RELIABILITY ANALYSIS OF PRECAST PRESTRESSED BRIDGE GIRDERS BASED ON AASHTO LRFD AND EUROCODE

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submitted by **CEYHAN ÇELİK** in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University** by,

Prof. Dr. Halil Kalıpçılar
Director, Graduate School of **Natural and Applied Sciences** ________________

Prof. Dr. İsmail Özgür Yaman
Head of Department, **Civil Engineering** ________________

Prof. Dr. Alp Caner
Supervisor, **Civil Engineering Dept., METU** ________________

Examining Committee Members:

Prof. Dr. Mehmet Utku
Civil Engineering Dept., METU ________________

Prof. Dr. Alp Caner
Civil Engineering Dept., METU ________________

Prof. Dr. Ahmet Türer
Civil Engineering Dept., METU ________________

Prof. Dr. Murat Altuğ Erberik
Civil Engineering Dept., METU ________________

Asst. Prof. Dr. Burcu Güldür Erkal
Civil Engineering Dept., Hacettepe University ________________

Date: ________________

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last Name : Ceyhan ÇELİK

Signature :

ABSTRACT

RELIABILITY ANALYSIS OF PRECAST PRESTRESSED CONCRETE BRIDGE GIRDERS BASED ON AASHTO LRFD AND EUROCODE SPECIFICATIONS

Çelik, Ceyhan
M.S., Department of Civil Engineering
Supervisor: Prof. Dr. Alp Caner

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Different bridge specifications are used in bridge designs depending on special conditions and needs. Since each bridge specification has its own specific design requirements and structural safety margins, different bridge cross-sections can appear for the same bridge spans. In the literature, it is seen that the reliability index is accepted as one of the basic measures in evaluating structural safety of bridges. In this thesis, the reliability levels of prestressed bridge girders designed according to American and European bridge specifications are calculated and compared. The main purpose of this study is to compare the American and European bridge specifications, which are used predominantly in bridge design practice, to determine the difference between the structural safety levels of these specifications. In this study, bridges with span lengths of 25 m, 30 m, 35 m and 40 m were taken into consideration. KGM-45 type truck loading which is frequently used for bridge design in Turkey was determined to be live load model designed according to AASHTO LRFD specification. Eurocode load model 1 has been taken into consideration as a live load model calculated based on EUROCODE provisions. The uncertainties regarding the load and resistance components required to calculate the reliability index have been determined by considering the data obtained from local sources and international studies.

Keywords: Reliability Analysis, Reliability Index, AASHTO LRFD, EUROCODE Precast Prestressed Concrete Bridge Girders.

ÖZ

PREFABRİK ÖNGERMELİ KÖPRÜ KİRİŞLERİNİN GÜVENİRLİK SEVİYELERİNİN AASHTO LRFD AND EUROCODE ŞARTNAMELERİ TEMELİNDE DEĞERLENDİRİLMESİ

Çelik, Ceyhan
Yüksek Lisans, İnşaat Mühendisliği
Tez Yöneticisi: Prof. Dr. Alp Caner

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Köprü tasarımlarında özel şartlara ve ihtiyaçlara bağlı olarak farklı köprü şartnameleri kullanılmaktadır. Her köprü şartnamesinin kendisine özel tasarım gereklilikleri ve yapısal güvenlik marjları bulunması sebebiyle aynı köprü açıklıkları için farklı köprü kesitleri ortaya çıkabilmektedir. Literatürde, güvenirlik indeksinin köprü yapı emniyetinin değerlendirmesinde temel ölçülerden biri olarak kabul edildiği görülmektedir. Bu tez çalışmasında, Amerikan ve Avrupa köprü şartnamelerine göre tasarlanan öngermeli köprü kirişlerinin güvenirlik seviyeleri hesaplanmış ve karşılaştırılmıştır. Bu çalışmadaki temel amaç köprü tasarım pratiğinde ağırlıklı olarak kullanılan Amerikan ve Avrupa köprü şartnamelerinin mukayese edilerek bu şartnamelerin yapısal güvenlik seviyeleri arasındaki farkın belirlenmesidir. Bu çalışma kapsamında, 25 m, 30 m, 35 m ve 40 m açıklıklarındaki köprüler dikkate alınmıştır. Türkiye'de köprü tasarımlarında sıklıkla kullanılan KGM-45 tipi kamyon yükü Amerikan şartnamesine göre yapılan tasarımlara esas olmak üzere hareketli yük olarak belirlenmiştir. Avrupa şartnamesine ilişkin hesaplamalara esas olarak ise aynı şartnamede belirtilen bir numaralı hareketli yük modeli göz önünde bulundurulmuştur. Güvenirlik indeksinin hesaplanabilmesi için gerekli olan yük ve dayanıma ilişkin belirsizlikler yerel kaynaklardan elde edilen veriler ve uluslararası çalışmalar dikkate alınarak belirlenmiştir.

Anahtar Kelimeler: Güvenirlik Analizi, Güvenirlik Endeksi, AASHTO LRFD, EUROCODE, Öngermeli Prefabrik Köprü Kirişleri.

To My Wife

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The author is also grateful for the encouragement and intimate support of his wife, parents and dearest friend Yusuf Dönmez.

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CHAPTER 1

INTRODUCTION

Bridges are the unique structures in the highway and railway network systems due to their critical role for the transportation. Therefore, the safety of bridges must be sustained and under controlled throughout the life time of structure.

Different design specifications and design approaches are used for that purpose in the world. For instance, AASHTO LRFD code is used in United States of America. Another specification currently applied in Europe is EUROCODE which is also based upon load and resistance factor method. The other approach is the Load Factor Design (LFD) based method that mostly preferred for the design of bridge in Turkey. However, the LFD method is an old fashioned approach and the new design concept is shifting to LRFD method.

The main difference between the LRFD and LFD specifications is the regulation process of the resistance and load. In LFD specification, factors for resistance and load are regulated according to general practice and experience. However, in LRFD specification, factors for resistance and load are regulated in accordance with the uncertainty of these parameters. Primary aim of this probability based design approach is to sustain standardized safety grade for each part of the construction.

The fundamental rule of structural engineering is designing a structure where the resistance is higher than the load effects. Nevertheless, this goal may not be achieved every time because of the uncertainties due to the effects of loads and resistance of materials. Therefore, in the design stage, these uncertainties must be considered. In LRFD method, this issue which can be called as probability of failure of structure is taken into account by calibrating the factors of resistance and load based on the local conditions. This probabilistic approach is widely used in the bridge safety measurement study. Reliability index, which is denoted by β, is used as an indicator for the structural safety. As the reliability index term increases, the probability of survival of structure increases.

Although the same design approach (LRFD) is taken as basis for the code-writing groups, different structural safety margins may arise for the same structure when the calculation was done by considering the different design specifications. This result is mainly due to the estimation of different sources of uncertainties for the load effects and resistance of materials to determine the factors for resistance and load.

1.1 Aim

The main purpose of this research is to compare the reliability levels of prestressed precast bridge girders designed based on the AASHTO LRFD and the EUROCODE-2 specifications.

In the design of girders, KGM-45 loading generated by Turkish General Directorate of Highways and Eurocode LM-1 loading defined in EUROCODE (1991-2:2003) are considered as live load models. These loading models are chosen for the design of girders based on the AASHTO LRFD and EUROCODE-2 specifications respectively.

For the determination of safety level of prestressed precast bridge girders, the statistical parameters related with elements of load and resistance are calculated. These statistical parameters are mainly the coefficient of variation and bias factor in accordance with the load and resistance components.

Statistical parameters for the live load models are determined by considering the local truck survey data taken from the General Directorate of Highways of Turkey. These data belong to the years between 2005, 2006 and 2013.

Other statistical parameters are estimated from the local and international studies that evaluate the uncertainties of the various load and resistance components for design and construction stages.

In this research, the prestressed precast bridge girders are designed for four different span lengths with a range of 25 m to 40 m. Moreover, the design of girders is carried out mainly for the flexural resistance at the mid span based on the strength I limit states of AASHTO LRFD and ultimate limit states of EUROCODE-2 specifications.

The reliability indexes are determined for the different bridge cross-sections specified for each span length under the KGM-45 and Eurocode LM-1 live loading. The Mean Value First Order Second Moment method is taken into consideration for the reliability analysis. As it is stated before, the main purpose is to estimate the differences between the reliability levels of bridge girders designed based on the AASHTO LRFD and the EUROCODE-2 codes.

1.2 Scope

The thesis content is constituted as below:

In Chapter 2, literature is reviewed. Calibration studies for load and resistance factors in AASHTO LRFD and Turkish LRFD are indicated. Moreover, the reliability analysis of different live load models and the comparison of reliability level of girders designed by different specifications such as AASHTO LRFD, EUROCODE, Spanish Norma, Chinese and Hong Kong provisions are presented.

In Chapter 3, loads and load combinations that are considered for the girder design are introduced. Truck survey data is evaluated and the 75 year maximum live load influence is calculated from extrapolation of these data. Then, the relevant statistical parameters of live loads, dead loads, dynamic load and impact factor are illustrated.

In Chapter 4, the uncertainties depending on the resistance of materials are discussed and the related statistical parameters of resistance are estimated.

In Chapter 5, the theory underlining the calculation of flexural resistance capacity of prestressed precast concrete girders are explained briefly. After that the detailed dimensions of the bridge girders are presented for each span length. Additionally, all the analyses and results are displayed.

In Chapter 6, the reliability evaluation method is explained. Moreover, the reliability evaluation of girders designed by AASHTO LRFD and EUROCODE-2 provisions are performed for both the KGM-45 and Eurocode LM-1 live loading. The comparison of reliability indexes is made in this part.

Finally, all the main results of the study are summarized and the thesis is concluded in Chapter 7. Recommendations are also made for the further studies.

CHAPTER 2

LITERATURE REVIEW

2.1 Literature Survey

Different design methods such as load factor and allowable stress design do not usually ensure a standardized and regular safety level for bridges. In order to get a consistent safety level for bridges, LRFD design concept has been developed. The main difference of this method is that it depends on a probabilistic approach.

In this study, procedures described in the report of National Cooperative Highway Research Program namely "Report 368: Calibration of LRFD Bridge Design Code" are followed. In literature, in comparison of different design specifications according to the reliability index, assumptions and statistical parameters described in this report were generally used. These parameters and assumptions mainly consist of the adjustment procedure, resistance and load models, reliability evaluation and determination of resistance and load factors. In the report of calibration, materials strength, dimensions and analysis are the three factors that have been considered for the resistance. The statistical parameters of resistance are also calculated by using the previous studies for the different special girder types and lognormal variate is used for the resistance.

LRFD calibration study has been conducted for nearly 200 constructed bridges in United States. Moments, shears, tensions and compressions effects have been determined and load capacities of each bridge members have been calculated. By using the local surveys and material tests, the database has formed for loads and resistance which were assumed as random variables. Since these variables are random, they are identified with regard to cumulative distribution functions (CDF). After that, live load and resistance models are constructed and structural reliability is evaluated based on the reliability index (β). This calculation procedure is an iterative situation and described by Fiessler and Rackwitz. Next, a target reliability index (β_T) is determined with respect to the safety level of the structures. Lastly, resistance and load factors are defined in view of the target reliability index (Nowak, 1999).

In the final stage of Calibration Report, reliability evaluation is assessed by taking into account the average daily truck traffic (ADTT) = 5000. By taking ADTT as 5000, AASHTO LRFD Strength Limit State I design equation become as;

$$1.25\ x\ D\ +1.50\ x\ D_A\ +1.75\ x\ (1\ +I)\ x\ L\ <\emptyset R \tag{2-1}$$

where, R is resistance, $\emptyset$ is factor for resistance, D is dead load because of cast-in place and factory made concrete structure, D_A is the dead load due to the asphalt plank wearing surface, I is impact factor and L is live load.

In another research conducted by Nowak, the reliability level of girders designed by AASHTO LRFD, EUROCODE and Spanish Norma specifications have been compared. Five prestressed concrete bridges have been selected with varying span from 20 to 40 m. The structures have been designed with typical Spanish precast concrete girders. The statistical parameters have been determined according to the adjusment report mentioned above. In reliability evaluation, statistical parameters of Spanish and Ontario truck surveys have been taken into consideration.

Extrapolated cumulative distribution functions of Ontario and Spanish trucks data are illustrated in Figure 2-1 and Figure 2-2. The cumulative distribution functions of moments have been extrapolated to obtain the statistical parameters of the maximum live load influence for futured periods of time which were taken as 75 years. And it is stated that the mean maximum annual live load supposed to pursue an extreme type I (Gumbell) distribution.

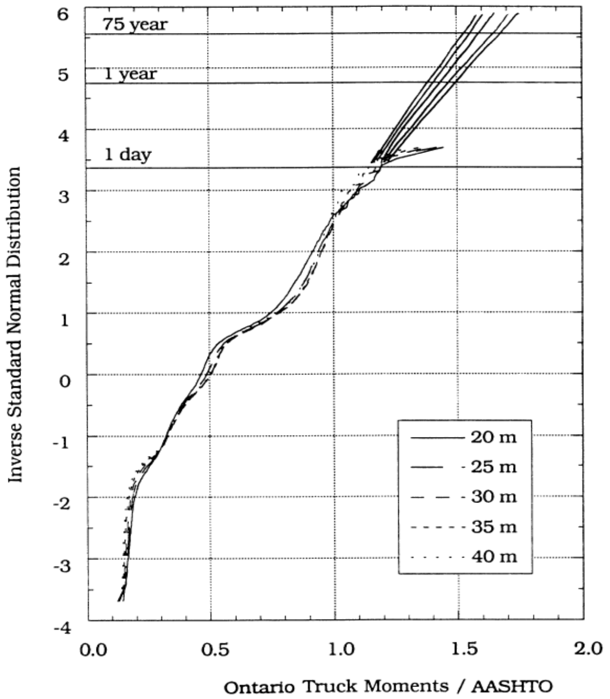


Figure 2-1 Moment Ratios Obtained from Ontario Truck Survey

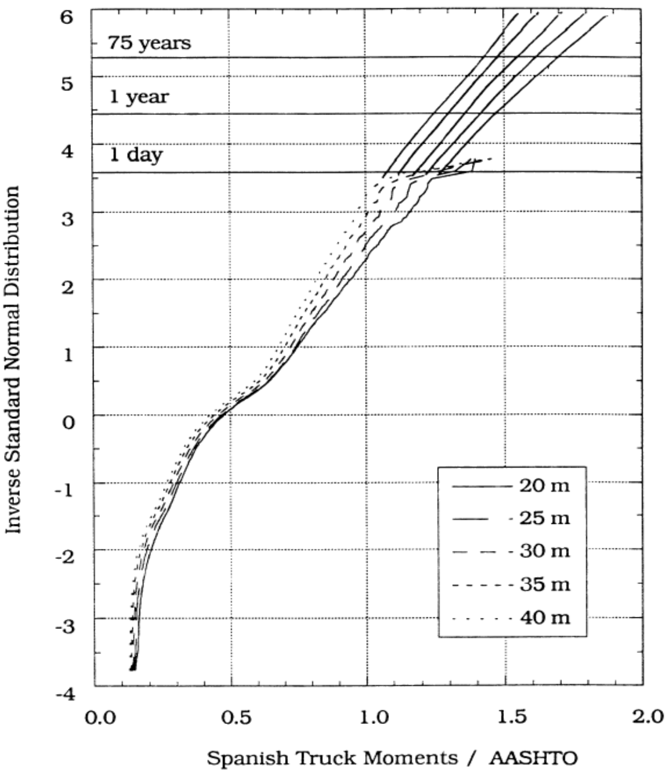


Figure 2-2 Moment Ratios Obtained from Spanish Truck Survey

Bias factors for the Ontario Truck Data and Spanish Truck Data are displayed in Figure 2-3. For the estimation of the reliability index, an iterative procedure has been followed as stated before. Reliability analysis were evaluated for both Ontario and Spanish trucks data as shown in Figure 2-4 and Figure 2-5.

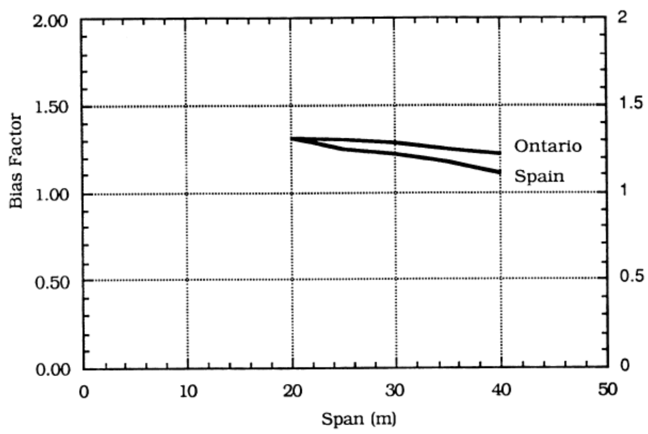


Figure 2-3 Bias Factors for the Moment per Girder for the Ontario Truck Data and Spanish Truck Data

The calculated reliability indices get changed considerably for the three different codes. The reliability indices (β) are varying between 7.0 and 8.0 for Eurocode, between 5.1 and 6.8 for Spanish Code, and between 4.5 and 4.9 for AASHTO. Based on these results, Eurocode is the most conservative specification and AASHTO LRFD is the most permissive provision. Furthermore, AASHTO LRFD ensures the most monotonous reliability level.

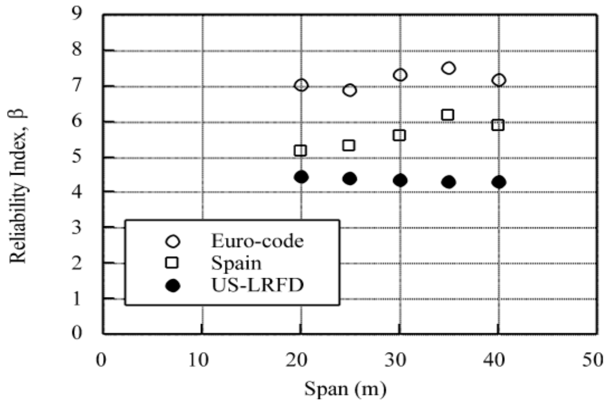


Fig. 10. Reliability indices for Ontario truck traffic.

Figure 2-4 Reliability Indices for Ontario Truck Traffic

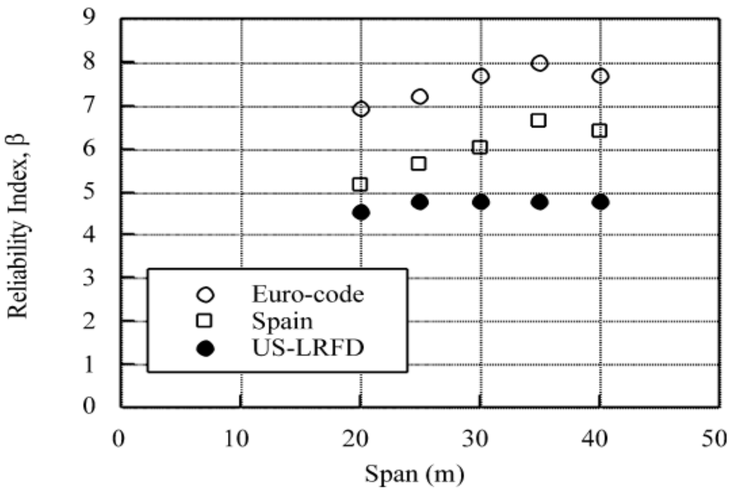


Fig. 11. Reliability indices for Spanish truck traffic.

Figure 2-5 Reliability Indices for Spanish Truck Traffic

Other studies reviewed in literature belong to the national sources. One of them is the Argınhan's (2010) thesis study namely "Reliability Based Safety Level of Turkish Type Precast Pre-Stressed Concrete Bridges Designed Accordance with LRFD". In the scope of this research, certain types of prestressed precast concrete girders with a length of 25 to 40 m were investigated. The statistical parameters of resistance and load were gathered from local database and existing literature studies. In order to determine the changes in the reliability index, various sets of resistance and load factors were used. For the live load models, Turkish live load, H30S24 and AASHTO LRFD live load, HL93 were taken as a basis for this model. Reliability indices were determined for each precast concrete girders. In Figure 2-6, reliability indices of girders for HL93 loading obtained by considering 15 various sets of resistance and load factors are shown.

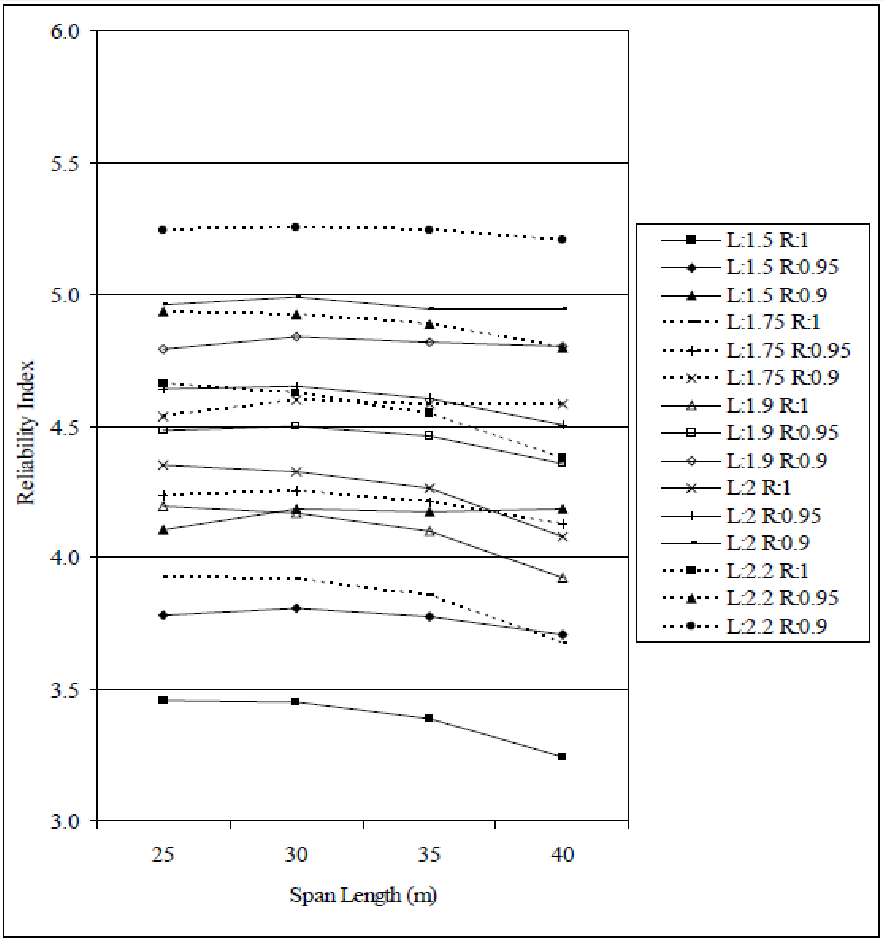


Figure 2-6 Reliability Indices for Different Sets of Load and Resistance Factors (HL93) (Argınhan, 2010)

By using the similar approach, Koç (2013) investigated reliability indices of steel bridges as thesis study which is namely "Calibration of Turkish LRFD Bridge Design Method for Slab on Steel Plate Girders". In his study, the bridges with span lengths of 50 to 80 m were used. Two types of truck loading namely HL-93 and AYK-45 determined as fundamental live loads. Later on, AYK-45 was renamed as KGM-45 by the Turkish General Directorate of Highways. In Table 2-1, reliability indices of girders for KGM-45 truck loading gained by considering 15 various sets of resistance and load factors are displayed.

Table 2-1 Reliability Indices for Different Sets of Load and Resistance Factors (KGM-45) (Koç, 2013)

Live Load (LL) and Resistance (R) Factors	Span Length (m)				Average
	50	60	70	80	β
LL: 1.50; R: 0.90	4.53	4.41	4.40	4.29	4.41
LL: 1.50; R: 0.95	4.31	4.14	4.11	3.99	4.14
LL: 1.50; R: 1.00	4.02	3.86	3.83	3.69	3.85
LL: 1.75; R: 0.90	4.89	4.74	4.74	4.60	4.74
LL: 1.75; R: 0.95	4.63	4.49	4.46	4.32	4.48
LL: 1.75; R: 1.00	4.39	4.23	4.19	4.03	4.21
LL: 2.00; R: 0.90	5.20	5.20	5.03	4.89	5.08
LL: 2.00; R: 0.95	4.97	4.80	4.77	4.62	4.79
LL: 2.00; R: 1.00	4.73	4.56	4.51	4.34	4.54
LL: 2.25; R: 0.90	5.47	5.48	5.30	5.14	5.35
LL: 2.25; R: 0.95	5.26	5.24	5.05	4.88	5.11
LL: 2.25; R: 1.00	5.03	4.84	4.80	4.62	4.82
LL: 2.50; R: 0.90	5.71	5.73	5.54	5.37	5.59
LL: 2.50; R: 0.95	5.51	5.50	5.30	5.12	5.36
LL: 2.50; R: 1.00	5.30	5.27	5.06	4.88	5.13

Dönmez (2015) investigated the reliability indices of the cable stayed bridges with span lengths of 420, 470, 520 and 550 meters. In his study, uncertainties of the construction and design methods in Turkey were examined. Statistical parameters for live load models were gathered by using the data of the General Directorate of Highways of Turkey. By defining the target reliability indices as 4.3, the resistance factor (Ø) was calibrated. In figures below, reliability index and the calibrated resistance factors for various live loads are illustrated.

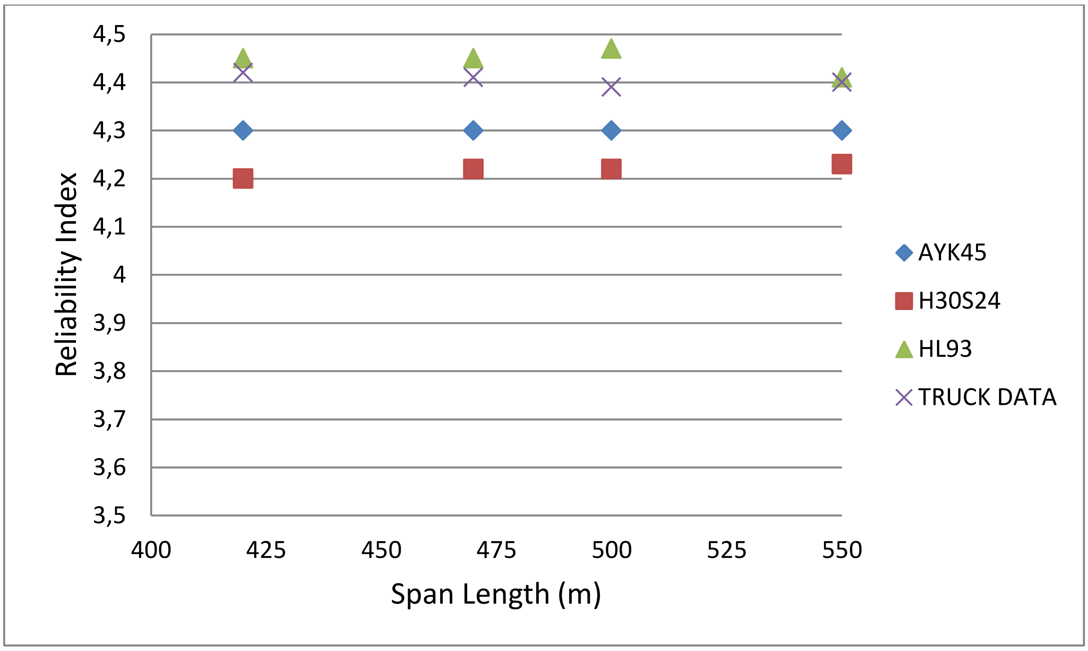


Figure 2-7 Reliability Indices Corresponding to Girder Design Performed for β_T=4.30 for KGM-45

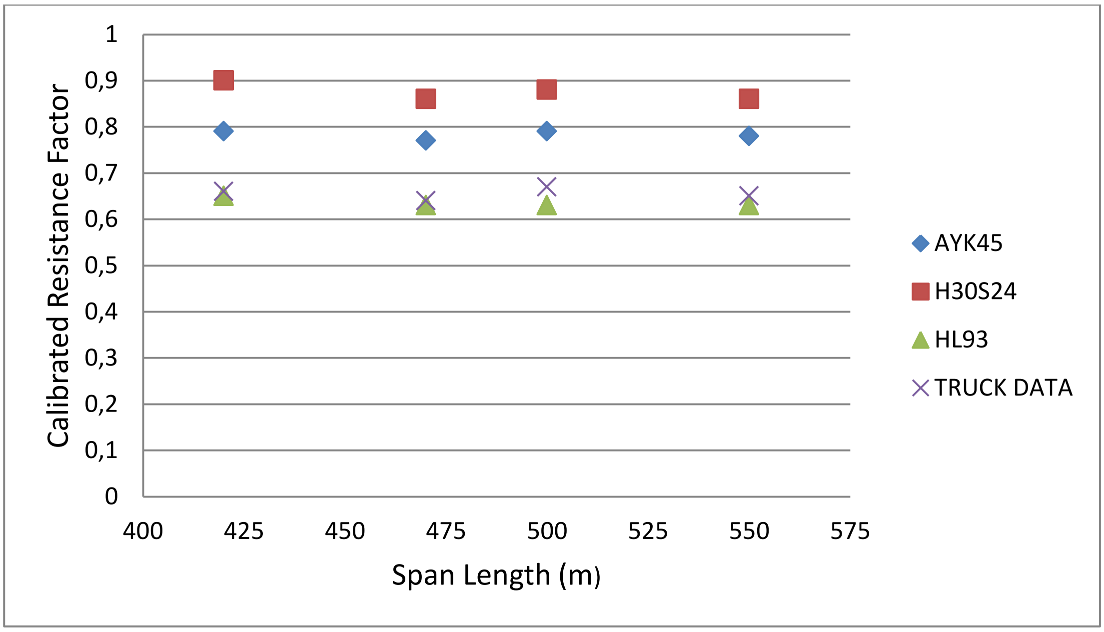


Figure 2-8 Calibrated Resistance Factor Corresponding to Girder Design Performed for β_T=4.30 for KGM-45

Çakır (2015) studied live load reliability indices for the balanced cantilever post-tensioned bridges with span lengths of 90, 120, 150 and 180 meters. Negative moment regions were grounded on this study since it is the most critical region for this type of bridges. KGM-45, H30-S24 and HL-93 truck loading were taken into account and target reliability index was defined as 4.5. As shown in figure 2-9, reliability indices of girders for KGM-45, HL-93 and H30-S24L are very close to each other. However, reliability indices of girder for H30-S24T truck are higher than the others because of the less force effects of this type truck on the fix designed bridge.

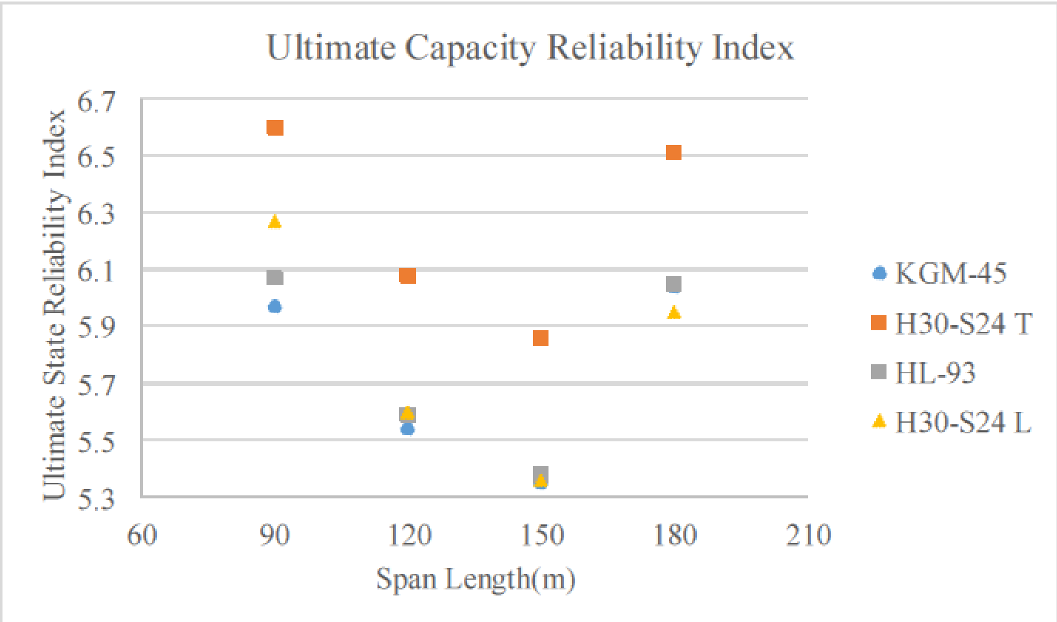


Figure 2-9 Reliability Indices Corresponding to Girder Design Performed for β_T=4.30 for KGM-45

CHAPTER 3

STATISTICS OF LOADS

In highway bridge design, the major design loads are the static and dynamic live loads, dead load, wind loads, earthquake loads, temperature loads and other loads that depend on traffic scenarios such as breaking and collision of vehicles. These loads occur throughout the service life of bridges. Therefore, in order to evaluate the reliability analysis of bridges, the statistical parameters of those loads must be taken into account. For the modeling of loads mainly available statistical data, surveys and observations have been mentioned in this study. The subject loads are randomly distributed and determined by taking account distribution of statistical parameters such as bias factor and coefficient of variation.

In this study, the reliability analysis of prestressed concrete girders has been fulfilled by using relevant AASHTO LRFD and EUROCODE load combinations.

3.1 AASHTO LRFD Load Combinations

In AASHTO LRFD, different load combinations are specified. For the design of prestressed concrete girders, Strength I and Service III limit state are generally used. Strength I limit state is a basic load combination relating with the normal vehicular usage of the bridge and used to determine the ultimate strength. On the other hand, Service III limit state is used for the crack check and to principal tension in the web of concrete girders (AASHTO LRFD 3.4.1).

Load combination of strength I limit state is shown as below:

$$Q = 1.25 \times DC + 1.50 \times DW + 1.75 \times LL \times (1 + IM) \times GDF \tag{3-1}$$

Load combination for Service III limit state is cited as following:

$$Q = DC + DW + 0.8 \times LL \times (1 + IM) \times GDF \tag{3-2}$$

where,

DC= Dead load of structural and non-structural components

DW= Dead load of wearing surface

LL= Vehicular live load

IM= Dynamic impact factor

GDF= Girder distribution factor

In this research, design of concrete girders is carried out with respect to these load combinations.

3.2 EUROCODE Load Combinations

European standards are basically gathered under ten different standards. The first standard was published at 1990 which is "Eurocode: Basis of Structural Design". Load combination rules for different bridges such as road bridges, footbridges and railway bridges were cited in this standard.

The combination of actions in ultimate limit states for structural members of road bridges is expressed as following (EN 1990 6.10a):

$$\sum_{j\geq 1}(\gamma_{G,j} G_{k,j}) \text{ "+" } \gamma_P P \text{ "+" } \gamma_{Q,1}\varphi_{0,1}Q_{k,1} \text{ "+" } \sum_{i>1}(\gamma_{Q,i}\varphi_{0,i}Q_{k,i}) \tag{3-3}$$

where,

γ_G is partial factors for permanent actions and considered as 1.35 (EN 1990 Table A2.4A).

γ_Q is partial factors for variable actions and taken as 1.35 (EN 1990 Table A2.4A).

γ_P is the value for prestressing actions and defined as 1.00 (EN 1990 Table A2.4A). $\varphi_{0,1}$ are the factors for the traffic loads and considered as 0.75 and 0.4 for relevant traffic loads (EN 1990 Table A2.1).

3.3 Dead Loads

In this study, dead loads are defined under four different components as shown below:

D_1 - Weight of factory made elements

D_2 - Weight of cast-in-place concrete

D_3 - Weight of wearing surface

D_4 - Weight of miscellaneous

For the AASHTO LRFD strength I limit state, the load factor of cast-in-place and factory made concrete elements is assigned to 1.25. The load factor of wearing surface and miscellaneous is set to 1.5. For the Eurocode ultimate limit state, since all the components must be considered under permanent actions and load factors are fixed to 1.35.

The statistical parameters of dead loads are obtained from Nowak's calibration report (1999). All the above mentioned variables related with the loads are assumed to be distributed normally and those parameters basically bias factor and coefficient of variation are shown below:

Table 3-1 Statistical Parameters of Dead Load

Component	Bias Factor	Coefficient of Variation
D_1	1.03	0.08
D_2	1.05	0.10
D_3	1.00	0.25
D_4	1.03-1.05	0.08-0.10

3.4 Live Load Models

In the scope of this study, two different live load models have been examined. One of them is KGM-45 loading which was taken from Turkish Highway Design Specifications. The other one is Eurocode Load Model 1. In addition to that the statistical parameters of live load models were calculated by using about 28.000 truck data that was taken from the Turkish General Directorate of Highways. In the forthcoming part of this section, above mentioned live load models and truck survey data are going to be discussed in detail.

3.2.1 KGM-45 Loading

KGM-45 is a new load model that was admitted by the Turkish General Directorate of Highways. This model includes both truck and lane loading. The number of "45" actually comes from the total weight of a heavy load truck in units of ton. This truck has a 3 axle with loads of 50 kN, 200 kN and 200 kN in turn. The distance between first and second axle is 4.25 meters and it is fixed. Furthermore, the distance between second and third axle is changing from 4.25 to 9.3 meters. This model also contains 10 kN/m uniform lane load. The subject model is illustrated in Figure 3-1.

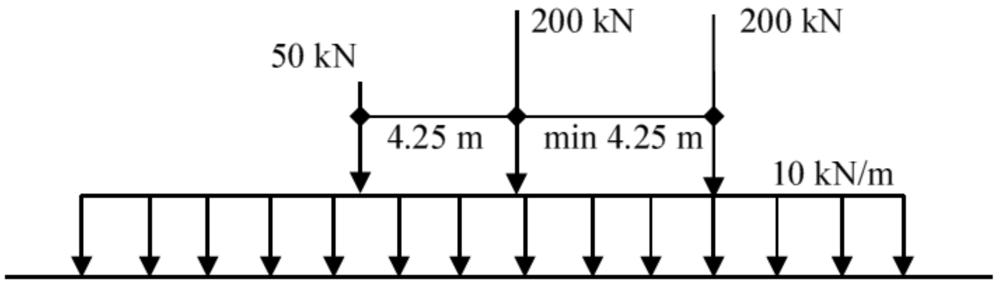


Figure 3-1 KGM-45 Live Load Model

3.2.2 Eurocode Load Model 1

Eurocode LM-1 was described in "Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges". Eurocode LM-1 loading consists of two partial systems which are double-axle concentrated loads and uniformly distributed load.

In double-axle concentrated loads which are named also as tandem system (TS), each axle weight is defined as $\alpha_Q Q_k$. In addition to this, α_Q is adjustment factor and Q_k is the axle.

The basic rules for the tandem system are,

- No more than one tandem system should be considered per lane.
- Tandem system should be taken into account completely.
- Each tandem system should be assumed to move centrally along the axes of lanes.
- Each axle of system considered as two identical wheels with a load of $0.5\alpha_Q Q_k$
- The wheel contact surface should be taken into account as 0.4 x 0.4 meter square.

Uniformly distributed load (UDL system) is described as $\alpha_q q_k$ per square meter for notional lane and α_q is defined as adjustment factor. The UDL loads should be determined only for the unfavorable parts of the influence surface. This application must be conducted both transversally and longitudinally.

LM-1 should be carried out on each notional lane and areas. The characteristic values of loads for each notional lane are shown in Table 3.2 (EN 1991-2:2003).

Table 3-2 Characteristic Values for Eurocode LM-1

Location	**Tandem system *TS***	***UDL* system**
	Axle loads Q_{ik} (kN)	AC1 q_{ik} **(or** q_{rk}) (kN/m^2) AC1
Lane Number 1	300	9
Lane Number 2	200	2,5
Lane Number 3	100	2,5
Other lanes	0	2,5
Remaining area (q_{rk})	0	2,5

The adjustment factors (α_Q, α_q) should be defined depending upon the traffic capacity and in the absence of this knowledge these factors should be taken as unity. In light of this information, adjustment factors are described as 1.0 for this study.

The details of Load Model 1 are displayed in Figure 3.2. In order to obtain the worst scenario for notional lanes, tandem system should be applied to most unfavorable location. If two tandem systems are considered on adjacent notional lanes, distance between the wheel axles should not decrease under the 0.5 meters. The location of tandem systems for this situation is shown in Figure 3.3.

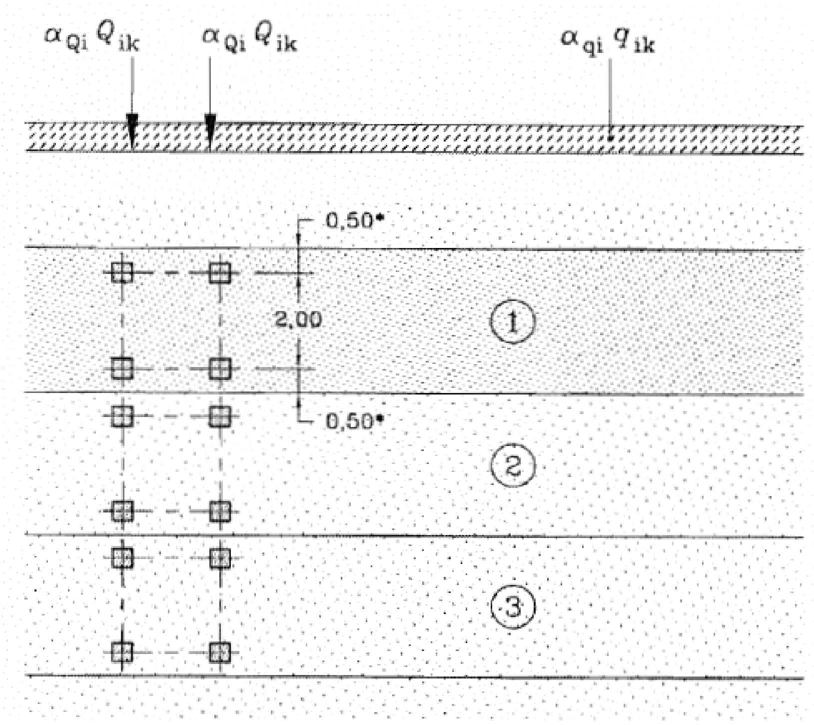


Figure 3-2 Application of LM-1 (EN 1991-2:2003)

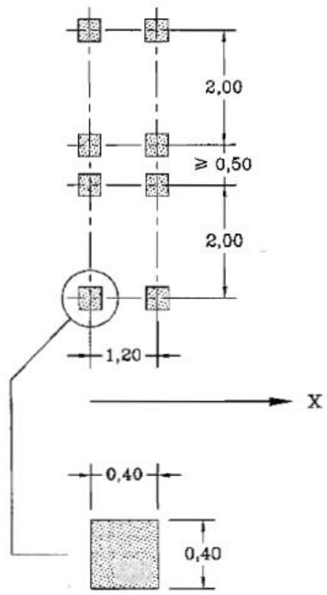


Figure 3-3 Application of tandem systems for LM-1 (EN 1991-2:2003)

3.2.3 Maximum Mid-Span Moments due to KGM-45 and Eurocode LM-1 Loading

Maximum mid-span moments under KGM-45 and Eurocode LM-1 load models have been calculated by using the moving load analysis. The span lengths of bridge were taken as 25, 30, 35 and 40 meters. LM-1 loading gives higher results comparing with KGM-45 loading for one lane as shown in Table 3-3. These moment values were considered for the determination of statistical parameters of live loads. Moments per lane values were taken into account for the design of girders which would be discussed in further chapter. In Figure 3-4, comparison of mid-span moments is illustrated as a bar graph.

Table 3-3 Maximum Moments due to Live Load Models for One Lanes

Span Length (m)	**Maximum Moment (kN.m)**	
	KGM-45	**Eurocode LM-1**
25	3027	5679
30	3933	7357
35	4902	9204
40	5933	11220

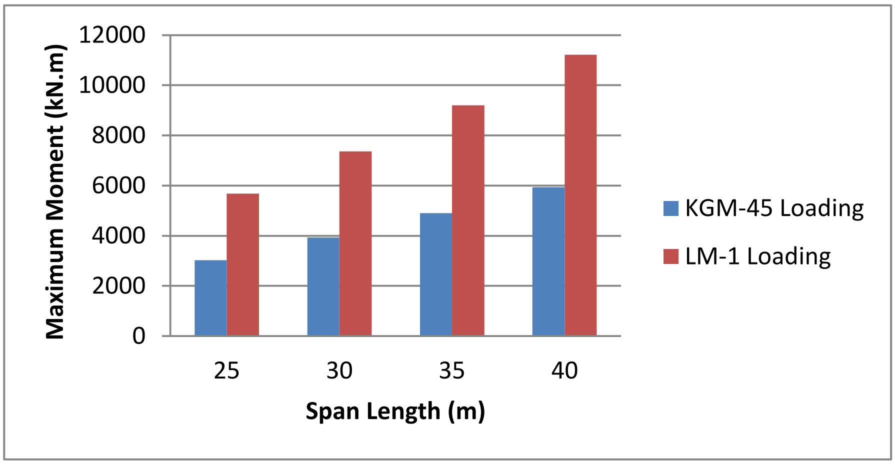


Figure 3-4 Comparison of Mid-Span Moments

3.2.4 Evaluation of Truck Survey Data

The truck survey data used to generate this live load model was taken from the Turkish General Directorate of Highways. This data belonging to about 28,000 truck measurements was gathered from various measurement stations belonging to highway in Turkey in 2005, 2006 and 2013. In Table 3-4, number and percentage of truck survey data are shown according to the axle count.

Table 3-4 Summary of Truck Survey Data

Axle Count	Number of Data	Percentage (%)
2 Axles	2905	10.4
3 Axles	15084	53.8
4 Axles	7351	26.2
5 Axles	2715	9.7
Total	**28055**	**100**

As illustrated in the above table, 3-axle trucks and 4-axle trucks frequently exist in real traffic comparing with 2-axle trucks and 5-axle trucks. In Figure 3-5, the distribution of surveyed truck is shown according to the axle configurations.

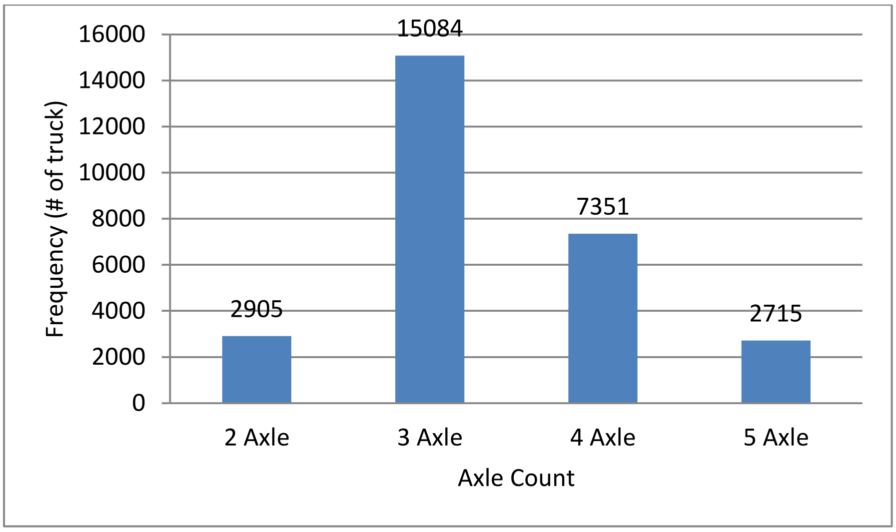


Figure 3-5 Histogram of Axle Configurations

The determination of gross weights of trucks is also important for the calculation of statistical parameters of live loads. Nearly half of the gross weights of surveyed truck data are in range of 6 tons and 18 tons. In Figure 3-6, the distribution of gross truck weights is presented.

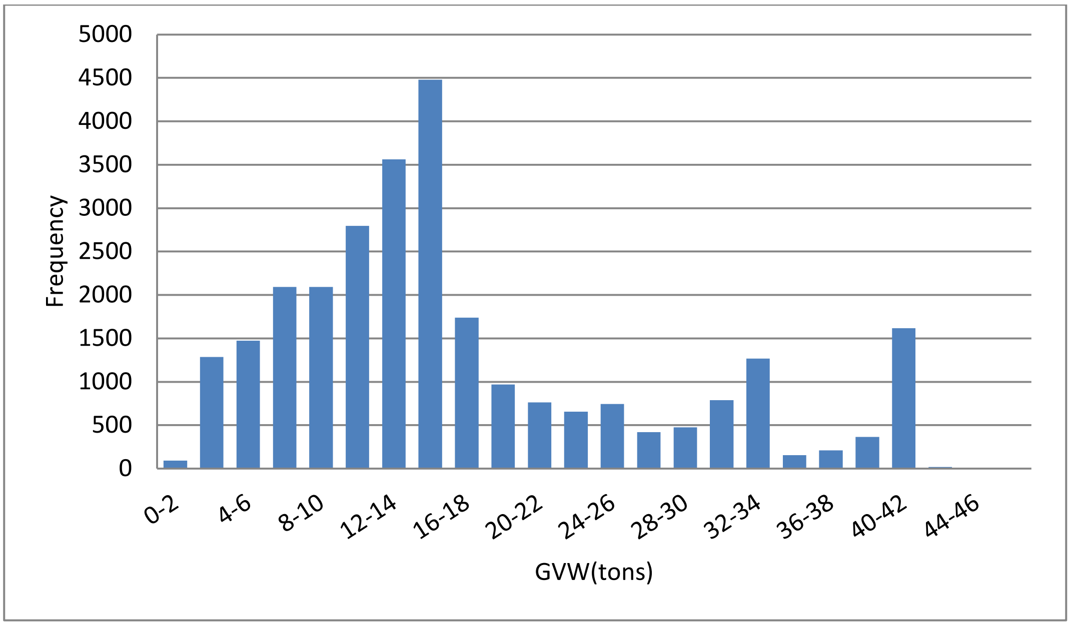


Figure 3-6 Histogram of Surveyed Trucks Weights

Maximum mid-span moments of each surveyed truck have been calculated in order to determine the statistical parameters of live load. The frequencies of moments are illustrated in Figure 3-7 to Figure 3-10.

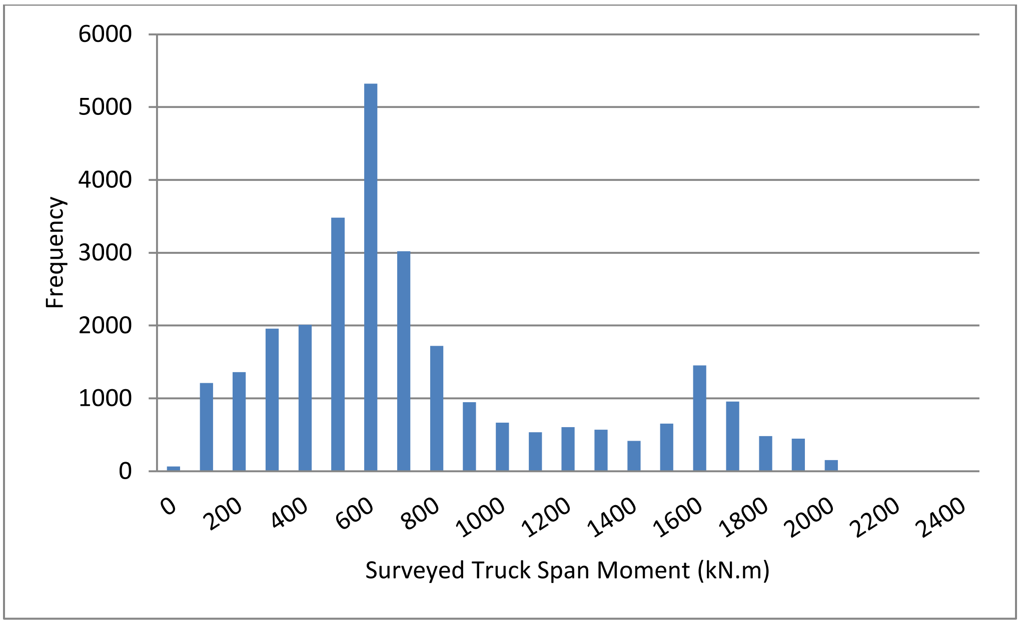


Figure 3-7 Histogram of Moments of Surveyed Trucks for 25 m Span Length

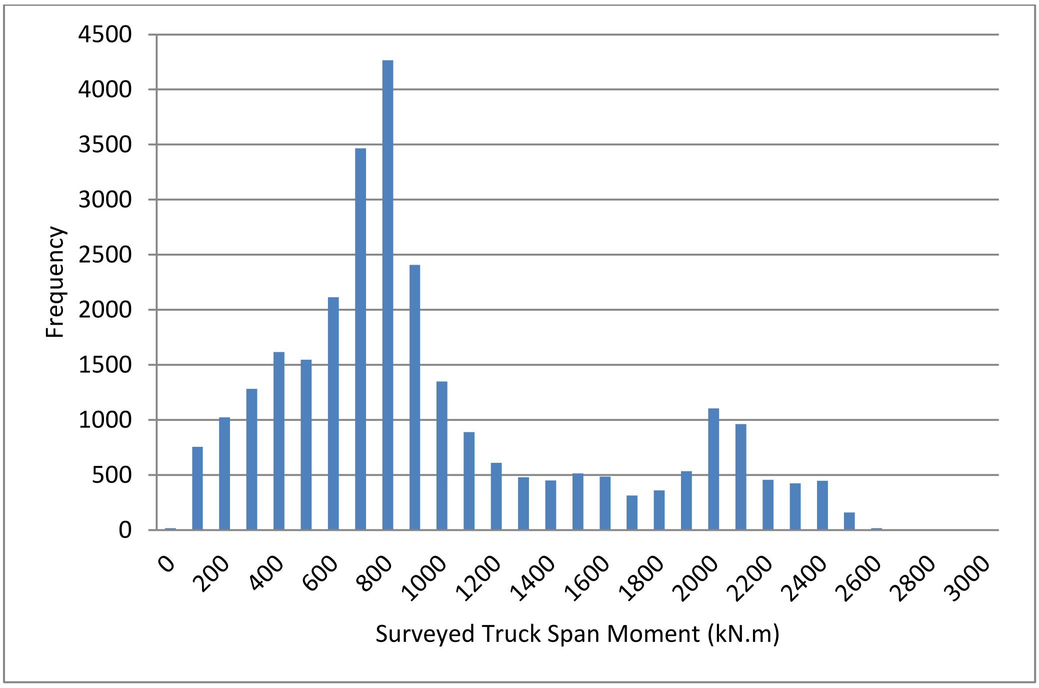


Figure 3-8 Histogram of Moments of Surveyed Trucks for 30 m Span Length

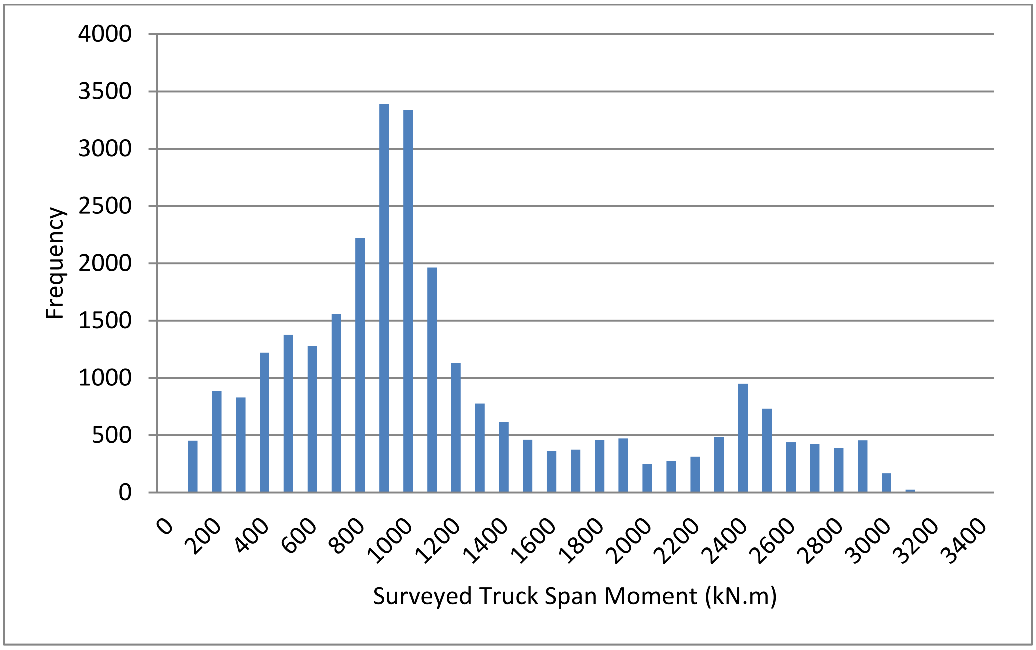


Figure 3-9 Histogram of Moments of Surveyed Trucks for 35 m Span Length

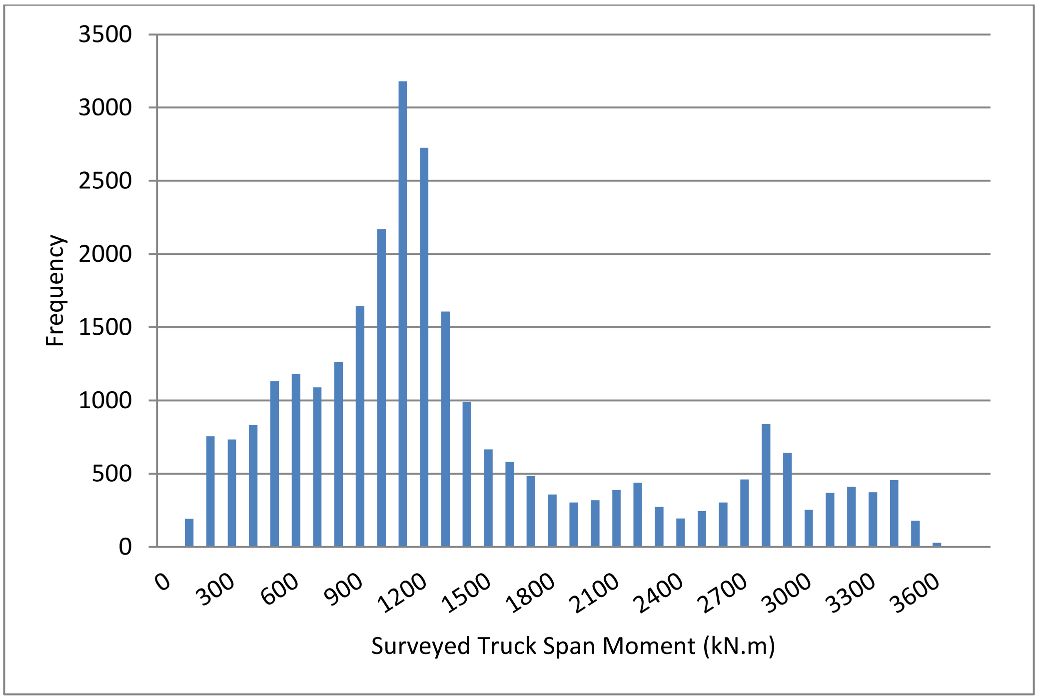


Figure 3-10 Histogram of Moments of Surveyed Trucks for 40 m Span Length

3.2.5 Assessment of Statistical Parameters of Live Load

In calibration of AASHTO LRFD, statistical parameters of live loads are evaluated by using extreme value theory. The basic idea of this theory is the estimation of future data by extending observed data to the longer periods. After this extrapolation, the aim is to get data that is more extreme than observed data.

For the assessment of statistical parameters of live load, three different cases have been taken under examination. The first one was the overall case which contains all the truck survey data. The second case was upper-tail that includes exceeding parts of 90% values of complete data. The last one was extreme case that considers isolated 10% highest values of data.

3.2.5.1 Fitting Straight Lines to the CDFs of Moments of Surveyed Trucks

For the above mentioned cases, the cumulative distribution functions of surveyed trucks to KGM-45 and Eurocode LM-1 loading are plotted on two types of probability papers such as normal and gumbel.

Moment ratios of surveyed truck data are plotted on normal probability papers and shown in Figure 3-11 to Figure 3-12. In these figures, vertical axis indicates the inverse of the standard normal distribution function (f) and defined as z;

$$z = \emptyset^{-1}[F(M)] \qquad (3\text{-}3)$$

where, F(M) is the cumulative distribution function of the mid-span moment (M) and $\emptyset^{-1}$ is the inverse standard normal distribution function. An inverse standard normal distribution is a way to find variables in terms of the distribution function F(x). In the subject figures, the horizontal axis indicates the mid-span moment ratio.

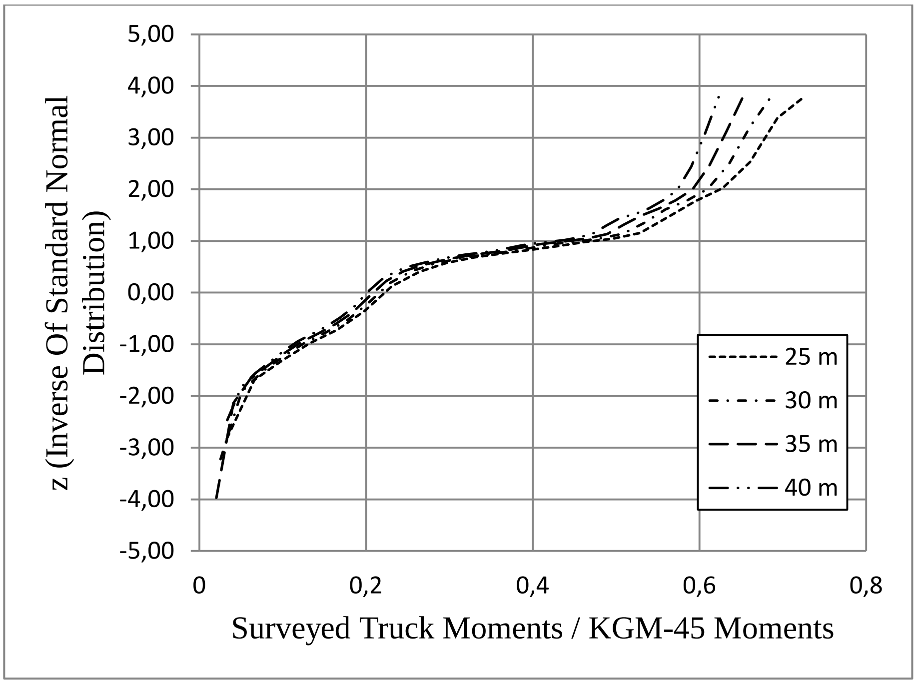


Figure 3-11 Moment Ratios of Surveyed Truck to KGM-45 on Normal Probability Paper

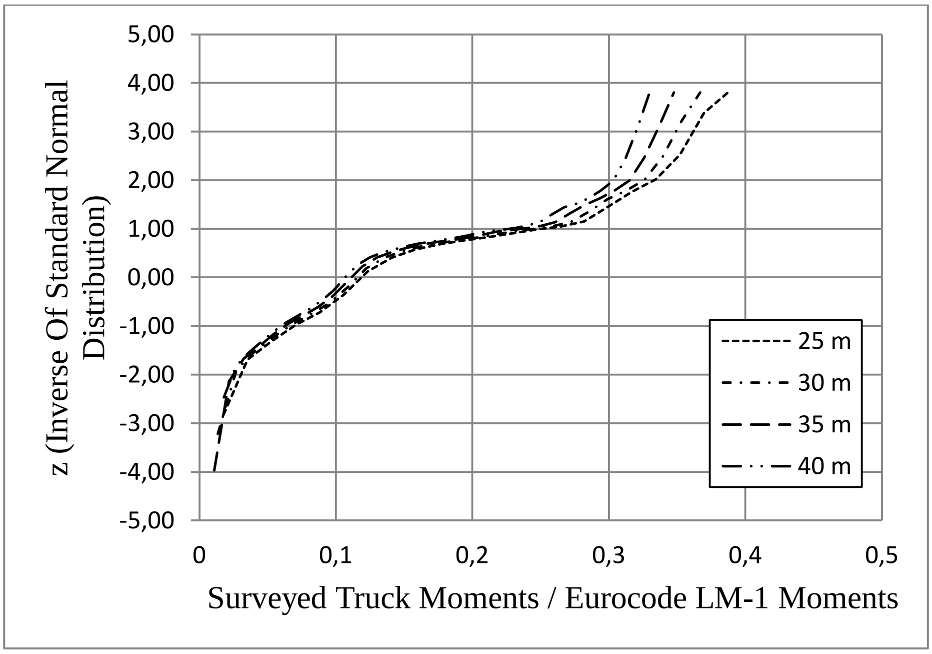


Figure 3-12 Moment Ratios of Surveyed Truck to Eurocode LM-1 on Normal Probability Paper

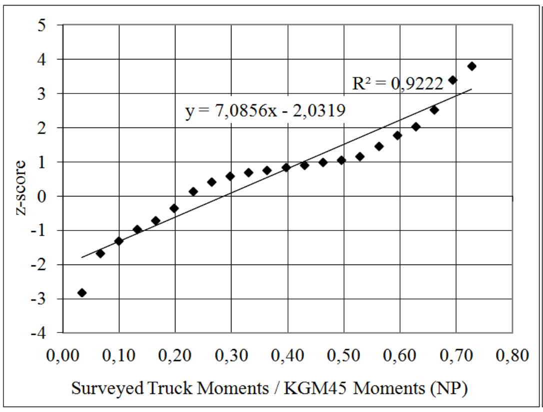
The other method studied in this study is Gumbel probability method which is used in the limited distribution of data. In the extreme value problems analysis, this method gives better results comparing with normal probability method. Different from the normal probability paper, vertical axis indicates the reduced variate (η) in Gumbel probability paper and is expressed as;

$$\eta = -\ln[-\ln[F(M)]] \tag{3-4}$$

where M is mid-span moment and F(M) is the cumulative distribution function of span moment. Besides, horizontal axis shows the moment ratio of the surveyed trucks to the KGM-45 and Eurocode LM-1 trucks.

In order to get the normal and Gumbel distribution function, the moment ratios of surveyed trucks to the live load models are plotted on normal and Gumbel probability papers for the three cases such as overall, upper-tail and extreme. After that, straight lines are fitted to the data points as shown in Figure 3-13 to Figure 3-36. These graphs show that there is no significant difference between two methods. Consequently, Gumbel probability method is preferred to be used in order to identify the statistical parameters belonging to the survey truck data.

The equations that are obtained from fitted straight lines on Gumbel probability papers will be used in the extrapolation of current data to the longer time periods. This extrapolation will be performed to estimate maximum live load effect in longer periods. Additionally, the equation constants will be used to describe the statistical parameters related with live loads.



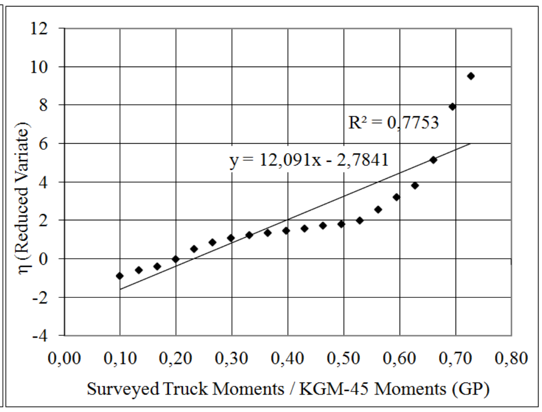
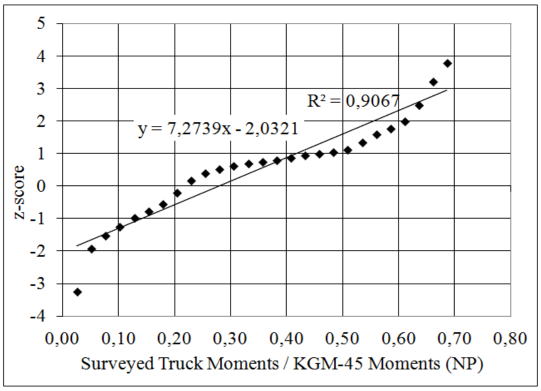


Figure 3-13 Straight Lines and Equations for Overall Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/25 m)



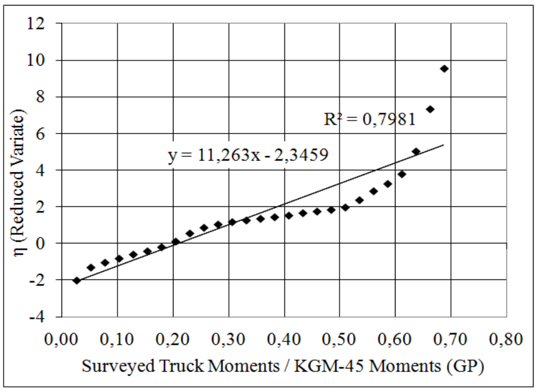
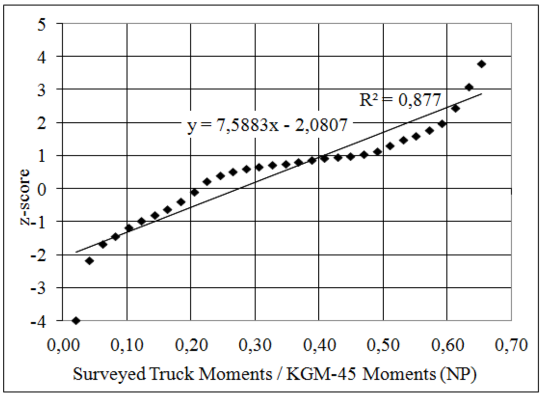


Figure 3-14 Straight Lines and Equations for Overall Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/30 m)



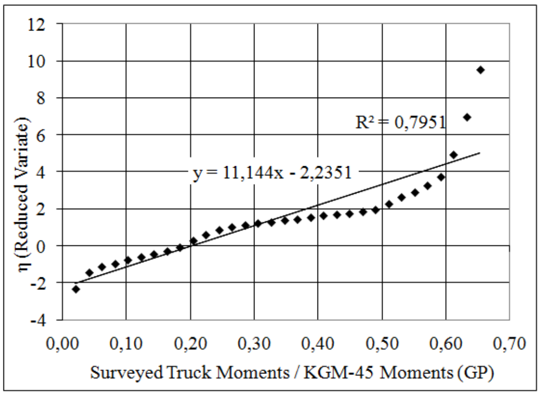


Figure 3-15 Straight Lines and Equations for Overall Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/35 m)

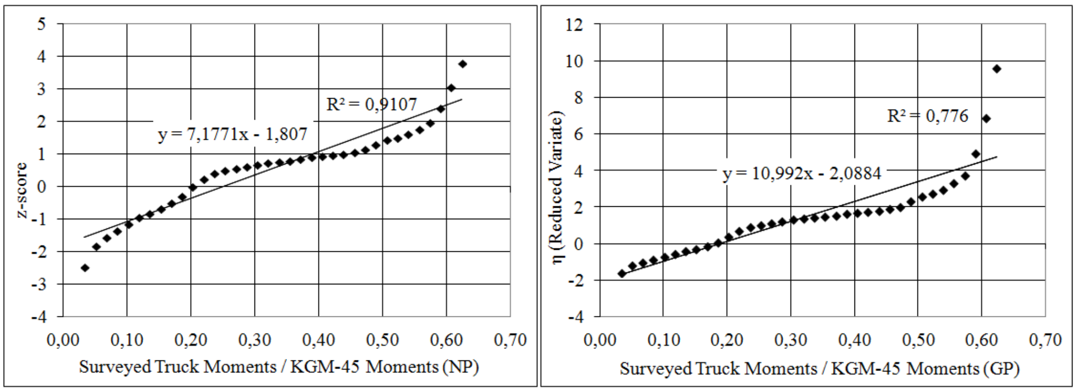


Figure 3-16 Straight Lines and Equations for Overall Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/40 m)

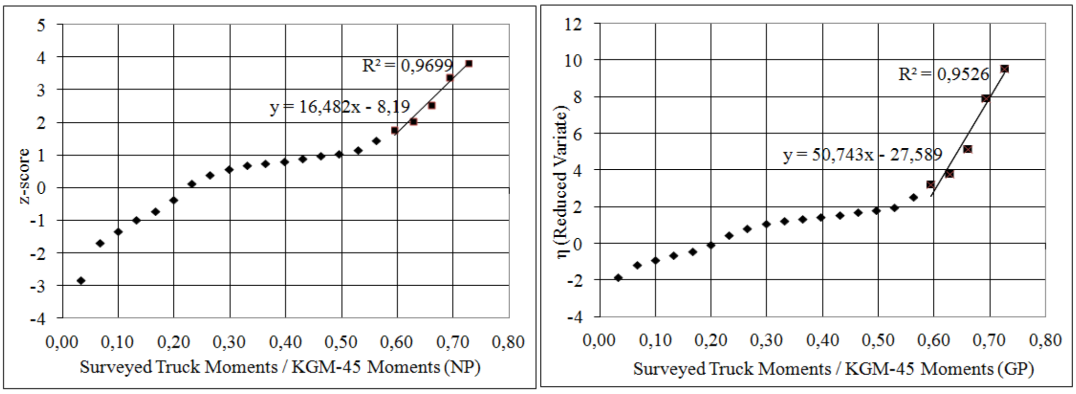


Figure 3-17 Straight Lines and Equations for Upper-Tail Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/25 m)

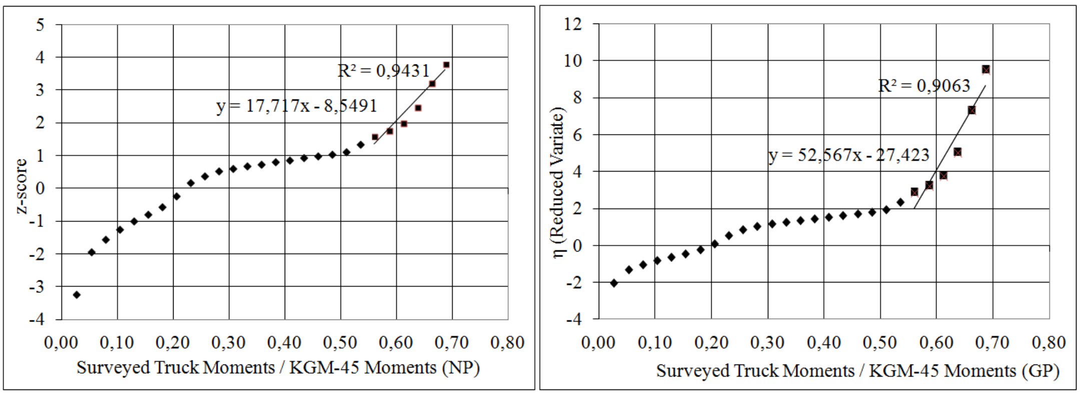


Figure 3-18 Straight Lines and Equations for Upper-Tail Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/30 m)

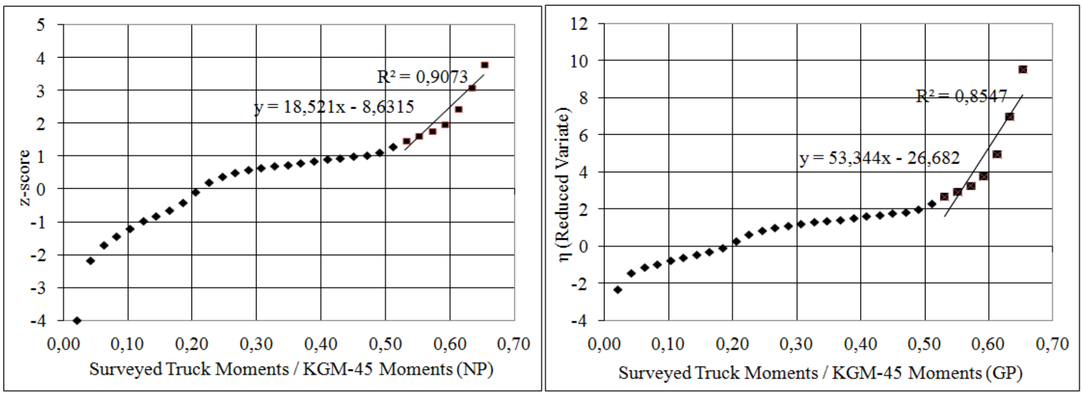


Figure 3-19 Straight Lines and Equations for Upper-Tail Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/35 m)

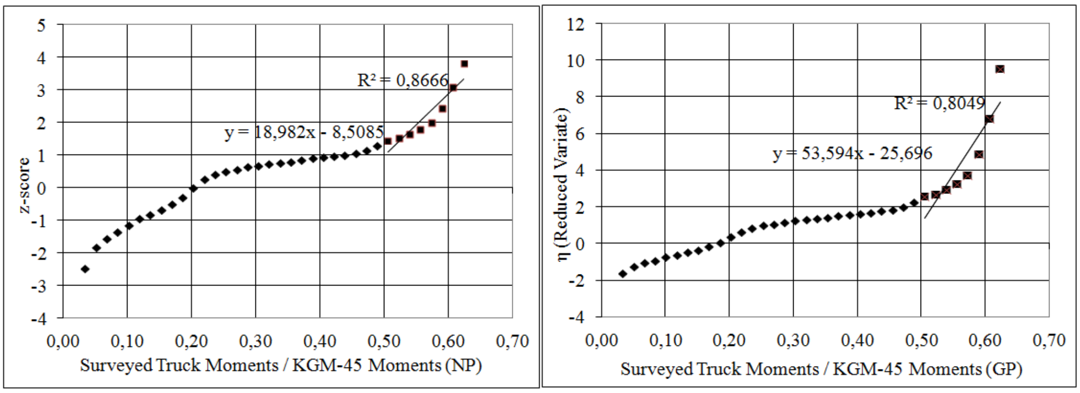


Figure 3-20 Straight Lines and Equations for Upper-Tail Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/40 m)

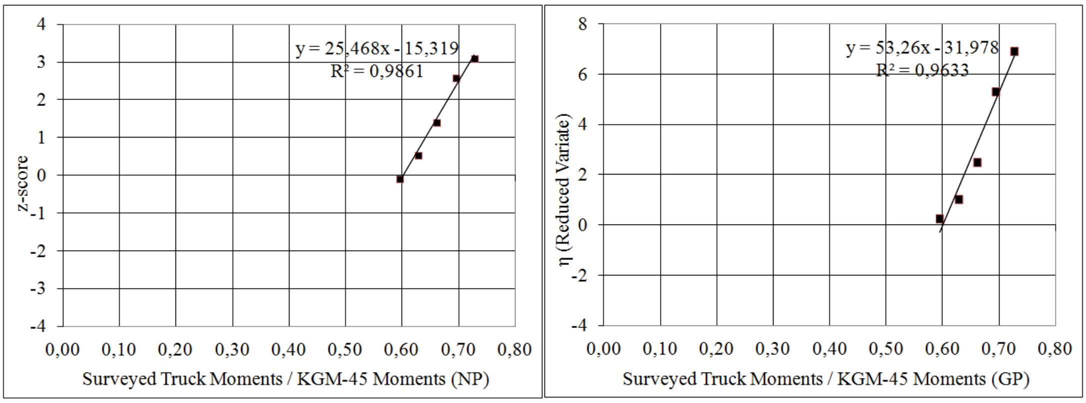


Figure 3-21 Straight Lines and Equations for Extreme Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/25 m)

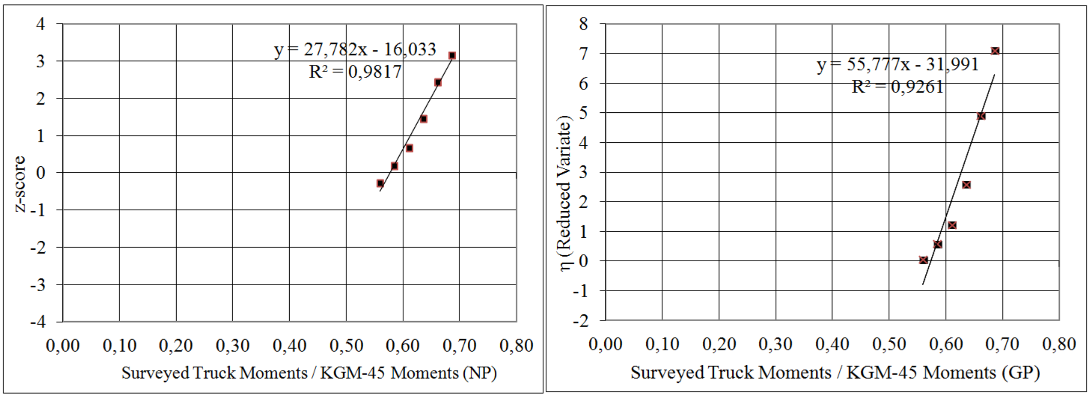


Figure 3-22 Straight Lines and Equations for Extreme Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/30 m)

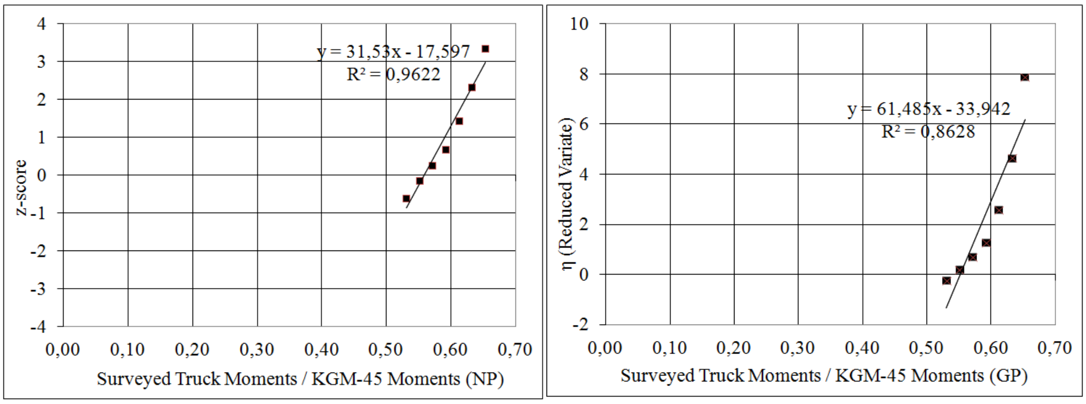


Figure 3-23 Straight Lines and Equations for Extreme Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/35 m)

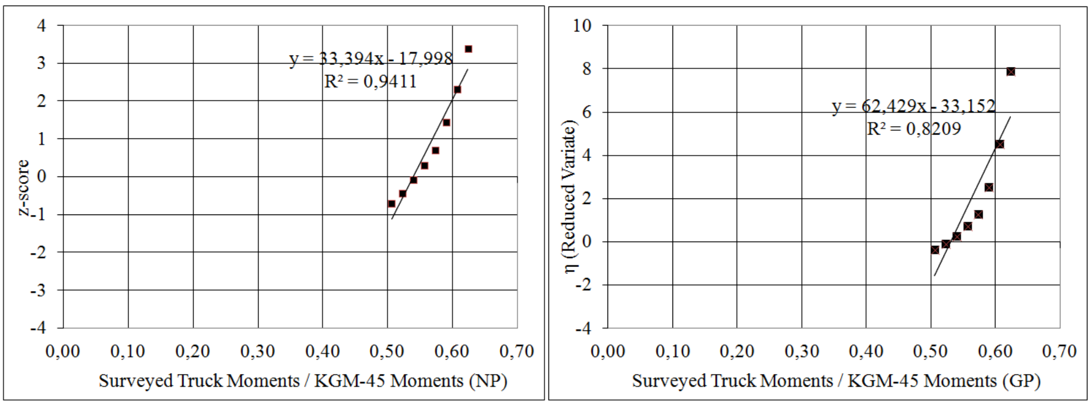


Figure 3-24 Straight Lines and Equations for Extreme Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (KGM-45/40 m)

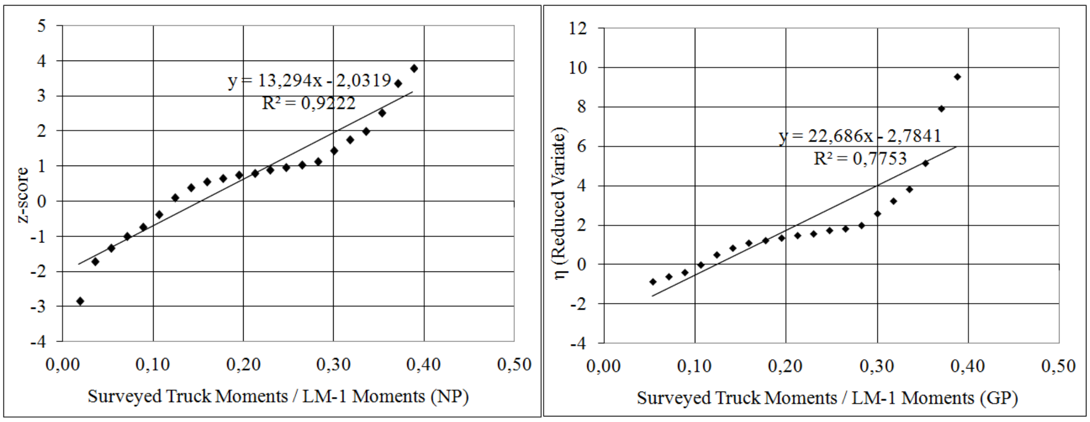


Figure 3-25 Straight Lines and Equations for Overall Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/25 m)

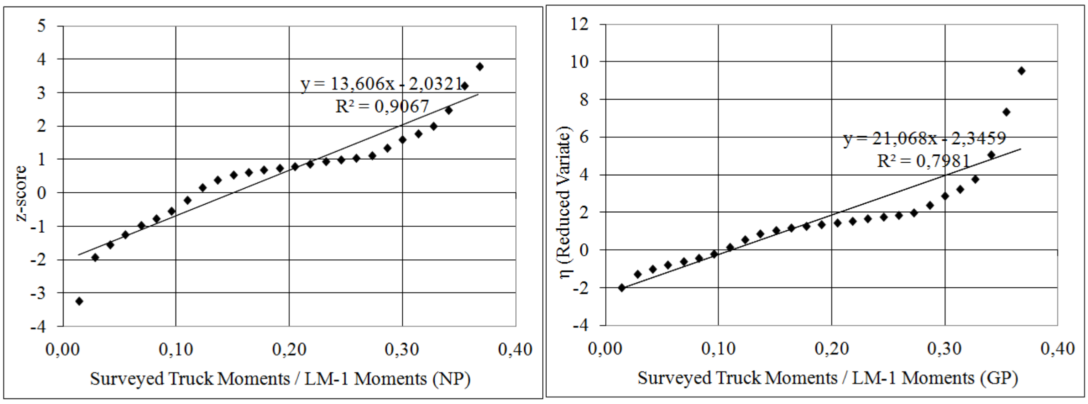


Figure 3-26 Straight Lines and Equations for Overall Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/30 m)

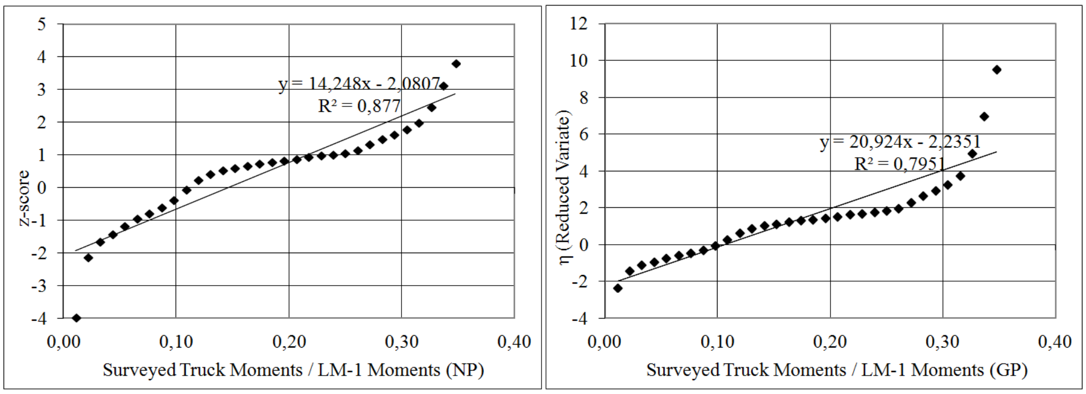


Figure 3-27 Straight Lines and Equations for Overall Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/35 m)

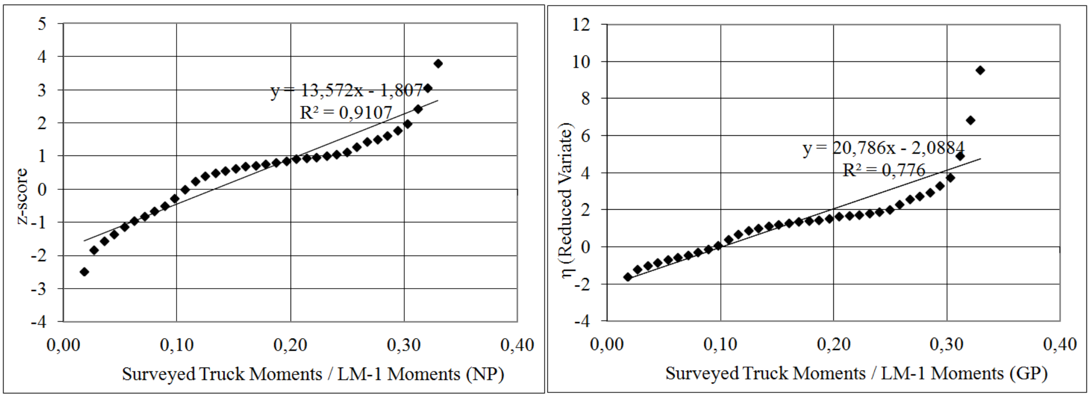


Figure 3-28 Straight Lines and Equations for Overall Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/40 m)

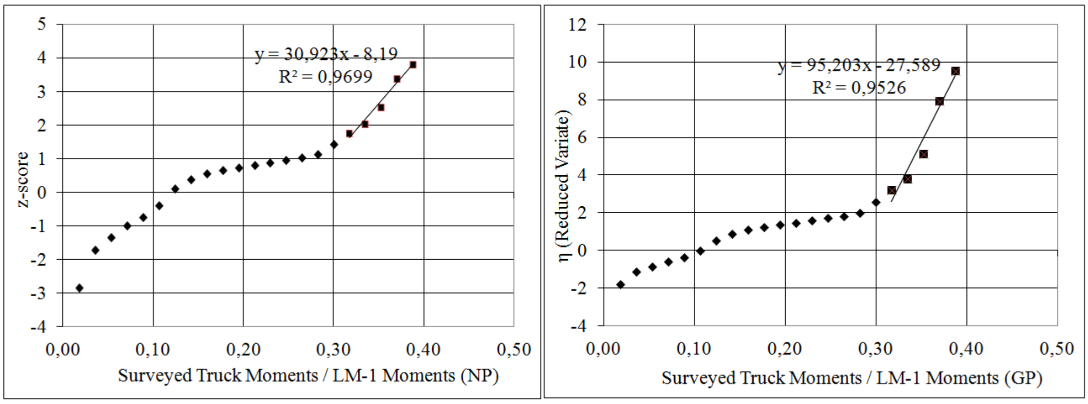


Figure 3-29 Straight Lines and Equations for Upper-Tail Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/25 m)

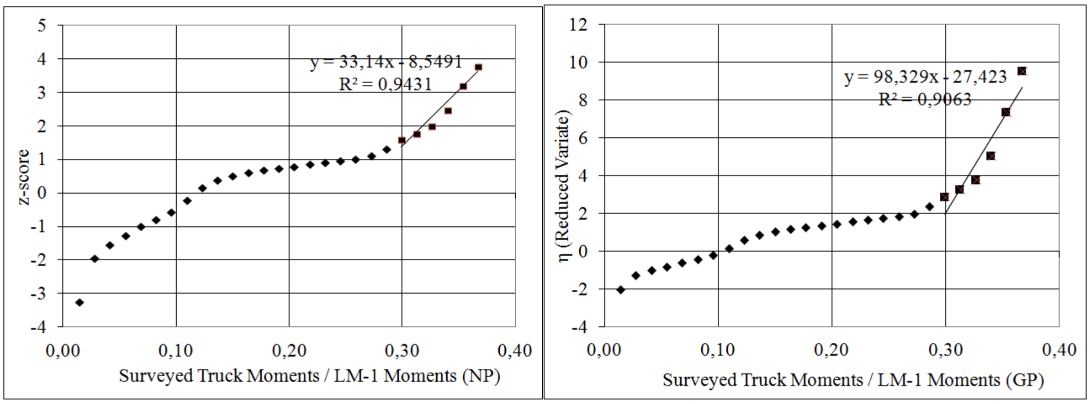


Figure 3-30 Straight Lines and Equations for Upper-Tail Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/30 m)

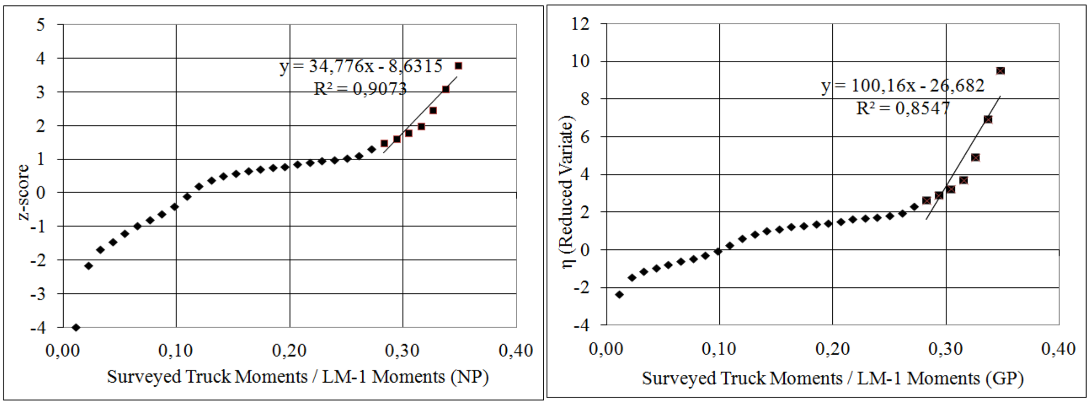


Figure 3-31 Straight Lines and Equations for Upper-Tail Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/35 m)

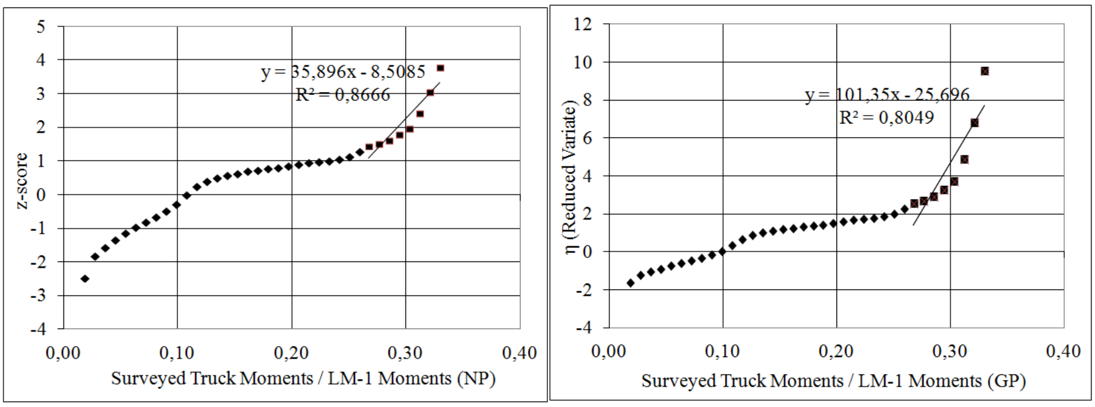


Figure 3-32 Straight Lines and Equations for Upper-Tail Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/40 m)

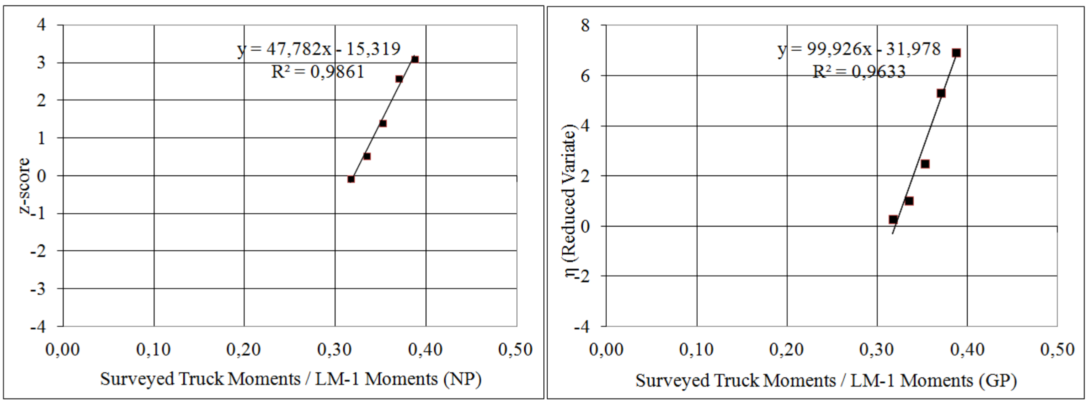


Figure 3-33 Straight Lines and Equations for Extreme Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/25 m)

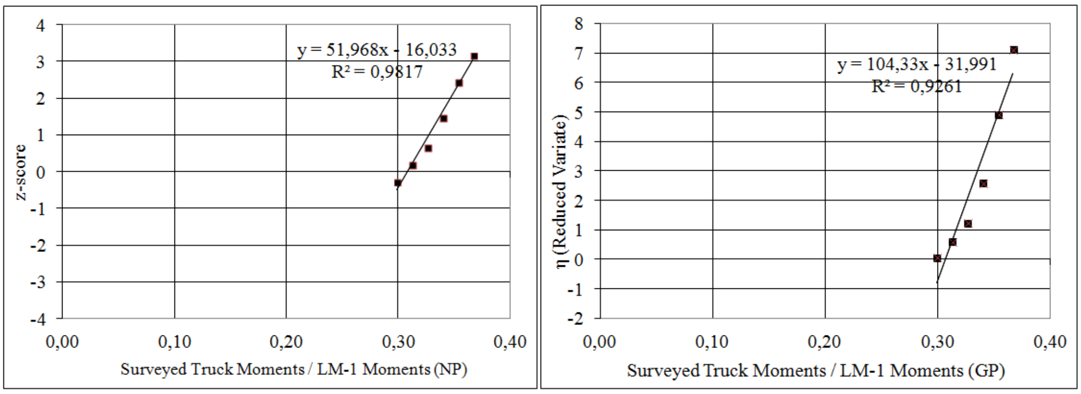


Figure 3-34 Straight Lines and Equations for Extreme Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/30 m)

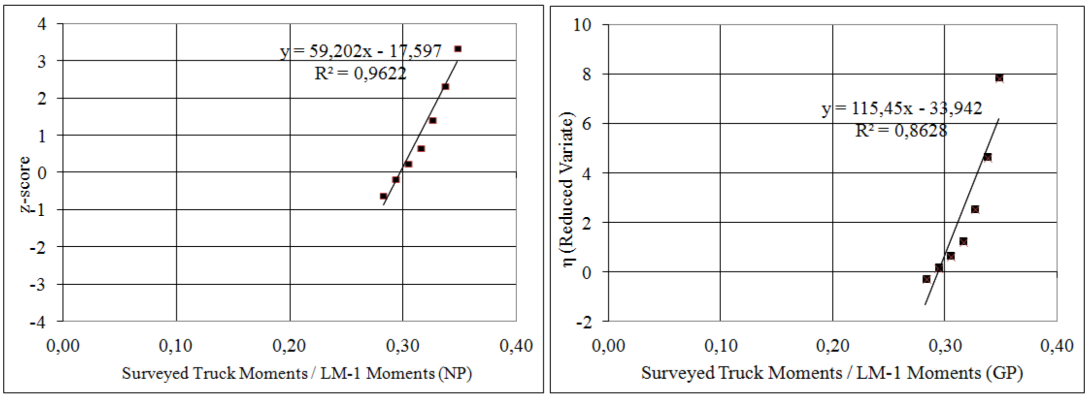


Figure 3-35 Straight Lines and Equations for Extreme Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/35 m)

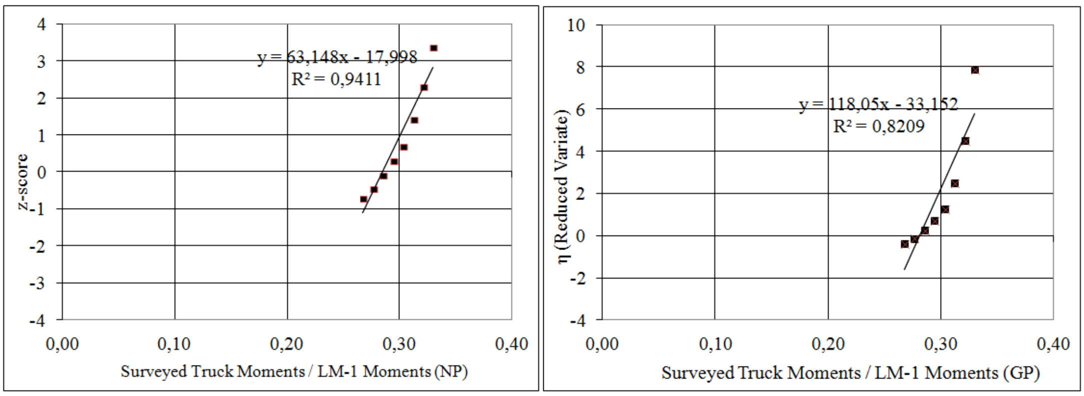


Figure 3-36 Straight Lines and Equations for Extreme Moment Ratios on Normal (NP) and on Gumbel (GP) Probability Papers (Eurocode LM-1/40 m)

3.2.5.2 Mean Maximum Moments Predicted by Extrapolation

In AASHTO LRFD, the return period of maximum live load is determined as 75 years. This time period also equals to the life time of a bridge. In order to get the live load effects in future, today's cumulative distribution functions are extrapolated to the 75 years. By using this projection, the maximum truck moments ratio for 1 day, 2 weeks, 1 month, 2 months, 6 months, 1 year, 5 years, 50 years and 75 years have been determined.

For extrapolation calculations, procedures and assumptions described in Nowak's calibration report have been taken as the basis. One of the main assumptions for extrapolation is that average daily traffic is about 1000 trucks. And the other assumption is that truck survey data shows heavy traffic on a bridge for two weeks.

The number of trucks, occurrence probabilities, inverse standard normal distribution and reduced variate values corresponding to the time periods are illustrated in Table 3-5.

Table 3-5 Time Period and Number of Trucks vs. Probability

Time Period T	# of Trucks N	Probability 1/N	ISND z	Reduced Variate η
75 years	20,000,000	$5x10^{-8}$	5.33	16.81
50 years	15,000,000	$7x10^{-8}$	5.27	16.52
5 years	1,500,000	$7x10^{-7}$	4.83	14.22
1 year	300,000	$3x10^{-6}$	4.50	12.61
6 months	150,000	$7x10^{-6}$	4.35	11.92
2 months	50,000	$2x10^{-5}$	4.11	10.82
1 month	30,000	$3x10^{-5}$	3.99	10.31
2 weeks	10,000	$1x10^{-4}$	3.72	9.21
1 day	1,000	$1x10^{-3}$	3.09	6.91

By solving the straight lines equations for Gumbel Distributions that are illustrated in Figure 3-37 to Figure 3-42, the extrapolated mean maximum moment ratios for the above mentioned time periods are determined. These extrapolated moment ratios of surveyed trucks to the KGM-45 and Eurocode LM-1 trucks are displayed and plotted in the below tables and figures.

Table 3-6 Mean Maximum Moment Ratios for KGM-45 (Overall)

Span (m)	**Surveyed Truck Moment / KGM-45 Moment**								
	1 day	**2 weeks**	**1 month**	**2 months**	**6 months**	**1 year**	**5 years**	**50 years**	**75 years**
25	0.80	0.99	1.08	1.13	1.22	1.27	1.41	1.60	1.62
30	0.82	1.03	1.12	1.17	1.27	1.33	1.47	1.68	1.70
35	0.82	1.03	1.13	1.17	1.27	1.33	1.48	1.68	1.71
40	0.82	1.03	1.13	1.17	1.27	1.34	1.48	1.69	1.72

Table 3-7 Mean Maximum Moment Ratios for KGM-45 (Upper-Tail)

Span (m)	**Surveyed Truck Moment / KGM-45 Moment**								
	1 day	**2 weeks**	**1 month**	**2 months**	**6 months**	**1 year**	**5 years**	**50 years**	**75 years**
25	0.68	0.73	0.75	0.76	0.78	0.79	0.82	0.87	0.88
30	0.65	0.70	0.72	0.73	0.75	0.76	0.79	0.84	0.84
35	0.63	0.67	0.69	0.70	0.72	0.74	0.77	0.81	0.82
40	0.61	0.65	0.67	0.68	0.70	0.71	0.74	0.79	0.79

Table 3-8 Mean Maximum Moment Ratios for KGM-45 (Extreme)

Span (m)	**Surveyed Truck Moment / KGM-45 Moment**								
	1 day	**2 weeks**	**1 month**	**2 months**	**6 months**	**1 year**	**5 years**	**50 years**	**75 years**
25	0.73	0.77	0.79	0.80	0.82	0.84	0.87	0.91	0.92
30	0.70	0.74	0.76	0.77	0.79	0.80	0.83	0.87	0.87
35	0.66	0.70	0.72	0.73	0.75	0.76	0.78	0.82	0.83
40	0.64	0.68	0.70	0.70	0.72	0.73	0.76	0.80	0.80

Table 3-9 Mean Maximum Moment Ratios for Eurocode LM-1 (Overall)

Span (m)	Surveyed Truck Moment / Eurocode LM-1 Moment								
	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
25	0.43	0.53	0.58	0.60	0.65	0.68	0.75	0.85	0.86
30	0.44	0.55	0.60	0.63	0.68	0.71	0.79	0.90	0.91
35	0.44	0.55	0.60	0.62	0.68	0.71	0.79	0.90	0.91
40	0.43	0.54	0.60	0.62	0.67	0.71	0.78	0.90	0.91

Table 3-10 Mean Maximum Moment Ratios for Eurocode LM-1 (Upper-Tail)

Span (m)	Surveyed Truck Moment / Eurocode LM-1 Moment								
	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
25	0.36	0.39	0.40	0.40	0.41	0.42	0.44	0.46	0.47
30	0.35	0.37	0.38	0.39	0.40	0.41	0.42	0.45	0.45
35	0.34	0.36	0.37	0.37	0.39	0.39	0.41	0.43	0.43
40	0.32	0.34	0.36	0.36	0.37	0.38	0.39	0.42	0.42

Table 3-11 Mean Maximum Moment Ratios for Eurocode LM-1 (Extreme)

Span (m)	Surveyed Truck Moment / Eurocode LM-1 Moment								
	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
25	0.39	0.41	0.42	0.43	0.44	0.45	0.46	0.49	0.49
30	0.37	0.39	0.41	0.41	0.42	0.43	0.44	0.47	0.47
35	0.35	0.37	0.38	0.39	0.40	0.40	0.42	0.44	0.44
40	0.34	0.36	0.37	0.37	0.38	0.39	0.40	0.42	0.42

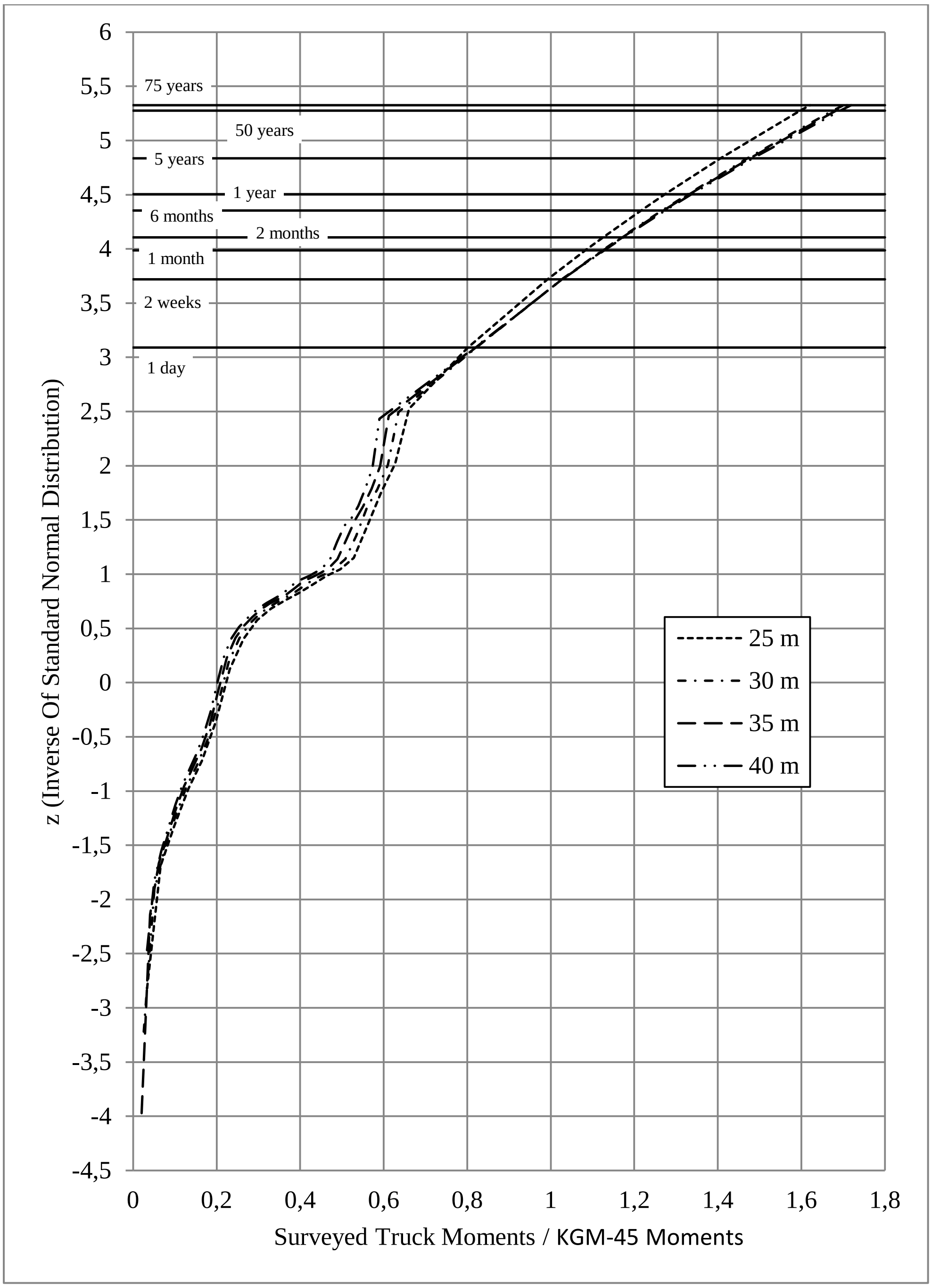


Figure 3-37 Extrapolated Moment Ratios for KGM-45 (Overall)

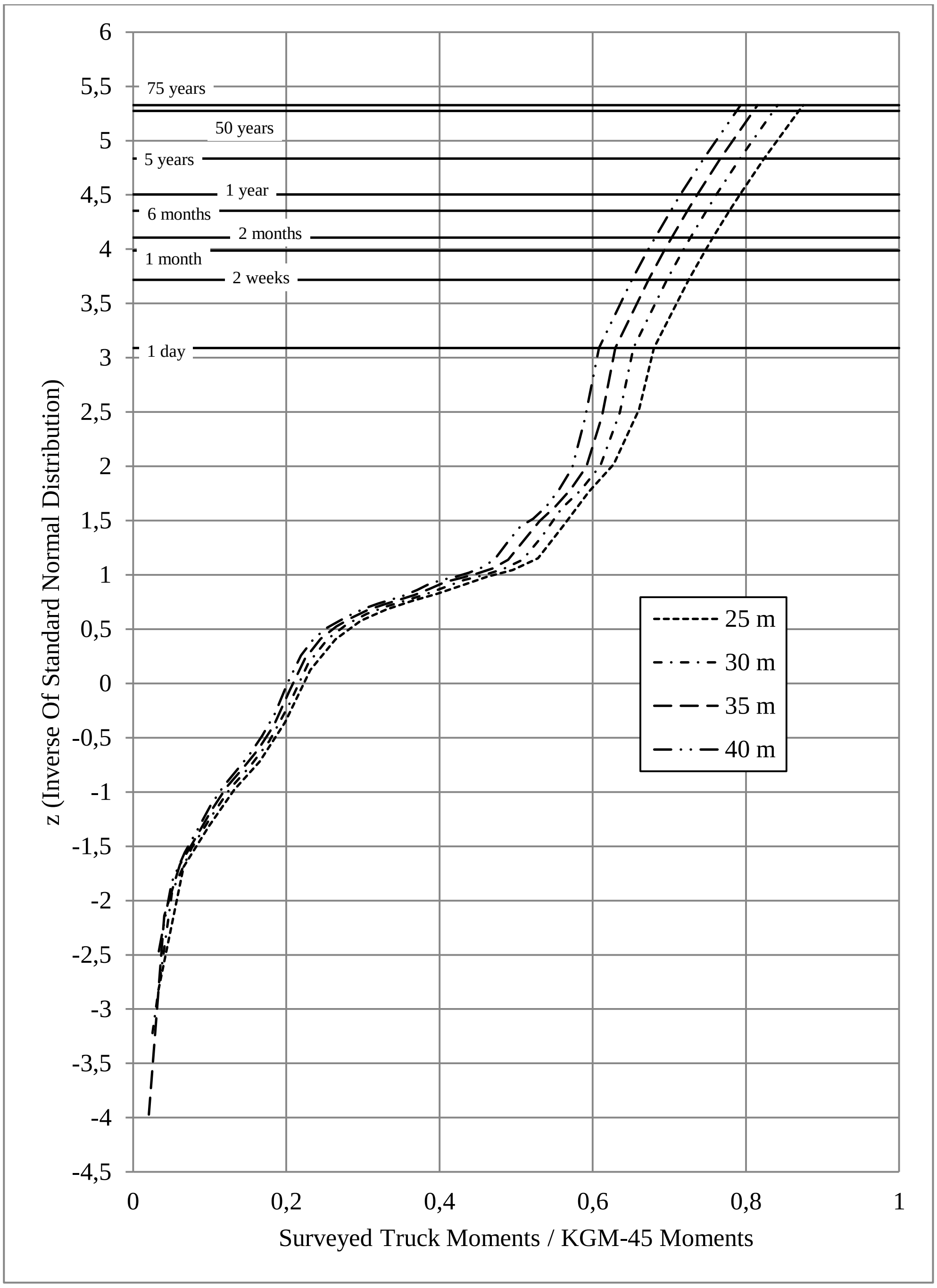


Figure 3-38 Extrapolated Moment Ratios for KGM-45 (Upper-tail)

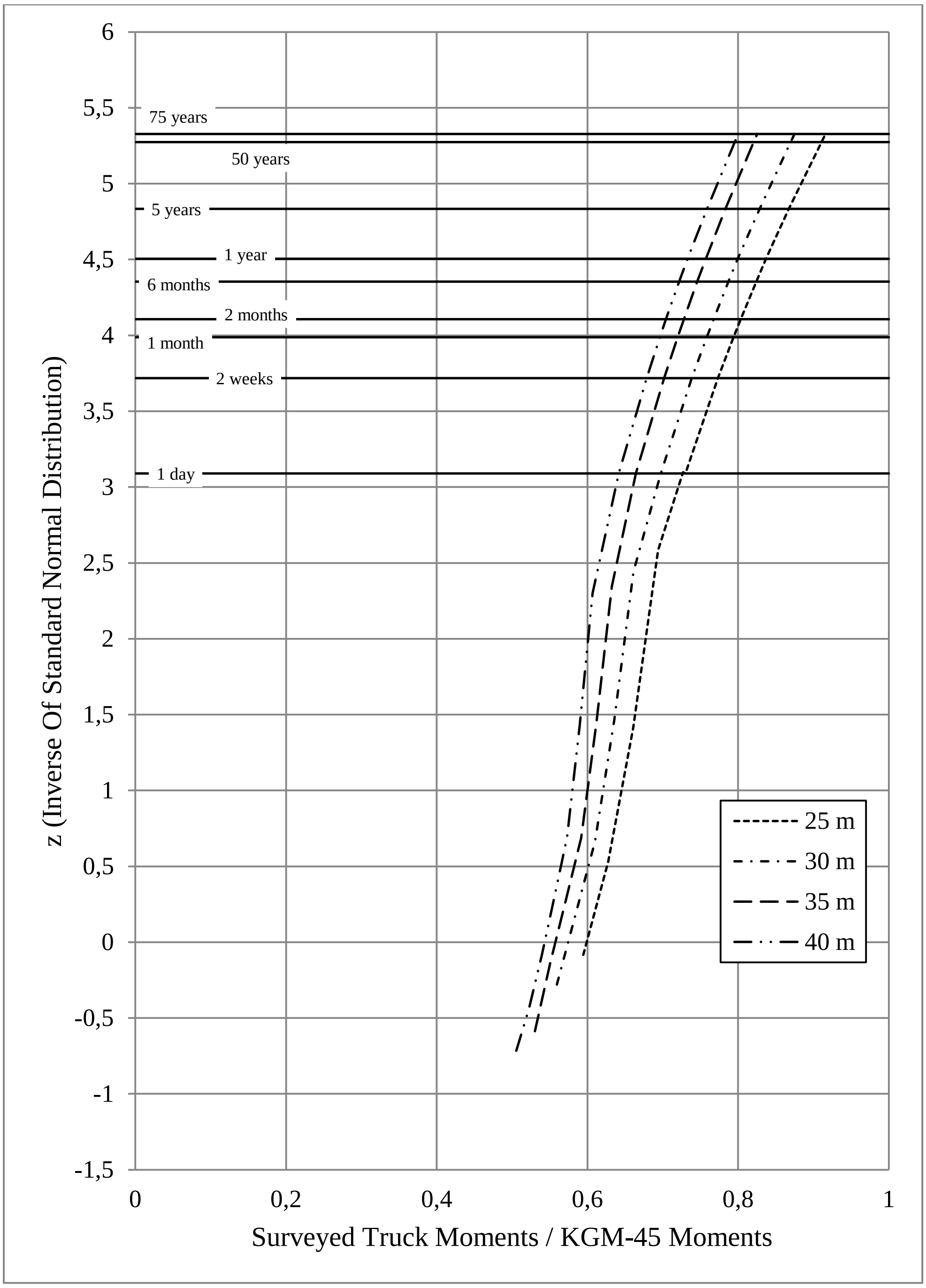


Figure 3-39 Extrapolated Moment Ratios for KGM-45 (Extreme)

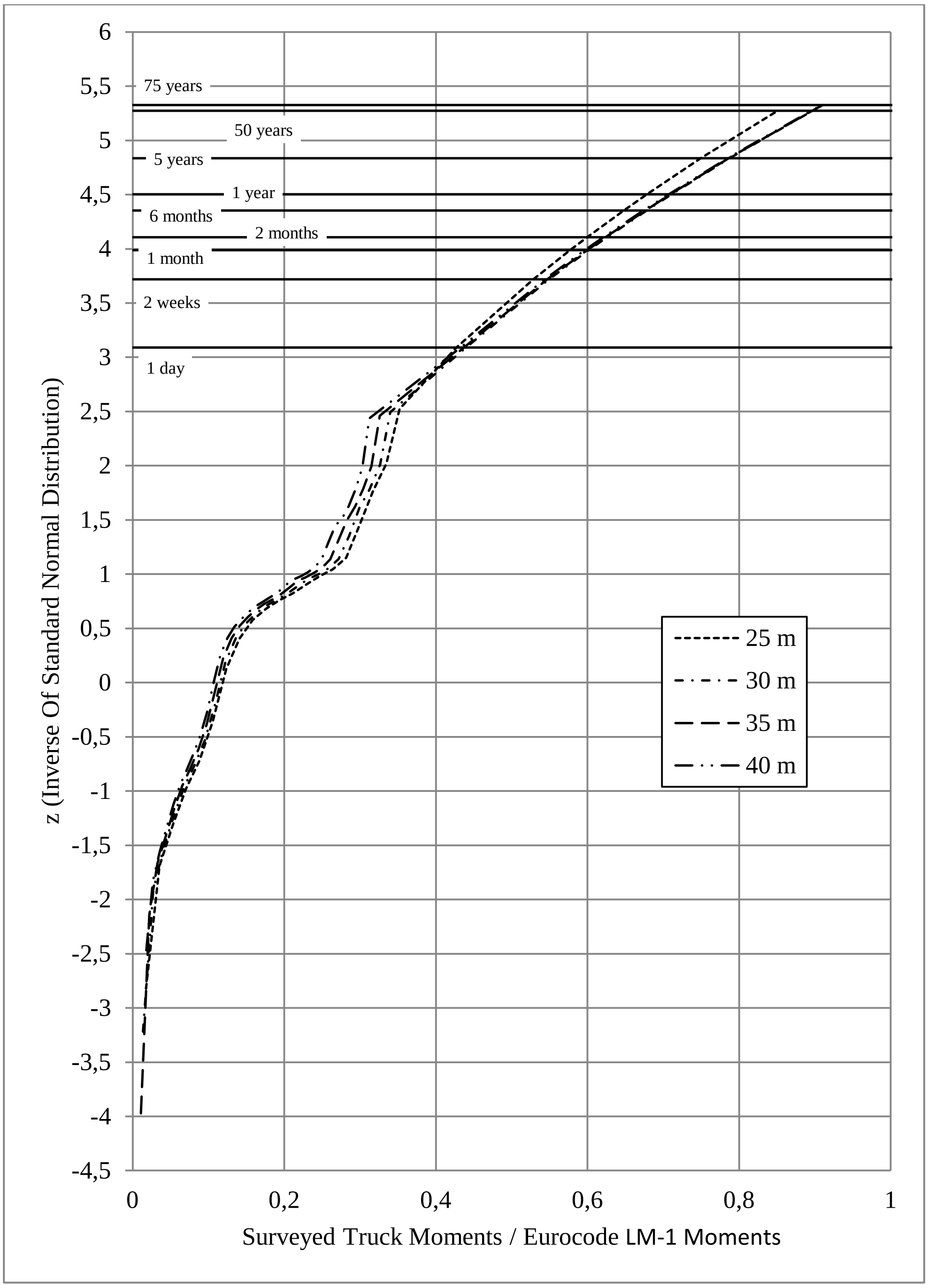


Figure 3-40 Extrapolated Moment Ratios for Eurocode LM-1 (Overall)

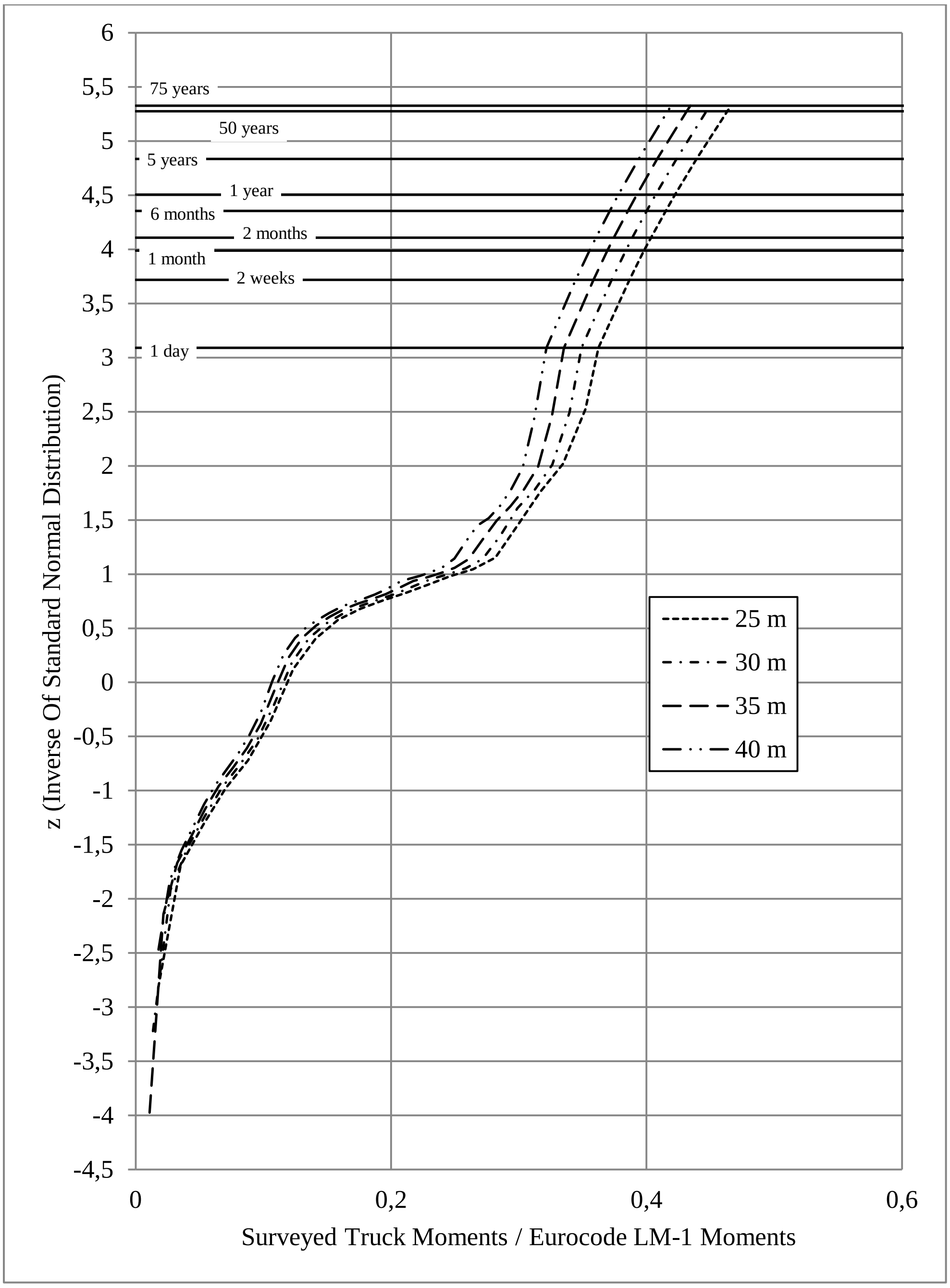


Figure 3-41 Extrapolated Moment Ratios for Eurocode LM-1 (Upper-tail)

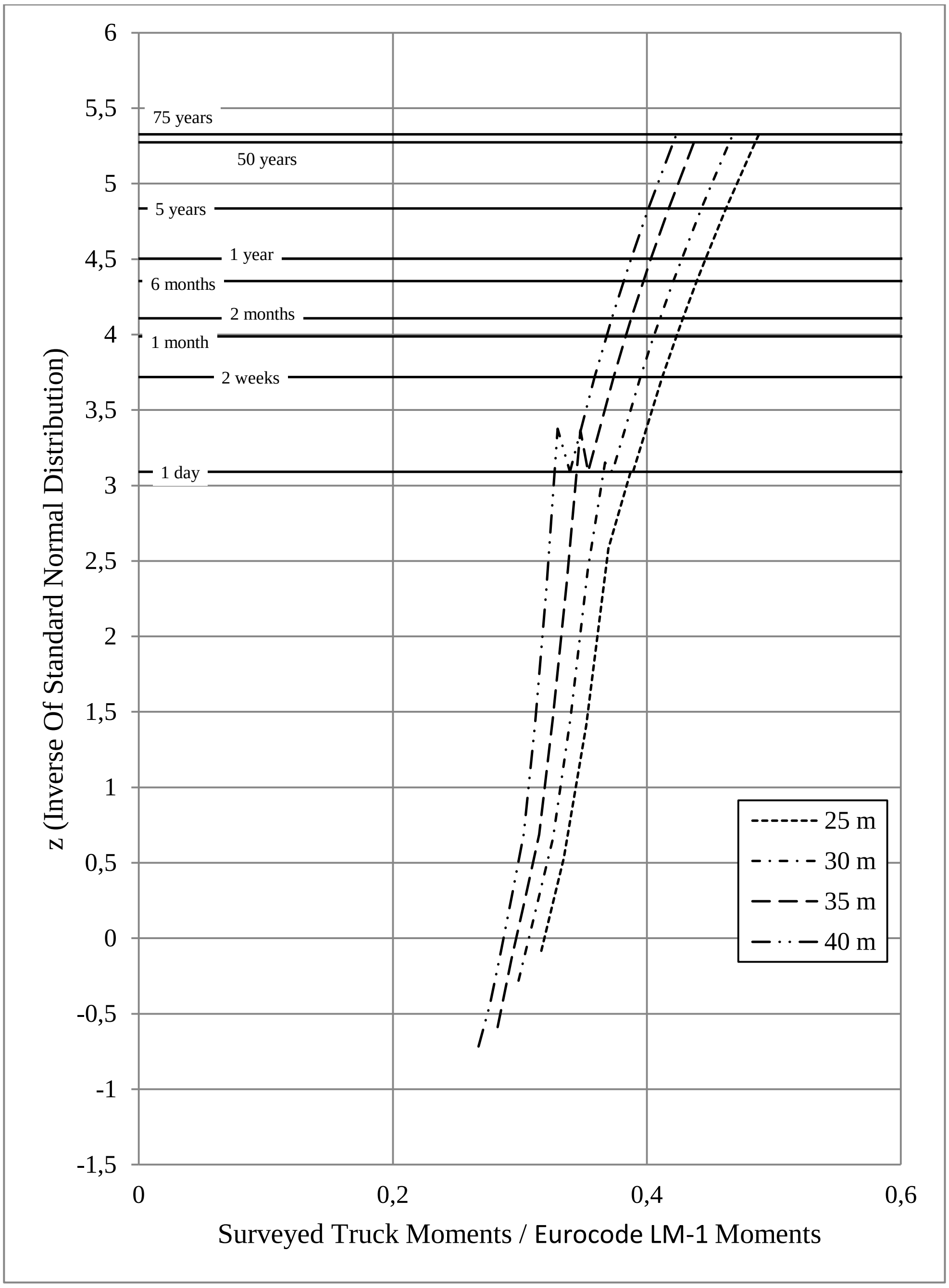


Figure 3-42 Extrapolated Moment Ratios for Eurocode LM-1 (Extreme)

3.2.5.3 Calculation of the Coefficient of Variation

The coefficients of variation for live loads have been calculated by using the Gumbel distribution method based on the truck survey data and a straight line fitted to these data points. The coefficients of variation are estimated by using the formulas below.

$$\mu = \frac{\sum M_i}{N} \quad , \quad \sigma^2 = \frac{\sum (M_i - \mu)^2}{N-1} \quad \rightarrow \quad COV = \frac{\sigma}{\mu} \qquad (3\text{-}6)$$

where μ is the mean value, σ is the standard deviation, , M_i is the i^{th} moment ratio , N is the total number of data and COV is the coefficient of variation.

The cumulative distribution function of the Gumbel distribution for maxima is defined by (Castillo, 1988);

$$y = F(x; \lambda, \delta) = \exp\left[-\exp\left[-\frac{x-\lambda}{\delta}\right]\right]; \qquad -\infty < x < \infty \qquad (3\text{-}7)$$

where λ and δ are the distribution parameters. After that, the straight line equation fitted to survey data points changes as below (Castillo, 1988);

$$\eta = h(y) = -\log\left[\log\left(\frac{1}{y}\right)\right] \rightarrow \quad \eta = \frac{x-\lambda}{\delta} \qquad (3\text{-}8)$$

λ and δ parameters can be defined by setting η = 0 and η = 1 (Castillo, 1988);

$$0 = x - \lambda \rightarrow x = \lambda \ \text{ and } \ 1 = \frac{x-\lambda}{\delta} \rightarrow x = \lambda + \delta \qquad (3\text{-}9)$$

"After fitting the straight line on Gumbel probability paper, the abscissas associated with ordinate values 0 and λ of the reduced variate, η , give the values of λ and l +δ , respectively. After obtaining the values of λ and the mean and variance of the

Gumbel distribution can be calculated by the following expressions" (Argınhan, 2010);

$$\mu = \lambda + 0.5772\delta \quad \text{and} \quad \sigma^2 = \frac{\pi^2 \delta^2}{6} \qquad (3\text{-}10)$$

where μ indicates the mean and σ shows the standard variation.

By using the above mentioned formulas, the coefficients of variation values have been calculated for the three cases such as overall, upper tail and extreme. The final results are illustrated in the table below.

Table 3-12 Parameters of Gumbel Distribution for Overall Case

Span (m)	**KGM-45**		**Eurocode LM-1**	
	λ	**δ**	**λ**	**δ**
25	0.230	0.083	0.123	0.044
30	0.208	0.089	0.111	0.047
35	0.201	0.090	0.107	0.048
40	0.190	0.091	0.100	0.048

Table 3-13 Mean, Standard Deviation and Coefficient of Variation Parameters for Overall Case

Span (m)	**KGM-45**			**Eurocode LM-1**		
	μ	**σ**	**COV**	**μ**	**σ**	**COV**
25	0.278	0.106	0.382	0.148	0.057	0.382
30	0.260	0.114	0.439	0.139	0.061	0.439
35	0.252	0.115	0.456	0.134	0.061	0.456
40	0.243	0.117	0.481	0.128	0.062	0.481

Table 3-14 Parameters of Gumbel Distribution for Upper-tail Case

Span (m)	KGM-45		Eurocode LM-1	
	λ	δ	λ	δ
25	0.544	0.020	0.290	0.011
30	0.522	0.019	0.279	0.010
35	0.500	0.019	0.266	0.010
40	0.479	0.019	0.254	0.010

Table 3-15 Mean, Standard Deviation and Coefficient of Variation Parameters for Upper-tail Case

Span (m)	KGM-45			Eurocode LM-1		
	μ	σ	COV	μ	σ	COV
25	0.555	0.025	0.046	0.296	0.013	0.046
30	0.533	0.024	0.046	0.285	0.013	0.046
35	0.511	0.024	0.047	0.272	0.013	0.047
40	0.490	0.024	0.049	0.259	0.013	0.049

Table 3-16 Parameters of Gumbel Distribution for Extreme Case

Span (m)	KGM-45		Eurocode LM-1	
	λ	δ	λ	δ
25	0.600	0.019	0.320	0.010
30	0.574	0.018	0.307	0.010
35	0.552	0.016	0.294	0.009
40	0.531	0.016	0.281	0.008

Table 3-17 Mean, Standard Deviation and Coefficient of Variation Parameters for Extreme Case

Span (m)	KGM-45			Eurocode LM-1		
	μ	σ	COV	μ	σ	COV
25	0.611	0.024	0.039	0.326	0.013	0.039
30	0.584	0.023	0.039	0.312	0.012	0.039
35	0.561	0.021	0.037	0.299	0.011	0.037
40	0.540	0.021	0.038	0.286	0.011	0.038

3.2.5.4 Comparison of the Extrapolation Cases

From Figure 3-37 to Figure 3-42, 75-year maximum moment ratios are plotted for all three cases. By using these values, coefficients of variation have been determined for KGM-45 and Eurocode LM-1 truck loading separately. The coefficients of variation values did not change according to the loading type since the same truck survey data has been taken into account. However, coefficients of variation are very high in overall case comparing with the upper-tail and extreme cases. In fact, this result was expected since the whole truck data (heavy and light trucks) were regarded in overall case. The comparison of coefficients of variation is shown in Figure 3-45.

In reliability evaluation that will be discussed in further parts of this study, statistical parameters of extreme case will be taken into account. The main reason of this preference is that extreme case data include higher bias factors and lower coefficient of variations values. By using the statistical parameters of extreme case, the most critical scenario for the reliability assessment can be handled. As a result, the mean bias factors which equal to the 75-year maximum mean moment ratio are taken as 0.80 to 0.92 for KGM-45 loading and 0.42 to 0.49 for LM-1 loading depending on each span length. On the other hand, the coefficient of variation is taken as 0.038 for both of the loading.

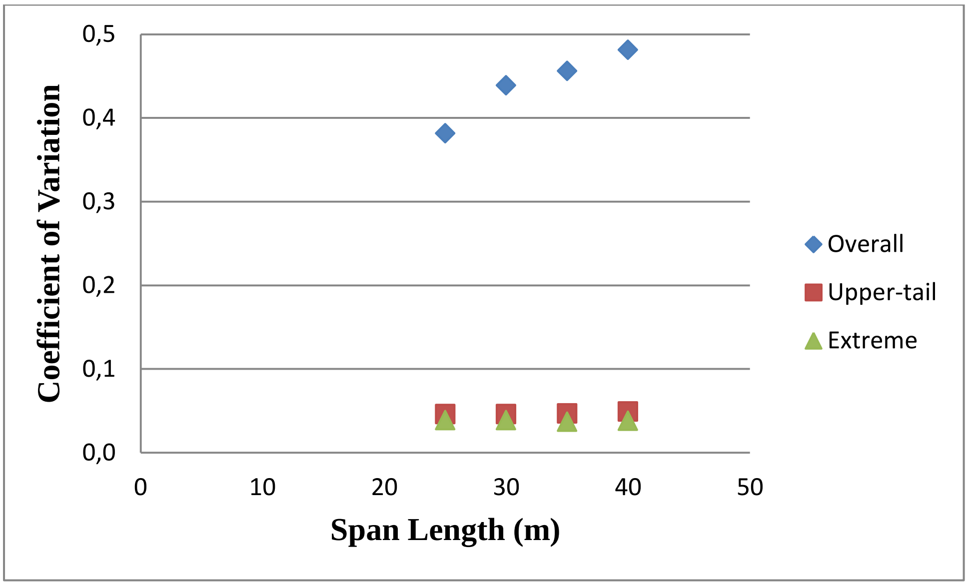


Figure 3-45 Comparison of the Coefficients of Variation for Different Cases

3.3 Dynamic Load

The dynamic load is mainly function of bridge span length, road roughness, and features of vehicles such as weight, type, axle configuration and position on the bridge. Therefore, it can be said that dynamic load is random and variable depending on time. The dynamic load is basically defined by dynamic load factor (DLF). Although different definitions take part in literature for dynamic load factor, it can be taken as the ratio of dynamic response to the static response. In this parameter, dynamic response equals to the absolute maximum dynamic response at points of stress, strain or deflection that were obtained from the test data. Static response indicates the maximum static response from the filtered dynamic response. In Figure 3-46, the static and dynamic response of a bridge under the load of 5 axle truck with a speed of 104 km/h is shown. This comparison indicates that dynamic load does not change with respect to static load. Therefore, dynamic load factor is low for heavier trucks since static response increases depending on weight of trucks and dynamic response remains constant.

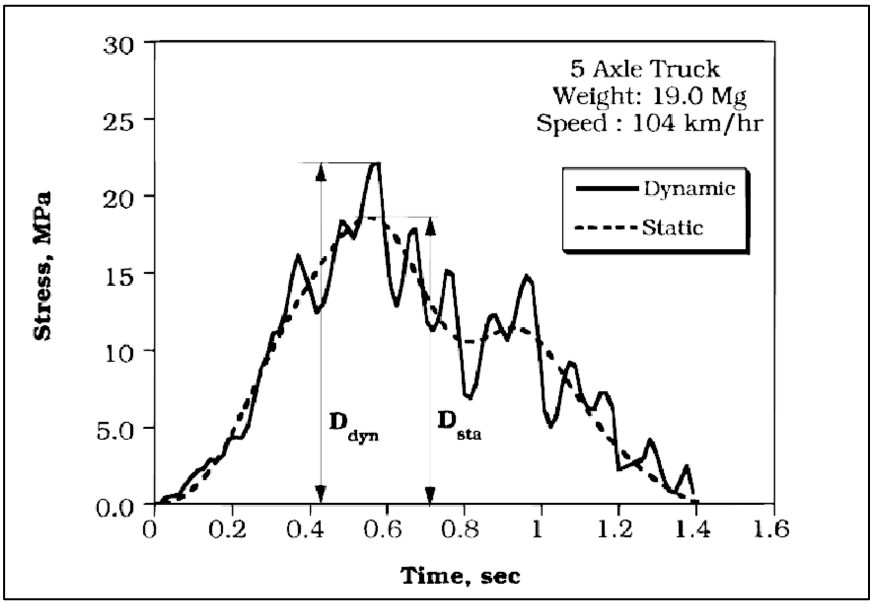


Figure 3-46 Static and Dynamic Response of a Bridge under the 5 axle Truck Loading (Nassif and Nowak, 1995)

In this study, dynamic load effect is considered as 33% so that static response of the truck is increased by this percentage as determined in ASHTO LRFD Bridge Design Specification. For the statistical parameters of dynamic load, mean value is taken as 0.15 and coefficient of variation is considered as 0.80 which are in line with the Nowak's calibration report (1999).

3.4 Girder Distribution Factor

The design moment is distributed to the each interior girder by using the girder distribution factor equation which is expressed in AASHTO LRFD Table 4.6.2.2.2b-1 as;

For one design lane loaded,

$$0.06 + \left(\frac{S}{4300}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{Lt_s^{\,3}}\right)^{0.1} \qquad (3\text{-}11)$$

For two or more design lanes loaded,

$$0.075 + \left(\frac{S}{2900}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{Lt_s{}^3}\right)^{0.1} \qquad (3\text{-}12)$$

where S is spacing of girder (mm), L is span length of girder (mm), K_g is longitudinal stiffness parameter (mm^4) and t_s is depth of concrete deck(mm). In AASHTO LRFD 4.6.2.2.1-1 and 4.6.2.2.1-2, longitudinal stiffness parameter is expressed as;

$$K_g = n\left(I + Ae_g{}^2\right), \qquad \text{in which} \qquad n = \frac{E_B}{E_D} \qquad (3\text{-}13)$$

where E_B is the modulus of elasticity of girder material (MPa), E_D is the modulus of elasticity of deck material (MPa), I is the moment of inertia of girder (mm^4), A is the cross sectional area of girder (mm^2) and e_g is the distance between the centers of gravity of the basic girder and deck (mm).

The bias factor and coefficient of variation have been taken as 1.0 and 0.12 respectively in the updating calibration for AASHTO LRFD code by Nowak (2007). These results are a bit conservative since they are calculated according to AASHTO LRFD. In another research where different design codes such as AASHTO LRFD, Spanish Norma IAP and Eurocode had been compared by Nowak (2001), the bias factor and coefficient of variation of GDF have been calculated as 0.93 and 0.12. Therefore, 0.93 and 0.12 values are taken as statistical parameters of girder distribution factor with a normal random distribution.

3.5 Summary of Statistical Parameters

Statistical parameters which are mainly bias factor and coefficient of variation and probability distribution for dead loads, live loads, impact factor and girder distribution factor are summarized in the table below.

Table 3-18 Summary of Statistical Parameters

Parameter	Description	Probability Distribution	Bias Factor	Coefficient of Variation
D1	Dead Load – Factory Made Members	Normal	1.03	0.08
D2	Dead Load – Cast in Place Members	Normal	1.05	0.10
D3	Dead Load – Wearing Surface	Normal	1.00	0.25
D4	Dead Load - Miscellaneous	Normal	1.05	0.10
LL	Live Load – KGM-45	Gumbel	0.80-0.92	0.038
	Live Load – Eurocode LM-1	Gumbel	0.42-0.49	0.038
IM	Impact Factor	Normal	0.15	0.8
GDF	Girder Distribution Factor	Normal	0.93	0.12

CHAPTER 4

STATISTICS OF RESISTANCE

In reliability analysis, flexural resistance capacities of precast prestressed concrete girders are determined relying on the nominal resistance values. In this part, the statistical parameters of resistance which basically depend on material properties of steel and concrete have been evaluated. In addition to this, uncertainties due to the dimensional error and theoretical behavior have been also elaborated. In this assessment, statistical data that was obtained from the national and international research has been used.

4.1 Material Properties

In the design of precast prestressed concrete girders, two main parameters must be evaluated to determine the resistance of structure. Those are material properties of steel and concrete which are going to be discussed in detail based on the statistical parameters.

4.1.1 Concrete

Concrete is the most widely used material in construction industry because of economic and practical reasons. The cement, aggregate and water are the basic materials in the composition of concrete. These materials can be gathered easily in most of the country. Due to that reason, concrete constitutes the main construction material in Turkey. The statistical report of European Ready Mixed Concrete Organization (ERMCO) in year 2015 indicates that Turkey is the leader country in the ready mixed concrete manufacturing among the European countries as shown in

Figure 4-1. According to the statistical report of Turkish Ready Mixed Concrete Association (THBB), RMC production capacity of Turkey is about 109 millions of m^3 in 2016 which is indicated in Table 4-1.

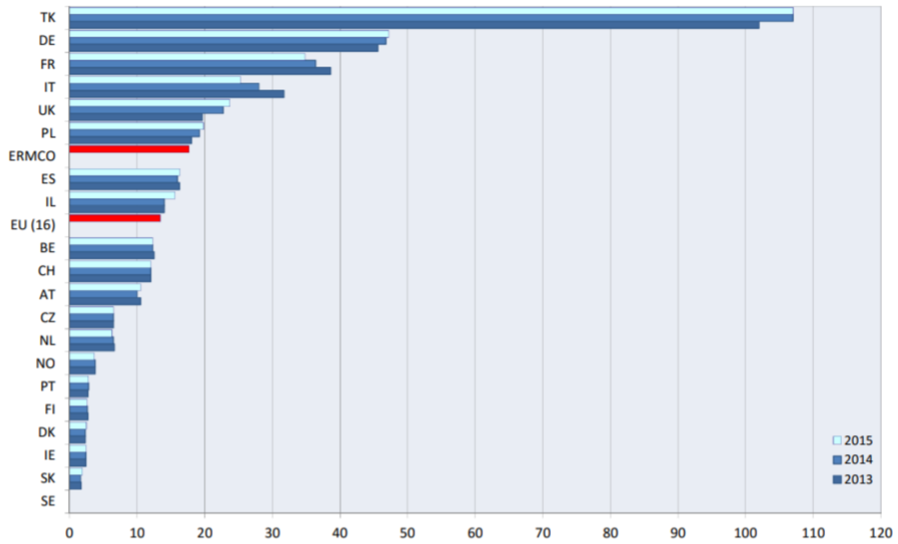


Figure 4-1 Ready Mixed Concrete Production per year in Europe (ERMCO, 2015)

Table 4-1 Ready Mixed Concrete Production of Turkey per year

Years	Ready Mixed Concrete Production (m^3)
1988	1.500.000
1993	10.000.000
1998	26.542.905
2003	26.828.500
2005	46.300.000
2006	70.732.631
2007	74.359.847
2008	69.600.000
2009	66.430.000
2010	79.680.000
2011	90.450.000
2012	93.050.000
2013	102.000.000
2014	107.000.000
2015	107.000.000
2016	109.000.000

The concrete classes are most important parameters for the resistance of structure. Before the 1999, concrete classes such as C14, C16 and C18 were the most widely used concrete types in Turkey. After these years, due to the construction of high rise buildings, bridges with long span lengths and deep subway systems, concrete quality has been improved. According to the report of Turkish Ready Mixed Concrete Association (THBB) which was published in 2013, production of high strength concrete increased in the last ten years steadily in parallel with the development of construction industry. This graph also indicates that the compressive strength of 25 MPa and above values compose nearly 80% of the total concrete production which is presented in Figure 4-2.

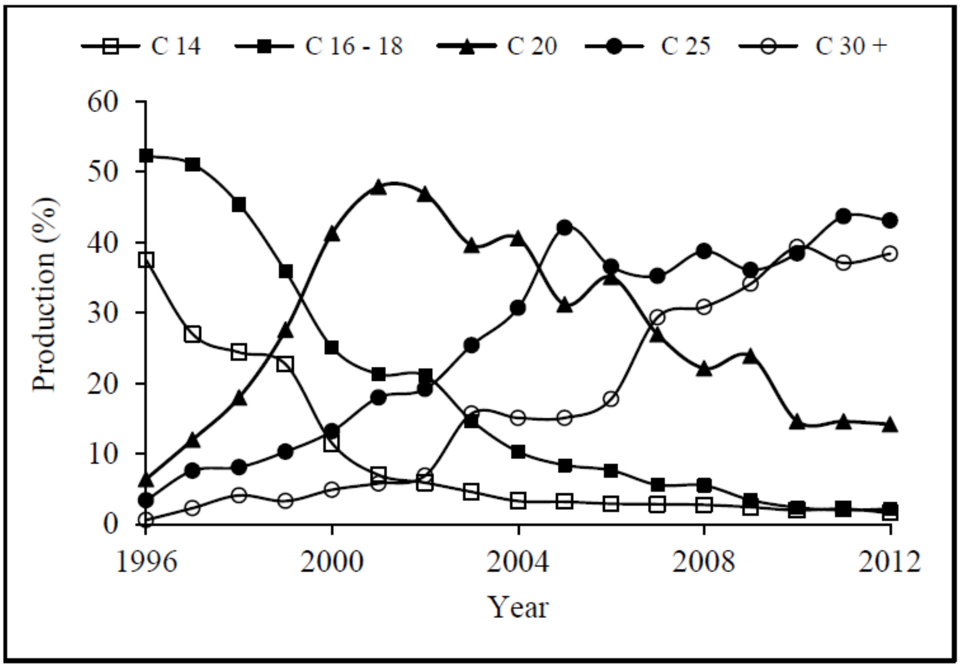


Figure 4-2 Production of Ready Mixed Concrete Classes in Turkey with respect to Years (THBB)

Although there are several effects such as compressive strength, durability, shrinkage and creep that need to be considered in the design of concrete structure, compressive strength is the primary property of concrete. In the construction of bridges, high compressive strength concretes are preferred. The compressive strength of concrete mainly depends on quality and ratio of cement, aggregates, water, and the chemical additives. There are also other stresses such as shear and tension in concrete; however, those stresses can be measured and defined in terms of compressive strength. For all this reason, the statistical parameters namely coefficient of variation, mean and standard deviation of compressive strength of concrete have been elaborated as important indicators.

4.1.1.1 Statistical Parameters of Compressive Strength of Concrete for Bridge Deck

According to AASHTO LRFD 2012 5.4.2.1., the minimum allowable concrete compressive strength for concrete deck is 4.0 ksi (27.6 MPa). While constructing the bridges in Turkey, C30 class concrete is generally used. Therefore, C30 grade concrete has been selected for the bridge deck in this study.

The concrete quality manufactured in Turkey was investigated by Fırat in 2006. In this study, 28-day compressive strength of 150x150x150 mm cubic test specimens which belong the years between 2000 and 2005 have been obtained from different test laboratories in Turkey. Additionally, the obtained results and the previous test results have been compared which are shown in Table 4-2 and Table 4-3.

Table 4-2 Statistical Parameters of Compressive Strength of Concrete (Fırat, 2006)

Year	Number of Samples	Mean (MPa)	COV	Number of Values Under Limit	Percentage of Values Under Limit (%)
94/95	417	20.60	-	58	13
2000	732	26.97	0.142	40	5.46
2001	535	30.97	0.107	23	4.30
2002	465	31.21	0.104	10	2.15
2003	644	30.78	0.131	36	5.59

Table 4-2 cont'd

2004	1283	28.87	0.123	30	2.34
2005	615	29.97	0.120	24	3.90

Table 4-3 Statistical Parameters of Compressive Strength of Different Concrete Grades (Fırat, 2006)

Grade of Concrete	Number of Samples	$f'_{c,cube}$ (MPa)	Mean (MPa)	COV	Number of Values Under Limit	Percentage of Values Under Limit (%)
C14	137	18	20.04	0.143	1	0.83
C16	755	20	25.11	0.144	13	1.73
C18	739	22	25.82	0.120	23	3.11
C20	5817	25	28.46	0.104	118	2.70
C25	2767	30	32.48	0.100	53	2.81
C30	870	37	40.07	0.079	14	2.47

Since C30 class concrete was used for bridge deck, statistics of this class are taken into consideration. As indicated in Table 4-3, the mean value of 28-day cubic compressive strength is 40.07 MPa and coefficient of variation is 0.079. On the other hand, these results do not include the epistemic uncertainties which will be discussed in the further part of this study.

4.1.1.2 Statistical Parameters of Compressive Strength of Concrete for Bridge Girder

In the construction of bridge girder, the commonly used concrete grade is C40 in Turkey. In addition to this, the minimum allowable concrete compressive strength for concrete girder is 4.0 ksi (27.6 MPa) according to AASHTO LRFD 2012 5.4.2.1. Therefore, C40 concrete grade has been preferred for design of precast prestressed concrete girder in this study.

In literature, there are several researches to obtain the statistical parameters of concrete by gathering different laboratory test results. In Argınhan's study (2010), the statistical parameters of C40 grade concrete produced in Turkey were revealed. In the scope of this study, standard 150x150x150 mm cube specimens and 150x300 mm cylinder specimens were obtained from two different firms. After that, 7-day and 28-day mean compressive strengths of these specimens had been observed. In the statistical calculations, two types of analyses had been performed. In the first analysis, all data were considered to calculate the mean and standard deviation of compressive strength of concrete. In the second analysis, the mean and standard deviation were determined by taking the average value of the grouped specimens. As a result of this study, similar mean values were obtained from two methods which are indicated in Table 4-4, Table 4-5 and Table 4-6.

Table 4-4 Statistical Parameters of C40 Concrete Class based on the 7 and 28 day Compressive Strength from First Firm Data (Argınhan, 2010)

Statistical Parameters	7-Day		28-Day	
	Overall	In-Batch	Overall	In-Batch
Max Value (MPa)	43.3	42.62	50.15	49.17
Min Value (MPa)	35.96	37.59	44.44	45.59
Mean (MPa)	39.58		47.57	
Standard Deviation (MPa)	1.86	1.70	1.39	1.26
Coefficient of Variation	0.047	0.043	0.029	0.026
Bias Factor (Mean/Nominal)	-		1.189	

Table 4-5 Statistical Parameters of C40 Concrete Class based on the 7 and 28 day Compressive Strength from Second Firm Data (Argınhan, 2010)

Statistical Parameters	7-Day		28-Day	
	Overall	In-Batch	Overall	In-Batch
Max Value (MPa)	47.52	44.68	60.2	57.78
Min Value (MPa)	30.85	31.9	40.87	42.72
Mean (MPa)	37.24		47.84	
Standard Deviation (MPa)	3.37	3.13	3.66	3.16
Coefficient of Variation	0.09	0.084	0.077	0.066
Bias Factor (Mean/Nominal)	-		1.196	

Table 4-6 Statistical Parameters of C40 Concrete Class based on the 7 and 28 day Compressive Strength from First and Second Firms Data (Argınhan, 2010)

Statistical Parameters	7-Day		28-Day	
	Overall	In-Batch	Overall	In-Batch
Max Value (MPa)	47.52	44.68	60.2	57.78
Min Value (MPa)	30.85	31.9	40.87	42.72
Mean (MPa)	37.37		47.82	
Standard Deviation (MPa)	3.34	3.11	3.57	3.08
Coefficient of Variation	0.089	0.083	0.075	0.064
Bias Factor (Mean/Nominal)	-		1.2	

In this study, the above mentioned statistical parameters are used for C40 class concrete.

4.1.1.3 Evaluation of Uncertainties for the Compressive Strength of Concrete

In addition to the aleatory uncertainties which have been estimated previously, epistemic uncertainties also affect the strength of concrete. These uncertainties can be listed namely as the difference between laboratory test conditions and in-situ conditions, the loading speed, human errors and the discrepancies of site batches and test batches.

The difference between strength of test specimen at site and laboratory can be defined by correlation factor (N_1). Bloem's research indicates that strength of concrete at site may be lower than the strength of concrete at laboratory with a percentage of 10% to 21% (1968, as cited in Ang and Tang, 1984). Moreover, according to report of Mirza (1979) ratio of core strength to actual strength varies between 0.74-0.96 which has an average value of 0.87. In accordance with this result, Ellingwood and Ang (1972) cited that this ratio changes between 0.83-0.92.

In the construction of bridge, quality control rules especially for the concrete strength have more dominant roles comparing with the ordinary building construction.

Therefore, distribution of the correction factor (N_1) for the concrete strength at site and laboratory accepted as an upper triangular distribution which has lower and upper limits as shown in Figure 4-3.

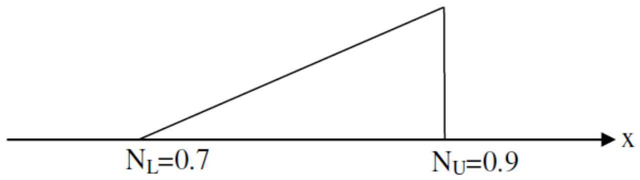


Figure 4-3 Probability Density Function (Upper Triangle)

The mean value and coefficient of variation are determined by using the above mentioned lower and upper limits and the equations expressed below:

$$\overline{\mathrm{N}} = \frac{1}{3}(\mathrm{N}_L + 2 . \mathrm{N}_U) \quad (4\text{-}1)$$

$$\Delta = \frac{1}{\sqrt{2}}\left(\frac{\mathrm{N}_U - \mathrm{N}_L}{2\mathrm{N}_U + \mathrm{N}_L}\right) \quad (4\text{-}2)$$

where N_L ad N_U are the lower and upper limit of the correction factor, $\overline{\mathrm{N}}$ is the mean correction factor and Δ is the coefficient of variation. By considering this distribution, mean correction factor ($\overline{\mathrm{N}}_1$) is calculated as 0.89 and coefficient of variation (Δ_1) is determined as 0.06. However, the COV value is taken as 0.1 in order to get conservative results.

Rate of loading is the other epistemic uncertainty that influences the concrete strength. The compressive strength of concrete will reach its maximum level as the loading rate increases. Mirza (1979) defined a correlation factor which is $\overline{\mathrm{N}}_2$ to determine this uncertainty. This correlation factor can be defined by using below formula:

$$\overline{N}_2 = 0.89x(1 + 0.08log_{10}(R)) \tag{4-3}$$

where R is loading rate and $\overline{\mathrm{N}}_2$ is the mean correction factor. $\overline{\mathrm{N}}_2$ is determined as 0.89 when R value is taken as 1 (psi/sec). Fırat (2007) also took mean correction factor ($\overline{\mathrm{N}}_2$) as 0.89 and epistemic uncertainty was ignored. In this study, same value is considered for the uncertainty of loading rate.

The last epistemic uncertainty that affects the strength of concrete is human error. These errors mostly arise from wrong application and lack of application of standard testing procedures or not selecting specimens randomly. Kömürcü (1995, as cited in Fırat, 2006) defined a mean correction factor ($\overline{\mathrm{N}}_3$) as 0.95 and coefficient of variation (Δ_3) as 0.05. In this study, mean correction factor was taken as 1.00 because of the high quality control in the construction of bridge and COV value was the same value as 0.05.

All the above mentioned epistemic uncertainties are expressed as shown below:

$$\overline{N}_{f\acute{c}} = \overline{N_1}\ x\ \overline{N_2}\ x\ \overline{N_3} = 0.89\ x\ 0.89\ x\ 1.00\ \cong\ 0.8 \tag{4-4}$$

$$\Delta_{f\acute{c}} = \sqrt{\Delta_1^2 +\ \Delta_2^2 +\ \Delta_3^2} = \sqrt{0.1^2 + 0^2 + 0.05^2} = 0.11 \tag{4-5}$$

Compression strength of concrete can be determined by using the below formula,

$$f_c = \overline{N}_{f\acute{c}}\ x\ \overline{f}_c \tag{4-6}$$

where f_c is compressive strength of concrete at site, $\overline{f}_c$ is compressive strength of concrete at the laboratory and $\overline{N}_{f\acute{c}}$ indicates the overall uncertainties.

Based on these data, compressive strength of C30 class concrete for cubic specimen is calculated as 0.8×40.07 = 32.1 MPa. The cubic compressive strength of C30 class concrete equals to the 37 MPa. Therefore, bias factor for compressive strength of C30 class concrete is 32.06/37 = 0.87.

For the C40 grade concrete, in-situ compressive strength is calculated as 0.8×47.82 = 38.3 MPa. The cylinder compressive strength of C40 grade concrete equals to the 40 MPa. Therefore, bias factor for compressive strength of C40 class concrete is found as 38.3/40=0.96.

The total uncertainty in terms of coefficient of variation can be defined as below,

$$\Omega_{f_c} = \sqrt{(\delta_{fc})^2 + (\Delta_{fc})^2} \quad (4\text{-}7)$$

where $\Delta f c'$ is the total epistemic uncertainty and $\delta f c'$ is inherent uncertainty.

Total uncertainty for C30 class concrete is calculated as $\sqrt{0.079^2 + 0.11^2} = 0.135$ and for C40 grade concrete it is calculated as $\sqrt{0.064^2 + 0.11^2} = 0.127$. The summary of statistics for compressive strength of concrete is indicated in Table 4-7.

Table 4-7 Summary of Statistics for Compressive Strength of C30 and C40 Concrete Grade

Statistical Parameters	Concrete Class	
	C30	C40
Laboratory Measured Mean (MPa)	40.07	47.8
In-situ Mean (MPa)	32.06	38.3
Nominal (MPa)	37	40
Bias Factor (Mean/Nominal)	0.87	0.96
Coefficient of Variation	0.135	0.127
Standard Deviation (MPa)	4.33	4.86

4.1.2 Prestressing Strands

In the construction and design of bridge girders, seven-wire prestressing strand is generally preferred. Depending on the manufacturing method, two types of product emerge, which are mainly stress-relieved and low relaxation strands. In the production of seven-wire strand in both of methods, six wires around one central wire are wrapped. In the stress relieving method, the residual stress comes to existence due to the twisting and the cooling of the wires which lead to a lower yield stress. This effect is removed by heating the strands up to 350°C and this situation allows them to cool gradually to eliminate the residual stresses due to the manufacturing process. On the other hand, in low-relaxation strand method, the strands are under the tension effect in the process of heating and cooling in order to reduce the relaxation of the strands.

The stress-strain level of seven-wire strand produced by different methods is indicated in Figure 4-4. In this study, the low relaxation strands are considered for the design of girders due to the higher yield strength. The yield strength of strand is calculated as 1674 MPa (0.9x1860) which is equal to 90% of ultimate tensile strength of strand (1860 MPa) according to the AASHTO LRFD Bridge Design Specifications (2012).

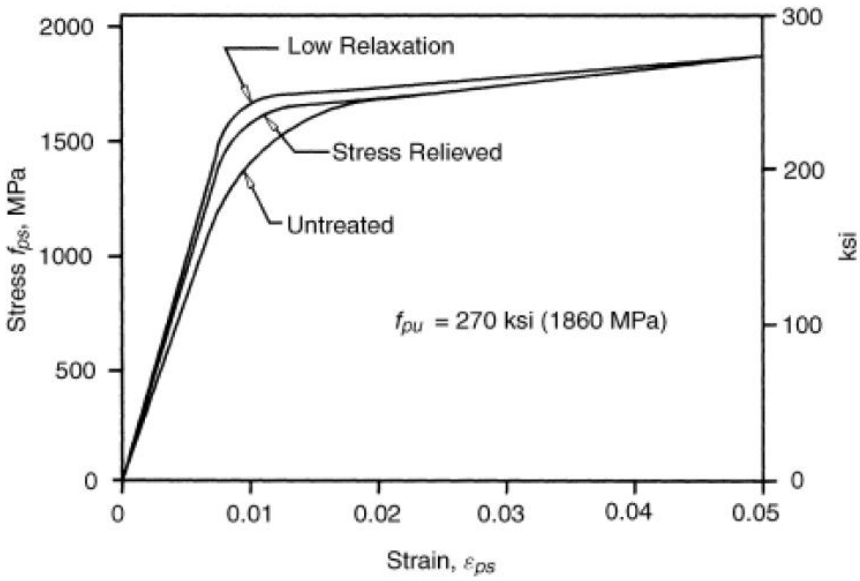


Figure 4-4 Stress Strain Level of Seven-Wire Strand Produced by Different Methods

4.1.2.1 Statistical Parameters of Prestressing Strands

Argınhan (2010) investigated one hundred forty six tensile strength test results for prestressing strands. The mean yield strength and coefficient of variation was determined as 1740 MPa and 0.021 respectively. As a result, the mean bias factor was calculated 1740/1674=1.04. Statistical parameters of yield strength are summarized in Table 4-8.

Table 4-8 Statistical Parameters of Yield Strength of Strand

Statistical Parameters	Values
Max Value (MPa)	1781
Min Value (MPa)	1599
Mean (MPa)	1740
Standard Deviation (MPa)	36.29
Coefficient of Variation	0.021
Bias Factor (Mean/Nominal)	1.04

In another research, Mirza (1980) determined the statistical parameters of yield strength of prestressing strands based on two hundred test records. According to this study, the mean yield strength of prestressing strands was estimated as 1.04 and coefficient of variation was found as 0.025. These results are in compliance with the Argınhan's study. Therefore, in this study, the mean bias factor is taken as 1.00 and coefficient of variation is considered as 0.03 by rounding of values.

The other uncertainty that must be taken into account for the prestressing strands is the cross sectional area. In the study of Al-Harty and Frangopol (1994) which was about the reliability analysis of prestressed precast concrete girders, the mean bias factor was determined as 1.01 and the coefficient of variation was calculated as 0.0125 for cross-sectional area of prestressing strands. In the light of this information, bias factor is taken as 1.00 by rounding and coefficient of variation is used as 0.0125. All the statistical parameters for prestressing strands are listed in Table 4-9.

Table 4-9 Statistical Parameters of Prestressing Strands

Statistical Parameters	Prestressing Strands	
	Yield Strength	Cross Sectional Area
Coefficient of Variation	0.03	0.0125
Bias Factor	1	1

4.2 Dimensions and Actual to Theoretical Behavior

In order to determine the variability and uncertainty in width and depth of the girders for both precast concrete and in-situ beams, Mirza (1979) recommended range and values for the statistical parameters of dimensions which are indicated in Table 4-10. From the values given below, the mean (μ_b) and coefficient of variation (δ_b) parameters of beam width can be determined by

(4-8)

$$\mu_b = b_n + 3.96mm. \quad \text{or} \quad b_n + 5/32in..$$

$$\delta_b = (6.35)mm./(b_n + 3.96mm.) \quad \text{or} \quad (1/4)in./(b_n + 5/32in.) \tag{4-9}$$

where, b_n is the nominal width of precast girder.

Table 4-10 Statistical Parameters of Girder Dimensions

<table>
<tr><th colspan="2" rowspan="2">Dimension description</th><th colspan="3">In-Situ beam</th><th colspan="3">Precast Beam</th></tr>
<tr><th>Nominal range</th><th>Mean deviation from nominal</th><th>Standard deviation</th><th>Nominal range</th><th>Mean deviation from nominal</th><th>Standard deviation</th></tr>
<tr><td rowspan="2">Width</td><td>Rib</td><td>11-12</td><td>+3/32</td><td>3/16</td><td>14</td><td>0</td><td>3/16</td></tr>
<tr><td>Flange</td><td>-</td><td>-</td><td>-</td><td>19-24</td><td>+5/32</td><td>1/4</td></tr>
<tr><td colspan="2">Overall Depth</td><td>18-27</td><td>-1/8</td><td>1/4</td><td>21-39</td><td>+1/8</td><td>5/32</td></tr>
<tr><td rowspan="2">Effective Depth</td><td>Top Rein.</td><td>1-1/2</td><td>+1/8
-1/4</td><td>5/8
11/16</td><td>2-2 1/2</td><td>0
+1/8</td><td>5/16
11/32</td></tr>
<tr><td>Bottom Rein.</td><td>3/4-1</td><td>+1/16
-3/16</td><td>7/16
1/2</td><td>3/4</td><td>0
+1/8</td><td>5/16
11/32</td></tr>
<tr><td colspan="2">Beam Spacing and Span</td><td>-</td><td>0</td><td>11/16</td><td></td><td>0</td><td>11/32</td></tr>
<tr><td colspan="8">- Dimensions are in inches.</td></tr>
</table>

Furthermore, the mean (μ_{dp}) and coefficient of variation (δ_{dp}) effective depth of beam can be expressed by

$$\mu_{dp} = d_{pn} + 3.18mm \quad \text{or} \quad d_{pn} + (1/8)in \tag{4-10}$$

$$\delta_{dp} = 8.73mm/(d_{pn} + 3.18mm) \quad \text{or} \quad (11/32)in./(d_{pn} + 1/8in.) \tag{4-11}$$

where, d_{pn} is the nominal effective depth of girder.

For example, according to the above formula, the mean and coefficient of variation values of beam with a width of 1000 mm equal to 1003.96 mm and 0.0063 respectively. If beam depth is assumed as 1200 mm, the mean and coefficient of variation values are calculated as 1203.18 mm and 0.0072. As the results show that the variations of dimensions are very small. In accordance with these results, the mean bias factor of both width and depth dimensions is taken as 1.00 and coefficient of variation is considered as 0.01 in this study.

The other factor that affects the resistance of girder is the variability and uncertainty of the actual and theoretical behavior due to the assumptions or approximations. For this reason, in reliability analysis, this uncertainty should be also considered. Nowak (1999) defined a multiplier for this uncertainty and named it as professional factor (PF). For the precast concrete girders, bias factor and coefficient of variation for theoretical behavior can be considered as 1.01 and 0.06 which are also used in this study.

4.3 Chapter Summary

The statistical parameters of resistance due the material (strength), fabrication (dimensions) and professional (actual-to-theoretical behavior) that are evaluated in this chapter are summarized in Table 4-11.

Table 4-11 Summary of Statistical Parameters of Resistance

Parameter	Bias Factor	COV	Distribution Type
Compressive Strength of Deck Concrete (C30)	0.87	0.135	Normal
Compressive Strength of Precast Girder (C40)	0.96	0.127	Log-normal
Yield Strength of Prestressing Strand	1.00	0.03	Normal
Cross Sectional Area of Prestressing Strand	1.00	0.0125	Normal
Width and Thickness of Precast Girder	1.00	0.01	Normal
Professional Factor	1.01	0.06	Normal

CHAPTER 5

DESIGN OF BRIDGE GIRDERS

In this chapter, the structural analysis and design procedure of the main girders of bridges have been explained. In the determination of flexural resistance capacities of girders, AASHTO LRFD (2012) bridge design specifications have been followed. The relevant design provisions will be mentioned in further parts. In the scope of this study, precast prestressed concrete girders have been designed based on AASHTO LRFD specifications. After that, the flexural resistance capacity of the designed girders has been designated according to the EUROCODE 2 (1992-1-1/1992-2) provisions.

5.1 The General Properties of Precast Prestressed Concrete Girders and Bridges

In this study, four different bridges have been specified with a length of 25 m, 30 m, 35 m and 40 m. The total widths of bridges have been taken as unique value which equals 11 m. The bridge girders have been selected as H100, H120, H140 and H195 types which are commonly used cross-sections for bridges in Turkey. The cross-sectional dimensions of these girder types are shown in Figure 5-1.

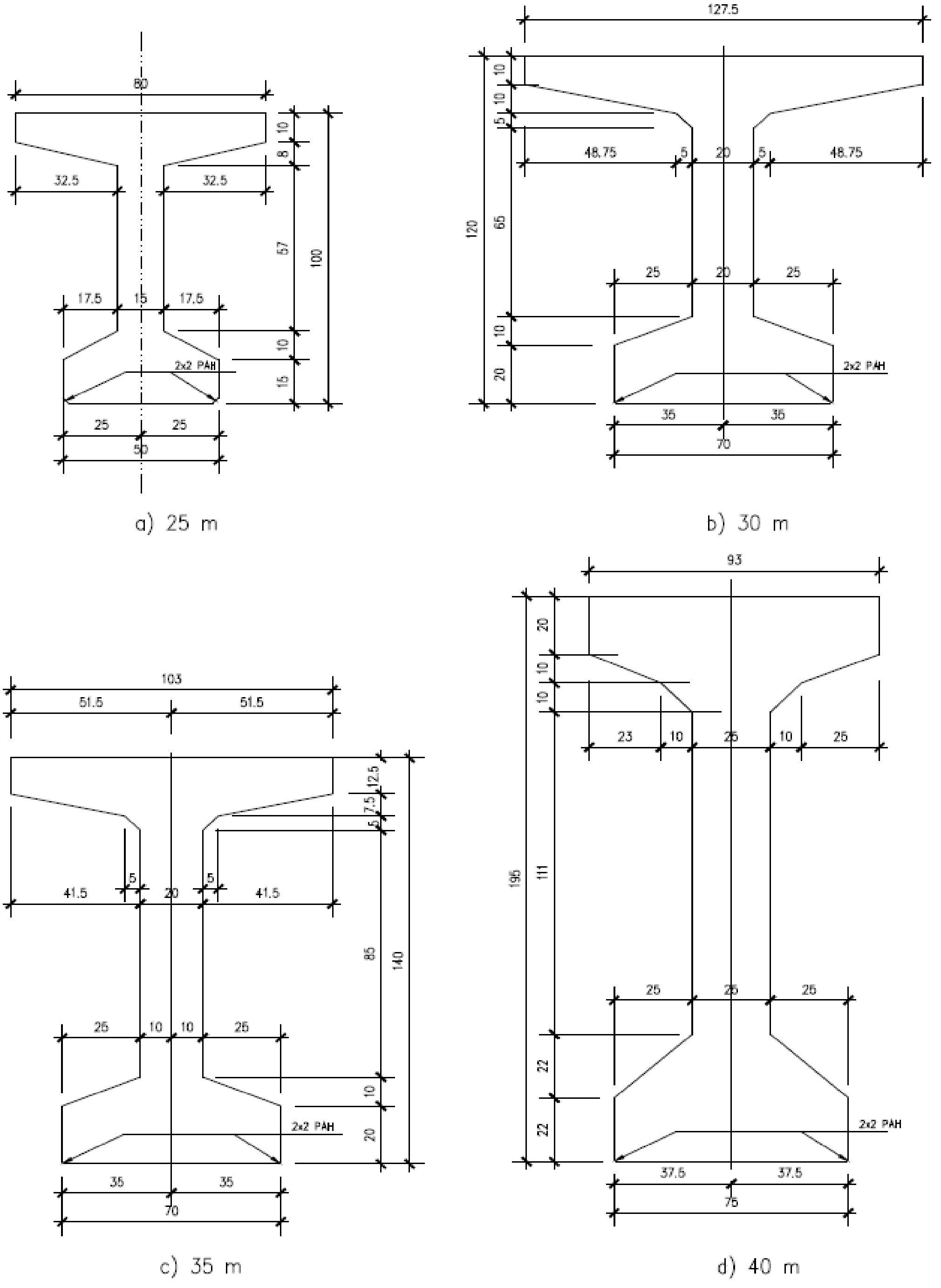


Figure 5-1 Cross-sectional Dimensions of Precast Prestressed Concrete Girders

3 design lanes have been arranged with a width of 3 m. This means that width of total roadway is 9 m. The remaining part of bridge has been separated for the pedestrian walk ways and barriers which have a width of 2 m. The number of precast concrete girders and the spacing distance of these girders are indicated in Table 5-1. The bridge cross-sections are also illustrated in between Figure 5-2 and Figure 5-5. Concrete deck thickness has been determined as 25 cm which is cast-in-place. For deck and precast girder, C30 and C40 grade concrete have been used respectively, with unit weight of 2400 kg/m^3. In addition to this, an asphalt layer has been specified as 5 cm with unit weight of 2200 kg/m^3. Lastly, the weight of barrier has been taken as 4.4 kN/m.

Table 5-1 The Number and Spacing of Precast Prestressed Concrete Girder Types

Bridge Span Length (m)	Girder Spacing (cm)	Girder Type	Number of Girders
25	137.5	H100	8
30	137.5	H120	8
35	137.5	H140	8
40	137.5	H195	8

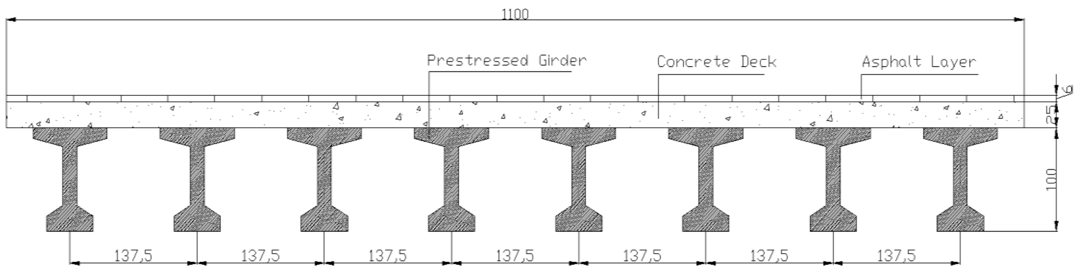


Figure 5-2 Bridge Cross-Section for 25 m

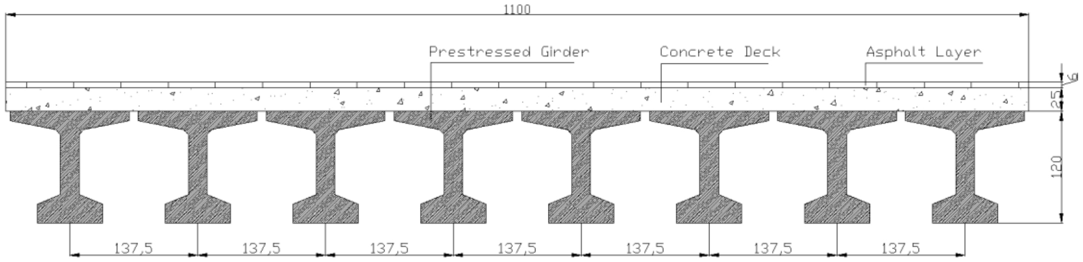


Figure 5-3 Bridge Cross-Section for 30 m

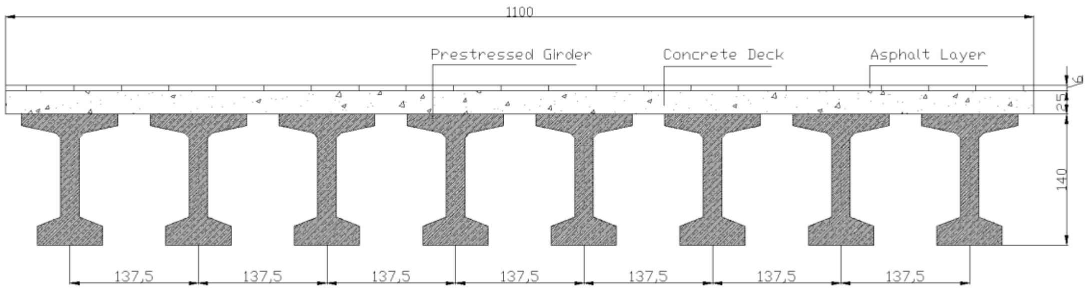


Figure 5-4 Bridge Cross-Section for 35 m

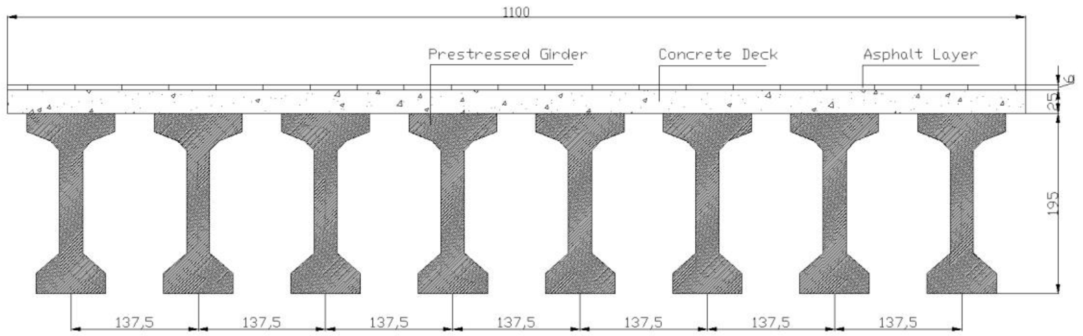


Figure 5-5 Bridge Cross-Section for 40 m

5.2 Flexural Resistance Capacity of Girders Based on the AASHTO LRFD Design Specification

In AASHTO LRFD (2012), design provisions for the flexural members were collected under the seventh part of section five which is called as concrete structure. The equations for flexural resistance capacity for bonded tendons are listed in AASHTO LRFD 5.7.3.1.1. The tendon which is also named as strand in prestressed concrete girders is fully bonded to concrete. Therefore, when the member is under the load effect, the amount of change in strain of the concrete is same with the amount of change in strain of the steel. In Figure 5-6, the forces that affect a reinforced concrete beam and strain changes are indicated.

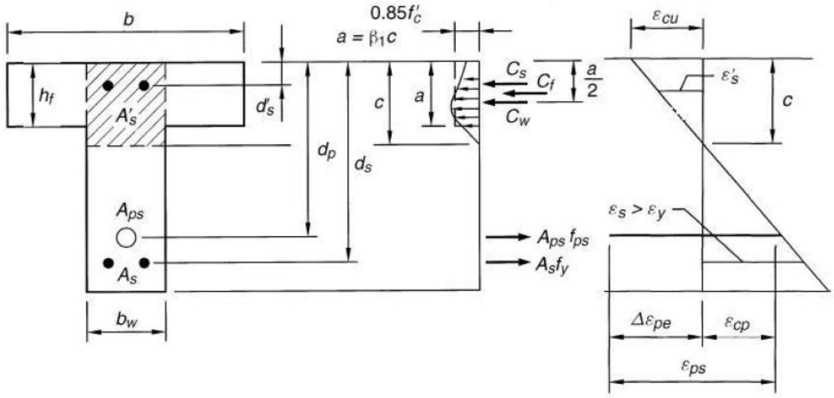


Figure 5-6 Forces and Strain Changes on Reinforced Concrete Beam

In the above shown figure, c indicates the depth of neutral axis and it can be determined by using the equilibrium equation of forces. C_s is the compressive force of rebar. C_w is the compressive force of concrete in the web and it is equal to 0.85. $\beta_1.f'_c.c.b_w$. Additionally, C_f is the compressive force of concrete in the flange and it is calculated by using 0.85. $\beta_1.f'_c.(b-b_w).h_f$ formula. Lastly, f_{ps} shows the average stress of prestressing steel and determined by using the below formulas (AASHTO LRFD 5.7.3.1.1-1-1).

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad (5\text{-}1)$$

In which (AASHTO LRFD 5.7.3.1.2):

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \quad (5\text{-}2)$$

k is equal to 0.28 for low relaxation strands (AASHTO LRFD Table C5.7.3.1.1-1).

For T-section behavior (AASHTO LRFD 5.7.3.1.3):

$$c = \frac{A_{ps} f_{pu} + A_s f_s - A'_s f'_s - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w + k A_{ps} \frac{f_{pu}}{d_p}} \quad (5\text{-}3)$$

For rectangular section behavior (AASHTO LRFD 5.7.3.1.4):

$$c = \frac{A_{ps} f_{pu} + A_s f_s - A'_s f'_s}{0.85 f'_c \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}} \quad (5\text{-}4)$$

"where:

A_{ps} *= prestressing steel area,*

f_{pu} *= specified tensile strength of prestressing steel,*

f_{py} *= yield strength of prestressing steel,*

A_s *= mild steel tension reinforcement area,*

A'_s *= compression reinforcement area,*

f_s *= stress in the mild steel tension reinforcement,*

f'_s *= stress in the mild steel compression reinforcement,*

b *= width of the flange section in compression,*

b_w *= web width,*

h_f = compression flange depth,

d_p = distance from extreme compression fiber to the centroid of the prestressing strands,

β_1 = the stress factor of compression block."

The nominal flexural resistance (M_n) can be determined by (AASHTO LRFD 5.7.3.2.2-1),

$$M_n = A_{ps} f_{ps} (d_p - \frac{a}{2}) + A_s f_s (d_s - \frac{a}{2}) - A'_s f'_s (d'_s - \frac{a}{2}) + 0.85 f'_c (b - b_w) h_f (\frac{a}{2} - \frac{h_f}{2}) \tag{5-5}$$

"where,

d_s = distance from extreme compression fiber to the centroid of non prestressed tensile reinforcement,

d'_s = distance from extreme compression fiber to the centroid of compression reinforcement,

$a = c\beta_1$; depth of the equivalent stress block."

For the rectangular section behavior and without mild tension or compression reinforcement, the flexural nominal resistance can be calculated from the below equation.

$$M_n = A_{ps} f_{ps} (d_p - \frac{a}{2}) \tag{5-6}$$

5.2.1 Tensile Stress Check

The number of strands is calculated by restricted tensile stresses of concrete at the bottom fiber in order to control the concrete cracking under 80% of truck and lane loads. Tensile stress at the bottom fiber can be determined by using the below formula.

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe} e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_b + M_{ws} + 0.8(M_{LT} + M_{LL})}{S_{bc}} \tag{5-7}$$

"where,

f_b = *the bottom fiber tensile stress,*

M_g = *the unfactored bending moment due to self-weight of beam,*

M_s = *the unfactored bending moment due to weight of slab,*

M_{ws} = *the unfactored bending moment due to weight of barrier,*

M_{LT} = *the unfactored bending moment due to truck load,*

M_{LL} = *the unfactored bending moment due to lane load,*

S_b = *the section modulus of the extreme bottom fiber for the non-composite precast beam,*

S_{bc} = *the composite section modulus of the extreme bottom fiber for the non-composite precast beam,*

P_{pe} = *the total prestressing force,*

e_c = *eccentricity of total prestressing force."*

In accordance with the provisions described in AASHTO LRFD Table 5.9.4.2.2-1, tensile stress limits in prestressed concrete at service limit state should not exceeded the $0.50\sqrt{f_c'}$ for the moderate corrosion conditions and the $0.25\sqrt{f_c'}$ for the severe corrosion conditions. In this formulation, f_c' indicates concrete strength of precast girder for 28-day in terms of MPa.

5.2.2 Prestressing Losses

The total prestressing force, shown by $P_{pe,}$ is equal to the multiplication of number of prestressing steels with the tensile stress capacity after prestressing losses. Total prestress loss is calculated from the following relationship (AASHTO LRFD 5.9.5.1-1),

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \tag{5-8}$$

"where,

Δf_{pT} *= total loss,*

Δf_{pES} *= sum of all losses or gains depending on the elastic shortening or extension during the application of prestress and/or external loads,*

Δf_{pLT} *= losses due to the relaxation of the steel, creep of concrete, and long-term shrinkage."*

The loses due to the elastic deformations of the member can be estimated by using the below equation (AASHTO LRFD 5.9.5.2.3a-1),

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp} \tag{5-9}$$

"where,

f_{cgp} *= the concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the selfweight of the member at the section of maximum moment,*

E_p *= modulus of elasticity of prestressing strand,*

E_{ct} *= modulus of elasticity of concrete at time of load application or transfer."*

The losses (Δf_{pLT}) due to the long-term shrinkage, creep of concrete and relaxation of the steel can be determined from the following formulations (AASHTO LRFD 5.9.5.3-1, 5.9.5.3-2, 5.9.5.3-3),

$$\Delta_{pLT} = 10.0\frac{f_{ps}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR} \qquad (5\text{-}10)$$

$$\gamma_h = 1.7 - 0.01H \qquad (5\text{-}11)$$

$$\gamma_{st} = \frac{5}{(1+f'_{ci})} \qquad (5\text{-}12)$$

"where,

f_{ps} *= prestressing steel stress before the transfer,*

H = the average annual ambient relative humidity (%) which is taken as 70% in this study,

γ_h *= correction factor for relative humidity of the ambient air,*

γ_{st} *= correction factor for specified concrete strength at time of prestress transfer to the concrete member,*

Δf_{pR} *= an estimate of relaxation loss which is taken as 17 MPa for low relaxation steel in this research."*

5.3 Flexural Resistance Capacity of Girders According to the EUROCODE 2 Design Specification

In EUROCODE 2 (1992-1-1) design specification, the ultimate resistance of prestressed concrete cross-sections is calculated under the similar assumptions that were lined up in AASHTO LRFD design provision. These assumptions are mainly based on ideas of plane sections remain plane, the strain in bonded prestressing strands is equal to the strain in the surrounding concrete and the tensile strength of the concrete is negligible. Possible strain distributions of reinforcing steel and concrete that were accepted in this specification is shown in Figure 5-7.

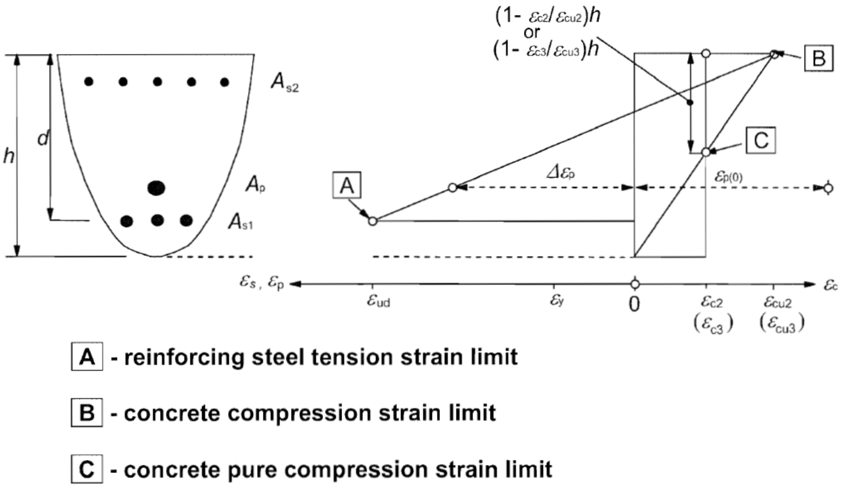


Figure 5-7 Possible strain distributions of Reinforced Concrete Beam

The total forces that are applied to a tendon during tensioning should not exceed the value gained from the following equation (EUROCODE 2/1992-1-1 5.41)

$$P_{max} = A_p \cdot \sigma_{p,max} \qquad (5\text{-}13)$$

"where,

A_p = Cross-sectional area of prestressing tendon,

$\sigma_{p,max}$ = the maximum stress in the tendon"

The mean prestress force ($P_{m,t}(x)$) of the tendon at any time is equal to the maximum force (P_{max}) minus the immediate losses ($\Delta P_i(x)$) and the time dependent losses($\Delta P_{c+r+s}(x)$). In addition to this, the initial prestressing force ($P_{mo}(x)$) value which is equal to maximum force (P_{max}) minus the immediate losses ($\Delta P_i(x)$) should not exceed the below given value (EUROCODE 2/1992-1-1 5.43),

$$P_{m0}(x) = A_p \cdot \sigma_{pm0}(x) \qquad (5\text{-}14)$$

"where,

$\sigma_{pm0}(x)$ = the stress in the tendon after tensioning or transfer."

After finding the mean prestress force of the tendon, the ultimate flexural capacity can be calculated by multiplying this prestress force with the lever arm of the prestressed tensile reinforcement. The material partial safety factors handledin this calculation are indicated in Table 5-2 (EUROCODE 2/1992-1-1 Table 2.1N).

Table 5-2 Partial Factors for Materials

Design situations	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

5.3.1 The Immediate and Time Dependent Losses of Prestressing Tendons

The immediate losses of tendons ($\Delta P_i(x)$) are due to the elastic deformation of concrete (ΔP_{el}), friction ($\Delta P_\mu(x)$) and anchorage slip (ΔP_{si}).

The loss of force in tendon because of the deformation of concrete is estimated by using the following formula (EUROCODE 2/1992-1-1 5.44),

$$\Delta P_{el} = A_p \cdot E_p \cdot \sum \left[\frac{j \cdot \Delta\sigma_c(t)}{E_{cm}(t)} \right] \qquad (5\text{-}15)$$

"where,

$\Delta\sigma_c(t)$ =the variation of stress at the centre of gravity of the tendons applied at time t,

j = the coefficient which equal to (n -1)/2n where n is the number of prestressed tendons."

The losses depending on the friction in tendons can be extracted from (EUROCODE 2/1992-1-1 5.45),

$$\Delta P_{\mu}(x) = P_{max}\,(1 - e^{-\mu(\theta + kx)}) \tag{5-16}$$

"where,

Θ = the sum of the angular displacements over a distance x,

μ = the coefficient of friction between the tendon and its duct,

k = an unintentional angular displacement for internal tendons

x = the distance along the tendon from the point where the prestressing force is equal to its maximum."

The time dependent losses of tendons are originated from the deformation of concrete due to creep and shrinkage and the relaxation of steel under tension. These reductions of stress can be calculated by using the below equation (EUROCODE 2/1992-1-1 5.46),

$$\Delta P_{c+s+r} = A_p \Delta\sigma_{p,c+s+r} = A_p \frac{\varepsilon_{cs} E_p + 0{,}8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}}\varphi(t,t_0).\sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}}\frac{A_p}{A_c}\left(1 + \frac{A_c}{I_c} z_{cp}^2\right)\left[1 + 0{,}8\,\varphi(t,t_0)\right]} \tag{5-17}$$

"where,

$\Delta\sigma_{c+r+s}$ = the absolute value of the variation of stress in the tendons due to relaxation, shrinkage and creep,

$\mathcal{E}_{cs}$ = the estimated shrinkage strain value,

E_p = the modulus of elasticity for the prestressing strand,

E_{cm} =the modulus of elasticity for the concrete,

$\Delta\sigma_{pr}$ = the absolute value of the variation of stress due to the relaxation of the prestressing steel,

$\varphi_{(t,t0)}$ = the creep coefficient,

$\sigma_{c,Qp}$ = the stress in the concrete adjacent to the tendons, due to self-weight, initial

prestress and other quasi-permanent actions,

A_p *= the area of all prestressing strands,*

A_c *= the area of the concrete member,*

I_c *= the second moment of area of the concrete member,*

Z_{cp} *= the distance between the centre of gravity of the concrete member and the strands."*

5.4 Analysis and Design Results

For the design of prestressed concrete girders, Microsoft Excel computer program has been used. The bridge girders have been designed according to the AASHTO LRFD Specifications. Then, flexural resistance capacities of designed girders have been calculated based on both AASHTO LRFD provisions and the EUROCODE-2 design specification. Since only the flexural moment resistances of girders are taken into consideration for the reliability evaluation in the scope of this study, the girders are designed from this standpoint.

Maximum span moments are calculated by considering the dead and live loads. After that, these moments are distributed based on the girder distribution factor described in AASHTO LRFD Specifications. However, there is no formulation related with the load distribution of girder in EUROCODE-2 design provisions. Therefore, in order to determine this load distribution, the study results of Çınar, A. Altunlu and Caner (2017) which is about EUROCODE live load distribution factors for multi simple span precast prestressed concrete girder bridges have been addressed. In the subject study, the girder distribution factors for LM-1 truck model of Eurocode were handled by taking into account both FEM analysis results and hand computations results. The results of this study are shown in Table 5-3 according to the span lengths and spacing of girders.

Table 5-3 The Estimated Girder Distribution Factors for LM-1 truck model of Eurocode

Span	Space 2.0	Space 1.7	Space 1.5	Space 1.33
15	0.396	0.339	0.320	0.274
20	0.372	0.317	0.283	0.255
25	0.367	0.316	0.275	0.251
30	0.344	0.295	0.258	0.234
35	0.336	0.294	0.256	0.230

By considering the above mentioned results and using linear interpolation, girder distribution factors for the studied bridges which have a girder spacing of 1.375 m have been calculated. All the girder distribution factors and the maximum span moments which are determined per girder are tabulated in Table 5-4.

Table 5-4 Maximum Moments per Girder and GDF Values

Load Type and GDF	Span Length			
	25 m	30 m	35 m	40 m
M_{DL1} (kN.m)	607.42	1501.17	2090.63	4237.50
M_{DL2} (kN.m)	671.39	966.80	1315.92	1718.75
M_{DW} (kN.m)	429.69	618.75	842.19	1100.00
M_{LL} (KGM-45) (kN.m)	1535.45	2028.10	2487.49	3148.39
M_{LL} (LM-1) (kN.m)	1461.60	1768.40	2180.35	2638.35
GDF for KGM-45 loading	0.407	0.417	0.414	0.435
GDF for LM-1 loading	0.257	0.240	0.237	0.235

In the above table, M_{DL1} shows the maximum moment because of the weight of precast concrete girder, M_{DL2} indicates the maximum moment depending on the weight of concrete deck, M_{DW} is the maximum moment due to the weight of asphalt layer and barrier, M_{LL} shows the maximum moment due to live load of KGM-45 and LM-1 loading and GDF is the girder distribution factor for both of the design loading.

The prestressed precast concrete girders are designed according to the subject dead and live loads. After that the minimum required numbers of prestressing tendons are determined for the defined cross-sections and material properties based on the AASHTO LRFD Specifications. Then, flexural resistance capacities of girders (M_n) are calculated according to the AASHTO LRFD specification and EUROCODE-2 design provision. Lastly, the minimum required resistances of girders (M_u) are determined by multiplying the moments of dead and live loads with the corresponding load factors of strength I limit state of AASHTO LRFD and ultimate limit states of EUROCODE-2 specifications which have been explained in Section 3.1 and 3.2. In the determination of the number of prestressing strands, the tensile stresses of concrete at the bottom fiber are also restricted in order to prevent the concrete cracking (see chapter 5.2.1). The number of strands for each girder type is determined as unique number under both of the truck loading in order to see the difference between AASHTO LRFD and EUROCODE-2 specifications in reliability analysis. The design results are indicated according to both the KGM-45 and LM-1 model loading in Table 5-5 to Table 5-6.

Table 5-5 Analysis and Design Results Based on the KGM-45 Truck Loading

Parameters	Span Length			
	25 m	30 m	35 m	40 m
M_u (kN.m)	4930.08	7562.26	9874.57	14605
M_n (kN.m)	6638.34	9575.93	12760.36	19771
Max Tensile Stress Limit (MPa)	3.16	3.16	3.16	3.16
Tensile Stress at Bottom (MPa)	2.51	2.69	2.87	3.16
Number of Strands	25	31	36	40

Table 5-6 Analysis and Design Results Based on the LM-1 Truck Loading

Parameters	Span Length			
	25 m	30 m	35 m	40 m
M_u (kN.m)	3529.85	5612.61	7480.66	11597.29
M_n (kN.m)	5335.01	7705.13	10270	15845.44
Max Tensile Stress Limit (MPa)	3.16	3.16	3.16	3.16
Tensile Stress at Bottom (MPa)	2.50	2.39	2.59	2.75
Number of Strands	25	31	36	40

The above results indicate that maximum moment per girder due to the Eurocode LM-1 truck loading is lower than the maximum moments per girder due to the KGM-45 truck loading. Although the LM-1 trucks have a higher weight comparing with the KGM-45 trucks, the lower load factors of ultimate limit state of EUROCODE-2 and the smaller girder distribution factors lead to these moment distribution. Moreover, flexural resistance capacities of girders calculated according to EUROCODE-2 specifications are also smaller than the capacities of girders estimated by AASHTO LRFD specifications.

On average, the ultimate moment, M_u, computed with Eurocode analysis method is about 25% less than the ultimate moment, M_u, computed with AASHTO LRFD analysis method. The Eurocode design capacity, M_n, is about 20% less than the design capacity, M_n, computed with AASHTO-LRFD method.

CHAPTER 6

RELIABILITY EVALUATION

6.1 Reliability Model

In engineering design, all the parameters related with loads and resistance of materials include some uncertainties and this situation affects the design requirements. Therefore, the reliability analysis is conducted by regarding these uncertainties. In Figure 6-1, the basic random variables which are namely load (S) and resistance (R) in failure function are indicated. Their randomness is defined by means (μ_S, μ_R), standard deviations (σ_S, σ_R) and corresponding density functions ($f_S(s)$ and $f_R(r)$).

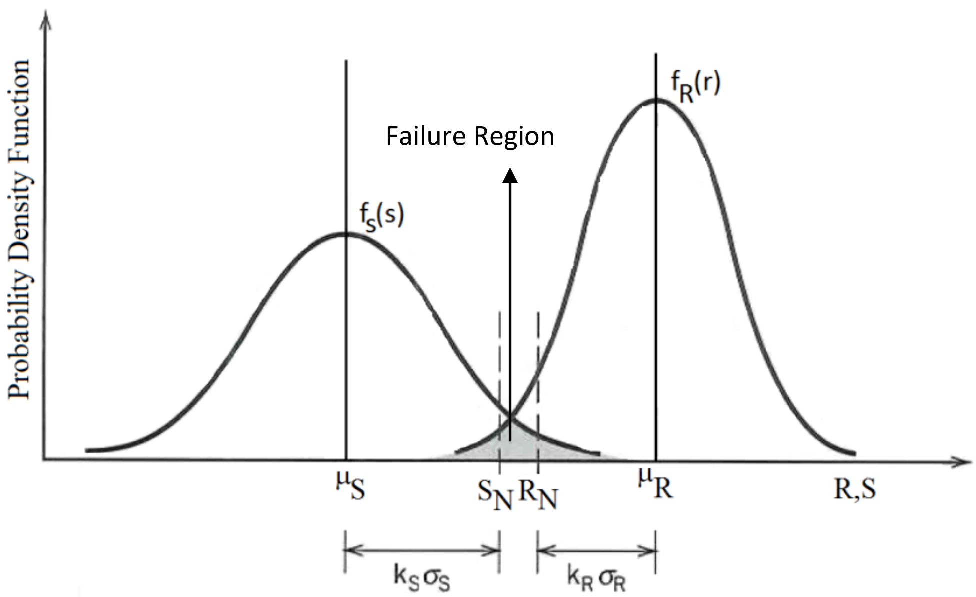


Figure 6-1 Basic Random Variables

The above figure indicates that even if the mean resistance is greater than the mean load effect, there is always possibility of failure for structure due to the uncertainties of loads and resistance of materials.

The reliability of structure can be expressed in terms of the probability of failure and probability of survival,

$$\begin{aligned} p_f &= P(failure) = P(R < S) \\ &= \int_0^{\infty} \left[\int_0^{s} f_R(r)dr\right] f_S(s)ds \\ &= \int_0^{\infty} F_R(s) f_S(s)ds \end{aligned} \qquad (6\text{-}1)$$

where $F_R(s)$ shows the cumulative distribution function of resistance. This equation is accepted as the primary equation in reliability-based design approach.

The fundamental basic random variables which are mainly load and resistance depend on many variables such as dimensional quantities, properties of material and effects of loads. In order to define these basic random variables, limit state function has been expressed as indicated below:

$$M = R - S = g(\boldsymbol{X}) = g(X_1, X_2, \dots, X_n) \qquad (6\text{-}2)$$

where M shows the safety margin (performance indicator) and X indicates the vector of random variables. The failure surface occurs when the g(X) and M are equal to zero. The failure surface which is also named as limit state function forms a boundary between survival and failure zones. As the M is greater than zero, the structure will survive. When the M is smaller than zero, it means that the structure will fail. The subject situation is illustrated in Figure 6-2.

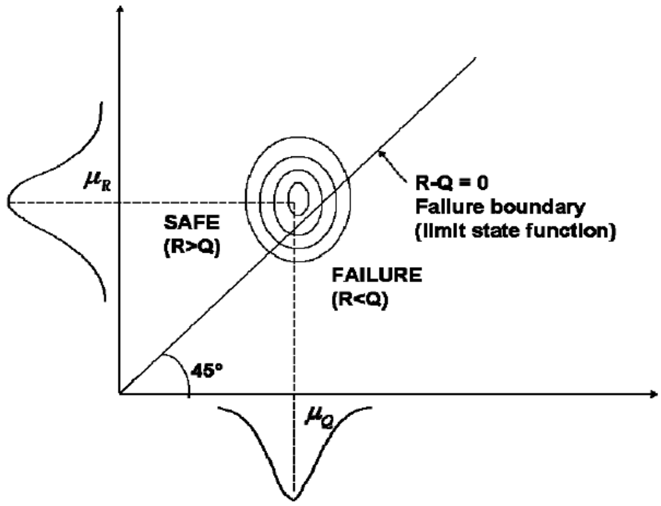


Figure 6-2 Failure Boundary

Probability of failure can be find by solving the below integration,

$$p_f = \int \dots \int_{g(\cdot)<0} f_X(x_1, x_2, \dots, x_n) dx_1 dx_2 \dots dx_n \qquad (6\text{-}3)$$

where $f_X(x1,x2,\dots,xn)$ indicates the joint probability density function and the integration is performed over the failure region. In determination of probability of failure, two main problems generally occur. The first one is the absence of sufficient data in order to obtain the joint probability density function. The second one is the difficulty in solution of multiple integrals. These problems can be overcome by using several approximate methods. In the scope of this study, MVFOSM (Mean Value First Order Second Moment) has been used for reliability evaluation.

6.1.1 Mean Value First Order Second Moment Method

The Mean Value First Order Second Moment Method is based on a Taylor series approximation from the first order. For the first time, it was emphasized in the study of Cornell (1969, as cited in Haldar and Mahadevan, 2000). This approximation is performed for failure function at the center of the mean values of random variables. As a result of the analysis of failure function, the mean (μ_g) and standard deviation (σ_g) of this failure function are determined. These parameters are essential for the computation of reliability index term which indicates both the probability of failure and the probability of survival. The reliability index physically equals to the shortest distance from the origin to the failure surface and illustrated in Figure 6-3.

Reliability index is expressed by the Greek letter β, and the formulation is indicated below,

$$\beta = \frac{M_r - M_u}{\sqrt{\sigma_n^2 + \sigma_u^2}} \tag{6-4}$$

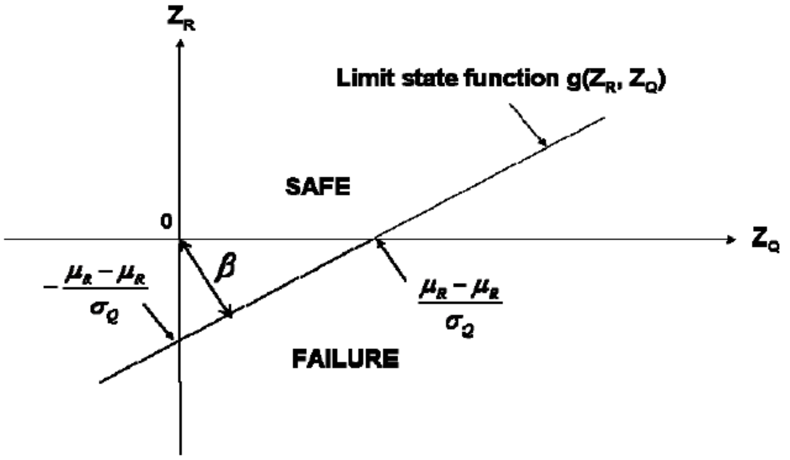


Figure 6-3 Physical Description of Reliability Index

It is understood from the above figure that as the reliability index increases, the distance between the failure surface and the mean will also increase. This means that the higher reliability index leads to the higher probability of survival.

With the assumption of normally distributed random variables, probability of failure can be determined in terms of reliability index as shown below,

$$P_f = \Phi(-\beta) = 1 - \Phi(\beta) \tag{6-5}$$

in which Φ indicates the standard normal cumulative distribution function. Different reliability indexes and their corresponding probability of failure values are listed in Table 6-1.

Table 6-1 Reliability Index vs Probability of Failure

Reliability Index, β	Probability of Failure, P_f
0	0.5
1	0.159
2	0.0228
3	0.00135
4	0.0000317
5	0.000000287
6	0.000000000987

If the failure function is assumed as linear, it can be expressed by the below formulation in terms of basic variables X_1, X_2,…, ,

$$g(\boldsymbol{X}) = a_0 + a_1X_1 + \cdots + a_nX_n \tag{6-6}$$

Mean value of failure function can be defined as;

$$\mu_g = g(\mu_X) = a_0 + a_1\mu_{X_1} + \cdots + a_n\mu_{X_n} \tag{6-7}$$

The variance of the function can be calculated as;

$$\sigma_g^2 = a_1^2\sigma_{X_1}^2 + \cdots + a_n^2\sigma_{X_n}^2 + \sum_{i=1}^{n}\sum_{j=1,j\neq i}^{n}\frac{\partial g}{\partial X_i}\frac{\partial g}{\partial X_j}Cov(X_i, X_j) \tag{6-8}$$

in which $Cov(X_i,X_j)$ indicates the covariance of X_i and X_j. Additionally, it can be determined by using the equation of $\rho_{XiXj}\sigma_{Xi}\sigma_{Xj}$ where ρ_{XiXj} shows the correlation coefficient between X_i and X_j. The exact values of mean and standard deviation can be calculated where the (***X***) is linear.

*"In case g(**X**) is nonlinear, the result of the mean and standard deviation would not be exact, and approximate values of those can be obtained by using a linearized function, which is constructed by expanding failure function in Taylor series centered at the mean values and keeping only the linear terms* (Koç,2013)."

Hence, linearized function will be defined by using the below formula;

$$g(\boldsymbol{X}) = g(\mu_X) + \sum_{i=1}^{n}\frac{\partial g}{\partial X_i}(X_i - \mu_{X_i}) \tag{6-9}$$

in which $\partial g/\partial X_i$ is carried out at mean values. Finally, approximate values of μ_g and σ_g are obtained from the following equations.

$$\mu_g \cong g(\mu_{X_1}, \ldots, \mu_{X_n}) \tag{6-10}$$

$$\sigma_g^2 \cong \sum_{i=1}^{n}\sum_{j=1}^{n}\frac{\partial g}{\partial X_i}\frac{\partial g}{\partial X_j}Cov(X_i, X_j) \tag{6-11}$$

6.2 Failure Function

The failure of structure depends on two main parameters which are resistance capacity and load effect. Therefore, the failure function can be defined as g = R – Q where R shows the flexural resistance capacity and Q indicates the load effect. This means that if the g is smaller than zero, the structure will fail.

Load effect Q is expressed as the following:

$$Q = D_1 + D_2 + D_3 + D_4 + LL\ (1+IM)\ GDF \qquad (6\text{-}12)$$

in which D1, D2, D3 and D4 show the various dead loads, LL indicates live load, GDF is girder distribution factor and IM is impact factor. It should be noted that these load components and flexural resistance capacity of girders are explained in Chapter 3 and Chapter 5 respectively.

6.3 Determination of Reliability Levels Based on the AASHTO LRFD and EUROCODE Specifications

The reliability index values are determined according to the AASHTO LRFD and the EUROCODE provisions separately. In the calculation of reliability indexes, the statistical parameters of demands and resistances that have been stated in the previous part of this study (see Table 3-18 and Table 4-11) are taken into account. The Mean Value First Order Second Moment Method is considered as a reliability analysis method.

For the calculation of reliability level for KGM-45 truck loading, strength I limit state load factors that are described in AASHTO LRFD specifications are used. Moreover, for the determination of reliability level for LM-1 model loading, ultimate limit state load factors that are stated in EUROCODE-2 provisions are elaborated. These load factors are summarized in Table 6-2.

Table 6-2 Load Factors Based on AASHTO LRFD and EUROCODE-2 Specifications

Load Type	Load Factors	
	AASHTO LRFD	EUROCODE-2
DC	1.25	1.35
DW	1.5	1.35
LL	1.75	1.35

The cross-section of girders and number of strands are fixed for the each loading case as mentioned in Chapter 5. The reliability indexes are calculated for four different span lengths under the load of KGM-45 and LM-1 trucks and the results are illustrated in the Table 6-3 and Figure 6-4.

Table 6-3 Reliability Index Values for KGM-45 and LM-1 Truck Loading

Span Length (m)	Reliability Index (β)	
	KGM-45 truck loading	LM-1 truck loading
25	4.35	4.63
30	4.36	4.82
35	4.35	4.77
40	4.29	4.63

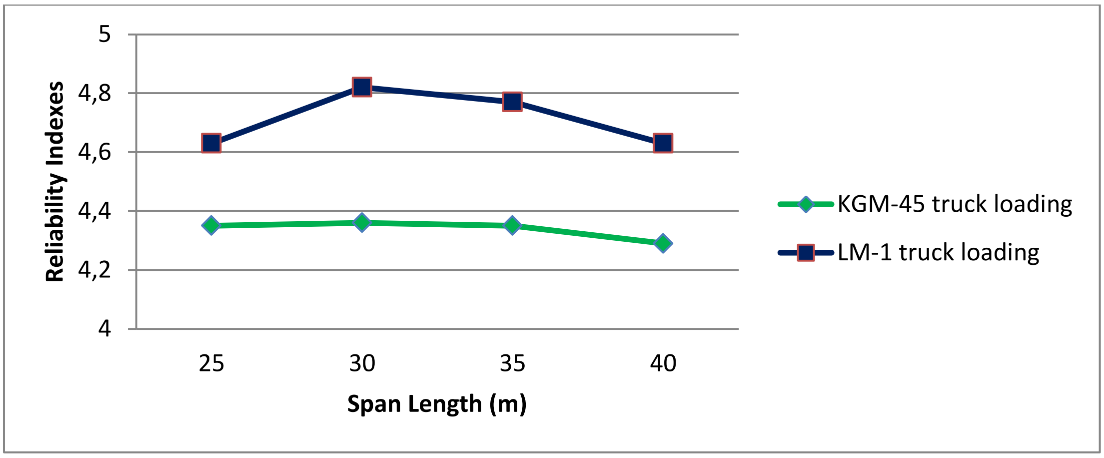


Figure 6-4 Reliability Index versus Span Length for Live Load Models

The above results indicate that the reliability indexes change between 4.29 and 4.36 for girders under the KGM-45 truck loading and vary between 4.63 and 4.82 for girders under the LM-1 truck loading. These results also show that reliability indexes of girders designed according to the EUROCODE is higher than the girders designed according to the AASHTO LRFD specifications. It is understood that the girders under the LM-1 truck loading have extra safety level comparing with the girders under KGM-45 truck loading.

In addition to the above results, the reliability index values for designed girders are also investigated by switching the truck loadings and design codes for each span length. The results are shown in the below tables and figure. According to these results, the reliability index values of girders increase when KGM-45 truck loading and EUROCODE specifications are taken into account simultaneously. On the other hand, the reliability indexes of same girders decrease while LM-1 truck loading and AASHTO LRFD specifications have been handling together.

Table 6-4 Reliability Index Values Calculated under Different Loads and Design Codes for Bridge (25 m)

CODE	Reliability Index (β)	
	KGM-45 truck loading	LM-1 truck loading
AASHTO LRFD	4.35	3.56
EUROCODE	4.81	4.63

Table 6-5 Reliability Index Values Calculated under Different Truck Loads and Design Codes for Bridge (30 m)

CODE	Reliability Index (β)	
	KGM-45 truck loading	LM-1 truck loading
AASHTO LRFD	4.36	3.76
EUROCODE	4.92	4.82

Table 6-6 Reliability Index Values Calculated under Different Loads and Design Codes for Bridge (35 m)

CODE	Reliability Index (β)	
	KGM-45 truck loading	LM-1 truck loading
AASHTO LRFD	4.35	3.82
EUROCODE	4.85	4.77

Table 6-7 Reliability Index Values Calculated under Different Loads and Design Codes for Bridge (40 m)

CODE	Reliability Index (β)	
	KGM-45 truck loading	LM-1 truck loading
AASHTO LRFD	4.29	3.79
EUROCODE	4.69	4.63

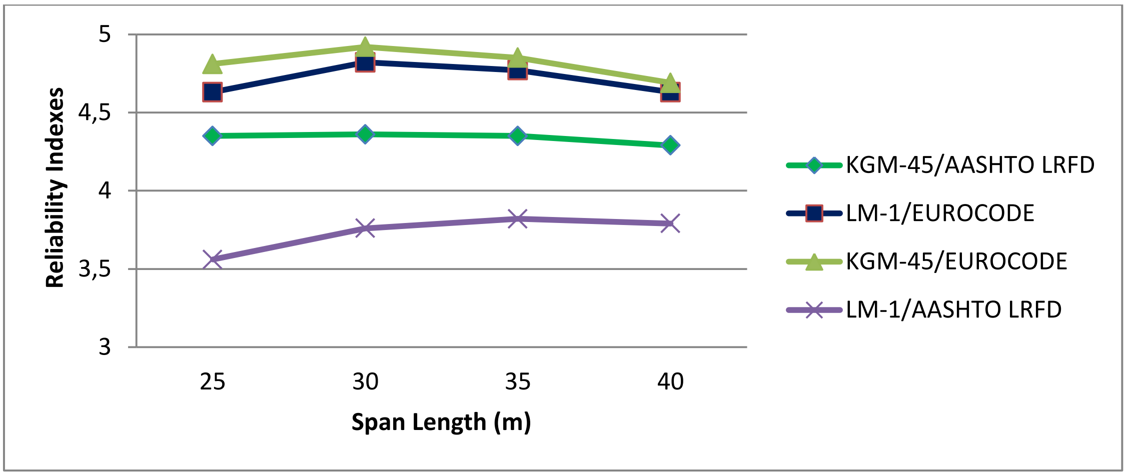


Figure 6-5 Reliability Index versus Span Length for Different Live Load Models and Design Codes

In the last chapter, whole study will be summarized and concluding comments will be made. In addition to this, relevant recommendations will take place for future researches.

CHAPTER 7

SUMMARY AND CONCLUSION

7.1 Summary and Concluding Comments

In the design of highway bridges, different design methods such as load factor design (LFD) and load and resistance factor design (LRFD) are used. The LRFD method is relatively new concept in bridge engineering applications. This method aims to reach a uniform safety level for the components of bridge. AASHTO LRFD and EUROCODE-2 specifications were calibrated based on the LRFD method. These specifications have been taken as basis for the design and reliability analysis of bridge girders in this research.

In the scope of this study, four different precast prestressed bridge girders mostly preferred in Turkey have been selected for the various bridge span lengths such as 25 m, 30 m, 35 m and 40 m. These girders have been designed and analyzed for the maximum positive moment at the mid span based on the strength I limit states of AASHTO LRFD and ultimate limit states of EUROCODE-2 codes. In the design of girders, one type girder has been designed having as the same number of prestressing tendons and girder spacing for each span length. Then, flexural resistance capacities of these girders at the mid-span have been calculated according to the AASHTO LRFD and EUROCODE-2 specifications.

In the design of girders, KGM-45 loading and Eurocode Load Model 1 are taken as live load models. On the other hand, same dead load components are considered for each girder type. For the evaluation of safety level of precast prestressed concrete girders, the statistical parameters which are mainly the probability distribution, bias factor and coefficient of variation of load and resistance elements have been determined. Load components have been taken as live load, dead load, impact factor and girder distribution factor and the resistance components have been thought as tensile strength of prestressing tendon, compressive strength of concrete, dimensions and theoretical behavior for the calculation of statistical parameters.

Statistical parameters for the live loads have been estimated by using the local truck survey data gathered in Turkey. This data includes the axle distance and weight of 28,000 trucks for the years of 2005, 2006 and 2013. For the determination of other statistical parameters, the results of local and international researches that process the different kinds of uncertainties related with load and resistance elements have been used.

In the evaluation of live loads statistical parameters, extreme value theory has been elaborated. In this context, the truck survey data is extended to the future in order to obtain more extreme data than the observed data. These extrapolations are performed by three different cases such as overall, upper-tail and extreme. In the reliability analysis, the results of extreme case have been taken into account since this case reflects the most critical scenario.

After all, the reliability indexes have been calculated on bridge cross-sections designed for each span length under the KGM-45 and Eurocode LM-1 live load models by using the Mean Value First Order Second Moment method.

According to the analysis and design results of this study, it can be concluded that the maximum moments per girder due to the KGM-45 truck loading is higher than the maximum moments per girder due to the Eurocode LM-1 truck loading. Total weight of KGM-45 truck is smaller than the heaviest lane load of Eurocode LM-1 truck. However, LM-1 load girder distribution factors of EUROCODE-2 provisions are smaller than the ones computed with the AASHTO LRFD specifications that results in high moments for KGM-45 truck as mentioned above.

Furthermore, flexural moment capacities of girders which have a same cross-section, number of prestressing tendons and girder spacing calculated based on the AASHTO LRFD specifications are greater than the moment capacities of girders determined by EUROCODE-2 specifications.

In accordance with the analysis and design results of bridge girders, similar conclusions can be drawn based on the reliability analysis results. It shall be noted that girder design is based on AASHTO-LRFD requirements in both LM-1 and KGM-45 analysis. As illustrated in the Table 6-3 and Figure 6-4, reliability index of girders are equal to the 4.35, 4.36, 4.35 and 4.29 for span length of 25 m, 30 m, 35 m and 40 m under the KGM-45 truck loading. Besides, reliability index of the same girders are equal to the 4.63, 4.82, 4.77 and 4.63 for span length of 25 m, 30 m, 35 m and 40 m under the Eurocode LM-1 loading. These results indicate that safety levels of girders analyzed per EUROCODE is greater than the same girders analyzed per AASHTO LRFD. In that case, it is apparent that use of AASHTO LRFD based cross-section design yields conservative results for EUROCODE load analysis checked with EUROCODE design capacities. On average, the ultimate moment, M_u, computed with Eurocode analysis method is about 25% less than the ultimate moment, M_u, computed with AASHTO LRFD analysis method. The Eurocode design capacity is about 20% less than the design capacity computed from AASHTO-LRFD method. Therefore, in a real design case, these two methods shall never be mixed such as taking loads from one specification and taking capacities from the other specification. For the AASHTO LRFD and EUROCODE, the largest values of reliability index (β) are for the span of 30 m length and decrease as the span lengths increase. Finally, these results also show that AASHTO LRFD has uniform reliability levels comparing with the EUROCODE.

7.2 Recommendations for Future Studies

The studied problematique in this thesis can be repeated for safety levels of various bridge types such as suspension bridges, arch bridges and truss bridges. In addition to this, same reliability comparison can be done for other types of precast prestressed concrete girders.

On the other hand, different than the flexural moment capacity of the girders at mid span, negative moments and shear force may be investigated. Moreover, these examinations can also be made for the other parts of the bridges such as piers, abutments, foundations and piles.

The benchmarking of reliability levels can be assessed by considering different live load models and various load combinations. Additionally, different limit states for the same design codes may also be considered.

Other than that the comparison of safety level can be performed based on the other design specifications.

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APPENDIX

In this part, reliability levels of girders designed under KGM-45 and Eurocode LM-1 truck loading have been verified. For this verification, Argınhan's (2010) thesis results were taken into account. In the subject study, the reliability levels of same types of precast concrete girders have been investigated for span lengths of 25, 30, 35 and 40 m. In the calculations, H30-S24 and HL-93 truck types were taken as live load models.

The reliability levels of girders were recalculated under H30-S24 and HL-93 truck loading for extreme case by using the statistical parameters of resistance and load in this research. Although designed girders and truck survey data used to generate live load models were different in each study, it was expected to obtain close safety levels for both of truck loading. The comparison of results is illustrated in the below tables and figures.

Table A-1 Comparison of Reliability Index Values for HL-93 Truck Loading

Span Length(m)	Reliability Indexes (β)	
	HL-93 (Recalculation based on this study)	HL-93 (Argınhan's study results)
25	3.90	3.92
30	3.87	3.92
35	3.83	3.85
40	3.67	3.67

Table A-2 Comparison of Reliability Index Values for H30-S24 Truck Loading

Span Length(m)	Reliability Indexes (β)	
	H30-S24 (Recalculation based on this study)	H30-S24 (Argınhan's study results)
25	4.44	4.48
30	4.35	4.49
35	4.35	4.43
40	4.29	4.21

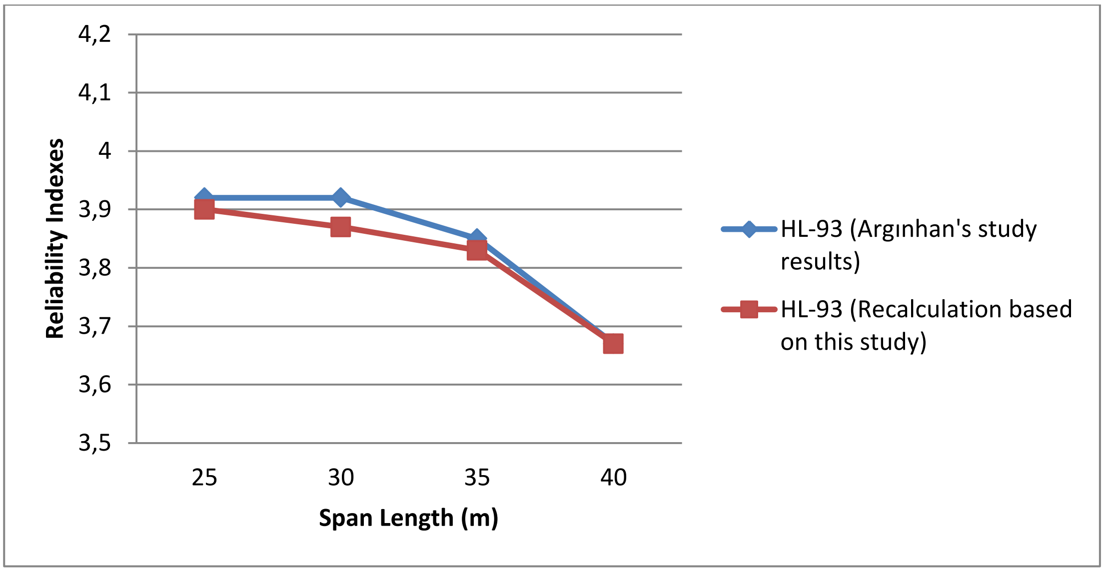

Figure A-1 Comparison of Reliability Indexes for HL-93 Truck Loading

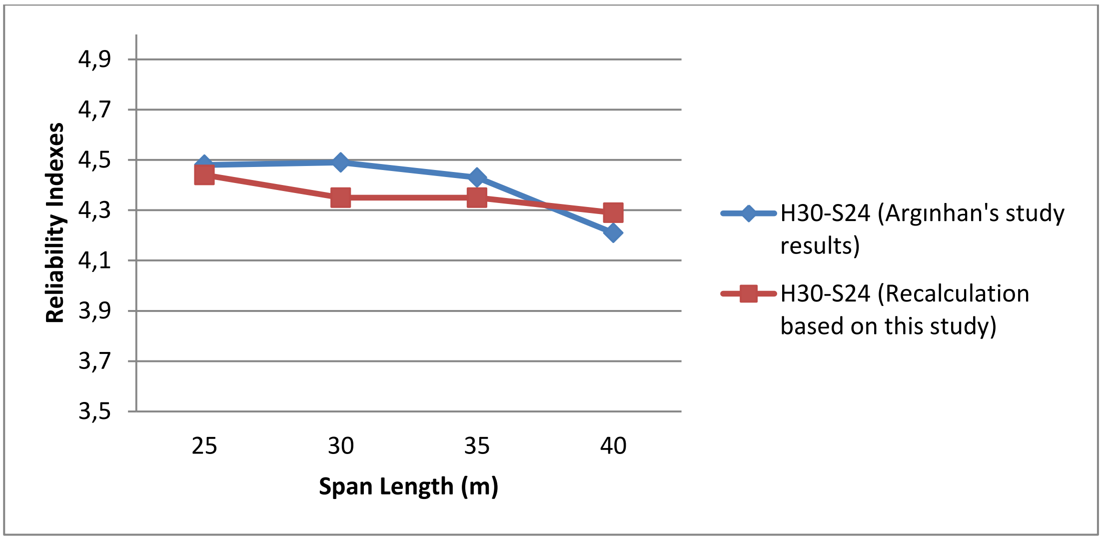

Figure A-2 Comparison of Reliability Indexes for H30-S24 Truck Loading

Consequently, these results indicate that girders designed in the scope of this study have uniform safety levels and give reasonable reliability index values under different live load models.