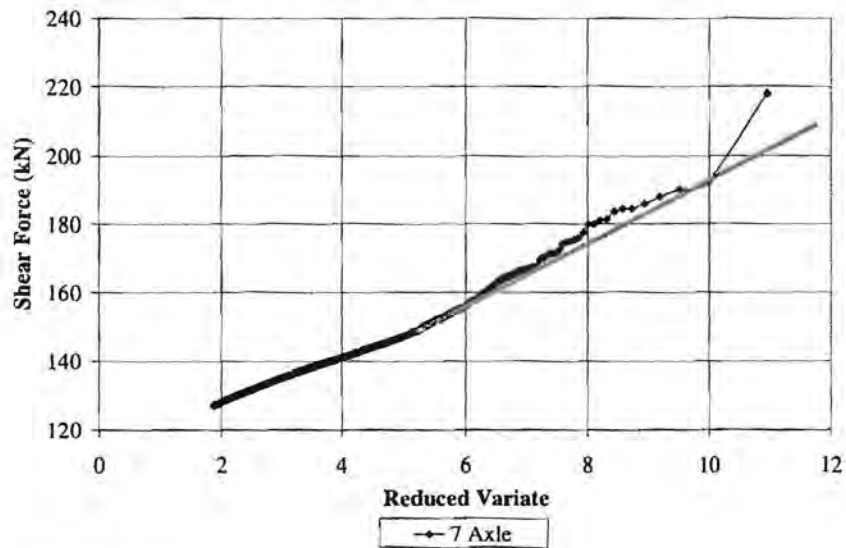


Figure B26: 5m span

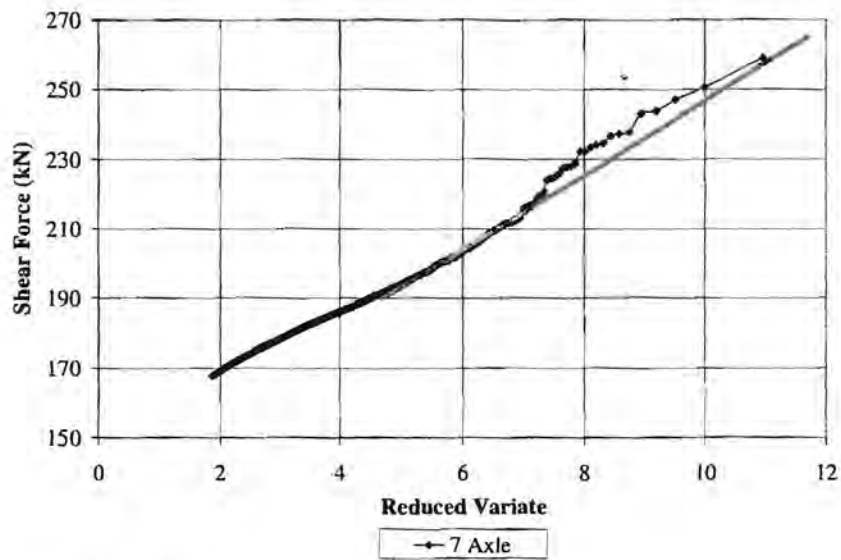


Figure B27: 10m span

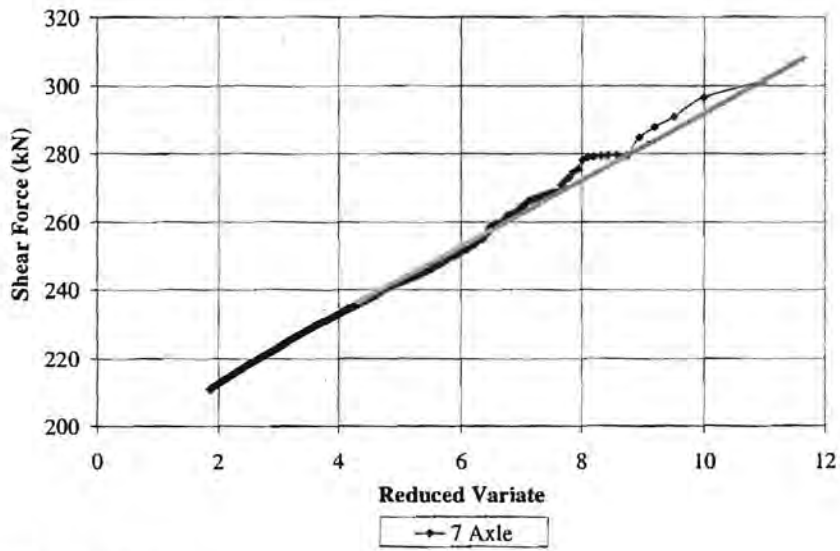


Figure B28: 15m span

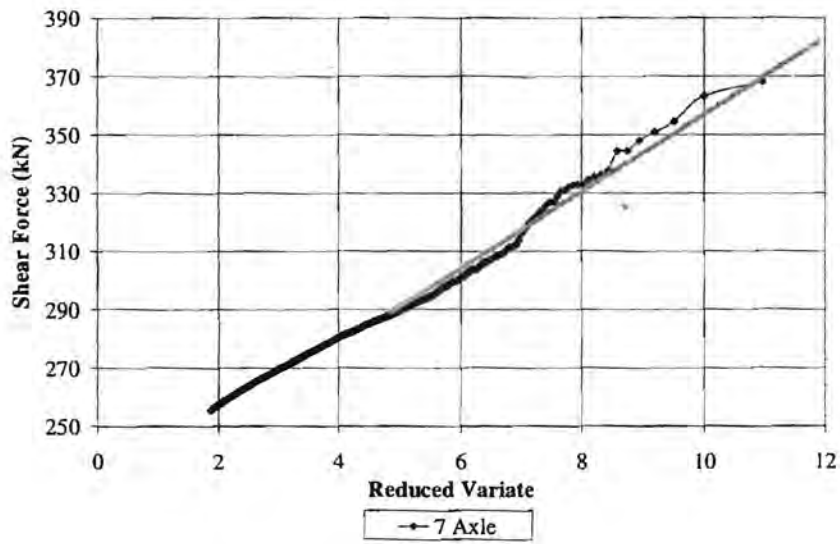


Figure B29: 20m span

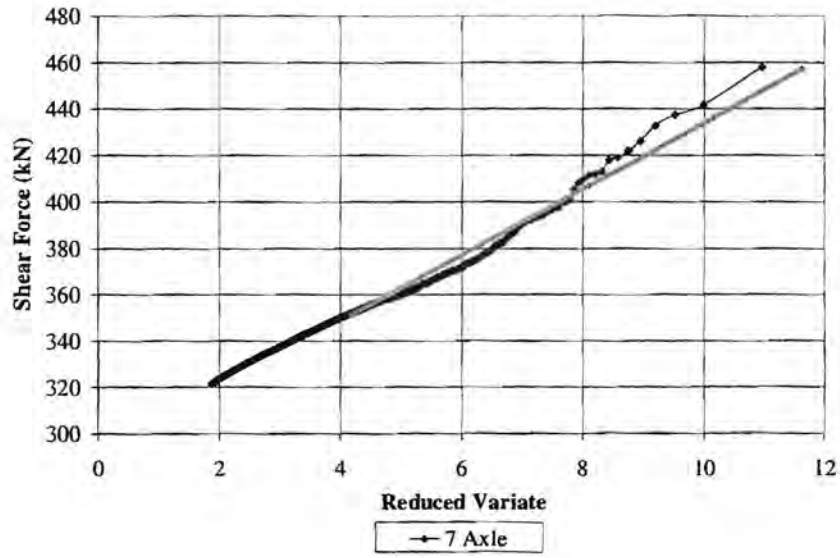


Figure B30: 30m span

## Frequency Distribution of Bending Moments of Legal Vehicles

### 7 Axle Vehicles

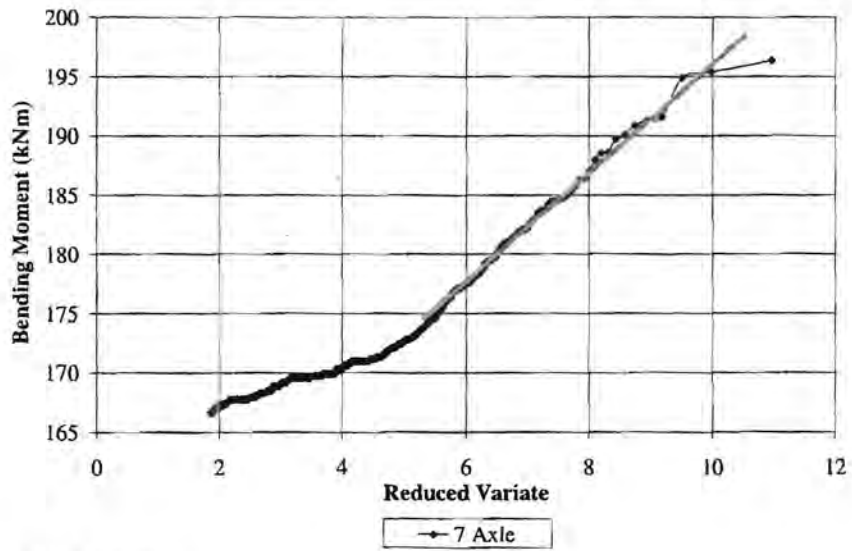


Figure B31: 5m span

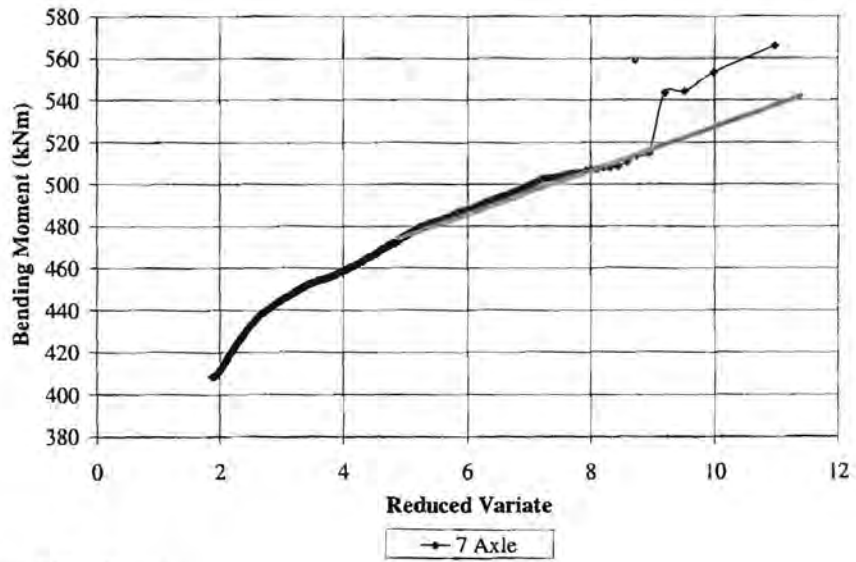


Figure B32: 10m span

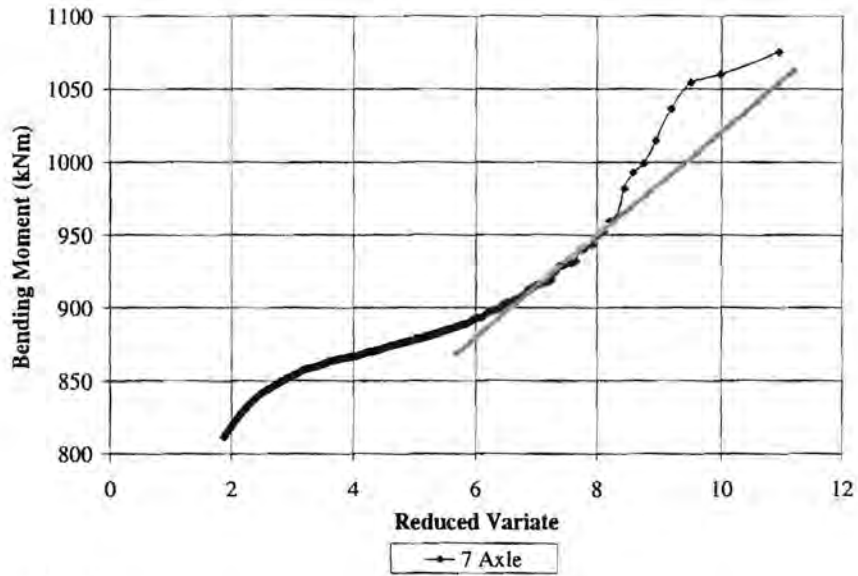


Figure B33: 15m span

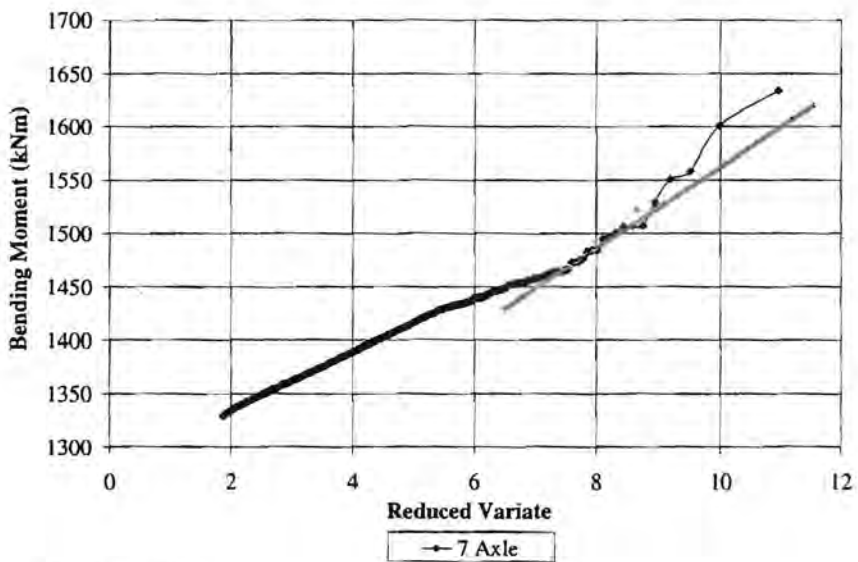


Figure B34: 20m span

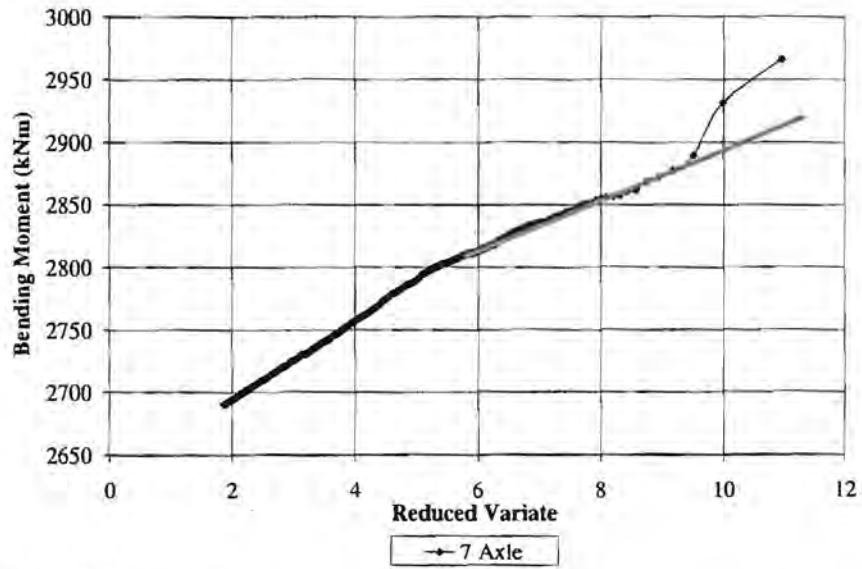


Figure B35: 30m span

## Frequency Distribution of Shear Forces of Legal Vehicles

### 7 Axle Vehicles

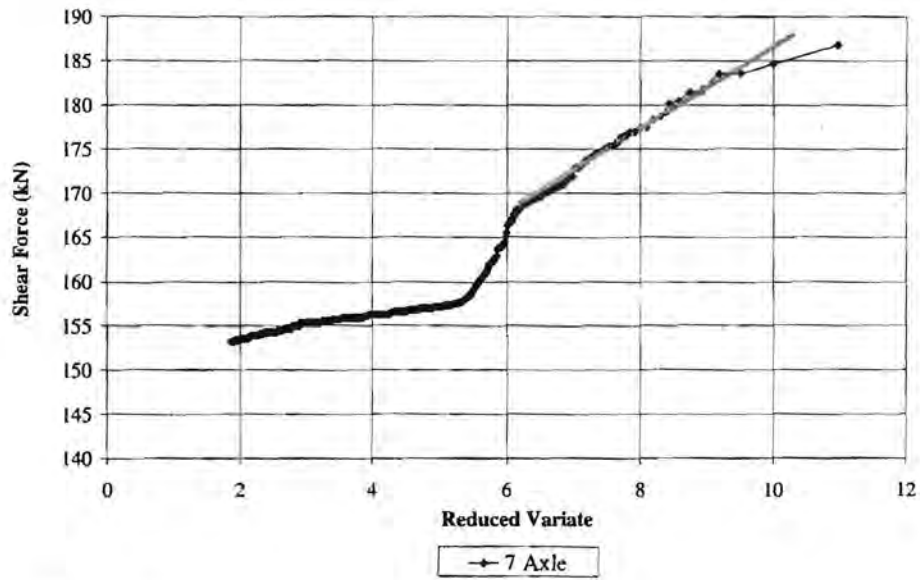


Figure B36: 5m span

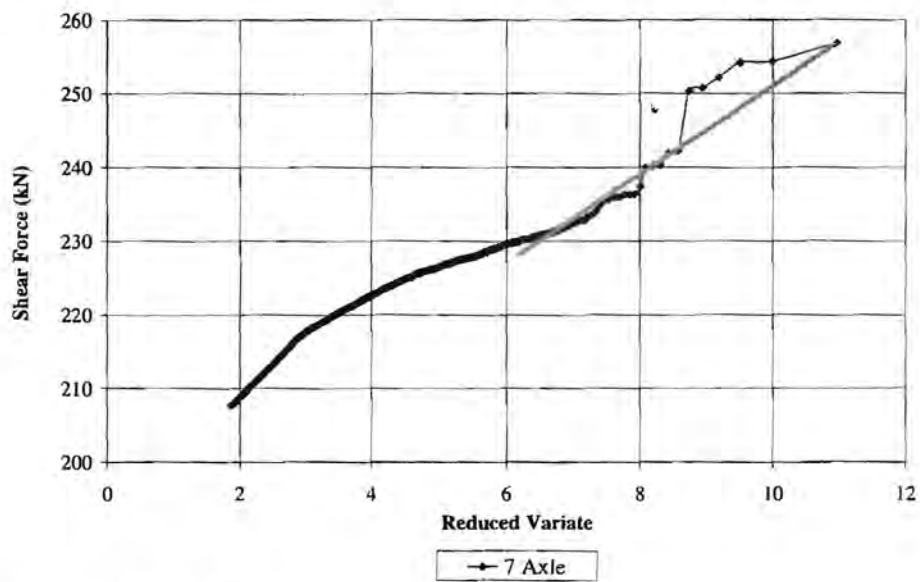


Figure B37: 10m span

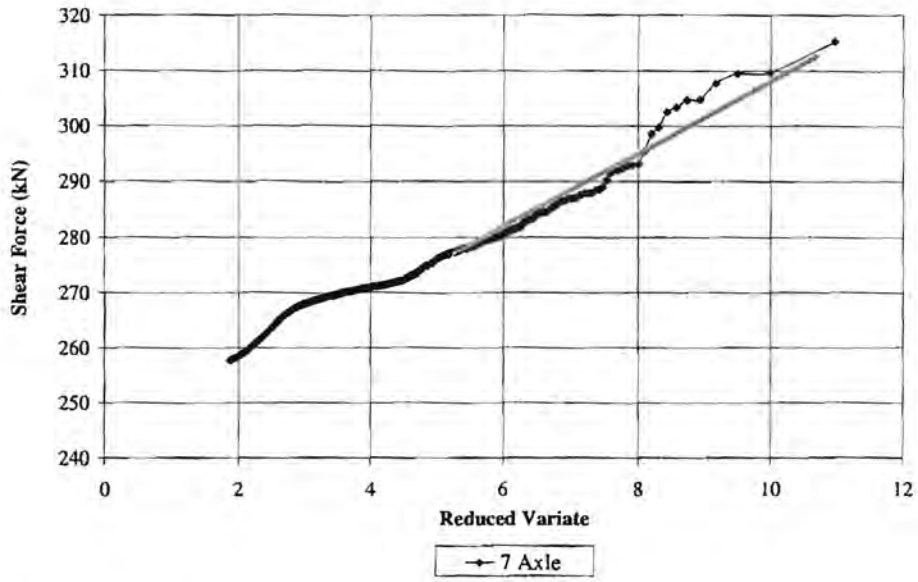


Figure B38: 15m span

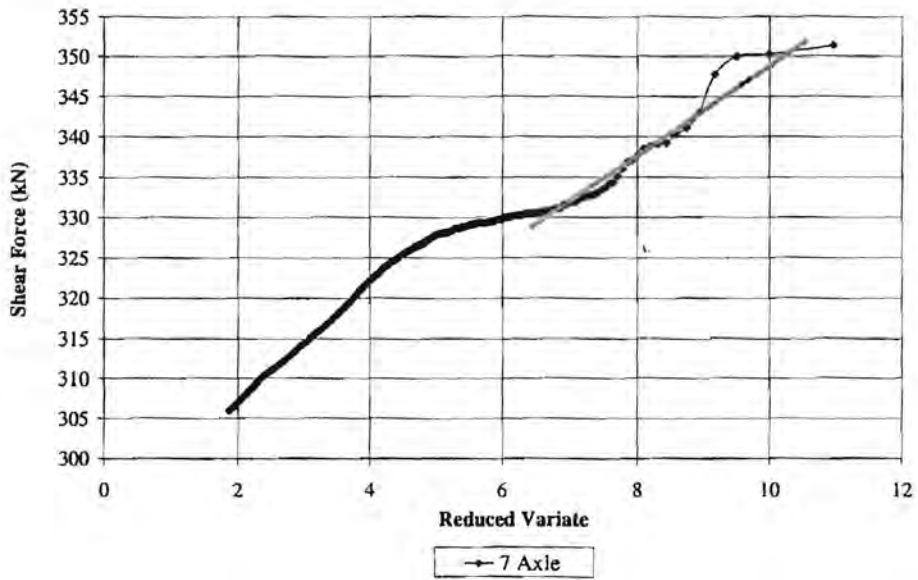


Figure B39: 20m span

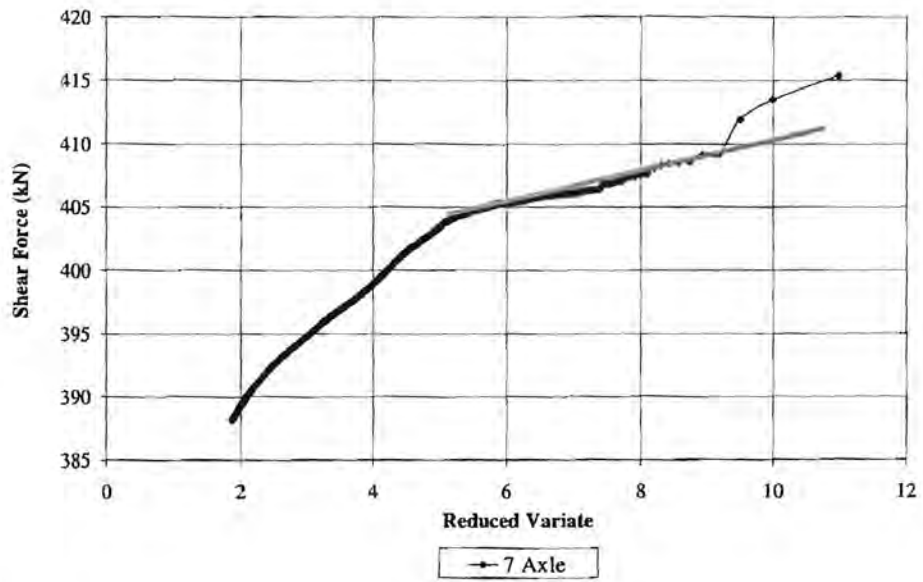


Figure B40: 30m span

## Frequency Distribution of Bending Moments of Actual Vehicles

### 8 Axle Vehicles

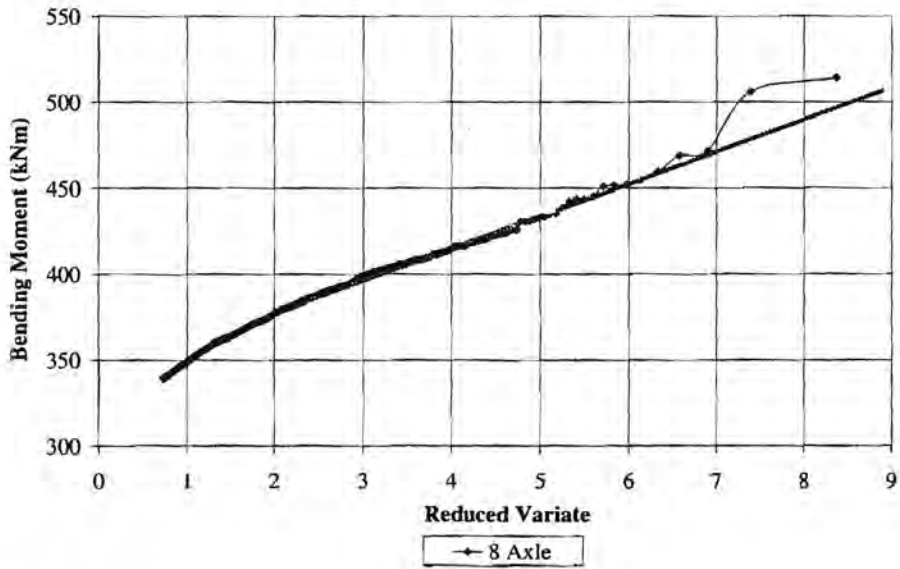


Figure B41: 5m span

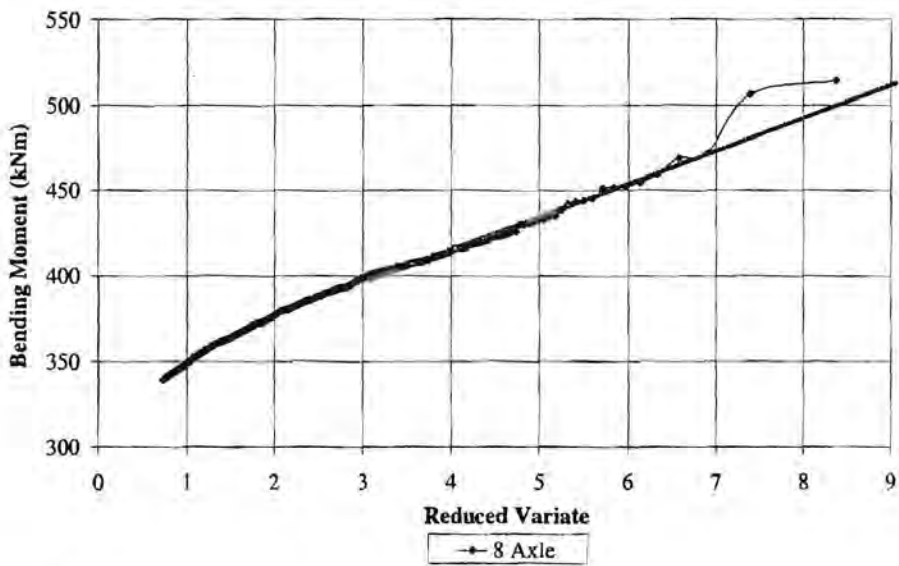


Figure B42: 10m span

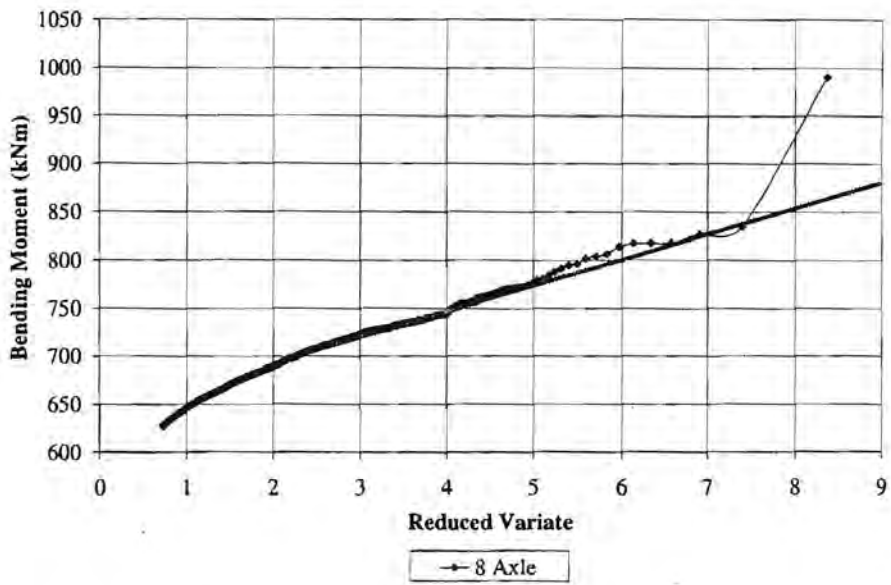


Figure B43: 15m span

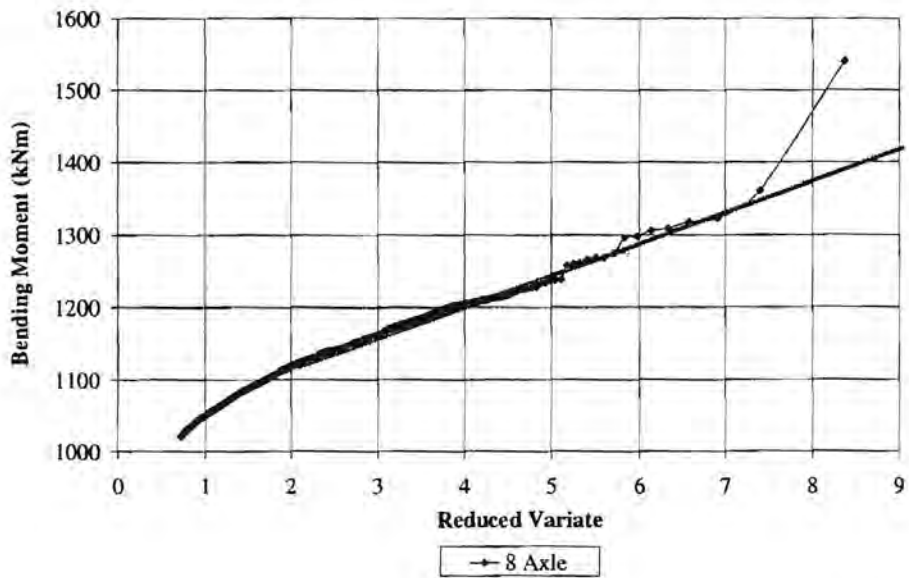


Figure B44: 20m span

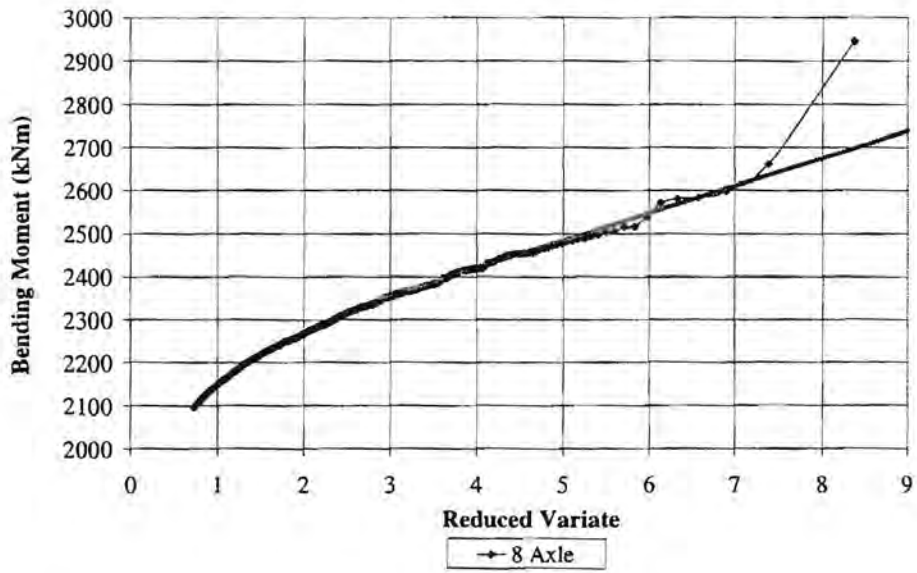


Figure B45: 30m span

## Frequency Distribution of Shear Forces of Actual Vehicles

### 8 Axle Vehicles

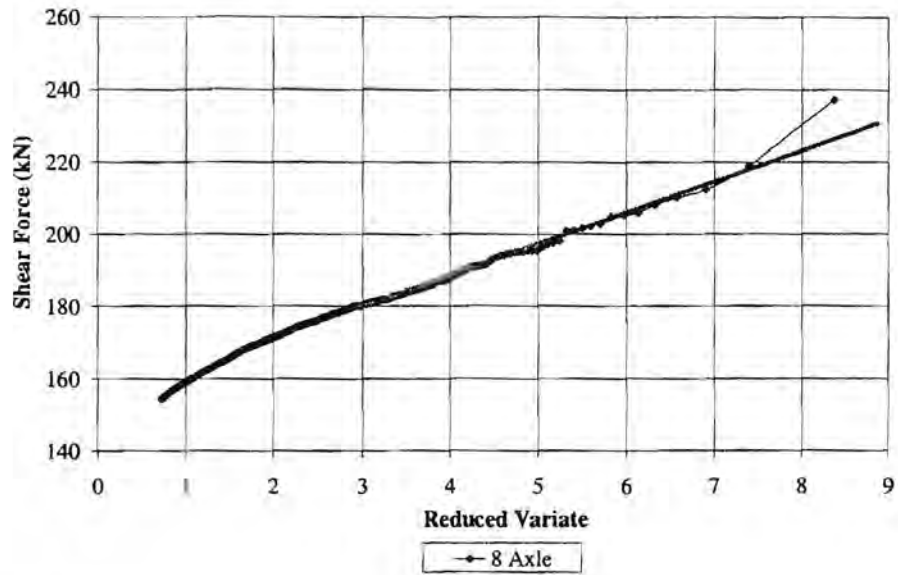


Figure B46: 5m span

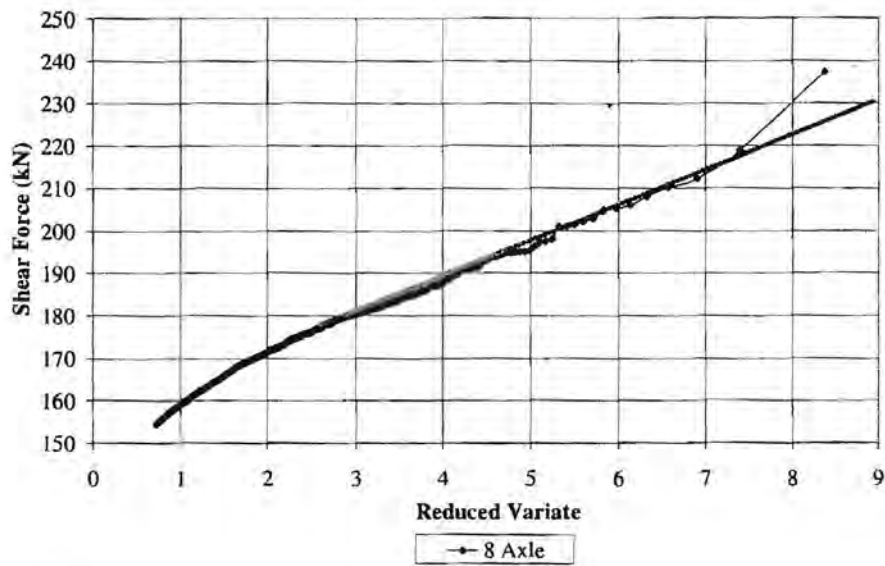


Figure B47: 10m span

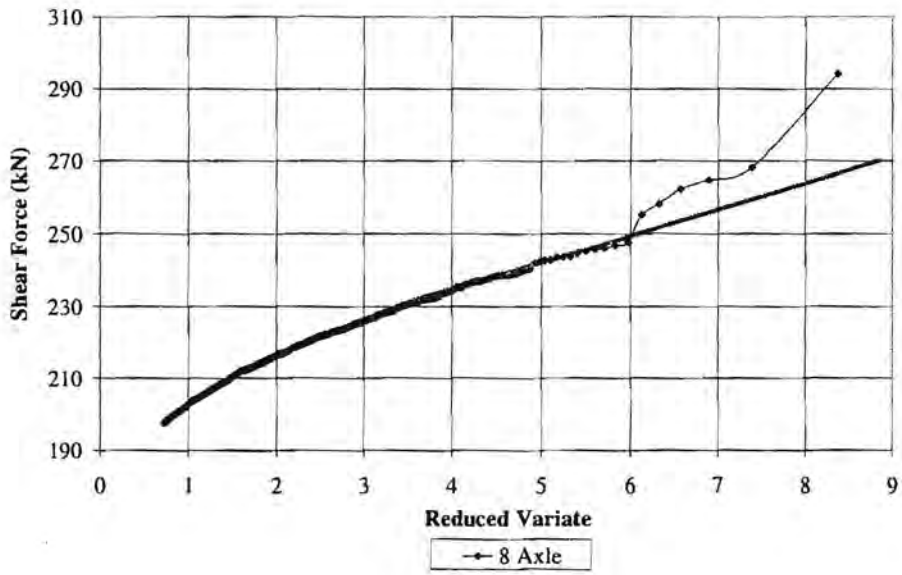


Figure B48: 15m span

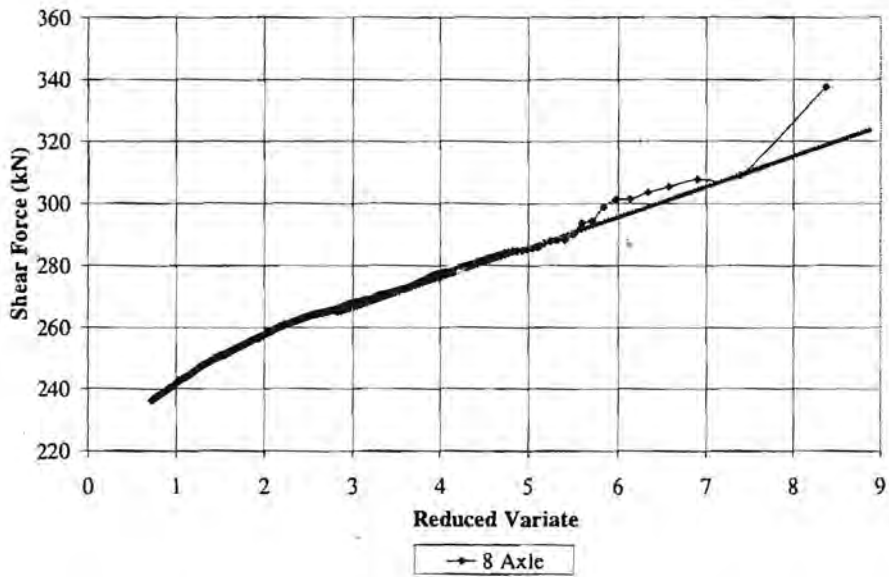


Figure B49: 20m span

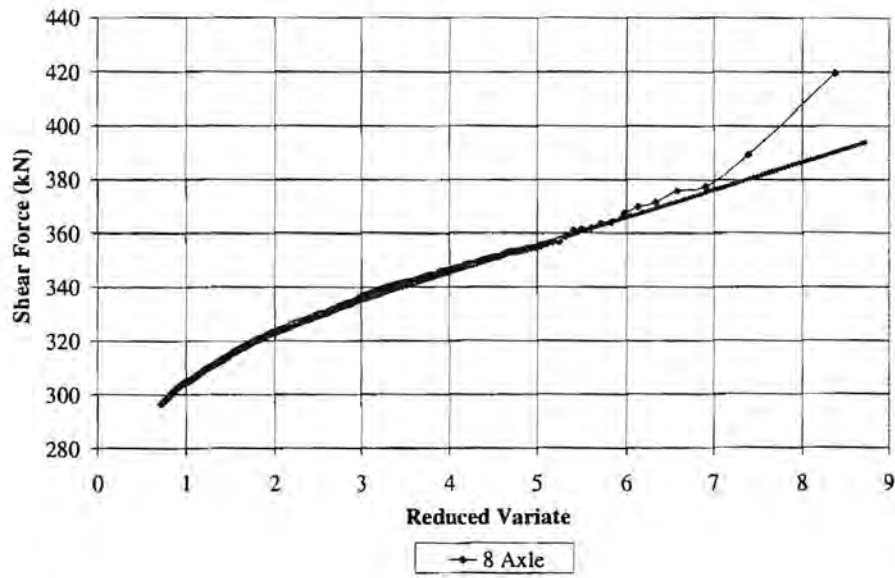


Figure B50: 30m span

## Frequency Distribution of Bending Moments of Legal Vehicles

### 8 Axle Vehicles

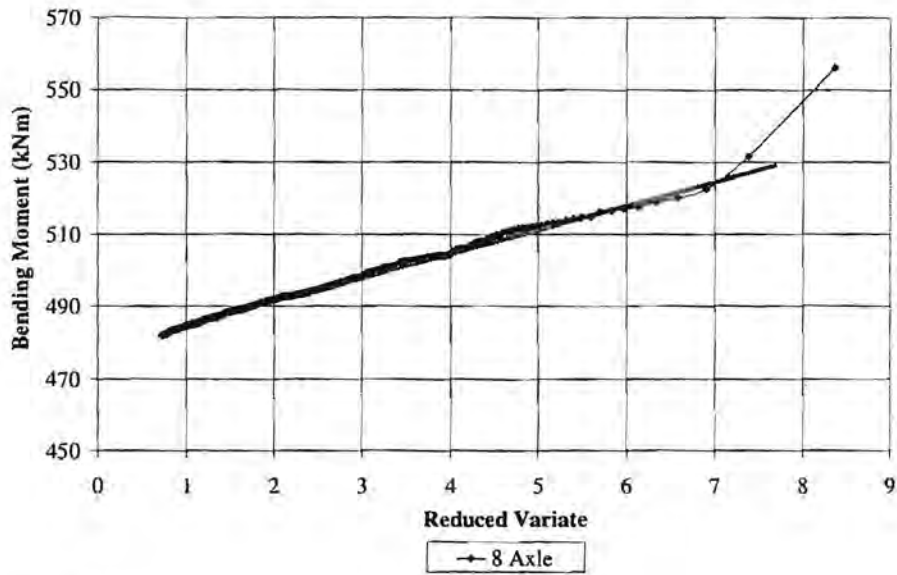


Figure B51: 5m span

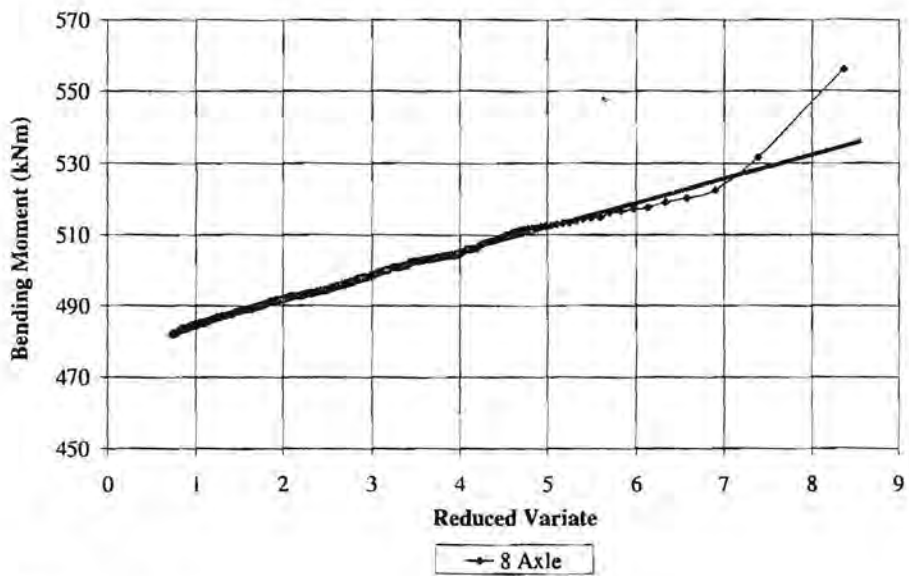


Figure B52: 10m span

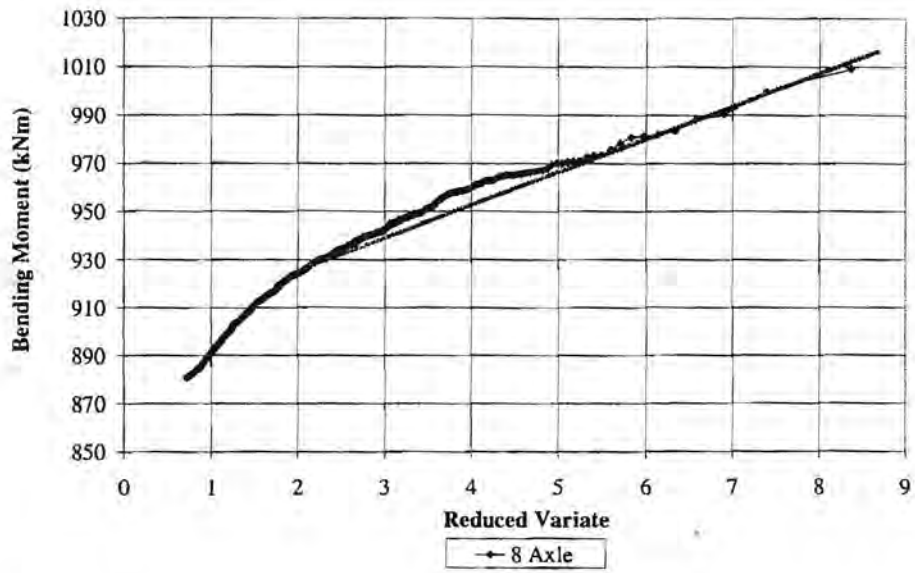


Figure B53: 15m span

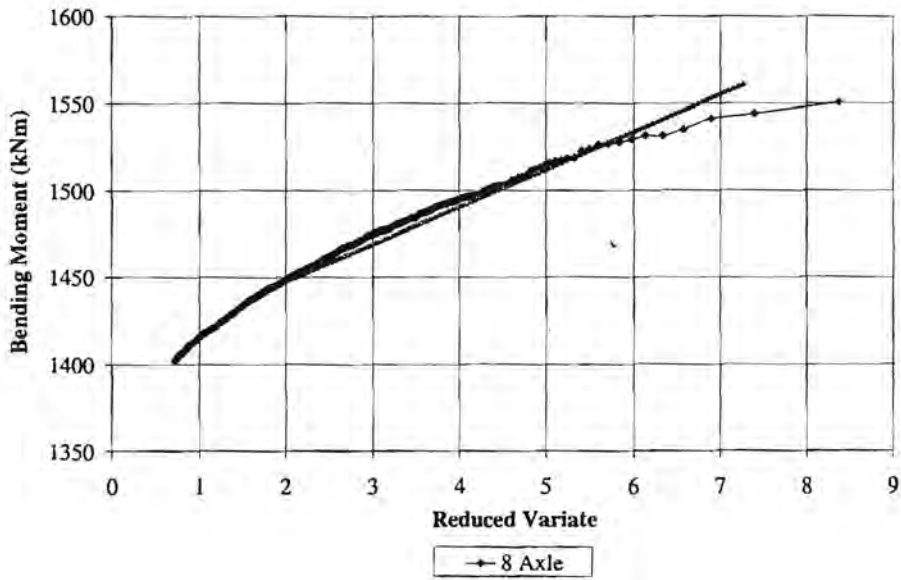


Figure B54: 20m span

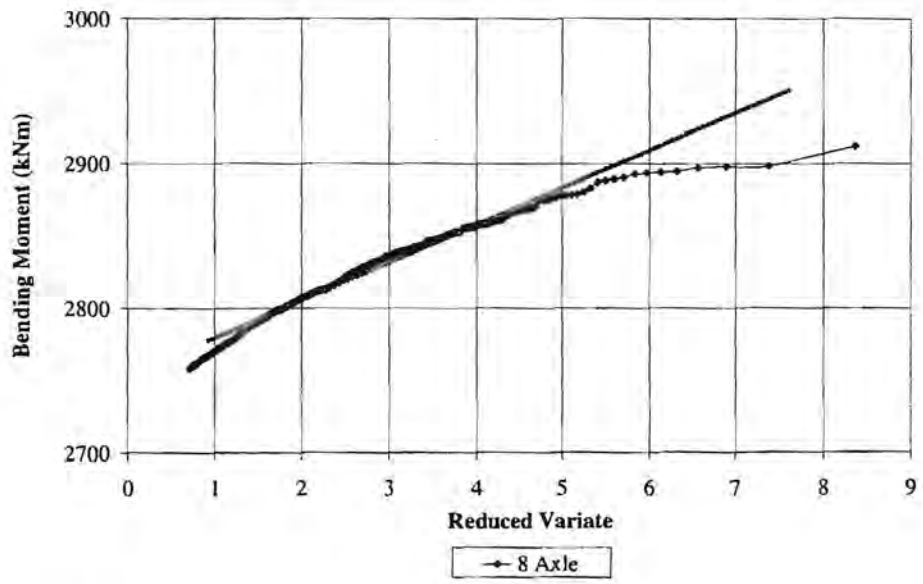


Figure B55: 30m span

## Frequency Distribution of Shear Forces of Legal Vehicles

### 8 Axle Vehicles

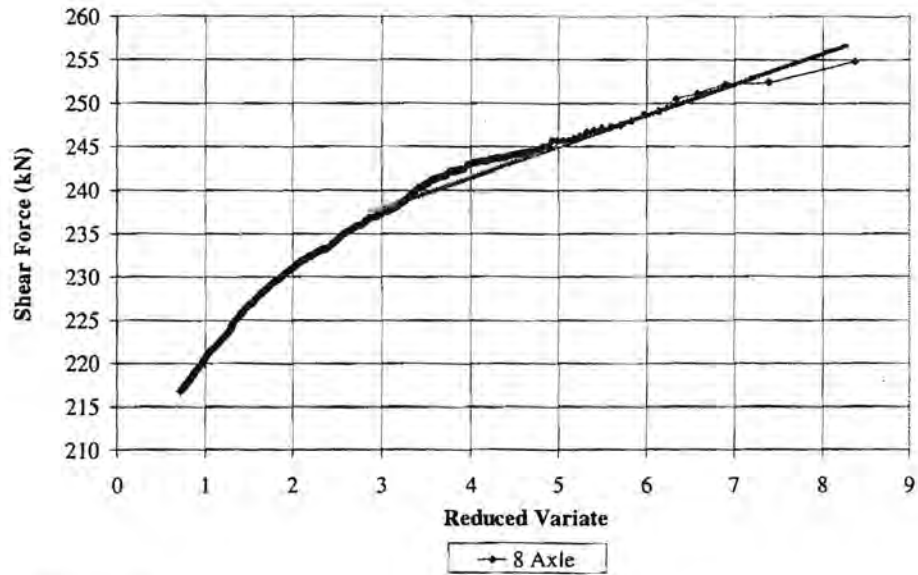


Figure B56: 5m span

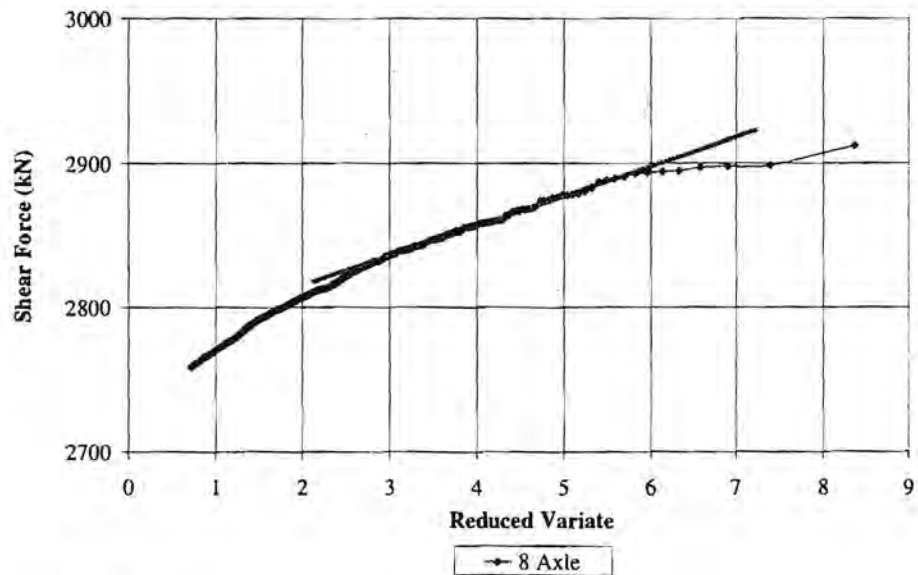


Figure B57: 10m span

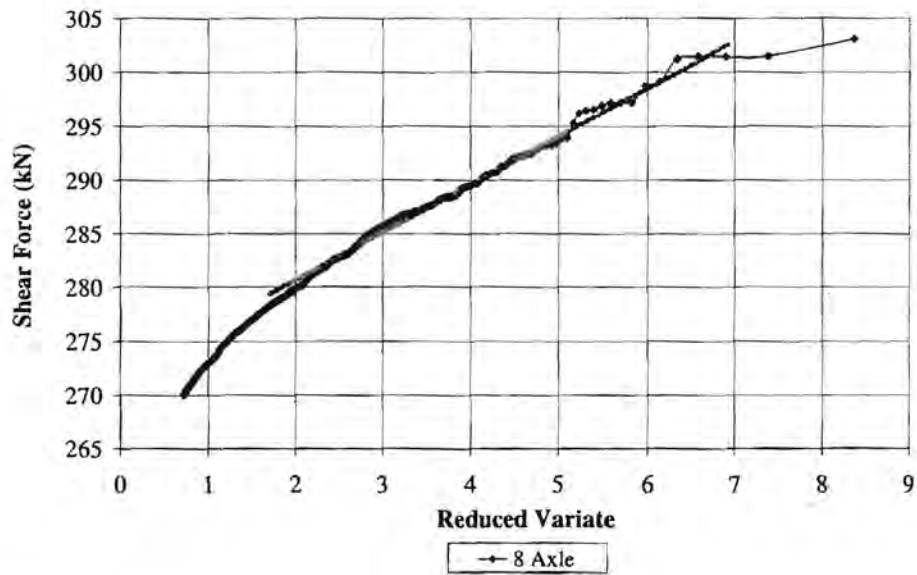


Figure B58: 15m span

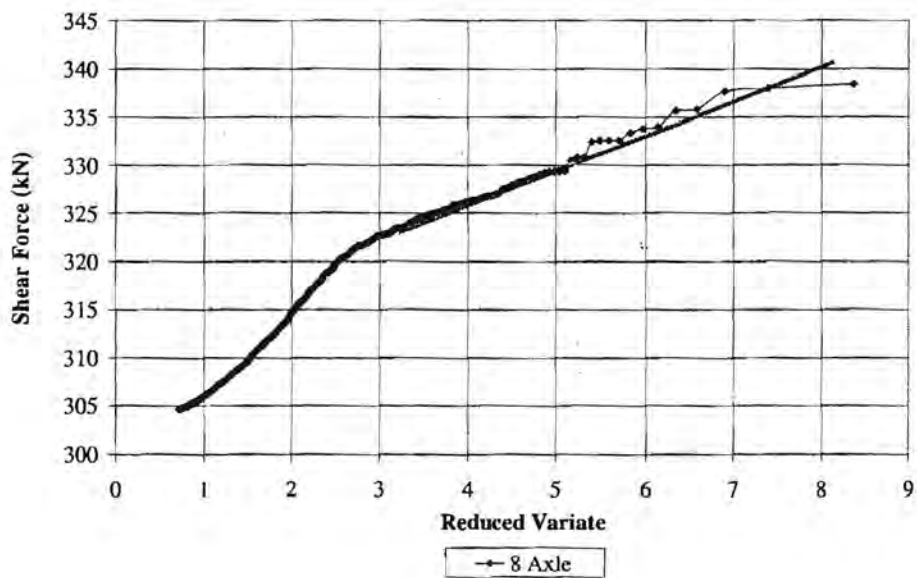


Figure B59: 20m span

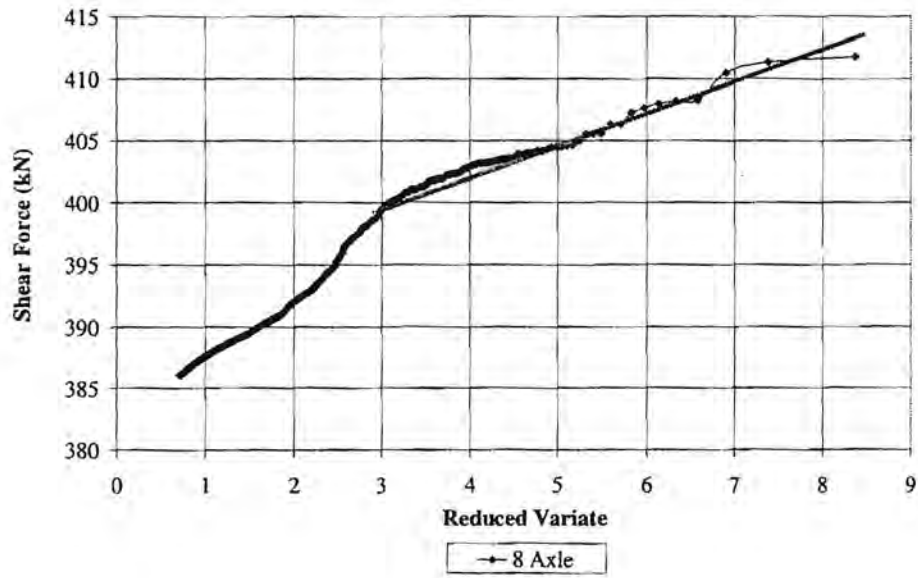


Figure B60: 30m span

### Distribution Graphs of Bending Moments of Actual Vehicles

#### 6 Axle Vehicles

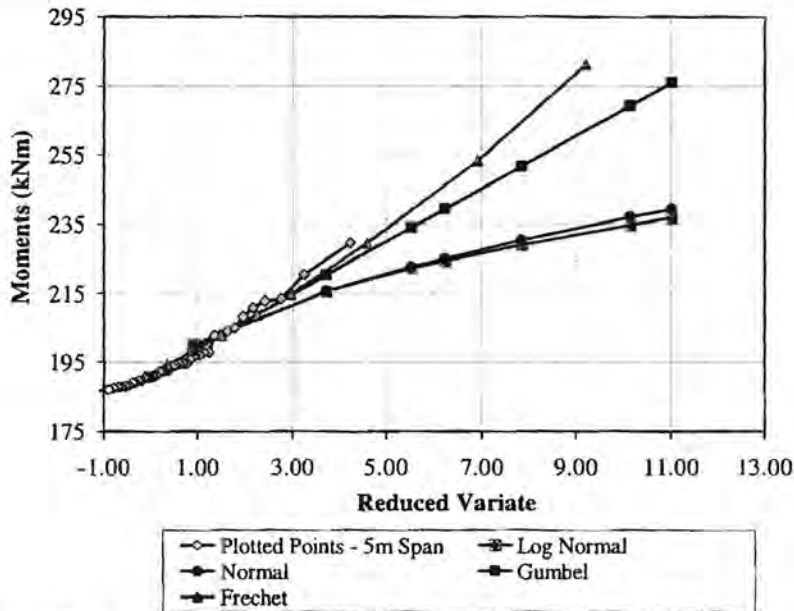


Figure B61: 5m span

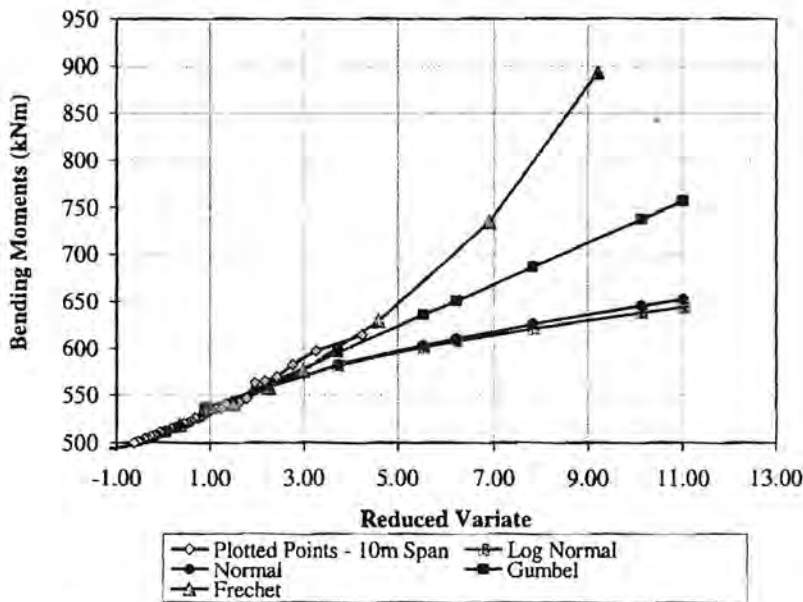


Figure B62: 10m span

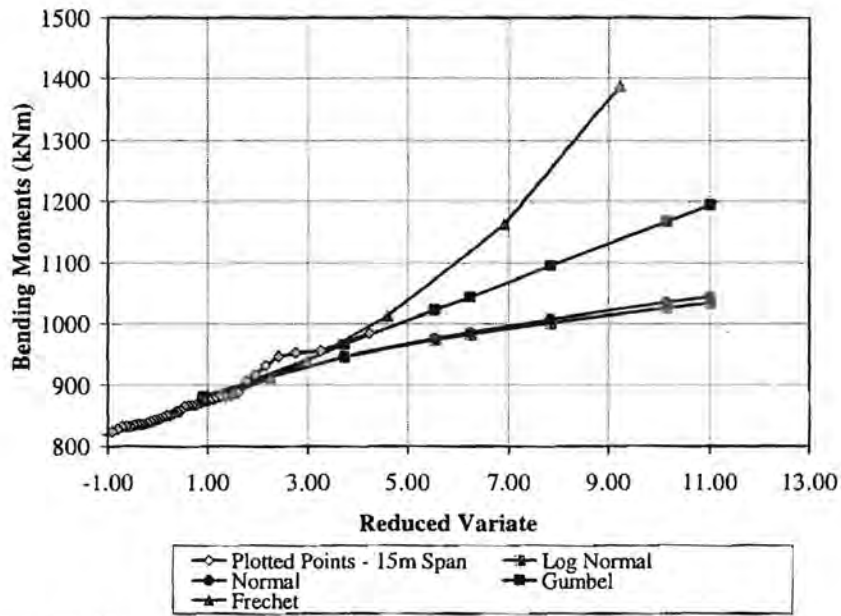


Figure B63: 15m span

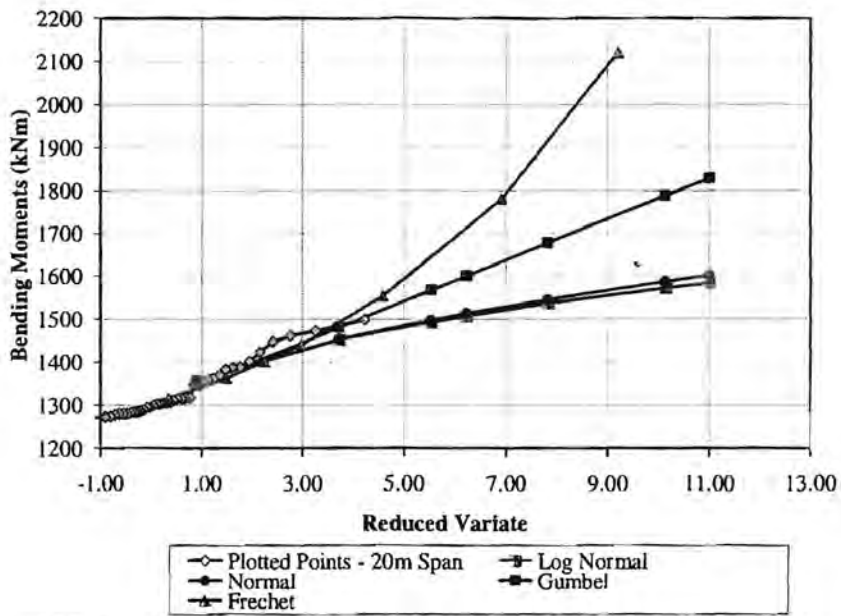


Figure B64: 20m span

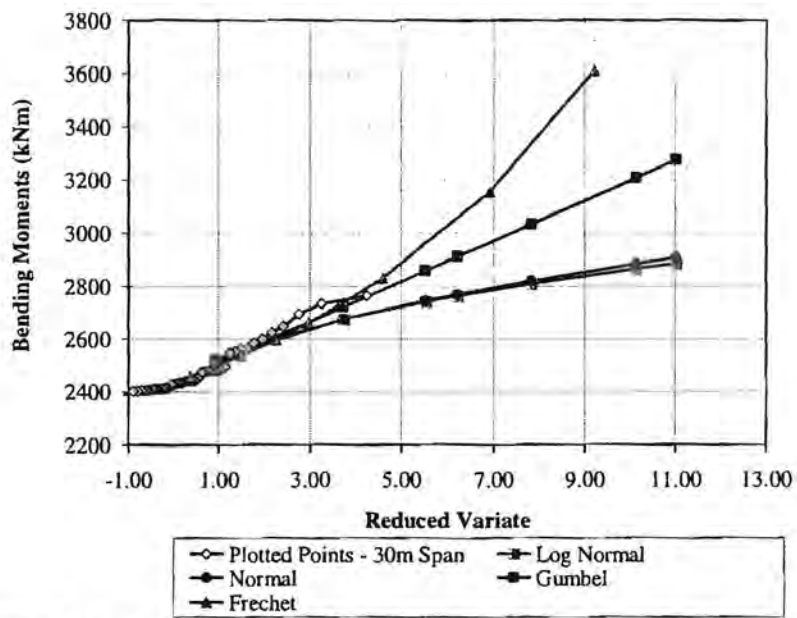


Figure B65: 30m span

### Distribution Graphs of Shear Forces of Actual Vehicles

#### 6 Axle Vehicles

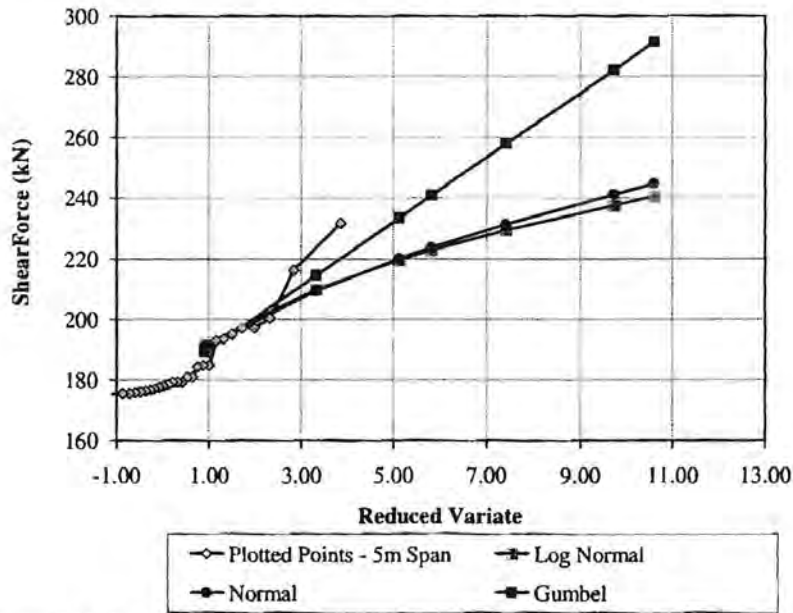


Figure B66: 5m span

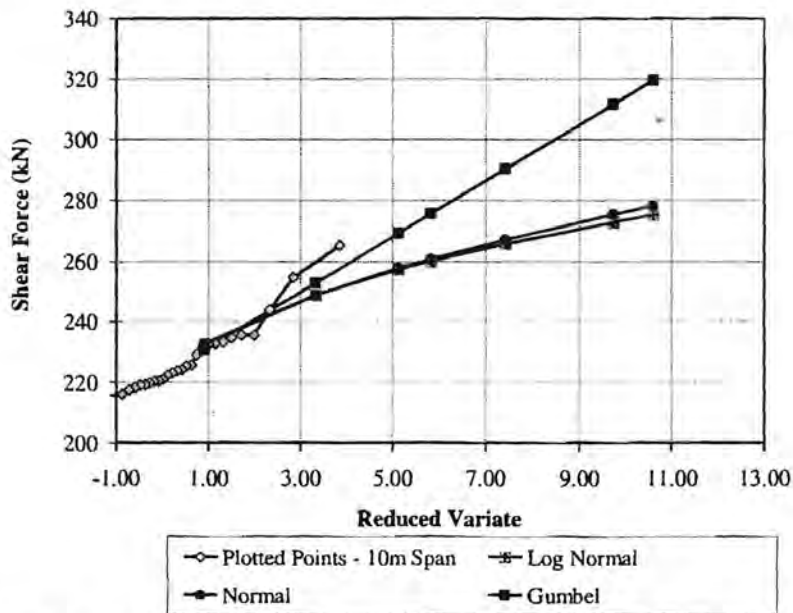


Figure B67: 10m span

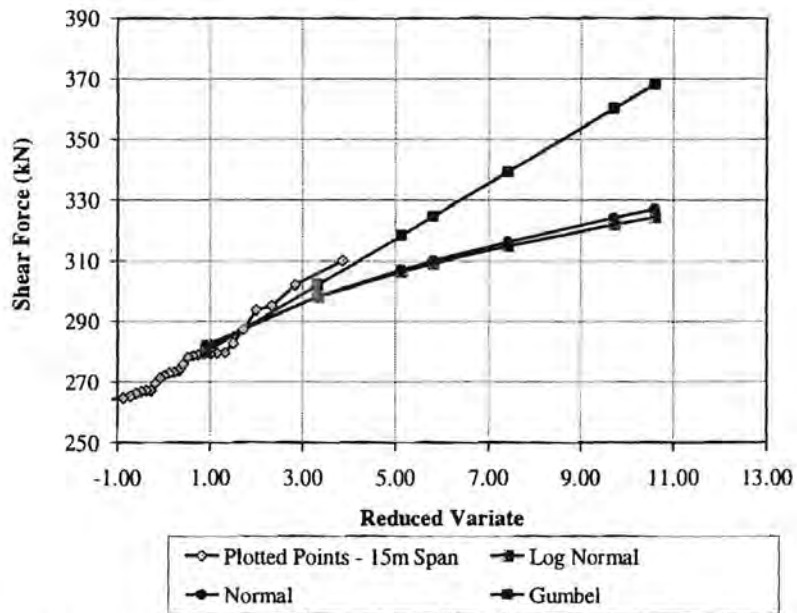


Figure B68: 15m span

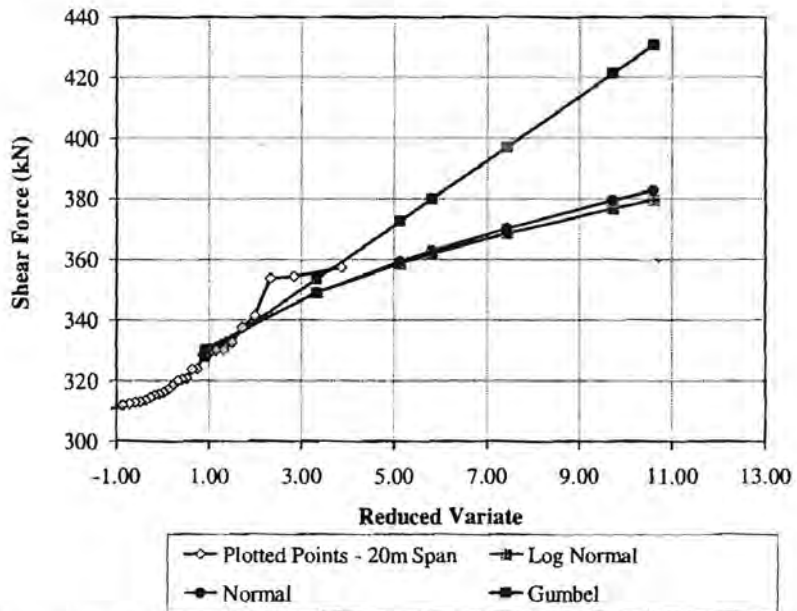


Figure B69: 20m span

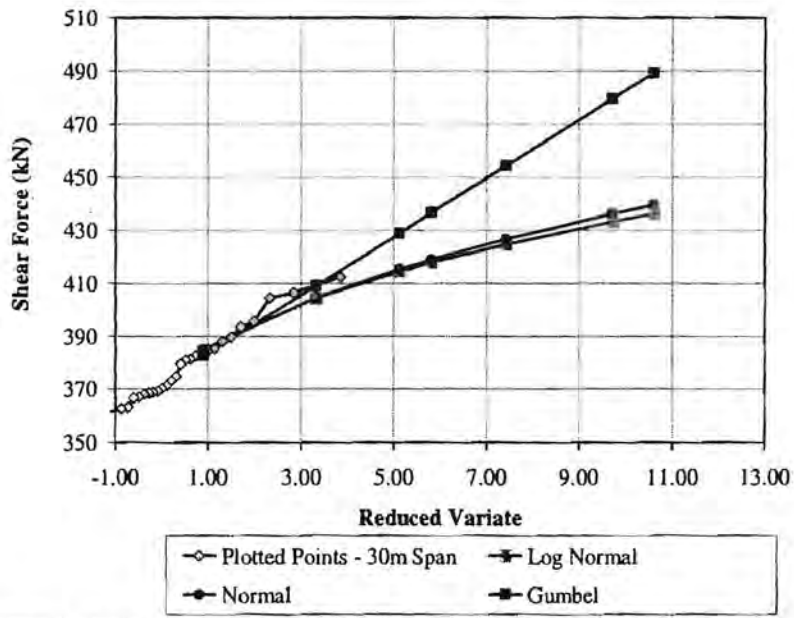


Figure B70: 30m span

## Distribution Graphs of Bending Moments of Legal Vehicles

### 6 Axle Vehicles

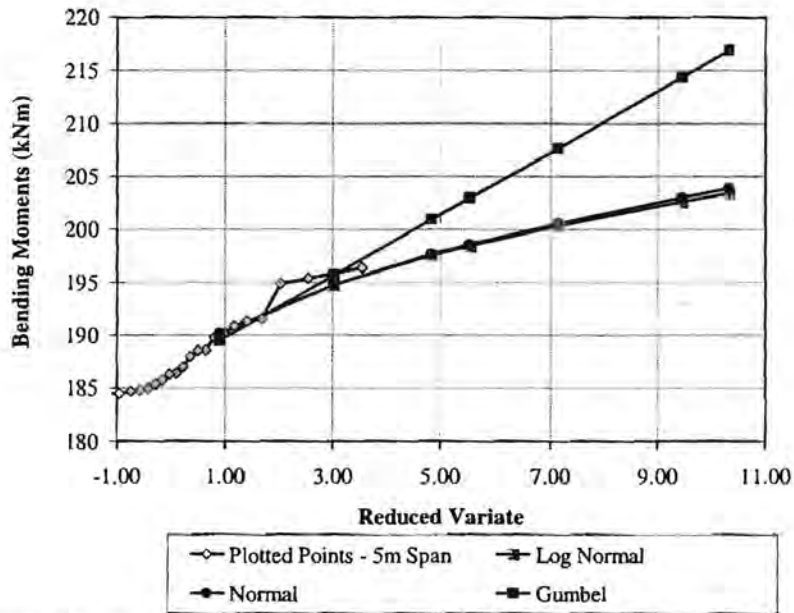


Figure B71: 5m span

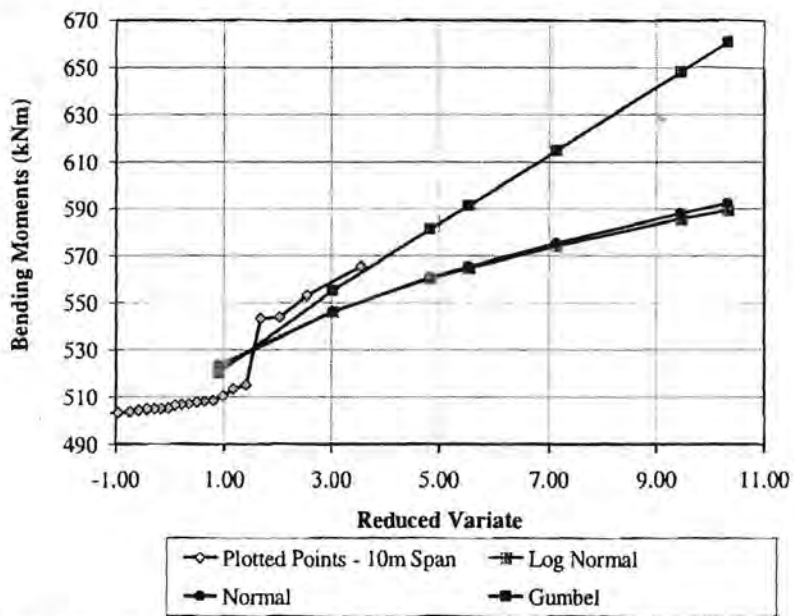


Figure B72: 10m span

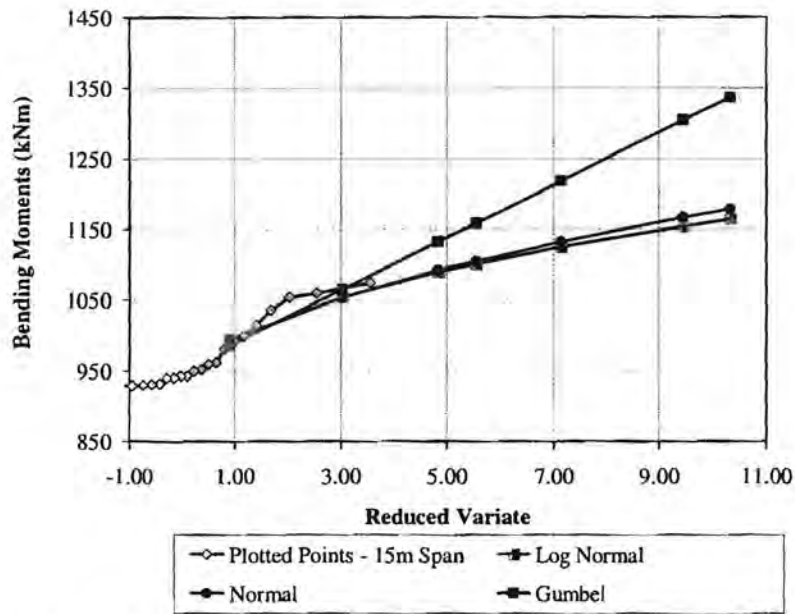


Figure B73: 15m span

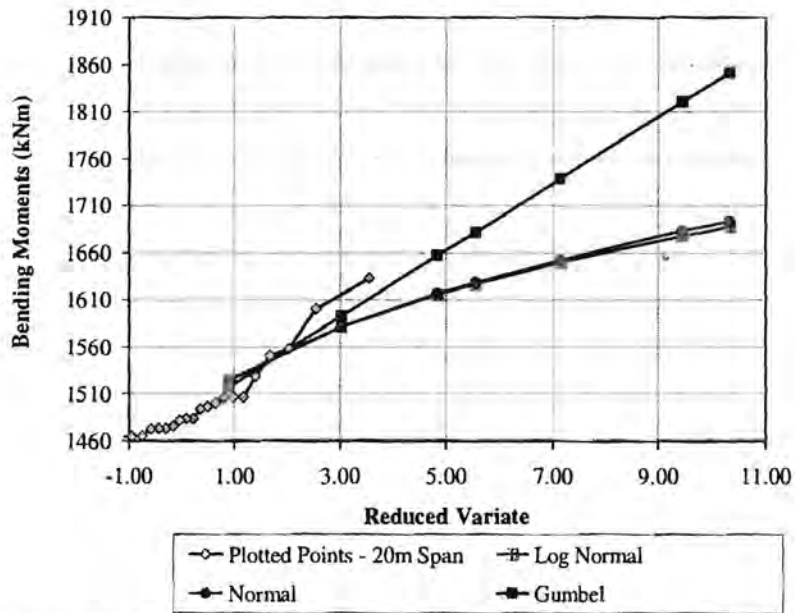


Figure B74: 20m span

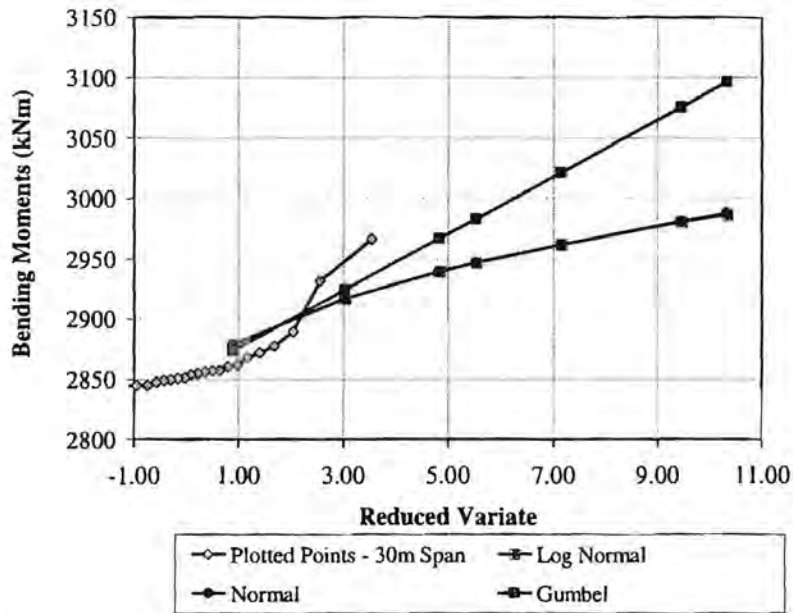


Figure B75: 30m span

### Distribution Graphs of Shear Forces of Legal Vehicles

#### 6 Axle Vehicles

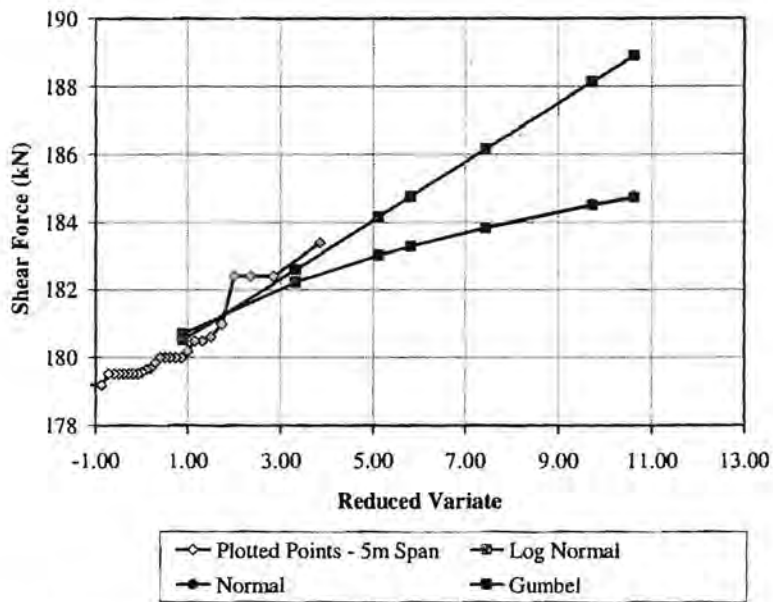


Figure B76: 5m span

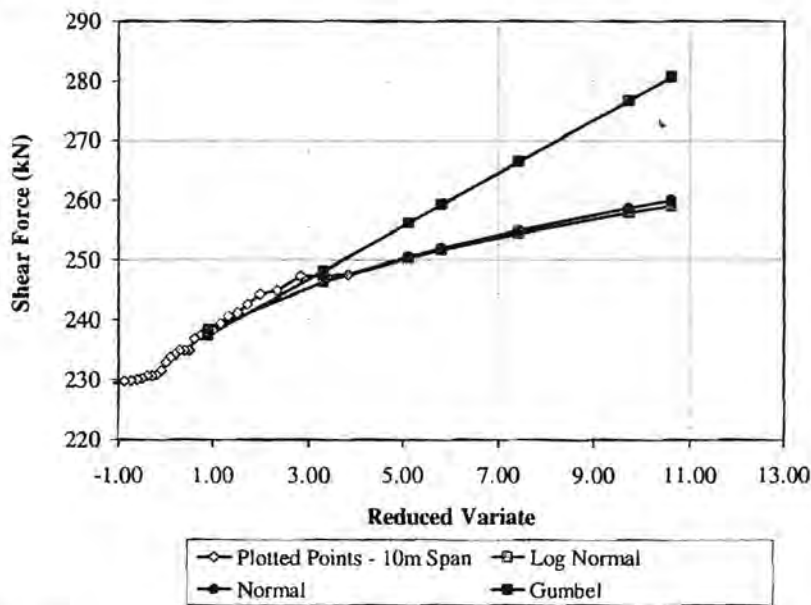


Figure B77: 10m span

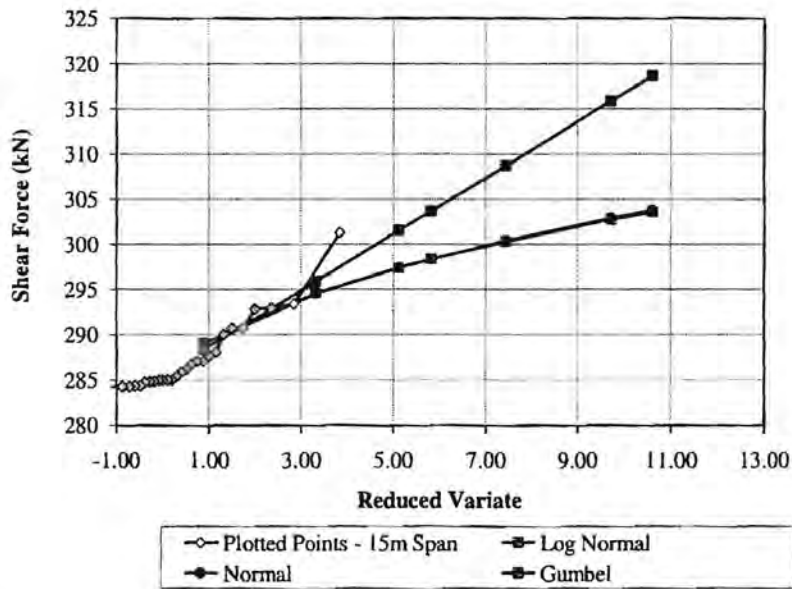


Figure B78: 15m span

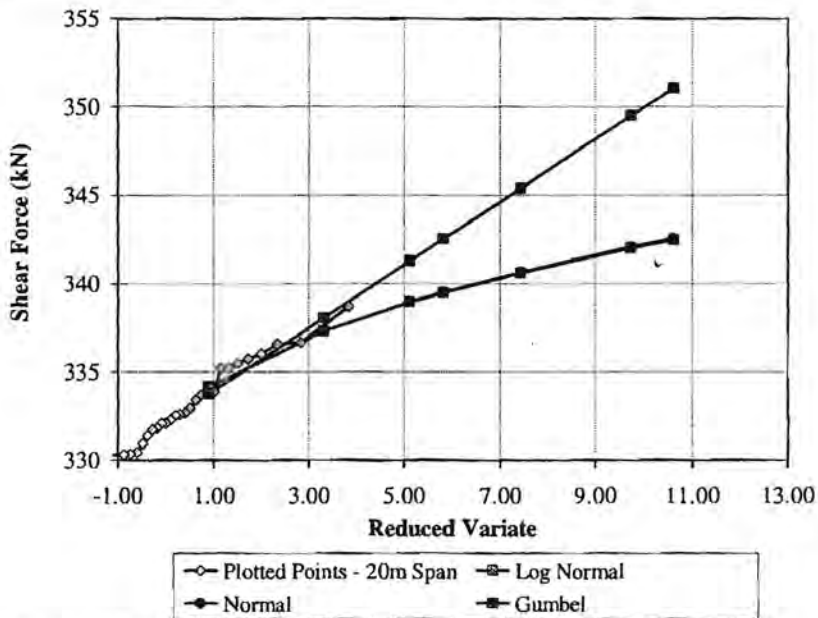


Figure B79: 20m span

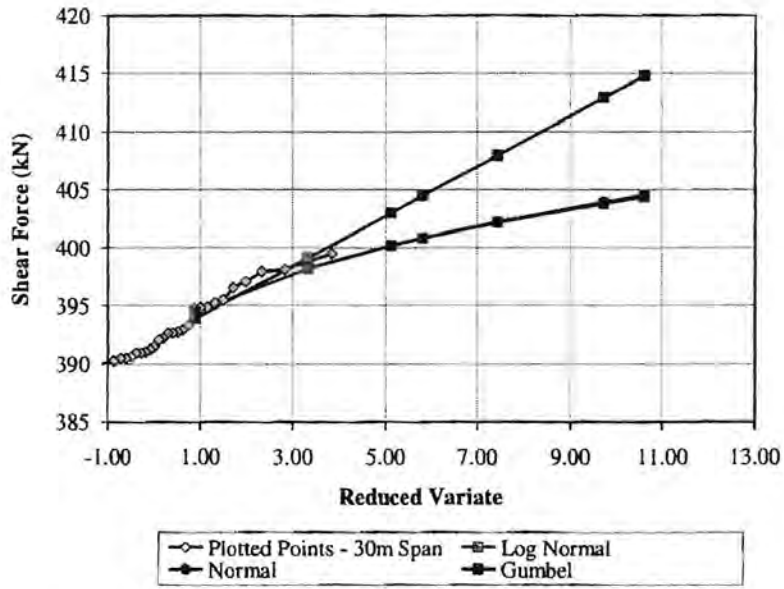


Figure B80: 30m span

### Distribution Graphs of Bending Moments of Actual Vehicles

#### 7 Axle Vehicles

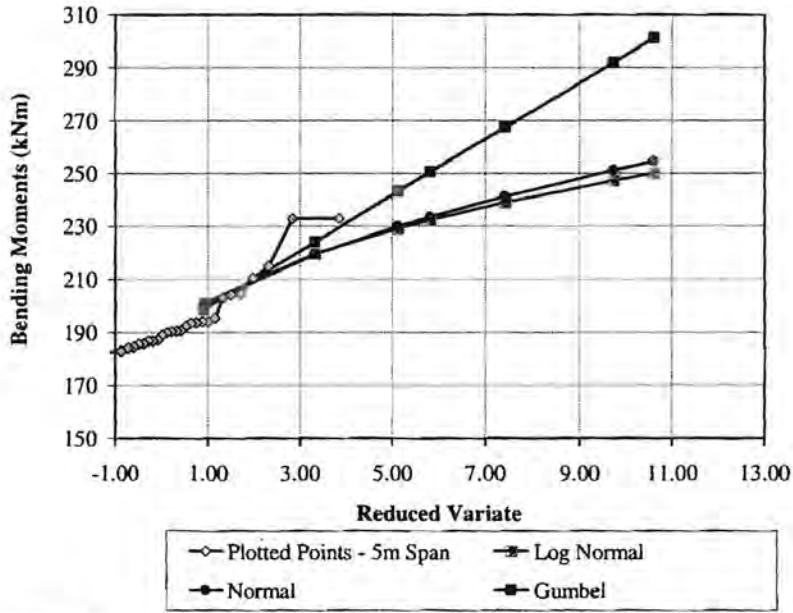


Figure B81: 5m span

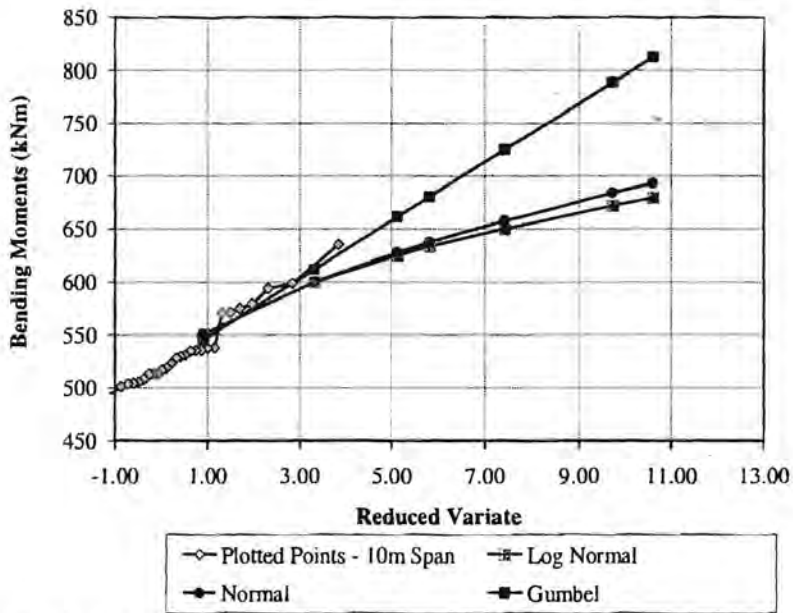


Figure B82: 10m span

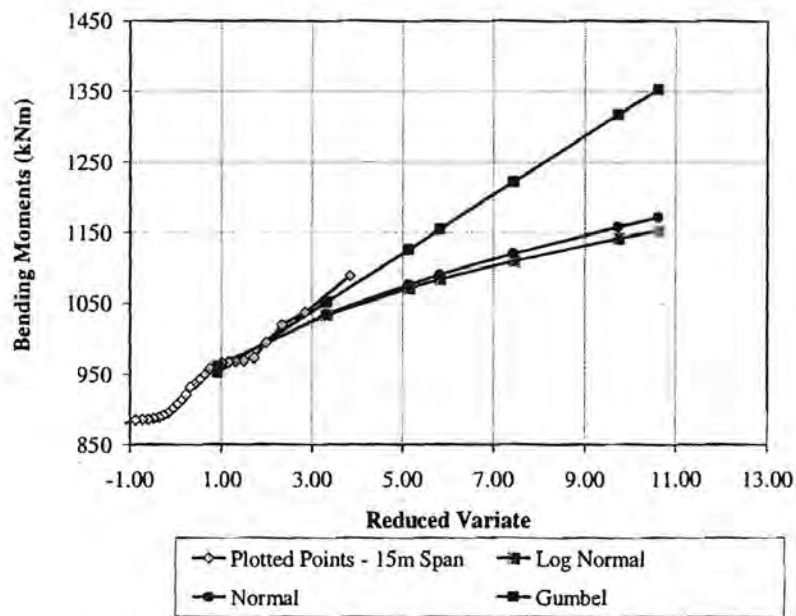


Figure B83: 15m span

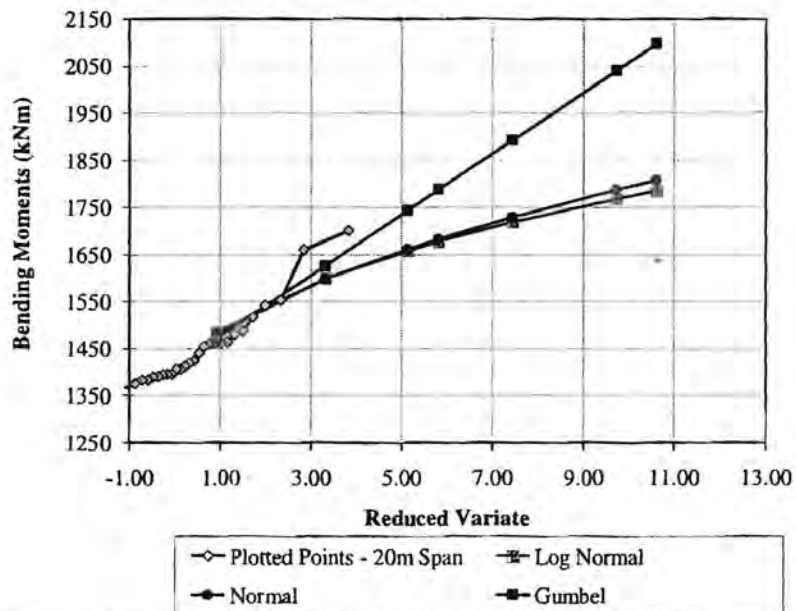


Figure B84: 20m span

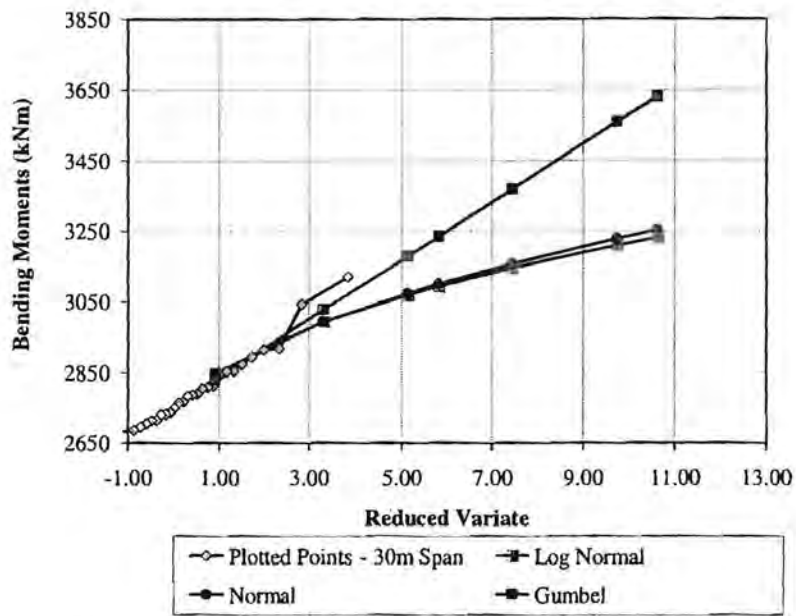


Figure B85: 30m span

### Distribution Graphs of Shear Forces of Actual Vehicles

#### 7 Axle Vehicles

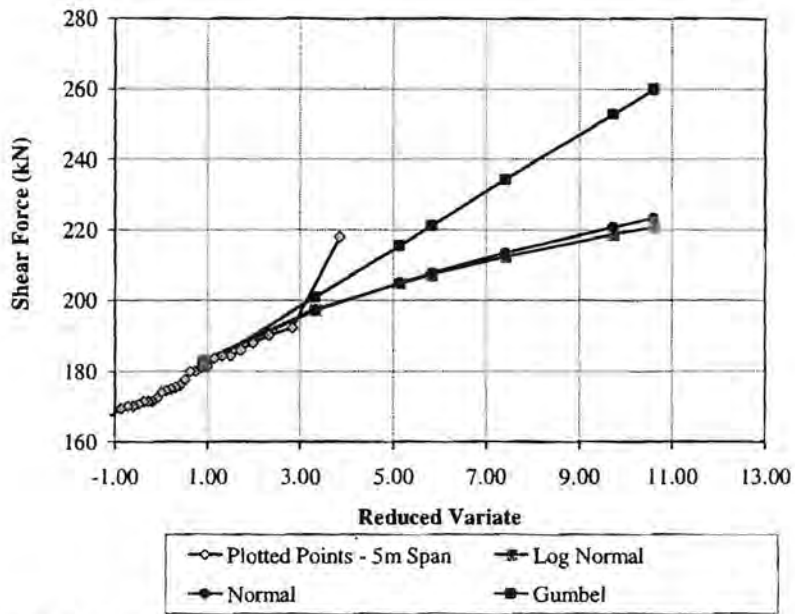


Figure B86: 5m span

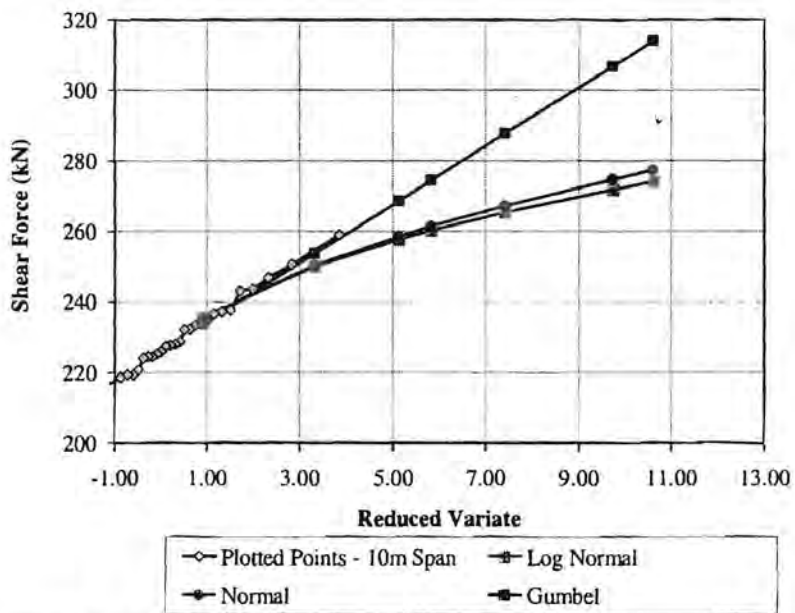


Figure B87: 10m span

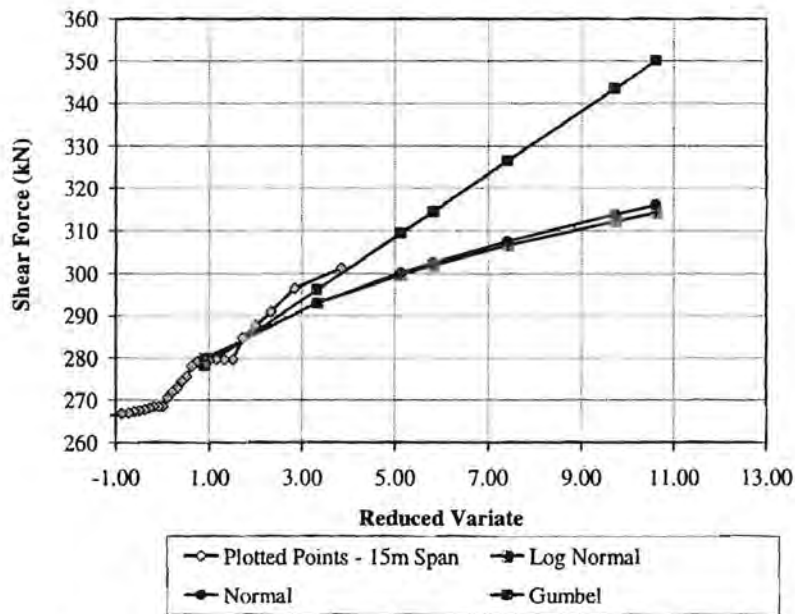


Figure B88: 15m span

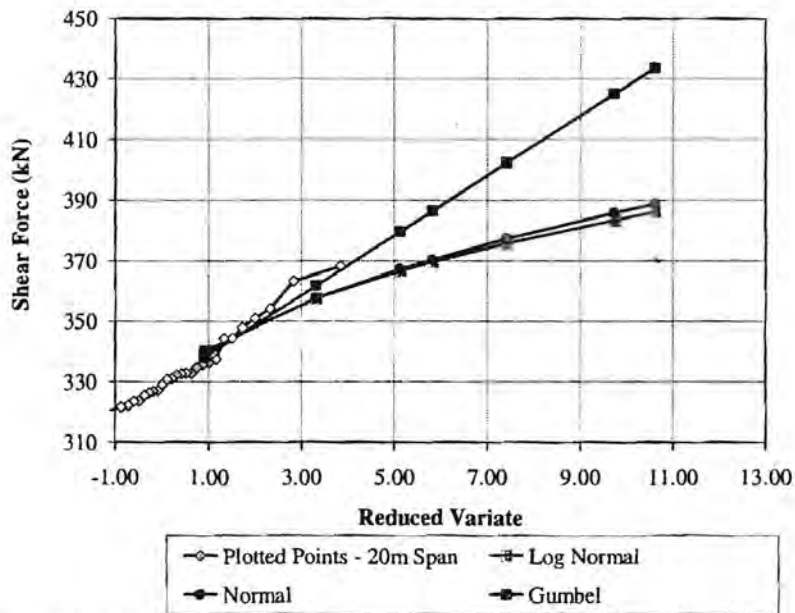


Figure B89: 20m span

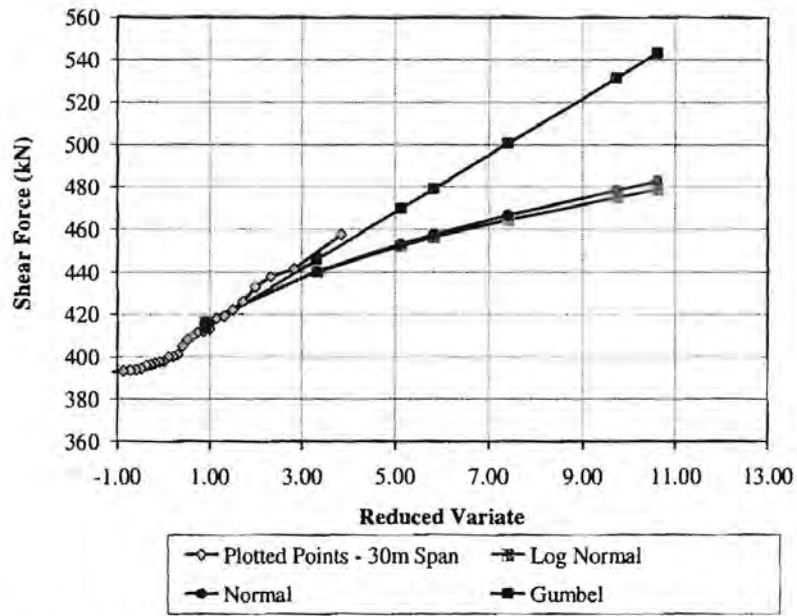


Figure B90: 30m span

## Distribution Graphs of Bending Moments of Legal Vehicles

### 7 Axle Vehicles

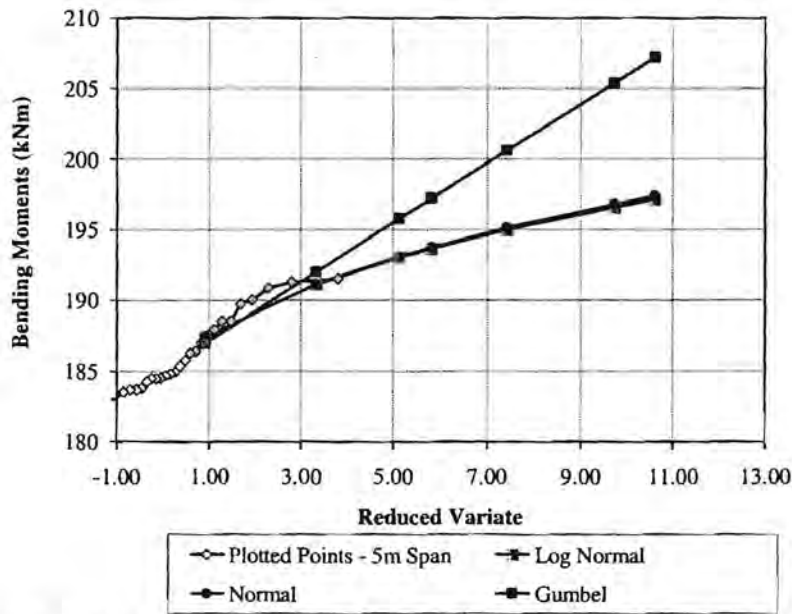


Figure B91: 5m span

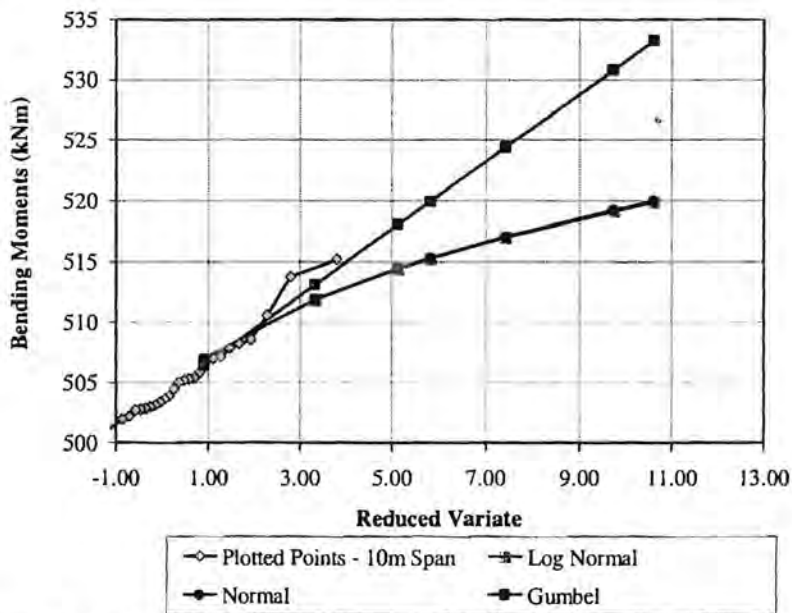


Figure B92: 10m span

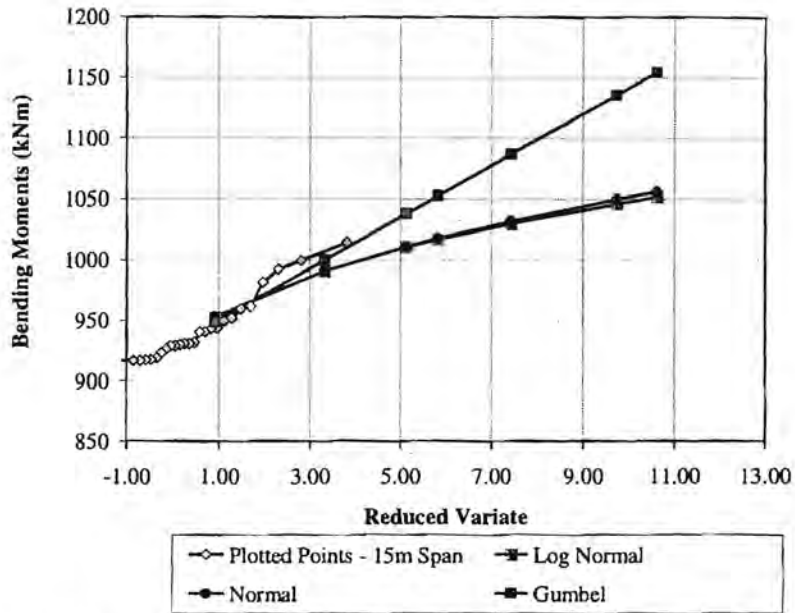


Figure B93: 15m span

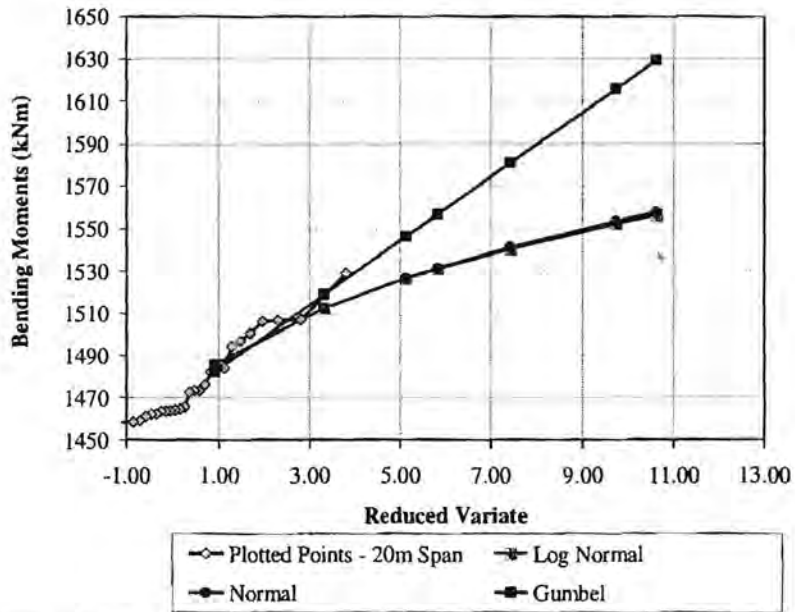


Figure B94: 20m span

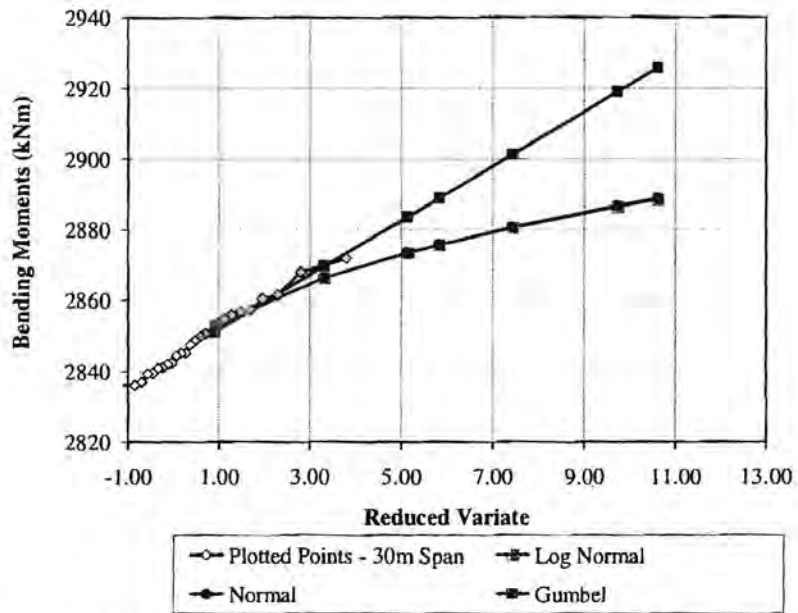


Figure B95: 30m span

## Distribution Graphs of Shear Forces of Legal Vehicles

### 7 Axle Vehicles

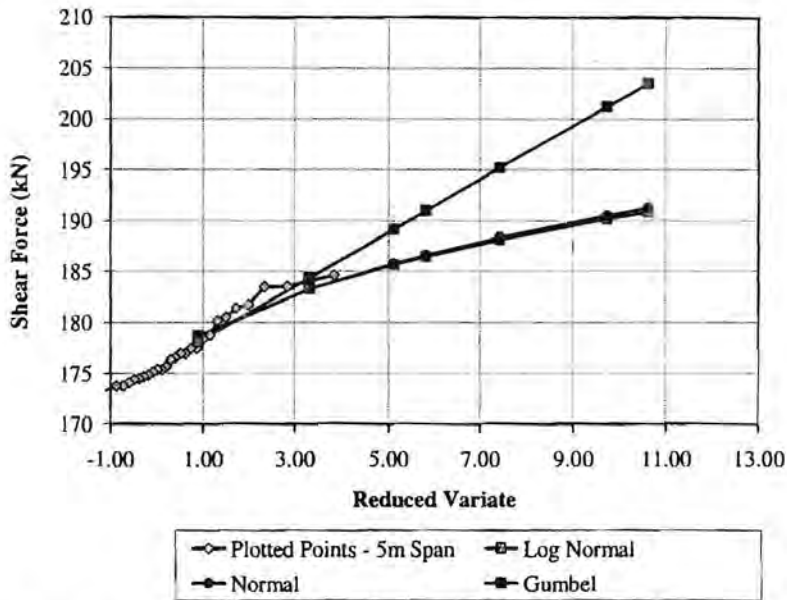


Figure B96: 5m span

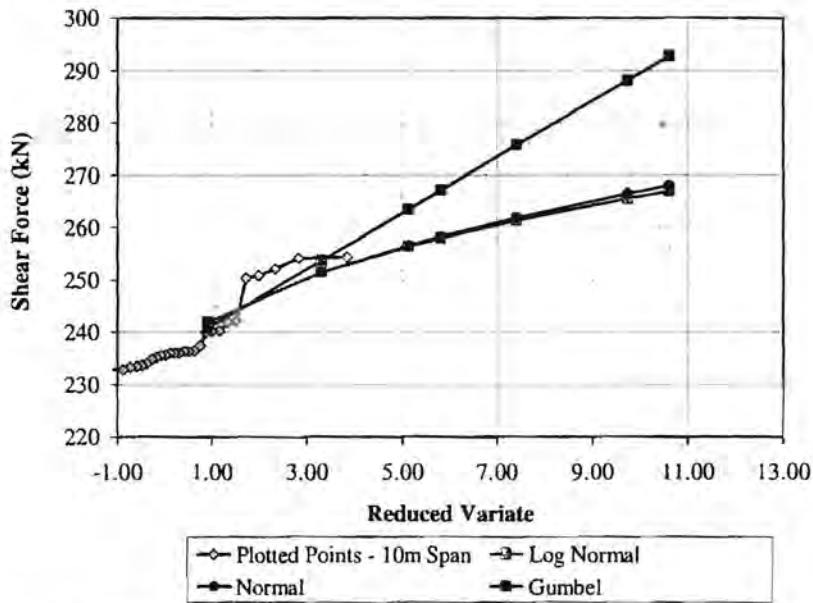


Figure B97: 10m span

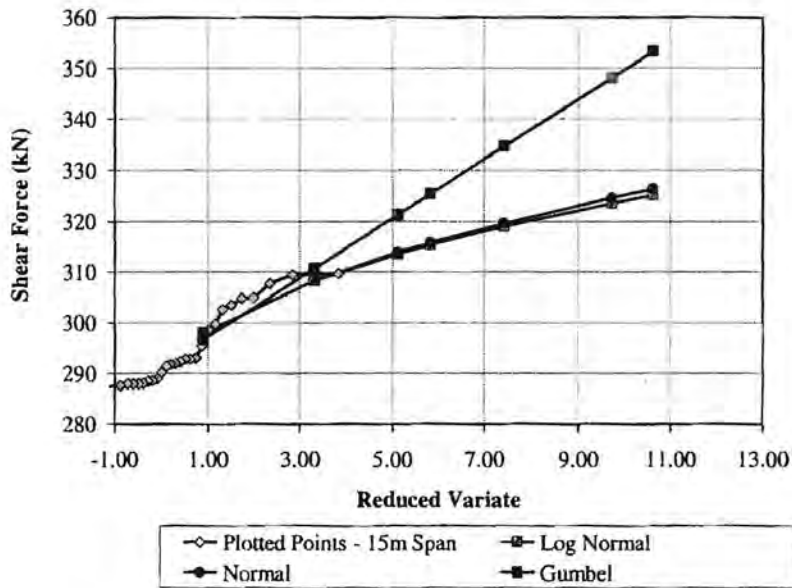


Figure B98: 15m span

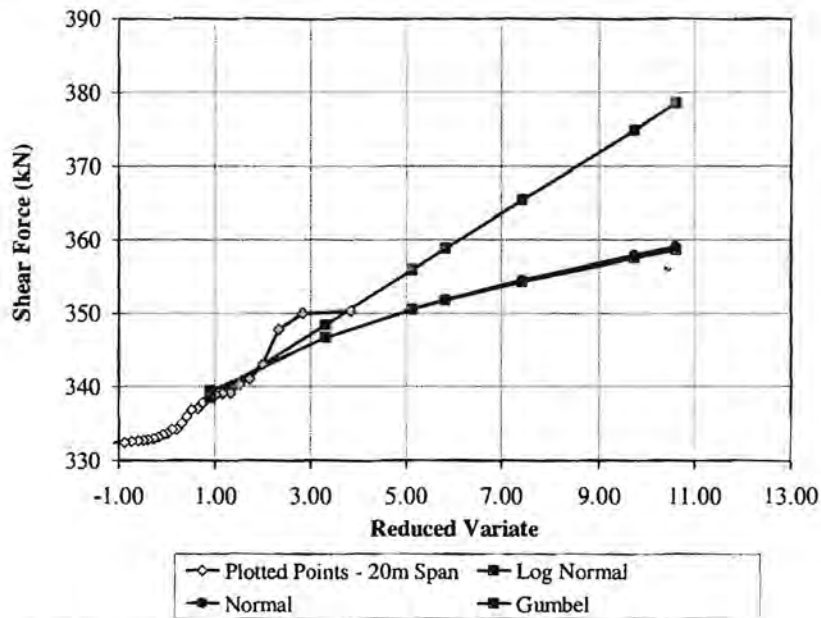


Figure B99: 20m span

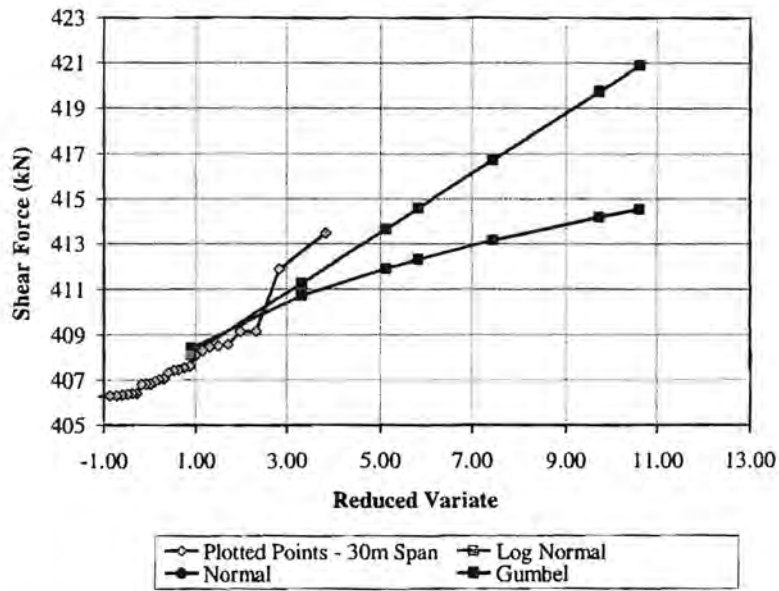


Figure B100: 30m span

## Distribution Graphs of Bending Moments of Actual Vehicles

### 8 Axle Vehicles

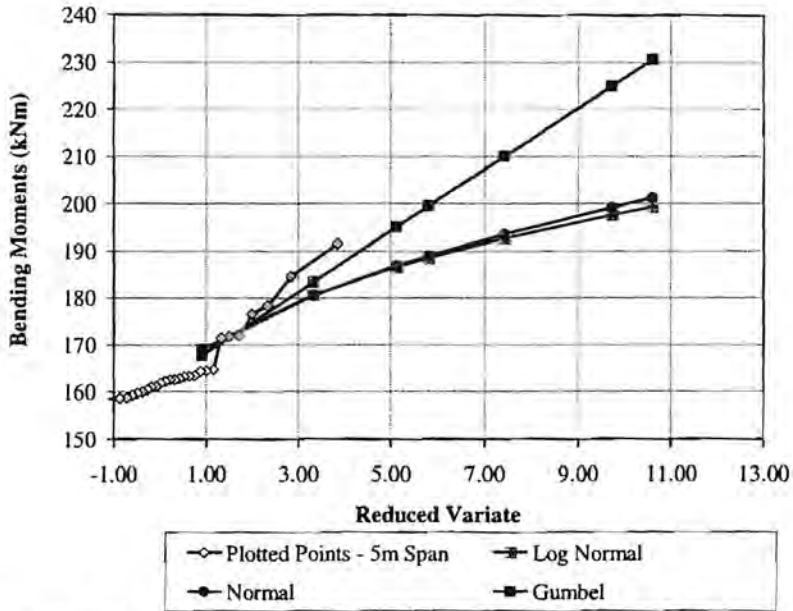


Figure B101: 5m span

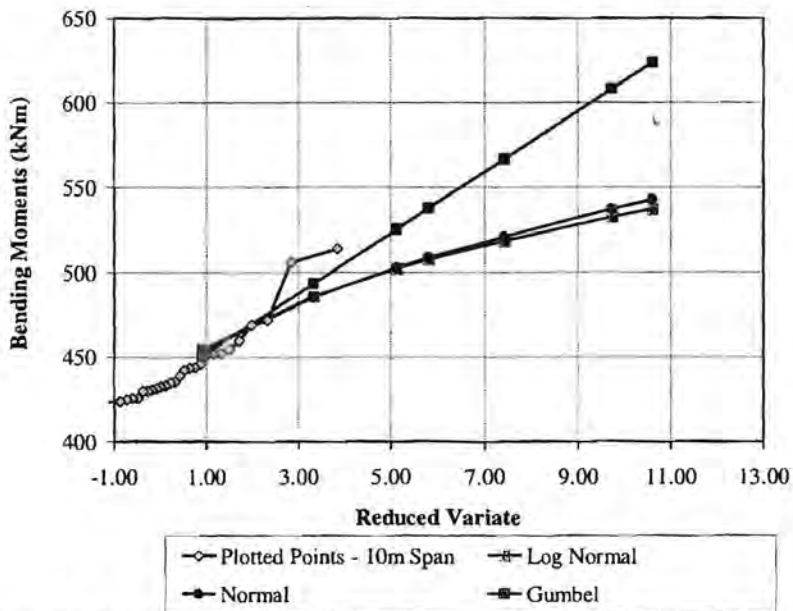


Figure B102: 10m span

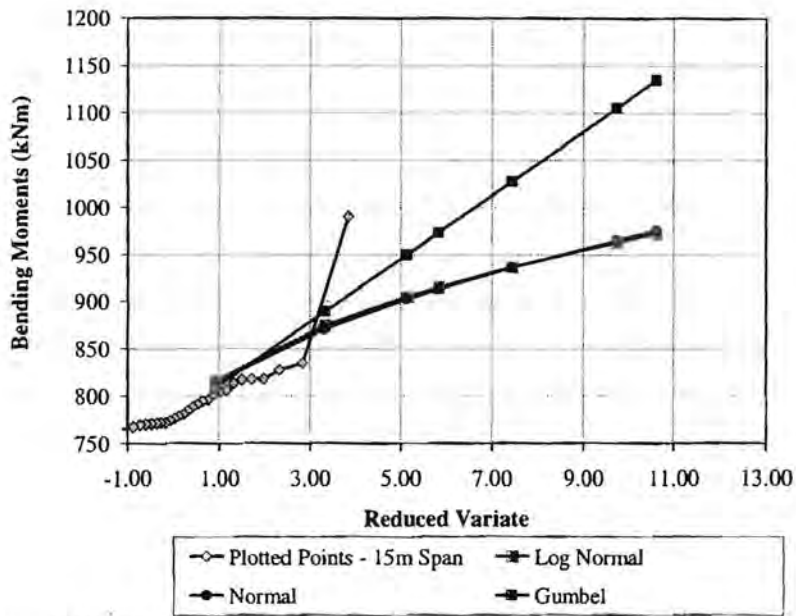


Figure B103: 15m span

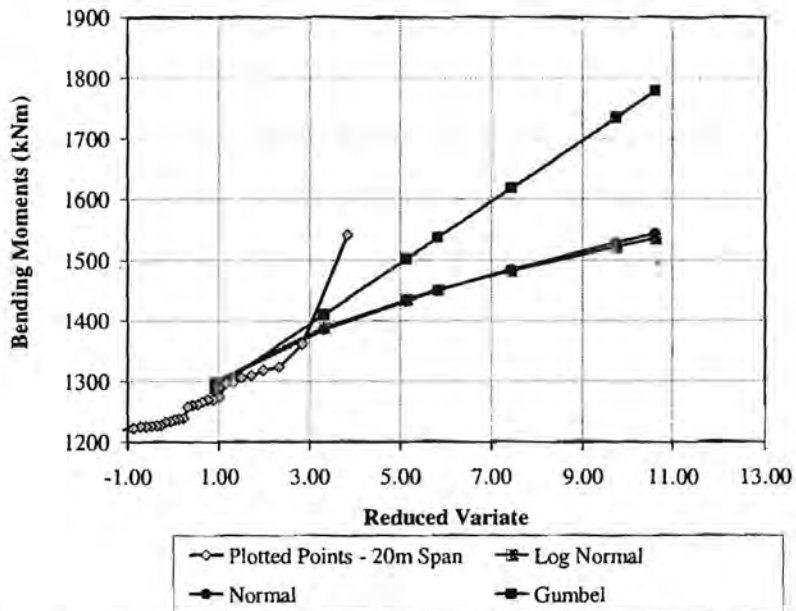


Figure B104: 20m span

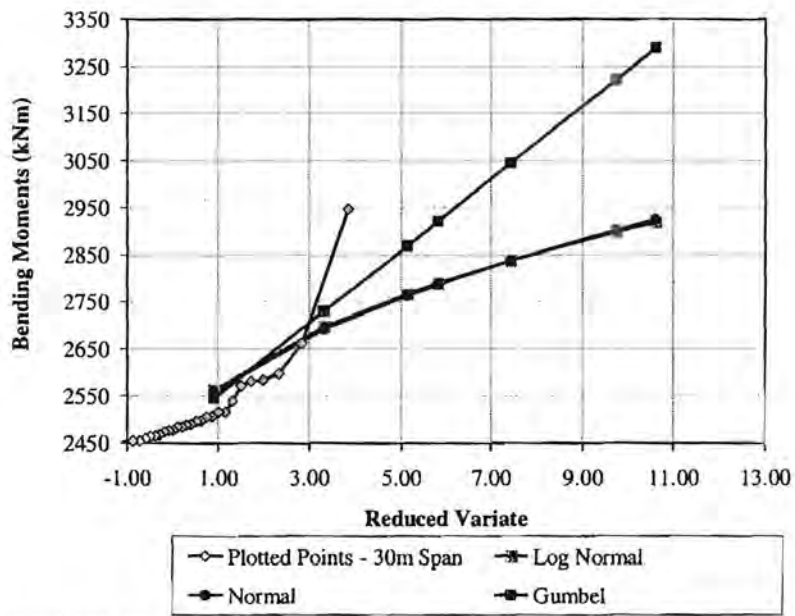


Figure B105: 30m span

### Distribution Graphs of Shear Forces of Actual Vehicles

#### 8 Axle Vehicles

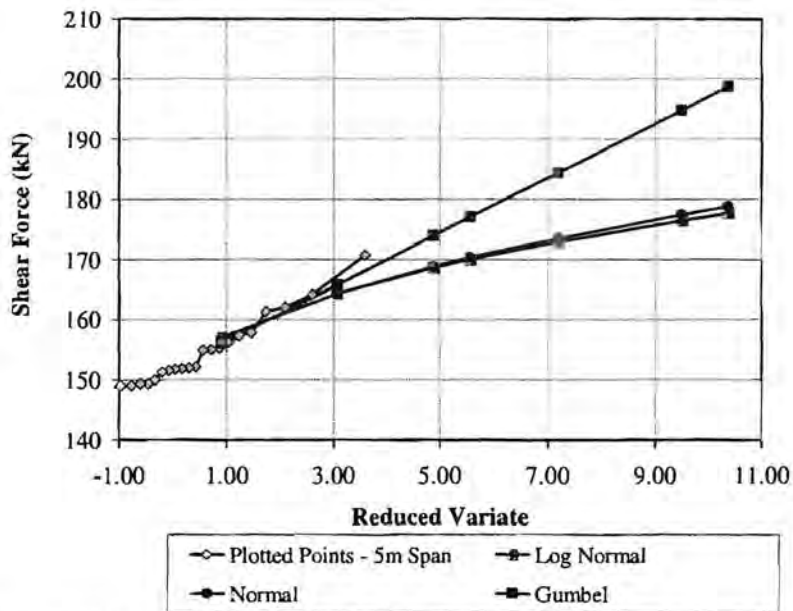


Figure B106: 5m span

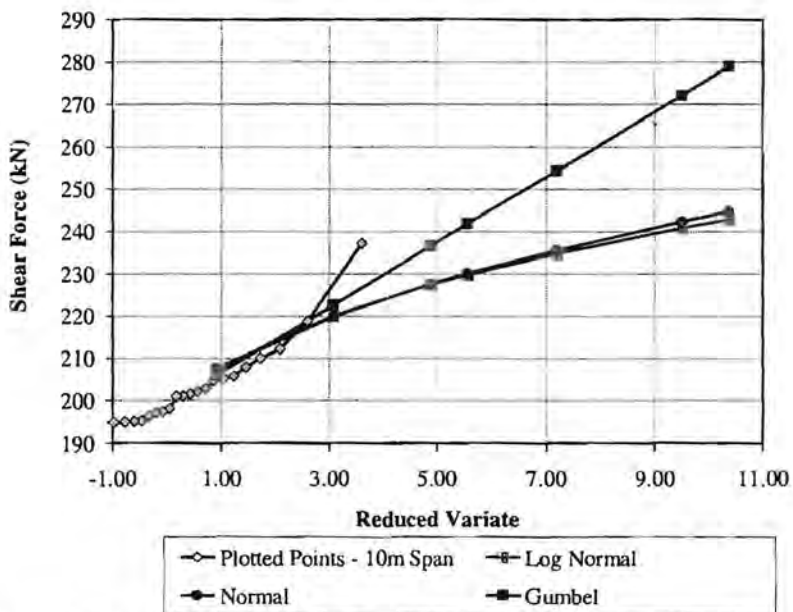


Figure B107: 10m span

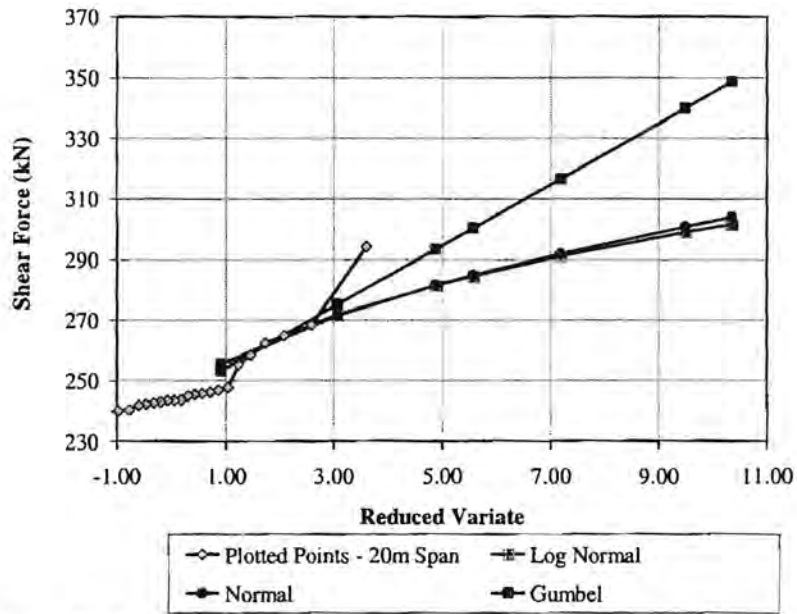


Figure B108: 15m span

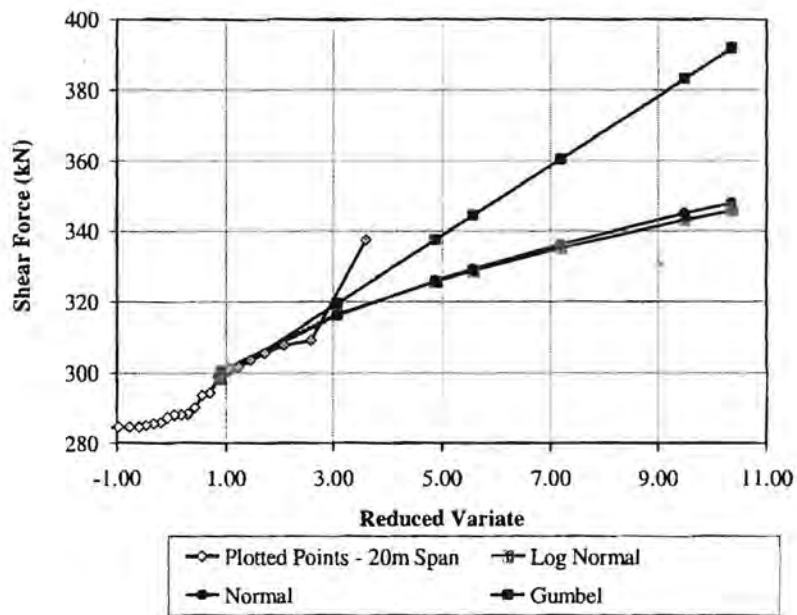


Figure B109: 20m span

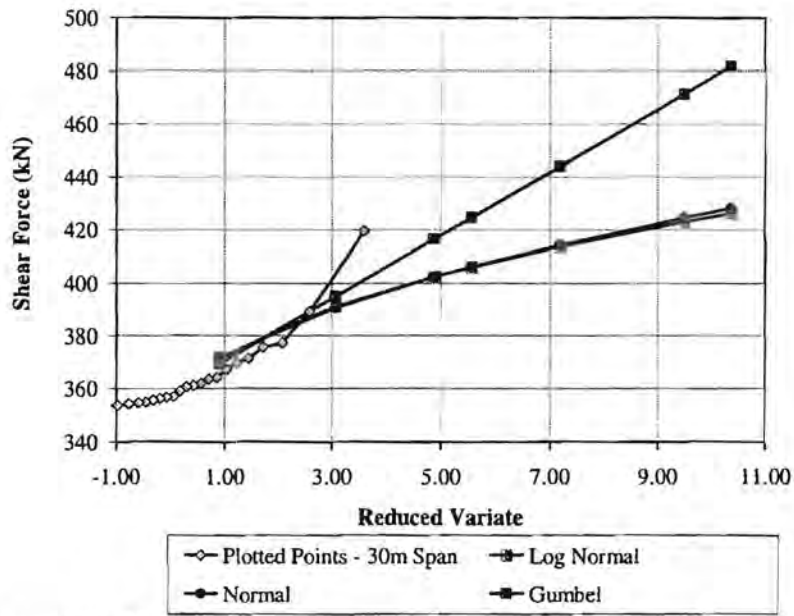


Figure B110: 30m span

### Distribution Graphs of Bending Moments of Legal Vehicles

#### 8 Axle Vehicles

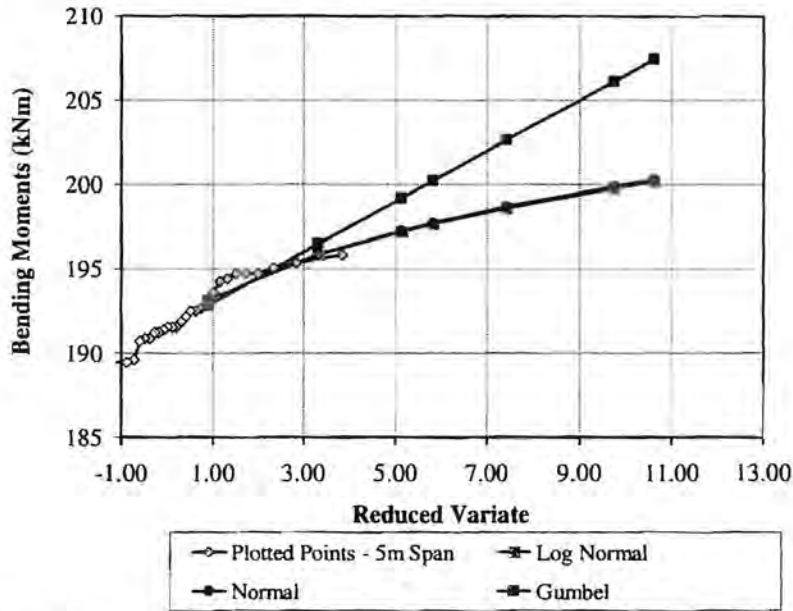


Figure B111: 5m span

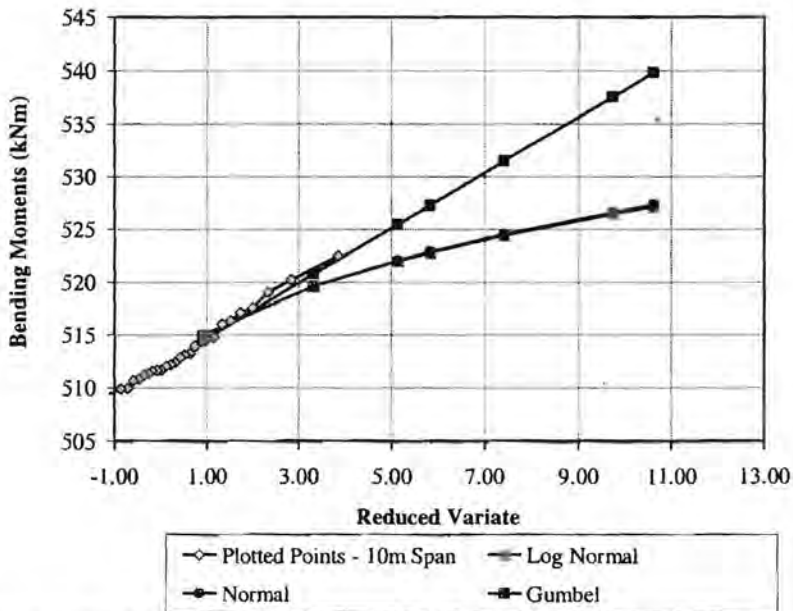


Figure B112: 10m span

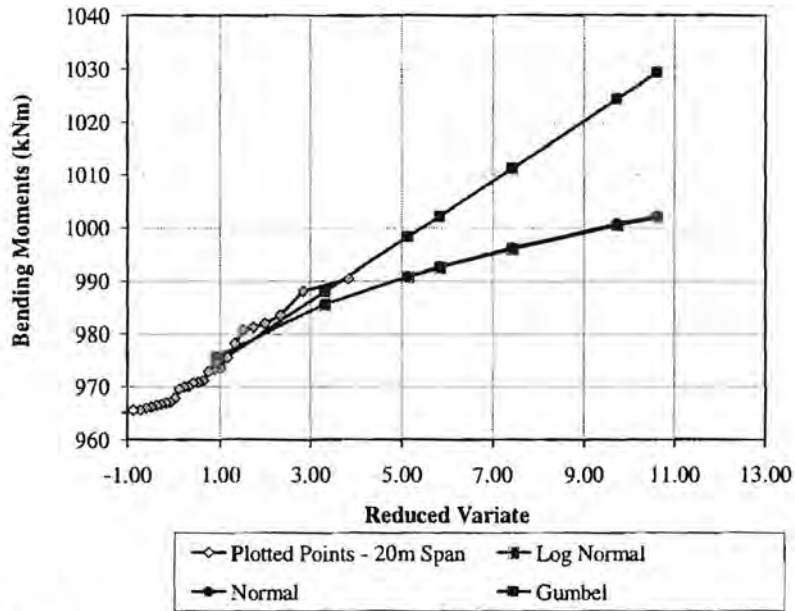


Figure B113: 15m span

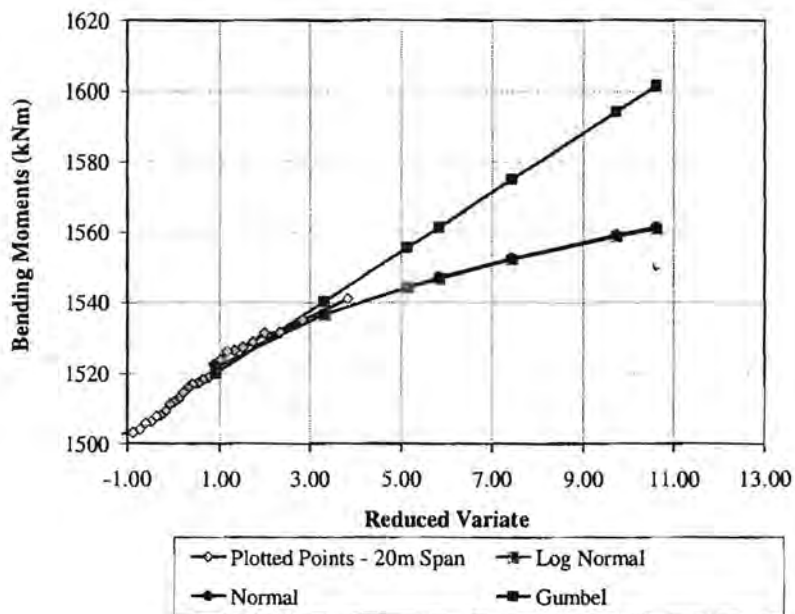


Figure B114: 20m span

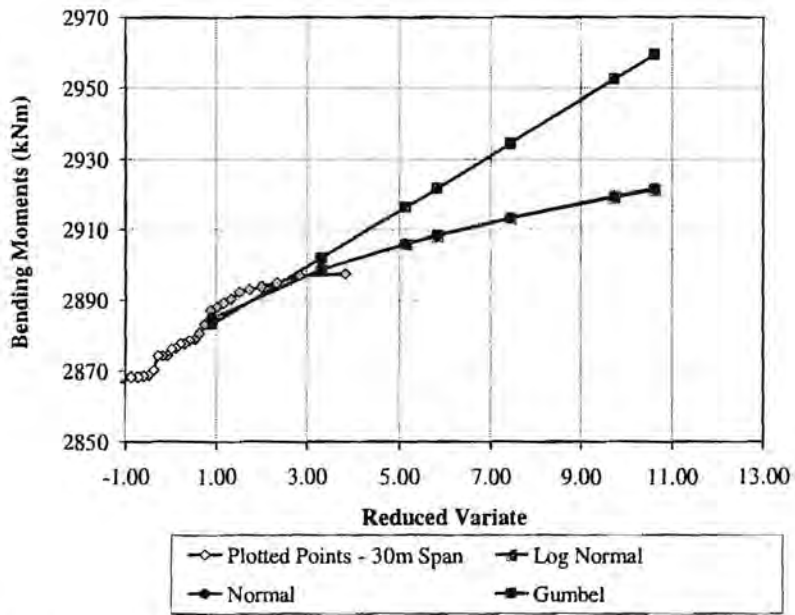


Figure B115: 30m span

### Distribution Graphs of Shear Forces of Legal Vehicles

#### 8 Axle Vehicles

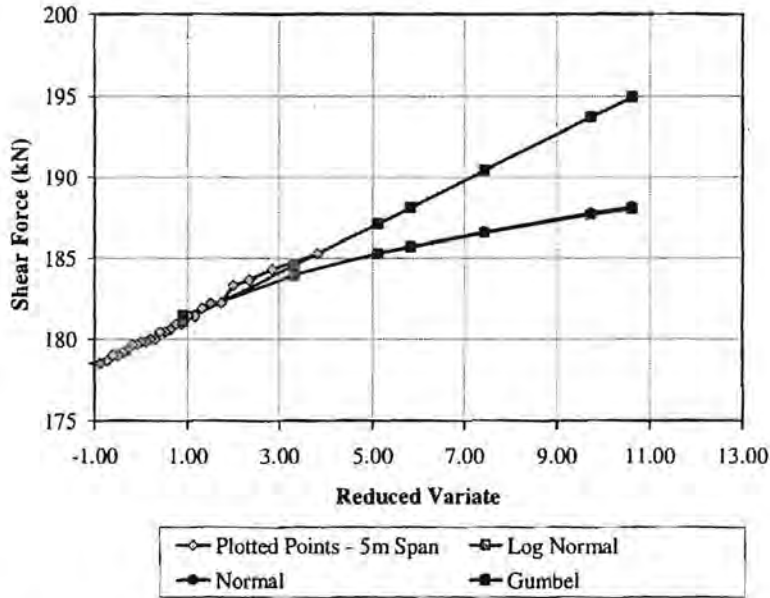


Figure B116: 5m span

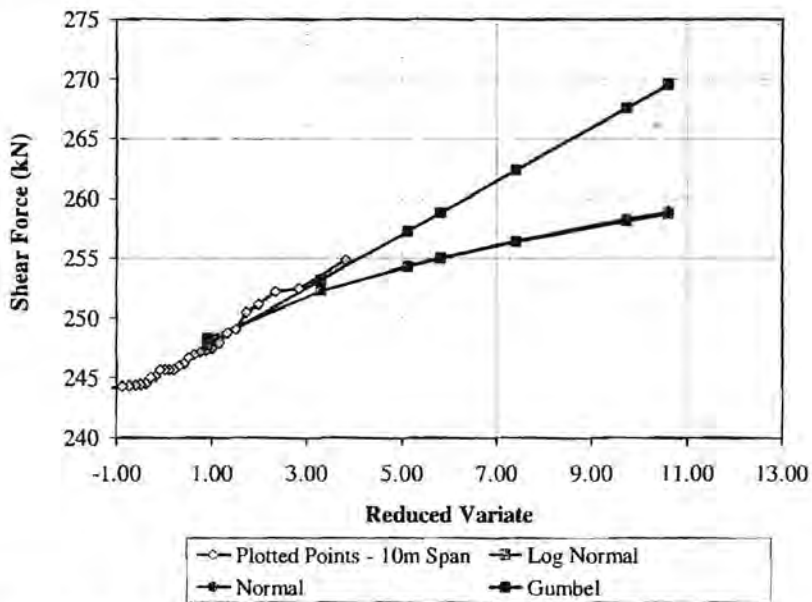


Figure B117: 10m span

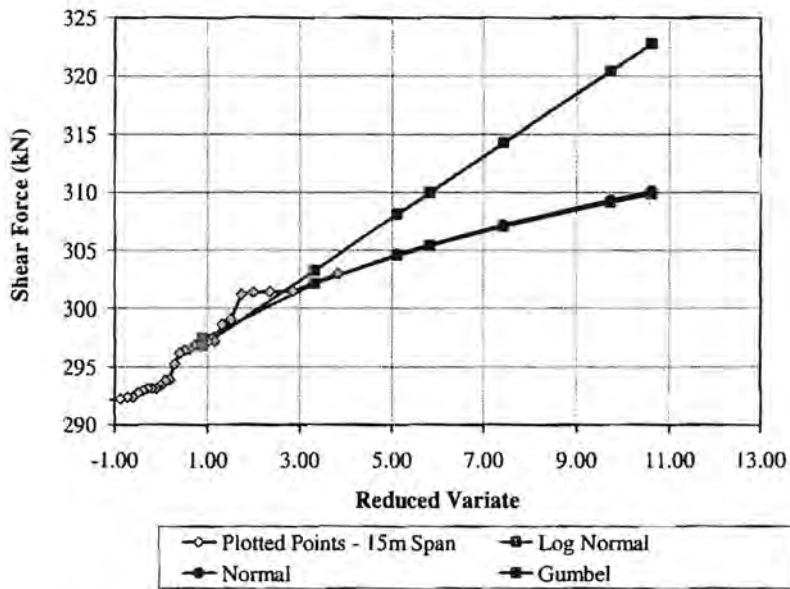


Figure B118: 15m span

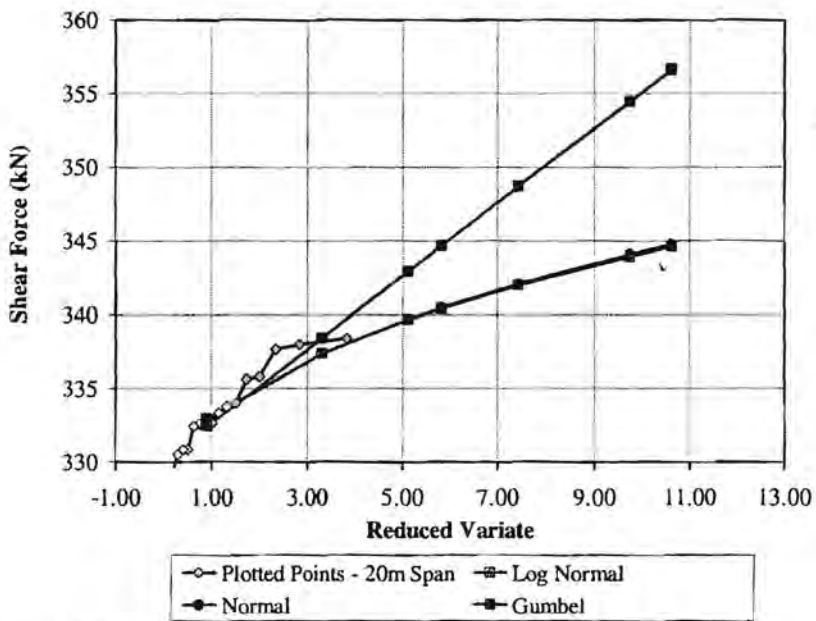


Figure B119: 20m span

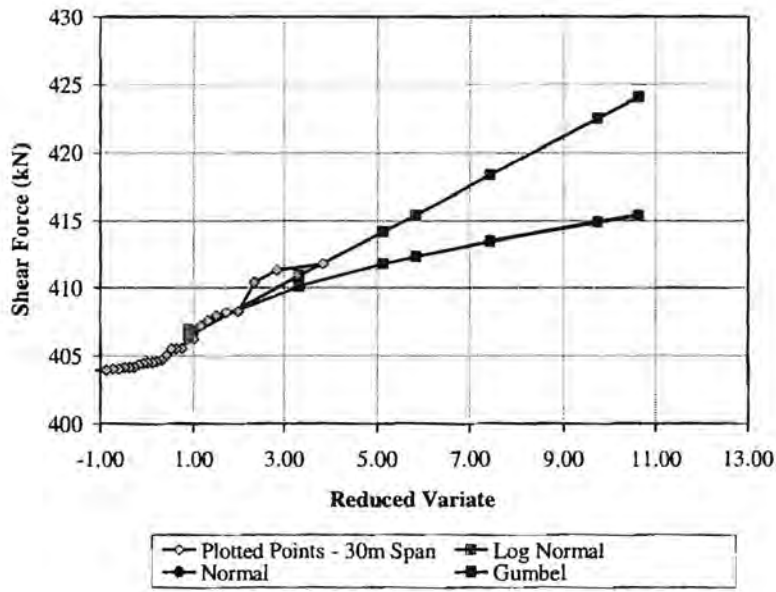


Figure B120: 30m span

**Appendix C:  
Liebenberg Combinations**

University of Cape Town

C1. LIEBENBERG & HENDERSON VEHICLE COMBINATIONS..... 1

C2. ACTUAL PROBABILITY OF COMBINATION J1 - 2 AXLE VEHICLES ..... 3

C3. ACTUAL PROBABILITY OF COMBINATIONS J2 - 3 AXLE VEHICLES ..... 4

**C1. LIEBENBERG & HENDERSON VEHICLE COMBINATIONS**



**C2. ACTUAL PROBABILITY OF COMBINATION J1 - 2 AXLE VEHICLES**

Sample Size

20689 vehicle/ month

No	Statistical Event	Probability	Calculation
1	Probability 2 axle vehicle is followed by another 2 Axle vehicle	0.22	
2	Probability 2 axle GVM >197	2.3E-05	
3	Probability 2 axle axle spacing is < 2.4m	0.0002	
4	Probability of (2) & (3) Co-existing	4.6E-09	(2)*(3)
5	Recurrence period	876 years	1/(4) / Sample Size / 12
6	Probability Critical Vehicle 1 is followed by a 2 axle vehicle	1.0E-09	(1)*(4)
7	Probability Critical Vehicle 1 is followed by Critical Vehicle 2	4.7E-18	(6)*(4)
8	Recurrence period	8.7.E+11 years	1/(7) / Sample Size / 12
9	Probability Critical Vehicle 2 is followed by a 2 axle vehicle	1.0E-18	(1)*(7)
10	Probability Critical Vehicle 2 is followed by Critical Vehicle 3	4.7E-27	(9)*(4)
11	Recurrence period	8.5.E+20	1/(10) / Sample Size / 12
12	Probability Critical Vehicle 3 is followed by a 2 axle vehicle	1.0E-27	(1)*(10)
13	Probability Critical Vehicle 3 is followed by Critical Vehicle 4	1.0E-36	(12)*(4)
14	Recurrence period	3.8E+30 years	1/(13) / Sample Size / 12
15	Probability Critical Vehicle 4 is followed by a 2 axle vehicle	2.3E-37	(1)*(13)
16	Probability Critical Vehicle 4 is followed by Critical Vehicle 5	1.1E-45	(15)*(4)
17	Recurrence period	3.8E+39 years	1/(16) / Sample Size / 12

**C3. ACTUAL PROBABILITY OF COMBINATIONS J2 - 3 AXLE VEHICLES**

Sample Size

9423 vehicle/month

No	Statistical Event	Probability	Calculation
1	Probability 3 axle vehicle is followed by another 3 Axle vehicle	0.11	
2	Probability 3 axle GVM >228 kN	0.0254	
3	Probability 2 axle axle spacing is < 2.8m	0.0485	
4	Probability of (2) & (3) Co-existing	0.0012319	(2)*(3)
5	Recurrence period	0.01 years	1/(4) / Sample Size / 12
6	Probability Critical Vehicle 1 is followed by a 3 axle vehicle	1.4E-04	(1)*(4)
7	Probability Critical Vehicle 1 is followed by Critical Vehicle 2	1.7E-07	(6)*(4)
8	Recurrence period	53 years	1/(7) / Sample Size / 12
9	Probability Critical Vehicle 2 is followed by a 3 axle vehicle	1.8E-08	(1)*(7)
10	Probability Critical Vehicle 2 is followed by Critical Vehicle 3	2.3E-11	(9)*(4)
11	Recurrence period	390,947	1/(10) / Sample Size / 12
12	Probability Critical Vehicle 3 is followed by a 3 axle vehicle	2.5E-12	(1)*(10)
13	Probability Critical Vehicle 3 is followed by Critical Vehicle 4	3.4E-16	(12)*(4)
14	Recurrence period	2.6E+10 years	1/(13) / Sample Size / 12
15	Probability Critical Vehicle 4 is followed by a 3 axle vehicle	3.7E-17	(1)*(13)
16	Probability Critical Vehicle 4 is followed by Critical Vehicle 5	4.6E-20	(15)*(4)
17	Recurrence period	1.9E+14 years	1/(16) / Sample Size / 12

## APPENDIX A

**Appendix D:  
Impact Formula**

University of Cape Town

D1.	MODEL FOR IMPACT EFFECTS ON BRIDGES FROM RR 91/004/01 & 02 .....	1
D1.1	CALCULATION MODEL .....	1
D1.2	BRIDGE IMPACT FACTOR.....	1
D1.3	VEHICLE MASS REDUCTION FACTOR .....	2
D1.4	VEHICLE SPEED REDUCTION FACTOR .....	2
D1.5	COEFFICIENT OF VARIATION .....	2

## D1. MODEL FOR IMPACT EFFECTS ON BRIDGES FROM RR 91/004/01 & 02

### D1.1 Calculation Model

Impact causes an increase in the loads which a vehicle applies to a bridge. The impact factor applied to the actual load effects was calculated using the following formula:

$$I_f = I_b f_m f_s \cdot (1 + \varepsilon)$$

Where

- $I_f$  = the final impact factor
- $I_b$  = the impact factor for the bridge
- $f_m$  = the reduction factor for the vehicles mass
- $f_s$  = the speed reduction factor
- $\varepsilon$  = the coefficient of variation.

### D1.2 Bridge Impact Factor

Based upon the currently available data, the expression proposed by Honda et al. (1986) for the calculation of the bridge impact formula is used. However, the factor is halved to simulate the response of concrete bridges under dynamic loads (RR 91/004/02, 1995). At span lengths with natural frequencies approaching 5Hz and less, the reduction factor of 2 will be excluded to allow for dynamic amplification due to the interaction between the vehicle and the structure.

For a simply supported bridge with a span of  $L$ :

$$I_b = \frac{3}{L}$$

For a continuous bridge with  $n$  spans, each  $L$  Long:

$$I_b = \frac{4,5}{L\sqrt{n}}$$

### D1.3 Vehicle Mass Reduction Factor

A heavier vehicle will have a smaller impact effect than a lighter vehicle. A mass reduction factor,  $f_m$ , is therefore used.

For a vehicle of mass T tons:

$$f_m = 0.819 \left[ 10^{0.436 - 0.02547T} + 0.15 \right] \text{ if } T \geq 16t$$

If the vehicle weight is less than 16t,  $f_m = 1.00$

### D1.4 Vehicle speed reduction factor

The faster a vehicle is travelling the higher its impact. The following expression was used to relate the impact factor and the vehicle's speed V (in km/h).

$$f_s = \left( \frac{V}{80} \right)^{0.75}$$

From this formula it can be seen that a speed of 80km/h will yield a value for  $f_s$  of 1.0.

### D1.5 Coefficient of Variation

Significant scatter exists in the data from research into the response of bridges to impact loading. The coefficient of variation,  $E$ , accounts for this scatter and is approximately 12% of the total dynamic response.

**Appendix E:  
Equivalent Vehicle Study**

University of Cape Town

<b>E1. EQUIVALENT VEHICLE STUDY .....</b>	<b>1</b>
E1.1 O'CONNOR'S APPRAISAL OF ONTARIO BASE LENGTH.....	1
E1.2 DEVELOPMENT OF ALTERNATIVE DERIVATION OF EQUIVALENT BASE LENGTH .....	1
E1.3 SENSITIVITY ANALYSIS.....	3
E1.4 SIMULATION STUDY .....	3
E1.5 ASSIGNMENT OF PARAMETERS TO SOUTH AFRICAN TRUCK SURVEY DATA.....	4
<b>E2. COMPARISON OF VIRTUAL AND LEGAL TRUCK POPULATIONS.....</b>	<b>5</b>
<b>E3. ALTERNATIVE SELECTION OF EXTREME EVENTS .....</b>	<b>5</b>

## E1. EQUIVALENT VEHICLE STUDY

The review of the WIM data recorded during a one month period, involved the processing of 106,917 vehicles. It is expected that a greater number of vehicles may be reviewed in future studies. A method of identifying the vehicles causing the most onerous force effects was therefore investigated. From the literature review the Ontario Equivalent Base Length concept and its further development by O'Connor (1981) was considered the most appropriate method.

### E1.1 O'Connor's Appraisal of Ontario Base Length

O'Connor (1981) reviewed the validity of the Ontario Equivalent Base Length as described in Section 2.4.3. The review focused on the validity of replacing survey data with a histogram of points in  $(W, B_m)$  space and the selection of a design vehicle on the basis that its equivalent base length follows the Maximum Observed Limit (MOL) curve.

In review of the Ontario Equivalent Base Length, it was recognised that a uniformly distributed load, placed about the centre of a span, will not necessarily simulate the maximum force effects caused by a heavy vehicle. In many instances, a group of axles at the rear of a vehicle will cause the maximum load effects. An alternative method to calculating the equivalent base length was therefore reviewed. A third parameter, the location parameter, was developed to ensure that the load model was correctly positioned on the span to produce the most onerous load effects.

### E1.2 Development of Alternative Derivation of Equivalent Base Length

In defining the location parameter, O'Connor (1981) utilises the influence lines associated with simply supported spans. A load group was moved across the span as shown in Figure E1. The load causing the maximum force effects when positioned at the maximum ordinate of the influence lines was identified. This load was classified as the central load. The load group was then divided into two sub-groups; a load group to left of the maximum ordinate and a load group to its right. The central load was then apportioned to the left and right so that both sub-groups sum to half of the total load group.

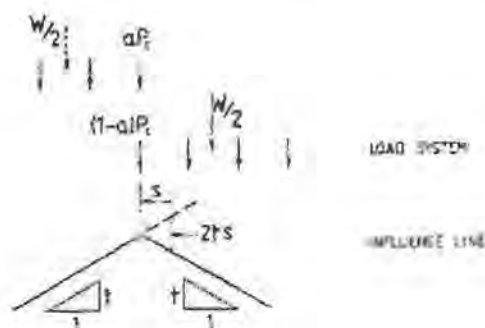


Figure E1 - Simply Support Span Influence Lines and Central Load (Source: O'Connor C., 1981)

A single concentrated load was then set to represent the subgroup, as shown in Figure E2. This load was positioned to generate the resultant moment of the individual loads within the sub group about the maximum ordinate. The distance from the load to the maximum ordinate is known as  $b_R$  and  $b_L$  respectively. The concentrated base length,  $b$ , is then equal to the sum of  $b_R$  and  $b_L$ .

$$b = b_L + b_R$$

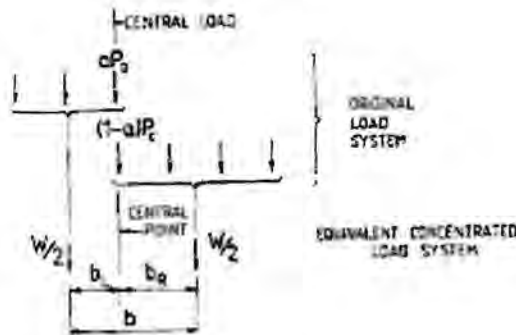


Figure E2 -Equivalent Concentrated Load System (Source: O'Connor C., 1981)

A location parameter,  $x$ , was derived to define the central point of the equivalent system and was calculated as the lesser of  $b_R/b$  and  $b_L/b$ .

In the case of the simplified derivation of  $B_m$ , the derivation of  $B_m$  is exactly twice the concentrated base length. However, the application of a uniformly distributed load will not necessarily cause the same force effects. This outcome is because the concentrated loads are not located equi-distant from the central load, as quantified by the location parameter. In using the concentrated base length, the central load may be located at the centre of the span and the concentrated loads will exactly generate the moments caused by the axle loads to its left and right. However, the centring of a uniformly distributed load on the centre of the span will not necessarily produce the maximum force effects. It is for this reason that O'Connor preferred the use of an equivalent concentrated load as shown in Figure E3.

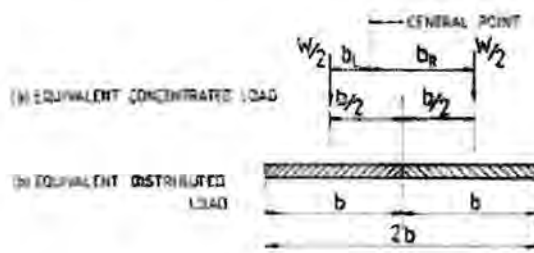


Figure E3 - Equivalent Concentrated Load (Source: O'Connor C., 1981)

### E1.3 Sensitivity Analysis

O'Connor identified that trucks of varying axle combinations may have similar total loads and concentrated base lengths. A sensitivity analysis was therefore undertaken comparing the force effects of five vehicles with varying axle configurations but with the same  $W$  and  $b$  values. The calculated maximum moments, for simply supported spans in excess of 10m, showed exact correlation. However, in the shorter spans the moments caused by the five trucks diverged.

In the case of continuous spans, differences in the moments produced were present on the longer spans. It was found that the trucks with the closest location parameters produced the best correlation of moments.

The sensitivity analysis was then taken a step further to compare a family of three trucks of varying axle configuration, but with identical  $W$ ,  $b$  and  $x$  values. Once more, the central bending moments in a simply supported span were consistent except for in the case of short spans. This result was to be expected, because in the case of short spans it is a single axle or group of axles that will produce the greatest bending moment. In the case of continuous spans, the results showed greater convergence on the longer spans. However, substantial differences in the shear force at the end of a continuous girder were observed. O'Connor stated the following conclusions from the studies described herein:

- (i) No equivalent vehicle is perfect for all cases;
- (ii) Vehicles with identical base length signatures can give different results;
- (iii) It is difficult to judge if the differences shown to exist between hypothetical vehicles are similar in magnitude to those that exist in practical vehicles; and
- (iv) There is some prospect of designing a satisfactory equivalent vehicle.

### E1.4 Simulation Study

To follow on from the conclusions reached in his preliminary studies, O'Connor undertook a simulation of the entire Ontario process. In the place of survey data, a population of vehicles was created with axle weights and axle configurations lying at the Australian legal limits.

From the  $W/b$  charts the critical  $W$  and  $b$  parameters were extracted and three equivalent vehicles were then generated. In simple terms, the shortest value of  $b$  with a  $W$  equal to the maximum legal load was chosen. These vehicles were then used to generate a maximum envelope of the five load effect functions. These envelopes were then compared against those of the parent population.

The study concluded that the description of a vehicle by the proposed three parameters does not exactly simulate its effects on single or continuous spans. However, in creating a design vehicle, the aim is to simulate the maximum force effects caused by a population of vehicles. It was considered that the method developed by O'Connor provides a means of identifying trucks with characteristics likely to produce the most onerous force effects. Those parameters being:

- (i) Maximum  $W$
- (ii) Shortest  $b$
- (iii) Centred  $x=0.5$

The outcome of the study was that it may be possible to use a single non-variable design vehicle with sufficient accuracy; this has subsequently happened in both the LFRD and the CSA-S06-00.

### **E1.5 Assignment of Parameters to South African Truck Survey Data**

In creating a credible population of possible axle configurations, O'Connor considered the subsets of adjacent loads within the 191 trucks created from the specified set of axle configurations. In all, 4,500 varying axle configurations were created. The aim of this study was to review the parameters associated with the axle configurations of the 106,917 actual vehicles recorded in the WIM data. Given that there are no legal constraints on axle spacing in South Africa (other than the bridge formula) it is considered that this is a valid population set when considering the derivation of an equivalent vehicle.

The legal vehicle population set developed in Chapter 3 was used for the purpose of the study. The mass of the vehicles, axle sets and individual axles was therefore compliant with the South African legal limits.

A virtual population of South African vehicles was also created following O'Connor's (1981) guidelines using the possible permutations of axle configurations. In assigning the axle masses, the South African maximum permissible axle masses were substituted. The aim of this exercise was to measure the variances associated with the use of a virtual population against a population of recorded legal vehicles.

The methodology developed by O'Connor was replicated in assigning the parameters  $W$ ,  $b$  and  $x$  to each vehicle. In addition, the bending moments and shears caused by the vehicle on a range of simply supported spans were calculated. The parameters associated with the most onerous bending moments and shears were then reviewed for the purpose of deriving a South African Equivalent Vehicle.

## E2. COMPARISON OF VIRTUAL AND LEGAL TRUCK POPULATIONS

In calculating the load effects, the vehicles were grouped in terms of their total number of axles. This grouping was done to compare the properties of the different vehicle classes.

The maximum bending moments generated from the virtual and legal truck populations for spans from 5m to 30m are shown in Table E1. A close correlation between the bending moments derived from both methods is observed. Table E2 shows the same correlation in the calculated shear forces.

Max. Bending Moments (kNm)			
Span (m)	Legal Vehicles	O' Connor Vehicles	% Difference Legal: O'Connor
5	197	189	-4%
10	547	562	3%
15	1054	1054	0%
20	1582	1626	3%
30	2917	2897	-1%

Table E1 - Bending Moments Comparison, O' Connors Vehicle versus Legal Vehicles

Max. Shear Forces (kN)			
Span (m)	Legal Vehicles	O' Connor Vehicles	% Difference Legal: O'Connor
5	184	173	-6%
10	252	258	3%
15	308	304	-1%
20	347	346	0%
30	411	409	-1%

Table E2 - Shear Force Comparison, O' Connors Vehicle versus Legal Vehicles

The close comparison was expected, as the axle weights of both populations were identical. However, the results indicate that the creation of a virtual population of vehicles adequately replicate the load effects of actual vehicles. These findings validate the use of virtual vehicles in Monte Carlo simulations.

## E3. ALTERNATIVE SELECTION OF EXTREME EVENTS

An alternative method of selecting the extreme set of vehicles was investigated. Using the parameters developed by O'Connor, the vehicles with the highest  $W/b$  ratios in each class were identified. A population set of 84 vehicles (28 from the 6, 7 & 8 vehicles classes), representing the extreme events over the period of the month, was then created.

The statistical properties of the extreme events were used to extrapolate a 1 in 120 year event using the Gumbel distribution. A comparison of the load effects from the  $W/b$  population with those of the legal population (Section 3) is shown in Table E3 & E4.

Span (m)	Bending Moments (kNm)		% Difference <i>W/b</i> : Legal
	<i>W/b</i> Extreme	Legal Vehicles	
5	293	301	-3%
10	800	812	-1%
15	1262	1364	-7%
20	1903	2097	-9%
30	3526	3631	-3%

Table E3: Bending Moment Comparison, *W/b* Vehicles versus Legal Vehicles

Span (m)	Shear Forces (kN)		%Difference <i>W/b</i> : Legal
	<i>W/b</i> Extreme	Legal Vehicles	
5	313	297	5%
10	337	320	5%
15	369	369	0%
20	438	433	1%
30	522	542	-4%

Table E4: Shear Force Comparison, *W/b* Vehicles versus Legal Vehicles

The results for both bending moments and shear forces show good correlation; this effectively validates the use of *W* and *b* parameters to identify the critical vehicles. The parameters, therefore, provide a means of sorting WIM data to reduce the number of results that require processing.

**Appendix F:  
Visual Basic Programs**

University of Cape Town

## Typical VB Program written to calculate the maximum bending moments caused by survey vehicles on varying simply supported spans

```
Sub centralload()  
  
Application.Calculation = xlCalculationManual  
  
Dim moment_array(2000, 2000) As Double  
Dim shear_array(3000, 3000) As Single  
Dim diff_array(100) As Single  
Dim mom_array(35000, 5) As Single  
Dim Rmax_array(35000) As Single  
Dim cum(11) As Single  
Dim span(5) As Integer  
  
Start = Range("b2").Value  
Finish = Range("b3").Value  
  
'assigns variable  
inc = 1  
span(1) = 5  
span(2) = 10  
span(3) = 15  
span(4) = 20  
span(5) = 30  
  
'loop for spans  
For s = 1 To 5  
  
For y = Start To Finish  
  
axle1 = Range("s" & y).Value  
axle2 = Range("t" & y).Value  
axle3 = Range("u" & y).Value  
axle4 = Range("v" & y).Value  
axle5 = Range("w" & y).Value  
axle6 = Range("x" & y).Value  
axle7 = Range("y" & y).Value  
axle8 = Range("z" & y).Value  
axle9 = Range("aa" & y).Value  
axle10 = Range("ab" & y).Value  
axle11 = Range("ac" & y).Value  
  
spac2 = Range("af" & y).Value  
spac3 = Range("ag" & y).Value  
spac4 = Range("ah" & y).Value  
spac5 = Range("ai" & y).Value  
spac6 = Range("aj" & y).Value  
spac7 = Range("ak" & y).Value  
spac8 = Range("al" & y).Value  
spac9 = Range("am" & y).Value  
spac10 = Range("an" & y).Value  
spac11 = Range("ao" & y).Value
```

```

cum(1) = spac2
cum(2) = cum(1) + spac3
cum(3) = cum(2) + spac4
cum(4) = cum(3) + spac5
cum(5) = cum(4) + spac6
cum(6) = cum(5) + spac7
cum(7) = cum(6) + spac8
cum(8) = cum(7) + spac9
cum(9) = cum(8) + spac10
cum(10) = cum(9) + spac11

```

```
Length = spac2 + spac3 + spac4 + spac5 + spac6 + spac7 + spac8 + spac9 + spac10
```

```
n = 1
```

```
Do While inc * n < span(s) + Length
```

```
pos = inc * n
```

```
'defines relative positions
```

```
pos1 = pos
```

```
If pos1 - cum(1) <= 0 Or cum(1) = 0 Then pos2 = 0 Else pos2 = pos1 - cum(1)
```

```
If pos1 - cum(2) <= 0 Or cum(2) = cum(1) Then pos3 = 0 Else pos3 = pos1 - cum(2)
```

```
If pos1 - cum(3) <= 0 Or cum(3) = cum(2) Then pos4 = 0 Else pos4 = pos1 - cum(3)
```

```
If pos1 - cum(4) <= 0 Or cum(4) = cum(3) Then pos5 = 0 Else pos5 = pos1 - cum(4)
```

```
If pos1 - cum(5) <= 0 Or cum(5) = cum(4) Then pos6 = 0 Else pos6 = pos1 - cum(5)
```

```
If pos1 - cum(6) <= 0 Or cum(6) = cum(5) Then pos7 = 0 Else pos7 = pos1 - cum(6)
```

```
If pos1 - cum(7) <= 0 Or cum(7) = cum(6) Then pos8 = 0 Else pos8 = pos1 - cum(7)
```

```
If pos1 - cum(8) <= 0 Or cum(8) = cum(7) Then pos9 = 0 Else pos9 = pos1 - cum(8)
```

```
If pos1 - cum(9) <= 0 Or cum(9) = cum(8) Then pos10 = 0 Else pos10 = pos1 - cum(9)
```

```
If pos1 - cum(10) <= 0 Or cum(10) = cum(9) Then pos11 = 0 Else pos11 = pos1 - cum(10)
```

```
'assigns axle weight = 0 if not on the beam
```

```
If pos1 >= span(s) Then axle1 = 0 Else axle1 = Range("s" & y).Value
```

```
If pos2 = 0 Or pos2 >= span(s) Then axle2 = 0 Else axle2 = Range("t" & y).Value
```

```
If pos3 = 0 Or pos3 >= span(s) Then axle3 = 0 Else axle3 = Range("u" & y).Value
```

```
If pos4 = 0 Or pos4 >= span(s) Then axle4 = 0 Else axle4 = Range("v" & y).Value
```

```
If pos5 = 0 Or pos5 >= span(s) Then axle5 = 0 Else axle5 = Range("w" & y).Value
```

```
If pos6 = 0 Or pos6 >= span(s) Then axle6 = 0 Else axle6 = Range("x" & y).Value
```

```
If pos7 = 0 Or pos7 >= span(s) Then axle7 = 0 Else axle7 = Range("y" & y).Value
```

```
If pos8 = 0 Or pos8 >= span(s) Then axle8 = 0 Else axle8 = Range("z" & y).Value
```

```
If pos9 = 0 Or pos9 >= span(s) Then axle9 = 0 Else axle9 = Range("aa" & y).Value
```

```
If pos10 = 0 Or pos10 >= span(s) Then axle10 = 0 Else axle10 = Range("ab" & y).Value
```

```
If pos11 = 0 Or pos11 >= span(s) Then axle11 = 0 Else axle11 = Range("ac" & y).Value
```

```
Wtotal = axle1 + axle2 + axle3 + axle4 + axle5 + axle6 + axle7 + axle8 + axle9 + axle10 + axle11
```

```
'calculate support reactions
```

```
'take moments about LHS
```

```
M1 = axle1 * pos1
```

```
M2 = axle2 * pos2
```

```
M3 = axle3 * pos3
```

```
M4 = axle4 * pos4
```

```
M5 = axle5 * pos5
```

```
M6 = axle6 * pos6
```

```
M7 = axle7 * pos7
```

```
M8 = axle8 * pos8
```

```

M9 = axle9 * pos9
M10 = axle10 * pos10
M11 = axle11 * pos11

Mtotal = M1 + M2 + M3 + M4 + M5 + M6 + M7 + M8 + M9 + M10 + M11

R2 = Mtotal / span(s)
R1 = Wtotal - R2

'calculate moments along length of the beam
x = 1
Do While x < span(s)
MR1 = x * R1
If x > pos1 Then MA1 = (pos1 - x) * axle1 Else MA1 = 0
If x > pos2 Then MA2 = (pos2 - x) * axle2 Else MA2 = 0
If x > pos3 Then MA3 = (pos3 - x) * axle3 Else MA3 = 0
If x > pos4 Then MA4 = (pos4 - x) * axle4 Else MA4 = 0
If x > pos5 Then MA5 = (pos5 - x) * axle5 Else MA5 = 0
If x > pos6 Then MA6 = (pos6 - x) * axle6 Else MA6 = 0
If x > pos7 Then MA7 = (pos7 - x) * axle7 Else MA7 = 0
If x > pos8 Then MA8 = (pos8 - x) * axle8 Else MA8 = 0
If x > pos9 Then MA9 = (pos9 - x) * axle9 Else MA9 = 0
If x > pos10 Then MA10 = (pos10 - x) * axle10 Else MA10 = 0
If x > pos11 Then MA11 = (pos11 - x) * axle11 Else MA11 = 0

m = MR1 + MA1 + MA2 + MA3 + MA4 + MA5 + MA6 + MA7 + MA8 + MA9 + MA10 + MA11
moment_array(x, n) = m
If R2 > R1 Then R = R2 Else R = R1
shear_array(x, n) = R
x = x + 1
Loop

n = n + 1
Loop

'finds position of max moment
maxrow = 0
maxcol = 0

For z = 0 To n
  For i = 1 To x
    If moment_array(i, z) > moment_array(maxrow, maxcol) Then maxrow = i
    If moment_array(i, z) > moment_array(maxrow, maxcol) Then maxcol = z
  Next i
Next z

Range("aq" & y).Value = moment_array(maxrow, maxcol)
mom_array(y, s) = moment_array(maxrow, maxcol)

```

```

'finds position of max shear
Rmaxrow = 0
Rmaxcol = 0

For z = 0 To n
  For i = 1 To x
    If shear_array(i, z) > shear_array(Rmaxrow, Rmaxcol) Then Rmaxrow = i
    If shear_array(i, z) > shear_array(Rmaxrow, Rmaxcol) Then Rmaxcol = z
  Next i
Next z
Range("ar" & y).Value = shear_array(Rmaxrow, Rmaxcol)
Rmax_array(y) = shear_array(Rmaxrow, Rmaxcol)

Next y

'sort moment array max value to top
maxmom = Start
For i = Start To Finish
  If mom_array(i, s) > mom_array(maxmom, s) Then maxmom = i
Next i
Range("at" & s + Start - 1) = mom_array(maxmom, s)
Range("av" & s + Start - 1).Value = maxmom

'sorts vehicle moments
For i = Start To Finish - 1
  Max = i
  For j = i + 1 To Finish
    If mom_array(j, s) > mom_array(Max, s) Then
      Max = j
    End If
  Next j
  temp = mom_array(i, s)
  mom_array(i, s) = mom_array(Max, s)
  mom_array(Max, s) = temp

Range("bd" & i).Value = mom_array(i, 1)
Range("be" & i).Value = mom_array(i, 2)
Range("bf" & i).Value = mom_array(i, 3)
Range("bg" & i).Value = mom_array(i, 4)
Range("bh" & i).Value = mom_array(i, 5)
Next i

'sort shear array max value to top
maxshear = Start
For i = Start To Finish
  If Rmax_array(i) > Rmax_array(maxshear) Then maxshear = i
Next i
Range("au" & s + Start - 1) = Rmax_array(maxshear)

Next s

Application.Calculation = xlCalculationAutomatic

End Sub

```