RELIABILITY BASED EVALUATION OF SEISMIC DESIGN OF TURKISH BRIDGES BY USING LOAD AND RESISTANCE FACTOR METHOD

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ABSTRACT

RELIABILITY BASED EVALUATION OF SEISMIC DESIGN OF TURKISH BRIDGES BY USING LOAD AND RESISTANCE FACTOR METHOD

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This study aims to evaluate the safety level of seismic design of Turkish highway bridge pier columns with respect to reliability theory. Evaluation of bridges was performed for four different codes of American Association of State Highway and Transportation Officials (AASHTO). Reliability indices were calculated for AASHTO Load and Resistance Factor Design 2010 (AASHTO LRFD 2010), AASHTO LRFD 2007, AASHTO Standard Specifications of Highway Bridges (LFD 2002) and Turkish modification of AASHTO LFD 2002 (current design code for highway bridges in Turkey). In the scope of project number of 110G093 “Development of Turkish Bridge Design Engineering and

Construction Technologies" associated with Middle East Technical University (METU) and Scientific and Technological Research Council of Turkey (TUBITAK) and Turkish bridge design authority General Directorate of Highways (KGM), a new seismic design chapter was proposed based on LRFD provisions. In this study, proposed specifications were also evaluated based on the structural reliability methodology. The statistical data of all components in seismic load demand and column carrying capacity were studied based on both local and international literature to assess the uncertainties. Seismic load demand was calculated with response spectrum analysis of simplified single degree of freedom models of various bridges. Load carrying capacity of pier columns were checked for bi-axial bending combined with axial compression. For that purpose, an algorithm based on Green's theorem was written for development of the interaction surface adopted from the "Interaction Surfaces of Reinforced – Concrete Sections in Biaxial Bending" (Fafitis, 2001). Reliability index was calculated with numerical Monte Carlo Simulation Method and compared with First Order Reliability Method and First Order Second Moment Reliability Methods.

Keywords: Reliability Index, Target Reliability, Seismic Design, Load and Resistance Factor Design

ÖZ

YÜK VE DAYANIM KATSAYILARI YÖNTEMİNE GÖRE KÖPRÜ ORTAAYAKLARININ TÜRK DEPREM TASARIMININA GÖRE GÜVENİLİRLİK BAZINDA DEĞERLENDİRİLMESİ

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Bu çalışma, Türkiye karayolu köprülerinin, ortaayak kolonları deprem tasarımının güvenilirlik teorisine göre değerlendirilmesini hedeflemiştir. Türkiye karayolları köprüleri American Association of State Highway and Transportation Officials, Standard Specifications for Highway Bridges 17th Ed. (AASHTO LFD 2002) kullanılarak tasarlanmaktadır. Ancak tasarım koşulları “Yük ve Dayanım Faktörü (LRFD)” metoduna göre güncellenmektedir. AASHTO LRFD 2010, AASHTO LRFD 2007, AASHTO LFD 2002,

Türkiye'de kullanılan revise edilmiş AASHTO LFD 2002'ye göre tasarlanmış köprülerin güvenilirlik seviyesinin belirlenmesi için çalışılmıştır. Karayolları Genel Müdürlüğü tarafından ODTU ve TUBITAK ile birlikte yürütülen 110G093 numaralı "Türkiye Köprü Mühendisliğinde Tasarım ve Yapımına İlişkin Teknolojilerin Gelirtirilmesi" projesinin kapsamında yeni bir deprem tasarımı bölümü oluşturulmuştur. Bu tez çalışmasında, devam eden proje kapsamında sunulması planlanan ilgili bölüme göre yapılacak köprülerin güvenilirlik seviyesi de belirlenmiştir. Gerçek yapısal durumları en uygun şekilde yansıtabilmek ve belirsizlikleri değerlendirmek için deprem yükleri ve kolon taşıma kapasitelerinin belirlenmesinde kullanılan tüm istatistiksel parametreler yerli ve yabancı literatürden elde edilmiştir. Deprem yükleri tek serbestlik dereceli köprü modellerinin tepki spektrumu analizi ile elde edilmiştir. Kolon taşıma kapasiteleri çift eksenli eğilme ve eksenel basınç tesirleri gözönüne alınarak belirlenmiştir. Bunun için, "Interaction Surfaces of Reinforced – Concrete Sections in Biaxial Bending" (Fafitis, 2001) de teorisi anlatılan, Green teoremi ile oluşturulmuş etkileşim yüzeyi algoritması hazırlanmıştır. Güvenilirlik katsayısı sayısal Monte Carlo simulasyonu yöntemi kullanılarak hesaplanmış, Birinci Mertebe Güvenilirlik Metodu ve Birinci Mertebe İkincil Moment Güvenilirlik Metodları kullanılarak analitik çözümler ile karşılaştırılmıştır.

Anahtar kelimeler: Güvenilirlik Katsayısı, Hedef Güvenilirlik Katsayısı, Deprem Tasarımı, Yük ve Dayanım Katsayısı Yöntemi

To my wife and family…

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CHAPTER 1

INTRODUCTION

1.1 General View

Bridges are one of the most important key elements of transportation systems. They are to be designed to sustain natural or human based load effects with a desired level of safety while retaining the construction cost as low as possible. The balance is provided with agreement of level of safety. Safety of a structure under the effects of design loads are decided by authorities of the country or the owner of the structure with respect to requirements of society.

Load effects are mostly uncertain and nominal parameters are defined with respect to statistical characteristics of the loads. Due to uncertainty of the components of structural system, safety shall be defined with statistical methods.

Level of the safety can be assessed based on reliability theory. Reliability index is a measure of safety in the probabilistic approach. Probability of failure can be determined based on the reliability index, considering the uncertainty of all the components of the system.

Many design specifications are based on reliability theory. In Turkey, highway bridges are designed according to modified version of American Association of State Highway and Transportation Officials Standard Specifications for Highway Bridges (AASHTO LFD). General Directorate of Highways (KGM) conducts a project to update the specifications to “Load and Resistance Factor Design” (LRFD). KGM aims to develop the existing design specifications and provide uniformity in safety levels of bridge designs. In the new specifications, seismic load definitions and capacity reduction factors are planned to be revised.

AASHTO LFD and LRFD were both prepared to provide safety level of bridges with respect to requirements for American society and economy. Load and resistance factors were calibrated with respect to conditions in the USA. Safety level of a bridge in Turkey, designed with respect to AASHTO codes, may be different. Thus, the code calibrations shall be checked with local uncertainties in design and construction parameters. The new specifications shall be recalibrated per local statistics of components to satisfy target level of safety.

1.2 Objectives

This study aims to evaluate the safety level of bridges designed with;

- AASHTO LRFD 2010
- AASHTO LFD 2002
- Modified AASHTO LFD 2002
- AASHTO LRFD 2007
- Modified AASHTO LRFD 2010 (proposed in the scope of the project number 110G093 “Development of Turkish Bridge Design Engineering and Construction Technologies” associated with Middle East Technical University (METU) and Scientific and Technological Research Council of Turkey (TUBITAK) and Turkish bridge design authority General Directorate of Highways (KGM).

1.3 Scope

Organization of the thesis is as follows:

In chapter 2, brief review of literature on structural reliability is given. Monte Carlo Simulation is one of the numerical methods for analyzing structural reliability problems. Monte Carlo simulation requires the generation of artificial test data of huge sizes, based on existing knowledge. Random number generation formulas are introduced for different distribution functions. Accuracy of the simulations is discussed at the end of the chapter.

In chapter 3, history of seismic design specifications is presented. The chapter aimed to show how the requirements of design specifications changed during the time.

In chapter 4, statistical parameters for components of resistance are discussed and assessed. Uncertainty in compressive strength of cast in place concrete, reinforced steel bars and section geometry are the variables in calculation of section capacity.

In chapter 5, statistical parameters for components of seismic load demand are discussed and determined. Seismic response coefficient, natural frequency, soil structure interaction, seismic mass, ductility capacity of the members and ground acceleration are the uncertain components of seismic load demand.

In chapter 6, failure function for each possible failure mode is defined. Reliability indexes are calculated with numerical Monte Carlo Simulation method and compared with analytical solutions. Seismic load demand is calculated with response spectrum analysis of single degree of freedom bridge models. Load carrying capacity of pier columns is checked for bi-axial bending with compression. Algorithm based on Green's theorem will be written for development of the interaction surface was adopted from the "Interaction Surfaces of Reinforced – Concrete Sections in Biaxial Bending" (Fafitis, 2001).

Reliability indexes for highway bridge pier columns are shown for different codes and designs with respect to the possible failure modes.

In chapter 7, the study and the results are summarized and conclusions are presented.

CHAPTER 2

LOAD AND RESISTANCE FACTOR DESIGN APPROACH

2.1 Structural Reliability

Structural design consists of various parameters of both demand and capacity, almost all of which are not deterministic, in other words, all parameters are random variables. The structural design has to consider a certain probability of target failure.

Target probability of failure, or probability of survival, is determined according to the requirements and expectations of the public and engineering community. For that purpose, specifications that lead to the minimum target expectations are imposed as design criteria in design codes. Specifications that consider the uncertainty of the parameters of design are typically called as reliability based design codes (Nowak & Kevin, 2000).

The structure's sufficiency to be in service without unplanned interruption and with expected properties during the pre-defined service life is referred as reliability of structure. The inability to fulfill that objective is called as failure, where the term failure does not mean collapse of a structure but implies loss of the expected service conditions.

Reliability approach has wide design code coverage, where codes are constructed on probabilistic modeling of load and resistance. Reliability approach can be used for not only for the design of new structures, but also for evaluation of existing designs. Some design codes that are based on structural reliability approach are American Institute of Steel Construction Load and Resistance Factor Design (AISC LRFD), Ontario Highway Bridge Design Code (OHBDC), American Association of State Highway and Transportation Officials (AASHTO LRFD), Canadian Highway Bridge Design Code, and Euro Codes.

Reliability of a structure can be a useful condition assessment parameter to decide whether to repair or demolish it. In most cases, failure of a member may not yield in the failure of whole structure due to alternative load paths. In reliability based approaches, a relation between member and system reliability can be established (Nowak & Kevin, 2000).

As all parameters of design are random variables, it is clear that resulting design has a certain probability of failure. For that reason, the aim is to achieve one of the following;

- For a limit cost, increase the reliability of design
- For a target reliability, decrease the cost of structure

Reliability of a design can be expressed by reliability index with a single value. Safety of a structure can always be improved; however, the probability of failure will never be zero. For that reason, design safety should have a minimum specified limit which will meet a certain probability of failure, with a minimum cost, as targeted. This aim requires an optimization of the economy and safety demands which will be discussed while assigning target reliability index (Nowak & Kevin, 2000).

Having the awareness that design problems involve random variables and focusing on reliability approach to consider the uncertainties of these random variables, designers and code makers can focus deeper in to the question of what are the uncertainties and which methods to apply for solution.

Uncertainties can be distinguished in two major components as aleatory and epistemic. Impossibility in prediction of loads such as earthquake, wind, live load etc. constitutes one of the causes of aleatory uncertainty. Variability of mechanical properties of materials is another cause for aleatory. Epistemic uncertainty arises from deviations in the form of the optimal design. These deviations can be illustrated as calculation errors, lack of knowledge, inadequate material or method of construction, revisions on site without analysis, inadequate maintenance etc.

Nowak & Kevin (2000) claimed that since loads and resistances are random variables, it is suitable to represent the loads and resistances as a combination of following three components. First is the physical variation factor which is the inherent variation of load and resistance (aleatory). Second is statistical variation factor which arises from the estimation of parameters from limited sample group (epistemic). As the sample size increases statistical variation factor decreases. Last is the model variation factor which is the uncertainty due to assumptions for simplification, unknown boundary conditions and other unknown variables (epistemic).

2.2 Monte Carlo Simulation Technique

It is possible to solve problems of reliability with analytical methods, as well as numerical methods. The numerical methods are typically used to overcome the weakness of analytical method with low number of samples. In this thesis study, Monte Carlo Simulation Method is used as the solution technique and the method will be introduced in this section.

As it can be inferred from the name, “Monte Carlo Simulation” method is used to simulate a parameter numerically and generate required number of observational data, without actual experiment. Limited amount of observed samples are analyzed to determine the statistical parameters. Determined statistical parameters are used to generate a large number of samples for Monte Carlo simulation. Using the generated samples, the problem can be solved numerically with a straight forward procedure to obtain the reliability index.

The simulation method is preferred in the cases where the failure function is complex to offer a closed form solution or a closed form solution does not exist.

The solution procedure is briefly as follows:

1. Samples randomly generated based on known information,
2. Failure function is calculated for every set of simulations using samples independently
3. Resulting number of failures are stored
4. Above steps are repeated until required number of simulations reached
5. Number of failures are divided to total simulation number to obtain probability of failure
6. Results of computations are plotted on probability papers (normal, lognormal etc.) to estimate the distribution characteristics.
7. In this study, both the seismic load demand and load carrying capacity of columns fitted in normal distributions. For that reason it is very easy to find the reliability index, β, by taking the inverse of the cumulative standard normal distribution (52). Details and example calculation is given in Chapter 6.2.

Many computer programs are capable of generating random numbers with certain distributions via their subroutines. However, it would be kept in mind that some routines are better than others, which means user should be careful in using them. For that reason, it is better to briefly describe the formulas to generate random variables.

Random variables can fit into uniform distribution, standard normal distribution, normal, lognormal distribution etc.

2.2.1 Uniformly Distributed Random Numbers

If one knows that sample u is a uniformly distributed random variable U between 0 and 1, samples x of uniformly distributed random variable X can be generated between any values a and b as follows;

$$x = a + (b-a)u \ , \ 0<u<1 \tag{1}$$

2.2.2 Standard Normal Random Numbers

In order to generate standard normal random numbers z, corresponding numbers u distributed uniformly should be generated between 0 and 1. Then using uniformly distributed u, standard normal distributed z can be generated as follows with inverse of normal distribution function Φ;

$$\mathrm{z} = \Phi^{-1}(\mathrm{p}) = -\mathrm{t} + \frac{c_0 + c_1 t + c_2 t^2}{1 + d_1 t + d_2 t^2 + d_3 t^3} \tag{2}$$

$$t = \sqrt{-\ln(u^2)} \tag{3}$$

where c_i and d_i are constants and have values given below;

c_0 =2.515517

c_1 =0.802853

c_2 =0.010328

d_1 =1.432788

d_2 =0.189269

d_3 =0.001308

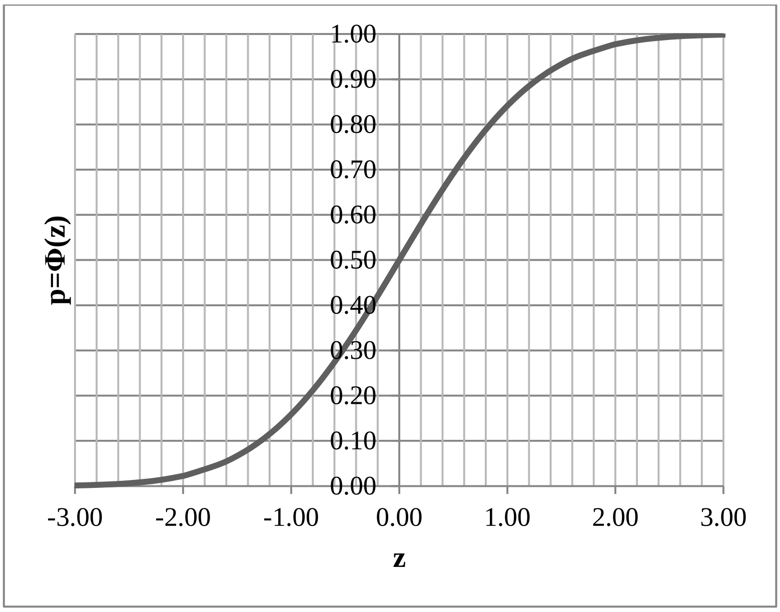

Figure 2-1 : Cumulative Distribution Function of Standard Normal Random Variables (z)

2.2.3 Normal Random Numbers

From the standard normal distributed random values as generated in previous section, normal random numbers can be generated. For a normally distributed random variable X, with mean μ_x and standard deviation σ_x, the relationship between X and standard normally distributed variable Z is as follows;

$$X = \mu_x + Z\,\sigma_x \quad (4)$$

2.2.4 Lognormal Random Numbers

For random variable X which is log-normally distributed with mean μ_x and standard deviation σ_x, normally distributed variable z could be generated based on standard normal distributed variable as described in previous section. Using the relation between normal and lognormal variables;

$$X = e^{[\mu_{\ln x} + z\sigma_{\ln x}]} \quad (5)$$

$$\sigma_{\ln x}^2 = \ln(V_x^2 + 1) \qquad (6)$$

$$\mu_{\ln x} = \ln(\mu_x) - \frac{1}{2}\sigma_{\ln x}^2 \qquad (7)$$

$$V_x = \frac{\sigma_x}{\mu_x} \qquad (8)$$

2.2.5 Accuracy of Simulation

Probability of failure can be estimated by using Monte Carlo simulation. Calculated failure probability is only a failure probability of the generated sample size in consideration. Failure probability may vary from sample size to sample size which indicates that calculated probability of failure using Monte Carlo simulation is also a variable which has a certain mean and variation for each sample size.

If P_{true} is the theoretically correct probability of failure that is attempted to be calculated, expected value, E[P'], variance, σ_p^2, coefficient of variation, V_P, of the calculated probability P' can be represented as follows (Soong & Mircea Grigoriu, 1993);

$$E[P'] = P_{true} \qquad (9)$$

$$\sigma_{P'}^2 = \frac{1}{N}[P_{true}(1 - P_{true})] \qquad (10)$$

$$V_{P'} = \sqrt{\frac{(1 - P_{true})}{NP_{true}}} \qquad (11)$$

where N is the number of simulations.

It is clear from the expressions above, as the number of simulations increases, uncertainty of the estimate decreases. Considering the inverse, required number of simulations can be found for desired accuracy as follows;

$$\mathrm{N} = \frac{(1 - P_{\mathrm{true}})}{\sigma_{\mathrm{P'}}{}^{2} P_{\mathrm{true}}} \tag{12}$$

CHAPTER 3

AN OVERVIEW OF SEISMIC DESIGN OF BRIDGES

Seismic design of highway bridges in Turkey is adopted from the American Association of State Highway Transportation Officials. The focus of this chapter is to review the seismic design philosophy of the Association.

In United States, design code for highway bridges is published by the American Association of State Highway and Transportation Officials (AASHTO). The first publication was in 1931 and requirements for seismic design were not included until 1940. In 1941, a newer version was released which included an awareness for seismic design of the highway bridges. However, no reference was listed to estimate the seismic loads. In order to meet the deficit, California Department of Transportation (Caltrans) elaborated equivalent static forces as lateral seismic design forces for various types of foundations. Allowable Stress Design philosophy (ASD) was adopted in member design checks. In 1961, AASHTO adopted the seismic load definitions by Caltrans with ASD design approach (Moehle, 1995).

Ger, J. & Cheng, F.Y. (2012) made an overview of seismic design of highway bridges as follows.

Publication in 1969 required that an equivalent lateral force to be applied as seismic load on the bridge analysis as follows:

$$EQ = CxD \tag{13}$$

where;

EQ : Lateral earthquake load applied at the center of gravity of the structure.

D : Dead load of the structure

C : Response Coefficient

- 2.00% for spread footings on soil rated as $4 kg/cm^2$ or more
- 4.00% for spread footings on soil rated as less than $4 kg/cm^2$
- 6.00% for structures founded on piles.

Equivalent lateral earthquake load was combined with dead load (D), earth pressure (E), buoyancy (B) and stream flow (SF) forces under the combination of Group VII.

$$Group\ VII = D+E+B+SF+EQ \tag{14}$$

ASD method allowed 33% increment for the allowable stress in seismic design forces. For the bridge columns in bending, code allowed $0.40f_c$' compressive stress at the extreme fibers and tension was not allowed.

In 1971, San Fernando Earthquake occurred. Many highway bridges designed per AASHTO seismic design requirements were heavily damaged or collapsed. Assessment studies following the earthquake incident revealed that the current requirements for seismic design of the time were insufficient for four reasons. First, the lateral force coefficients given in (13) were insufficient for California.

Second, column capacity demand exceeded the actual capacity. Thirdly, ductility requirements were inadequate that many brittle failure modes observed. Lastly, energy degradation was very low.

San Fernando earthquake revealed that soil effects on seismic load had to be included and dynamic properties of the structure had to be considered. In addition, ductility shall be improved by increasing the transverse reinforcements of the columns and seating length should be improved not to experience unseating type of collapses.

In 1975, AASHTO released a new publication of seismic design requirements based on Caltrans which had been used until 1992.

The equivalent static load method was valid in that code also.

$$EQ=CxFxW \tag{15}$$

where;

EQ : Lateral earthquake load applied at the center of gravity of the structure.

F : Framing factor

1.0 for structures supported with single columns

0.80 for structures supported with continuous frames

W : Dead weight of the structure

C : Response coefficient

$$C = A\ R\ \frac{S}{Z} \tag{16}$$

where;

A : Maximum ground acceleration expected. (PGA)

(Introduced with a new seismic risk map for United States) (Figure 3-1)

R : Normalized acceleration response spectral value for rock sites

S : Soil amplification factor

Z : Force reduction factor (accounts for ductility degree of the system)

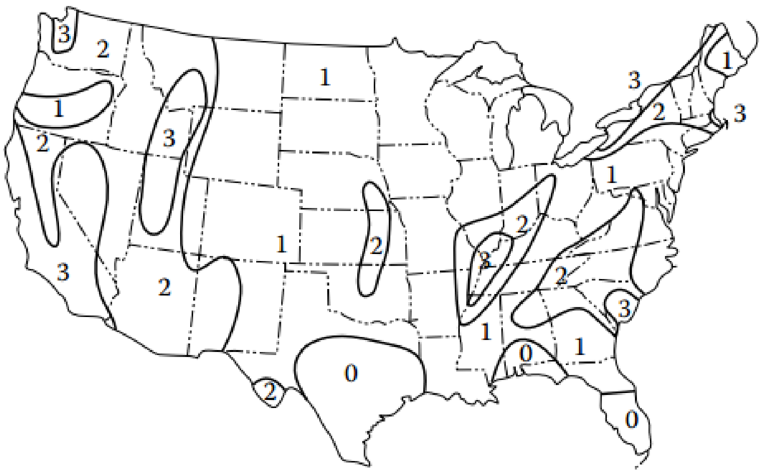

Figure 3-1 : National Seismic Risk Map (Standard Specifications for Highway Bridges, 12th Ed., 1977)

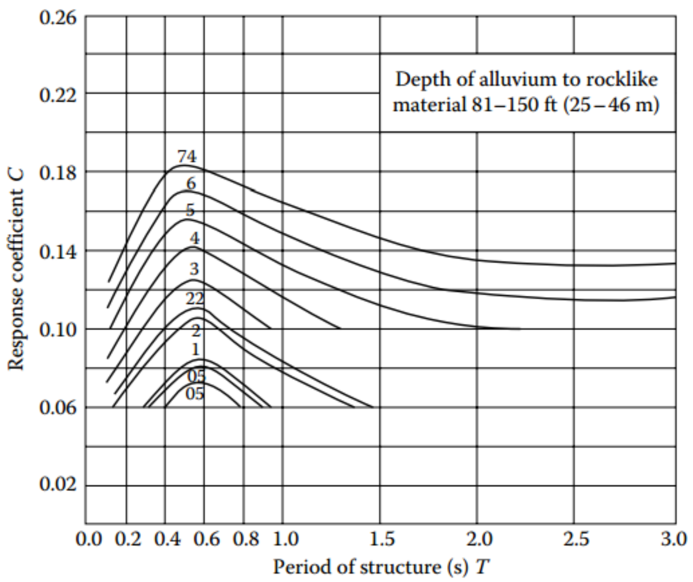

Figure 3-2 : Response Coefficient for Different Structural Periods (Standard Specifications for Highway Bridges, 12th Ed., 1977)

It was the first time; the seismic risk map presented in Figure 3-1 was included in design code. Soil amplification factor, spectral response and reduction factors were mentioned but no guidance was referenced. As it had been used in the previous ASD code, 33% increment was also allowed on allowable stresses for members for Group VII loading.

Table 3-1: Maximum Expected PGA for Different Zones (Standard Specifications for Highway Bridges, 12th Ed., 1977)

Maximum Expected PGA for Different Zones	
PGA Value	Zone
0.09	1
0.22	2
0.5	3

Experiences gained during the San Fernando earthquake in 1971, lead the code makers to introduce the load factor design (LFD) with ductile columns.

$$Group\ VII = \gamma(\beta_D D + \beta_E E + B + SF + EQ) \tag{17}$$

where γ is the combination factor and β_D, β_E,are the load factors.

D : Dead Loads

E : Earth Pressure

SF : Stream Flow Pressure

B : Buoyancy

EQ : Earthquake Load

In 1992, procedures in seismic design of bridges were improved considerably. In that edition, the seismic design requirements were based on the "Seismic Design

Guidelines for Highway Bridges" published by Applied Technology Council in 1981 and that design procedure were used until 2008.

First major change in 1992 document was focused on the analysis methods. The 1992 code required the structures to be analyzed with elastic response spectrum methods in substitute for equivalent lateral static earthquake forces adopted in the previous code. Soil was classified from hard soils to soft soils and the soil classification at the bridge location was included in formulation of elastic response spectrum. The seismic loads used in the design were required to be applied in two horizontal axis of the bridge, and these orthogonal seismic forces needed to be combined. The member bending moment forces obtained on columns were to be divided by a response modification factor, R, to consider the ductility of the columns (Table 3-2). The columns were designed to have a minimum transverse reinforcement required by the code to provide a minimum ductility of the member.

Unlike the previous editions, strength reduction factor, ϕ, was numerically defined for the designer to be able to consider the estimated column ductility capacity. Subsequent to that Group VII load combination was defined as follows;

$$Group\ VII = 1.0(D+E+B+SF+EQM) \qquad (18)$$

where EQM is the modified earthquake load.

Table 3-2 : Response Modification Factors (AASHTO Standard Specifications for Highway Bridges 16th Ed. 1996)

Response Modification Factors	
Substructure	R
Wall-Type Pier	2
Reinforced Concrete Pile Bents	
- Vertical Piles only	3
- One or more battered piles	2
Single Columns	3
Steel or composite and steel and Concrete Pile Bents	
- Vertical Piles only	5
- One or more battered piles	3
Multiple Column Bent	5

A new approach was included in this edition for formulation of elastic seismic response coefficient C_s for the m^{th} mode is as follows;

$$C_{\mathrm{sm}} = \frac{1.2AS}{T_{\mathrm{m}}{}^{2/3}} \tag{19}$$

where;

A : Peak ground acceleration taken from Seismic Risk Map for 475 year return period. (Figure 3-3)

S : Site coefficient (ranges from 1 to 2 from hard soils, S_1, to softest, S_4)

T_m : Natural period of the member in the *m*th mode.

Design spectrum was determined mainly using the earthquake data in the United States for soil types S_1, S_2, S_3 and S_4. AASHTO published a more refined seismic risk map compared to the ones in the previous versions (Figure 3-3). An importance classification parameter was added in the equation which accounted for the importance of the bridge (Table 3-3).

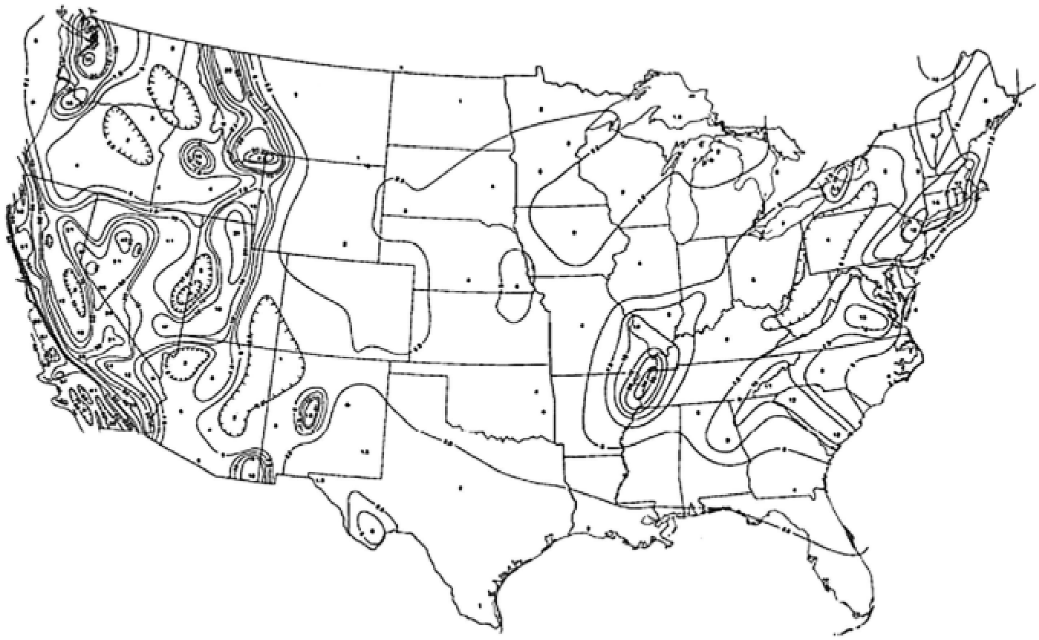

Figure 3-3 : PGA Acceleration Coefficient (AASHTO Standard Specifications for Highway Bridges 16th Ed. 1996)

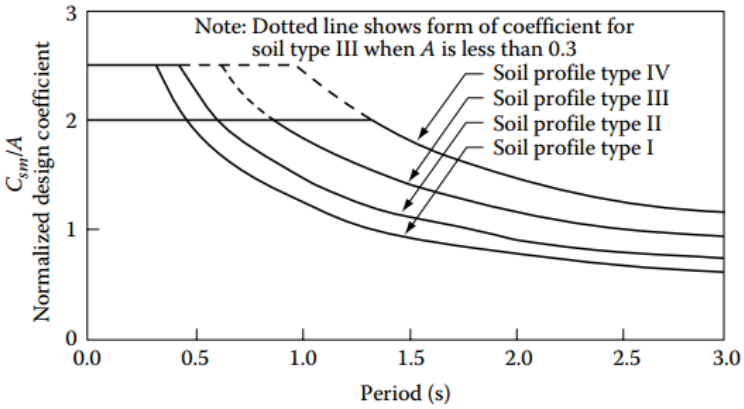

Figure 3-4 : Normalized Seismic Response Spectrum (AASHTO Standard Specifications for Highway Bridges 16th Ed. 1996)

Importance classification was modified in further editions. The importance classes expanded from two (where the classes were named as essential (I) and other (II)) to three where classes named as critical, essential and other. Critical bridges were aimed to be in service to all kinds of traffic after design earthquake where essential bridges are aimed to be in service for emergency and security traffic after the earthquake.

In new LFD code requirements, four classes of seismic performance categories were defined from A to D. For each zone, design methods and reinforcement requirements were different.

Table 3-3: Seismic Performance Category

Seismic Performance Category		
Acceleration Coefficient (g)	Importance Coefficient	
A	I	II
A≤0.09	A	A
0.09<A≤0.19	B	B
0.19<A≤0.29	C	C
A<0.29	D	C

LFD method was developed based on synthesis of theoretical knowledge, test evaluations and gained experiences from the past earthquakes. The recent trend in design is to develop a statistical background for the code requirements.

Load and Resistance Factor Design (LRFD) code mainly based on reliability theory and load combinations were founded on the following formula,

$$Q = \sum \eta_i \gamma_i Q_i \tag{20}$$

where;

η_i : Load modifier due to ductility, redundancy and importance for the load component, *i*

γ_i : Load factor for the load component, *i*

Q_i : Force effect of the load component, *i*

Using equation (19), Extreme Event I limit state corresponding to the old Group VII loading was combined as follows;

$$Q = \eta[\gamma_{\mathrm{DC}}DC + \gamma_{\mathrm{DW}}DW + \gamma_{\mathrm{EQ}}LL + WA + FR + EQM] \qquad (21)$$

where;

DC : Dead load of structural members

DW : Dead load of wearing surfaces and railings

LL : Live load

WA : Water load

FR : Friction load

EQM : Earthquake Load

where bending moment forces were modified with response modification factors given in Table 3-4.

In 2008 AASHTO LRFD Interim bridge design specifications, a major change in calculation of elastic seismic forces was adopted where 1000 year return period of earthquake USGS seismic risk maps were included for peak ground acceleration, 0.2 seconds and 1.0 second periods from Frankel et al. (1996). Ger, J. & Cheng, F.Y. (2012) claimed that the site effects considered in a much more realistic manner in the design spectrum. In the interim report, main basis for site effects was the studies performed after the Loma Prieta earthquake in California (1989). Many recommendations were found in different codes such as Uniform Building Code (UBC, 1997), National Earthquake Hazards Reduction Program Building Provisions (NEHRP, 1998) and International Building Code (IBC, 2000) were evaluated in developing the AASHTO LRFD.

Table 3-4 : Response Modification Factors (AASHTO LRFD Bridge Design Specifications for Highway Bridges 4th Ed. 2007)

Response Modification Factors, R			
Substructure	Critical	Essential	Other
Wall-Type Pier, larger dimension	1.5	1.5	2
Reinforced Concrete Pile Bents			
- Vertical piles only	1.5	2	3
- With batter piles	1.5	1.5	2
Single Columns	1.5	2	3
Steel or composite and steel and Concrete Pile Bents			
- Vertical Piles only	1.5	3.5	5
- With batter piles	1.5	2	3
Multiple Column Bent	1.5	3.5	5

Table 3-5 : Seismic Zones (AASHTO Standard Specifications for Highway Bridges)

Seismic Zones	
Acceleration Coefficient (g)	Seismic Zone
A≤0.09	1
0.09<A≤0.19	2
0.19<A≤0.29	3
A<0.29	4

Figure 3-5 shows the design spectrum in 2008 Interim report where short period and long period spectral accelerations were calculated as follows;

$$A_s=F_{pga}PGA \tag{22}$$

$$S_{DS}=F_aS_s \tag{23}$$

$$S_{Dl}=F_vS_l \tag{24}$$

where;

F_{pga} : Site factor for PGA coefficient

S_s : 0.2 second spectral acceleration from 0.2s map

F_a : Site factor for S_S

S_l : 1.0 second spectral acceleration from 1.0s map

F_v : Site factor for S_{Dl}

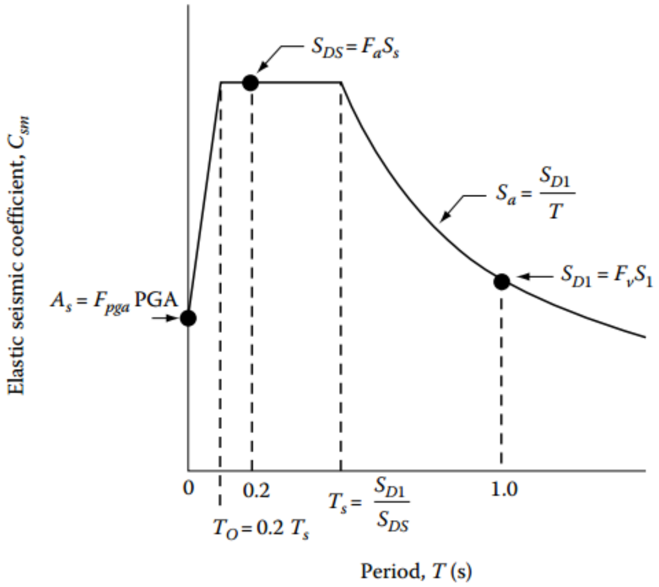

Figure 3-5 : Design Response Spectrum (AASHTO LRFD Interim Report, 2008)

1.0second spectral acceleration coefficient is used to define the seismic zone as given in Table 3-6.

Table 3-6 : Seismic Zones (AASHTO LRFD, 2010)

Seismic Zones	
Acceleration Coefficient $S_{D1}=F_vS_1$	Seismic Zone
$S_{D1}\leq 0.15$	1
$0.15<S_{D1}\leq 0.30$	2
$0.30<S_{D1}\leq 0.50$	3
$S_{D1}<0.50$	4

CHAPTER 4

ASSESSMENT OF THE STATISTICAL PARAMETERS FOR SECTION RESISTANCE

4.1 Statistical Parameters of Cast in Place Concrete

Piers were typically constructed using cast in place concrete construction in Turkish engineering practice. The design 28-days compressive cylinder strength varied from 25 to 35 MPa for various bridge projects in Turkey. After the 1999 Marmara Earthquake, construction with ready-mix concrete was allowed, only. This decision led to a very good quality control and standardization of the concrete. The results obtained from the thesis study of (Fırat, 2007) revealed this fact.

In the mentioned study above, 150x150x150mm cubic specimens supplied from various laboratories of different cities were investigated and mean and coefficient of variation of the compressive strength of concrete were listed for each year in Table 4-1. The mean compressive strength increased from approximately 20 MPa to30 MPa between 1995 and 2005, and a constant trend was achieved in 2000s. It could be concluded that the uncertainty in compressive concrete strength decreased significantly.

Table 4-1 : Statistical Parameters of 28-Days Compressive Strength Data According to Years for Turkey (Fırat, 2007)

Years	Number of Samples	Mean (N/mm^2)	Coefficient of Variation	Number of Values Under the Limit	Percentage of Values Under the Limit (%)
94/95	417	20.60	---	58	13.00
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
2004	1283	28.87	0.123	30	2.34
2005	615	29.97	0.120	24	3.90

C30 class concrete was accepted for all the designs used in this study since it was the most common concrete class found in Turkish bridges. Table 4-2shows the statistical parameters for various classes. It is remarkable that all mean values are greater than the characteristic strengths. Improved quality control in production with strict requirements in material codes, positive effects of chemical additives, development in curing and vibrating techniques resulted in better mean strength than expected characteristic one.

Table 4-2 : Statistical Parameters of 28-Days Compressive Strength Data According to Concrete Class for Turkey (Fırat, 2007)

Concrete Class	Number of Samples	f_{ck}, cyl, (f_{ck}, cube) (N/mm^2)	Mean (N/mm^2)	Coefficient of Variation	Number of Values Under the Limit	Percentage of Values Under the Limit (%)
C16	755	16 (20)	25.11	0.144	13	1.73
C18	739	18 (22)	25.82	0.120	23	3.11
C20	5817	20 (25)	28.46	0.104	118	2.7
C25	2767	25 (30)	32.48	0.100	53	2.81
C30	870	30 (37)	40.07	0.079	14	2.47
Overall	11085		29.87	0.105	222	2.65

The values obtained from concrete cube test were converted to cylinder compressive strength by dividing 1.23 since all the provisions and equations are related to cylinder compressive strength for the design. Converted values were given in Table 4-3.

Table 4-3 : Mean 28-Days Compressive Strength Values Converted to Cylinder Compressive Strength (Fırat, 2007)

Concrete Class	Mean (N/mm2)
C14	16.29
C16	20.41
C18	20.99
C20	23.14
C25	26.41
C30	32.57

It can be observed from the Table 4-3 that C30 class concrete has a mean compressive strength of 32.57MPa and coefficient of variation 0.079. This means that C30 class concrete has 2.57MPa of standard deviation which is the mean value times the coefficient of variation. It can also be calculated that C30 class concrete has a mean bias of 1.09 which is the ratio of mean compressive strength to the nominal compressive strength.

The aleatory uncertainty in compressive strength of concrete was summarized above and the epistemic uncertainty shall be considered. The study of Ellingwood & Ang(1972) showed that compressive strength of test specimens may be higher by 10% to 20% from the actual core concrete compressive strength. Another experimental study of Mirza et al. (1979) showed similar results for lower and upper bound as 4% and 36% respectively. The reasons of this epistemic uncertainty were improper placing and curing of concrete ($\overline{N}_{c1}$), difference of rate of loading ($\overline{N}_{c2}$), and human errors ($\overline{N}_{c3}$). Based on the literature data given, the epistemic uncertainty was taken into account by using the lower and upper mean biases 0.83 and 0.92, respectively from the study of Ellingwood and Ang (1972).

The probability distribution may be uniform, triangular or arbitrary based on the data. In this thesis study, upper triangle probability density function was preferred to define the epistemic uncertainty. Unlike the common reinforced concrete works in Turkey, bridge industry has sufficient defined quality control and concrete knowledge as per specified in KTS (2013)[1]. Figure 4-1 represents the upper triangle probability density function between the given limits.

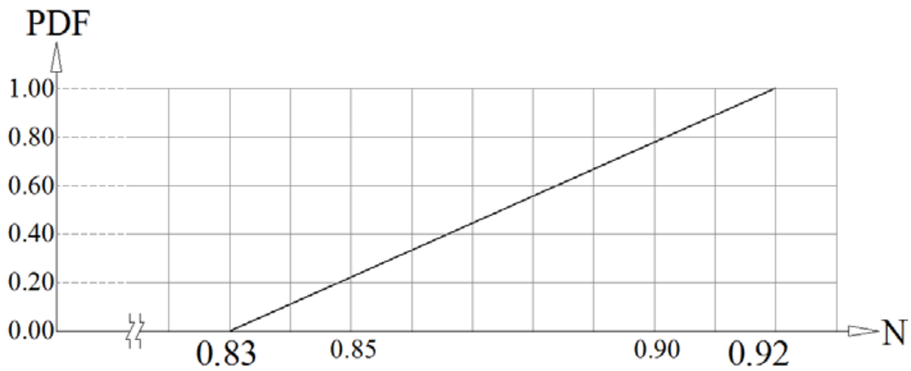

Figure 4-1 : Probability Distribution Function (PDF) For Bias of In Situ Concrete Strength

The correction factor $\overline{N_{c1}}$ can be calculated by using the upper triangle probability density function in Figure 4-1 with the equation (25) where N_{cU} and N_{cL} are the upper and the lower limits taken from Figure 4-1 as 0.92 and 0.83, respectively.

$$\overline{N}_{c1} = \frac{1}{3}(N_{cL} + 2N_{cU}) = \frac{1}{3}(0.83 + 2x0.92) = 0.89 \quad (25)$$

[1] Highways Technical Specifications, General Directorate of Highways, 2013, Ankara

$$\Delta_{c1} = \frac{1}{\sqrt{2}}\left(\frac{N_{\mathrm{U}} - N_{\mathrm{L}}}{2N_{\mathrm{U}} + N_{\mathrm{L}}}\right) = \frac{1}{\sqrt{2}}\left(\frac{0.92 - 0.83}{2x0.92 + 0.83}\right) = 0.024 \qquad (26)$$

Using equations 25 and 26, the mean value of the correction factor $\overline{\mathrm{N}_{c1}}$ was calculated as 0.89 and epistemic uncertainty Δ_{c1} as 0.024.

Rate of loading is usually higher than the actual site loading. This rate leads to an increase in the compressive strength calculated for test specimen. Kömürcü (1995) suggested a correction factor, $\overline{\mathrm{N}_{c2}}$, for accounting the effect of rate of loading with a mean value of 0.88 and with ignoring epistemic uncertainty (i.e. Δ_{c2}=0.0).

In addition to the mentioned uncertainties above, the human factor must be taken into account to predict the in-situ compressive strength of the concrete. The case of laboratory technician not to apply the testing procedure properly, or the difference of the laboratory mix and the mix for the actual construction can be accounted by a third correction factor, $\overline{\mathrm{N}_{c3}}$, which is taken from the study of Kömürcü (1995) as 0.95 (mean value) with a uncertainty of 0.05 (Δ_{c3})in terms of coefficient of variation. Using first order second moment approach, the correction factors above can be combined to find the mean bias resulting from the various discrepancies between the in-situ and laboratory conditions.

$$\overline{N_{\mathrm{fc}}} = \overline{N_{\mathrm{c1}}}\,\overline{N_{\mathrm{c2}}}\,\overline{N_{\mathrm{c3}}} = 0.89x0.88x0.95 = 0.744 \qquad (27)$$

As a result, the compressive strength of C30 class concrete placed in-situ was predicted and accepted as a base in the calculations given in this study as;

$$f_{\mathrm{c}} = \overline{f}_{\mathrm{c}} x \overline{N_{\mathrm{fc}}} = 32.57x0.744 = 24.24MPa \qquad (28)$$

The total epistemic uncertainty is the square root of sum of squares of the three uncertainties was calculated in equation (29).

$$\Delta_{\mathrm{fc}} = \sqrt{0.024^2 + 0.00^2 + 0.05^2} = 0.055 \tag{29}$$

Remember that aleatory uncertainty (coefficient of variation) was 0.079. Total uncertainty of compressive strength of in-situ concrete is also the square root of sum of squares of the epistemic and aleatory uncertainty;

$$\Omega_{fc} = \sqrt{0.055^2 + 0.079^2} = 0.095 \tag{30}$$

At that point, the previously mentioned bias factor shall also be revised for in-situ concrete. Bias factor is the ratio of nominal strength to mean compressive strength of in-situ concrete. Mean value for compressive strength of C30 class concrete was found as 24.24MPa where nominal strength is 30MPa. As a result the mean bias in compressive strength of C30 class concrete is 0.808.

4.2 Statistical Parameters of Reinforcement Steel

The iron and steel industry was established in the early years of new Turkish Republic. The first iron and steel plant was at Kirikkale region and in the following years, more developed Kardemir integrated plant had started production which is one of the backbones of Turkish industry and economy. Together with the economic and industrial growth and developments, other integrated plants such as Erdemir and Isdemir which are the biggest plants after Kardemir, were built to satisfy the demand. In recent years, steel export has increased significantly. Turkish iron industry exports products to over 100 abroad countries every year.

Turkey is one of the biggest ranking steel producing countries of the world and especially of the Europe. Export steel products have an important percentage in the production size. For that reason, products satisfying various standards and classes around the world can be found. It can be easily stated that the production quality and product variability has improved in Turkish iron and steel industry during the last decades. (Fırat, 2007)

The industry releases BCIII(a) type reinforcement steel to the local market which satisfies the requirements of TS708 (Turkish Standards of Reinforcement Steel for Reinforced Concrete).The evaluations in the following sections are made for this class of steel bars. The data conducted by Fırat (2007) will be used in this thesis.

Fırat (2007) investigated seven different iron and steel plants and collected the data for evaluating mean yield strength, coefficient of variation, ultimate strength and elongation. The data collected from these plants, data from the past experiments performed in the laboratories of Istanbul Technical University (ITU), Middle East Technical University (METU) and Selçuk University (SU) were combined.

The mean values and coefficient of variation for the yield strength, ultimate strength and elongation were reported in Table 4-4 to 4-6.

Table 4-4 : Mean and Coefficient of Variation of Yield Strength of BCIII(A) Rebars (Fırat, 2007)

		Manufacturers						
		Habas	Icdas	Ekiciler	Colakoglu	Egecelik	Yesilyurt	Kroman
Variables	Number of Samples	9619.00	1400.00	2390.00	530.00	1073.00	3024.00	1673.00
	Yield Strength (N/mm2)	530.01	516.79	480.94	473.63	489.71	464.40	460.71
	Coefficient of Variation	0.03	--	0.05	0.05	0.05	0.04	0.04

Table 4-5 : Mean and Coefficient of Variation of Ultimate Strength of BCIII(a) Rebars (Fırat, 2007)

		Manufacturers						
		Habas	Icdas	Ekiciler	Colakoglu	Egecelik	Yesilyurt	Kroman
Variables	Number of Samples	1400	2390	530	1073	1673	3024	10090
	Ultimate Strength (N/mm2)	621	563.81	617.95	631.45	675.68	593.1	609.1
	Coefficient of Variation	--	0.047	0.065	0.057	0.04	0.043	0.045

Table 4-6 : Mean and Coefficient of Variation of Elongation Percent of BCIII(a) Rebars (Fırat, 2007)

		Manufacturers						
		Habas	Icdas	Ekiciler	Colakoglu	Egecelik	Yesilyurt	Kroman
Variables	Number of Samples	1400	2390	530	1073	1673	3024	10090
	Elongation (%)	18.32	18.78	19.25	19.02	23.69	21.7	20.46
	Coefficient of Variation	--	0.076	0.094	0.097	0.105	0.081	0.087

The variability of ultimate and yield strength of steel bars are related to many parameters such as cross-sectional area of the rebar, rate of loading, yield strain and mechanical properties of bars for different bar diameters, in addition to variation of strength of the material. Although the strength variation does not considerably differ for a single bar along its length, the strength can be significantly varied for a group of product. This variation can be due to the difference in the manufacturing and also the bar diameter. Kömürcü (1995) indicated that for a sample batch taken from same manufacturer, the variation in yield strength ranged from 2% to 7% for several bar sizes in Turkey whereas this difference ranged from 1% to 4% in literature. In addition Mirza and Macgregor (1979) identified the range as 5% to 8%.

The mean yield strength and coefficient of variation in yield strength were tabulated in Table 4-4. The sample size of the data taken varies from one manufacturer to other. For this reason, the coefficient of variation was identified as the weighted average of the tabulated data for the inherent variability in the yield strength of the individual bar sizes.

Mean value of yield strength can be identified by introducing the mean correction factors for the rate of loading ($\overline{N_{s1}}$), defined yield strain ($\overline{N_{s2}}$) and prediction errors ($\overline{N_{s3}}$). It is a fact that as the rate of loading or rate of strain increases, obtained yield strength also increases contrary to the actual yield strength. It is also another fact that the rate of loading in reinforced concrete structures and testing standards are different. Loading rate and strain rate is higher in test procedures to determine the yield strength of steel rebars. For that reason yield strength is overestimated for rebars (Mirza, S.A., Hatzinikolas, M., & MacGregor, J.G., 1979). It is accounted by Ellingwood and Ang (1972) that mean bias $\overline{N_{s1}}$ is 0.90. In addition, Kömürcü (1995) claimed the prediction error Δ_{s1} to be negligible.

Kömürcü (1995) assumed the estimation error, Δ_{s2}, as 9% due to the fact that considering upper or lower yielding limit for determination of yield strength causes difference in results. This error was accounted as 5% in the study of Ellingwood and Ang (1972). However Mirza and Macgregor did not account for this factor. Fırat (2007) evaluated that as 5% prediction error (Δ_{s2}) with a mean value $\overline{N_{s2}}$ of 1.00.

Maximum value for prediction uncertainty was estimated as 11.2% (Fırat, 2007). However, it was stated that that uncertainty will be 0.00 in the case when the structure was built with rebars of only from one manufacturer. This does not reflect the truth for most structures. For that reason, an average of 5.60% prediction error (Δ_{s3}) was taken into account with a mean $\overline{N_{s3}}$, as 1.00.

Considering data from Table 4-4, the weighted average of the mean yield strength was calculated as 501.37 MPa. When the correction was made due to the above evaluations, the mean correction factor was computed as:

$$\overline{N_s} = \overline{N_{s1}}\,\overline{N_{s2}}\,\overline{N_{s3}} = 0.90x1.00x1.00 = 0.90 \tag{31}$$

As a result, the yield strength of BCIII(a) class reinforcement steel bars was predicted and accepted as a basis in this study as;

$$f_s = \overline{f}_y x \overline{N_s} = 501.37x0.90 = 451.23MPa \tag{32}$$

To calculate the total uncertainty, Δ_s, in correction factor, the square root of sum of squares of the three uncertainties mentioned above was calculated. The epistemic uncertainty was;

$$\Delta_s = \sqrt{0.05^2 + 0.00^2 + 0.056^2} = 0.075 \tag{33}$$

Average aleatory variability within a group of manufacturers was calculated from Table 4-4 as 3.80% (δ_s). Total uncertainty of the yield strength of BCIII(a) class reinforcement steel bars is also the square root of sum of squares of the epistemic and aleatory uncertainty;

$$\Omega_s = \sqrt{0.038^2 + 0.075^2} = 0.085 \tag{34}$$

Kömürcü (1995) used log-normal distribution. Nowak and Szersen (2003) and Topcu and Karakurt (2006) used normal distribution. Fırat (2007) stated that normal distribution was more suitable for the data collected.

4.3 Statistical Parameters for Geometry of Cross Section

Fırat (2007) determined that the uncertainty in the area of reinforcement bars differ for each diameters of bars. It was also assumed that all possible errors, apart from fabrication errors can be ignored. 1522 BCIII(a) reinforcement bars were tested and results were reported in Table 4-7.

Table 4-7 : Statistical Result of Reinforcement Areas (Fırat, 2007)

Bar Size	Number of Observations	Mean Bias	Standard Deviation	Mean of Aleatory Variablity
8	185	1.01	0.16	0.020
10	172	0.99	0.10	0.010
12	256	1.00	0.15	0.013
14	126	1.01	0.15	0.011
16	185	1.00	0.13	0.008
18	106	1.00	0.19	0.011
20	284	0.99	0.26	0.013
22	112	0.99	0.26	0.012
25	96	1.03	0.31	0.012
Overall	1522	1.00	0.20	0.012

In evaluation of the tabulated results, the mean bias was 1.00. Mean aleatory variability was 1.20%. In addition, Ellingwood and Ang(1972) and Kömürcü (1995) suggested prediction error of 3% and combining with aleatory uncertainty, the total uncertainty in reinforcement areas were:

$$\Omega_{As} = \sqrt{0.012^2 + 0.03^2} = 0.03 \qquad (35)$$

Due to the natural imperfections in workmanship of reinforced concrete structures, the design dimensions and locations may differ in the site. There are several reasons for this imperfection that can be summarized in the following explanations. The column construction starts during the construction of foundation which means that the column reinforcement must be placed before foundation is fully completed. In that duration, the rebar locations might be affected from the pouring, or other steel works. The reinforcement bars might not be placed perfectly at the beginning. When concrete hardens in foundation, location of the column reinforcement is fixed but considering the fact that no splicing is allowed in hinging locations, the length of the bars can extend up to 6.00-8.00 meters. For that reason, locating the bars in their proposed position in formwork is also a work that can include possibility of imperfection. One of the most common problems is vibration and pouring process. The process of pouring and the weight of the concrete itself create a pressure towards to outside of the formwork and expand the dimensions of the formwork. As the rigidity and fixity of the assembly of the formwork weakens the uncertainty in the external dimensions increases.

Using statistical parameters obtained directly from the literature search of worldwide studies was not reliable since local factors affect the uncertainty significantly. The uncertainty is directly affected by human errors and in situ quality control which can vary significantly for each country. It has been determined that no comprehensive study of statistical parameters of member dimensions apart from the very valuable study conducted by Fırat (2007).

Fırat (2007) collected measurements of 4216 building sites. It was reported that column cross-section height varied between 20 to 80cm where width ranged between 35 and 120cm.Common column dimensions of bridge piers were greater and direct suitability of the data in this study will be discussed later on this section.

In the evaluation of the collected data, Fırat (2007) reported that mean to nominal ratio varied between 0.932 to 1.027 for cross section width and 0.922 to 1.033 for cross section height. Mean values of nominal to mean ratio for width and height is 1.007 and 1.013, respectively. Fırat (2007) also founded that aleatory variability ranged between 1.80% and 9.30% for cross section width, w, where the results varied between 1.20% and 7.20% for cross section height, h. To conclude, mean aleatory variability was found to be 3.2% for width and 2.4% for depth.

To consider the expansion of the forms during the pouring stage, a prediction error N shall be determined. In order to find that, measurements taken from formwork which was in the exact clear dimensions before pouring and compared with the measurements after the forms removed for 42 samples by Fırat (2007). In evaluation, the prediction error was recommended to be 1.02 with ignoring the epistemic uncertainty in N. Bias factor were revised for the prediction error as 1.02x1.007=1.02 for width and 1.02x1.013=1.03 for height.

Finally, total uncertainty in cross section dimensions, namely width and height were calculated as follows:

$$\Omega_w = \sqrt{0.032^2 + 0.00^2} = 0.032 \tag{36}$$

$$\Omega_h = \sqrt{0.024^2 + 0.00^2} = 0.024 \tag{37}$$

It is also reported by Fırat (2007) that, aleatory variability for effective depth has a mean of 2.70%. In addition, it was claimed that external dimension of the column and depth of steel cage was perfectly correlated and mean value of mean to nominal ratio of the column effective depth was found as 1.01 where mean value of aleatory variability of 0.025.

Finally, total variability in the effective depth of a column was calculated as follows:

$$\Omega_{\mathrm{eff,w}} = \sqrt{0.025^2 + 0.027^2} = 0.037 \tag{38}$$

CHAPTER 5

ASSESSMENT OF STATISTICAL PARAMETERS FOR SEISMIC LOAD

5.1 Statistical Parameters for Design Spectra

Design spectra is usually generated based on the study of average response spectra developed from a large earthquake mapping as suggested by National Cooperative Highway Research Program Report 489 (NCHRPP 489, 2003). According to this document, at low seismic regions, design spectral acceleration coefficients have an average coefficient of variation of 15%. At high seismic regions, the average coefficient of variation is 40%. In another study conducted on probabilistic seismic hazard assessment in Delhi, coefficient of variation in design spectral accelerations varied between 15% and 60% (Joshi, G.J. & Sharma, M.L., 2011). Another research showed that coefficient of variation in California varies between 10 and 15%, coefficient of variation is between 20 and 30% in San Andreas, and 40-60% in Southern California. (Cao, T., Petersen, M.D., & Frankel, A.D., 2005).

Cao, T. et al. (2005), stated that coefficient of variation of design spectral accelerations decreases as return period increases.

A detailed earthquake mapping is out of scope of this study. The literature shows that upper and lower boundaries for coefficient of variation in design spectral accelerations are 60 and 15% in average.

5.2 Statistical Parameters for Natural Frequency

Determination of seismic load with elastic response spectrum analysis is directly related to the determination of natural frequency to find corresponding spectral acceleration.

Natural frequency of a system is related to stiffness of the member and mass of the system which means uncertainty can arise from uncertainty of the cross-section geometry, pier length, material property and seismic mass which will be discussed later. Structural modeling assumption can cause significant variation. To illustrate, considering soil structure interaction, one can calculate a lower natural frequency by 8%. Takada, Ghosn, and Shinozuka (1989) suggested this bias factor with coefficient of variation 0.20 for building type structures. Since this data account for soil structure interaction, it can be used for statistical calculations in this study where soil structure interaction was not taken into account.

5.3 Statistical Parameters for Seismic Mass

For the purpose of calibration of LRFD bridge design code, Nowak (1999) studied statistics of dead load and proposed bias factors and related coefficient of variations. For pre-fabricated concrete members, the ratio of the real dead load to the nominal dead load is 0.97 in average which corresponds to a mean bias factor of 1.03 and coefficient of variation of 0.08. For cast in place concrete members, the ratio of the real dead load to the nominal dead load is 0.95 in average which corresponds to a mean bias factor of 1.05 and coefficient of variation of 0.10. However, it was stated that this coefficient of variation considers both the uncertainty of weight and effect of structural analysis. Coefficient of variation only in member weight is 0.05. Unlike the concrete members, dead load of wearing surface has a higher uncertainty. Coefficient of variation in dead load of wearing surface is 0.25.

Probability distribution was suggested to be normal by both Nowak (1999) and Ellingwood (1980) in their researches.

5.4 Response Modification Factor

Common types of bridge piers are designed with linear elastic analysis of the system. Linear elastic forces which are excessively high for a column cross section to provide adequate capacity. Instead of designing a cross section to overcome that excessive load, the column cross section is allowed to reach the ultimate flexural capacity; provided that ductility capacity is not reached. If the member is capable to meet the high deformations without collapsing, it would survive the earthquake. In other words, earthquake motion will enforce the structure to displace in a small time interval rather than resulting in temporary flexural forces, so the capability to resist the required displacement without reaching ultimate flexural capacity can be defined as the ability of the member to resist earthquake impacts. For that reason, it is almost impossible to design a section to remain elastic during earthquakes, in fact, it is not needed to. Considering the ductility capacity of a member in the design will lead designer to design a member with adequate capacity.

In the pier design procedure, ductility demand is introduced with response modification factor. This factor is introduced as “response modification factor” in U.S. National Earthquake Hazard Reduction Program (NEHRP) and American Association of State Highway and Transportation Officials (AASHTO), “system modification factor” in Uniform Building Code (UBC).

Response modification factor called R has two main components;

- Nonlinear behavior of the system
- Structural over strength

The component of response modification factor, R_μ is basically described as the ratio of elastic flexural capacity which is yield strength of the section to the ultimate flexural capacity. The ultimate flexural capacity in this study is not defined as the collapse strength but the strength where the tolerable displacement capacity is reached.

For a defined earthquake load, required displacement capacity is fixed and as the yield strength decreases, the ductility demand increases. In other words, as the response modification factor increases, ductility demand increases and these parameters are proportional to each other. For that reason, the aim of a design is to define the response modification factor such that, the minimum flexural capacity requirement will be obtained whereas ductility requirements larger than the defined response modification factors will not take place.

Miranda and Bertero (1994) described the situation graphically as given in Figure 5-1.

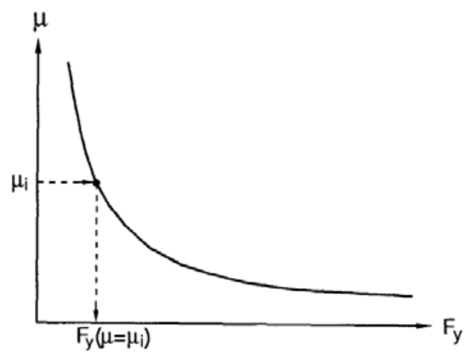

Figure 5-1 : Relationship Between Flexural Strength Of System And Displacement Ductility Demand, (Miranda & Bertero, 1994)

Miranda and Bertero (1994), combined and summarized previous 13 studies of Newmark and Hall (1993), Riddell and Newmark (1979), Elghadamsi and Mohraz (1987), Riddell, Hidalgo and Cruz (1989), Arias and Hidalgo (1990), Nassar and Krawinkler (1991), Vidic, Fajfar and Fischinger and Miranda (1992) on reduction factor due to nonlinear behavior and damping of the system in Figure 5-2.

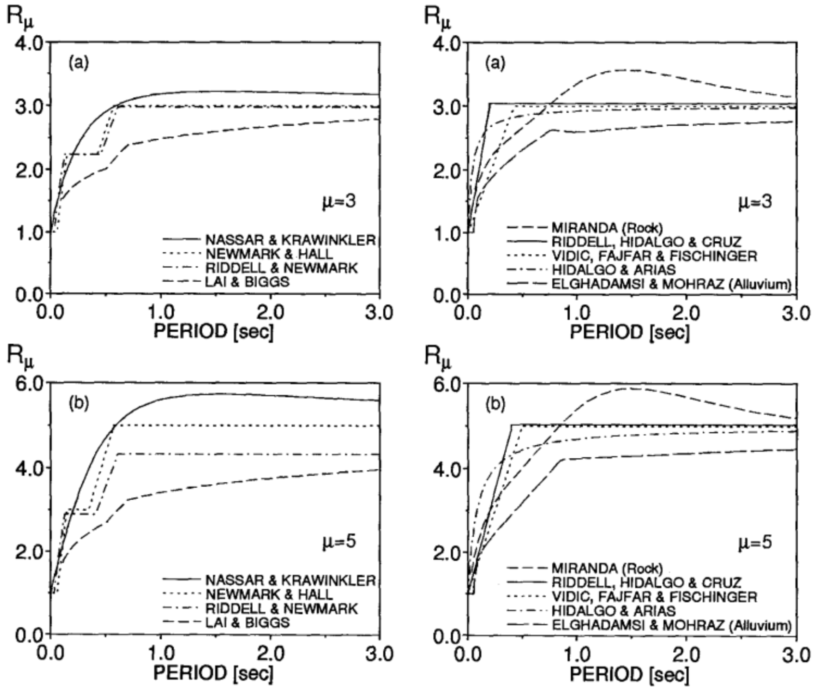

Figure 5-2 : Comparison of Strength Reduction Factors Of Previous Studies (a) M=3 and (b) M=5 (Miranda & Bertero, 1994)

Comparison of mean strength reduction factors for 111 different sites and ground motions is given in Figure 5-3 and the result revealed that reduction factor due to nonlinear behavior does not differ significantly for different sites and seismic regions (Miranda & Bertero, 1994).

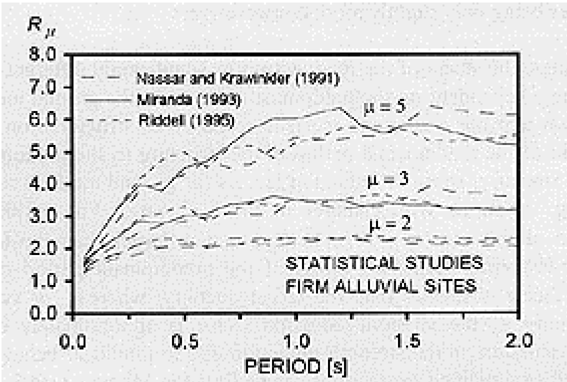

Figure 5-3 : Comparison of Mean Response Modification Factors (Miranda & Bertero, 1994)

In addition to the studies of Miranda et al.(1994) where the relation of response modification factor and ductility capacity of the columns are discussed, the accuracy in calculating ductility capacity have to be considered. For that purpose, observations of the study of Priestley and Park (1987) to determine ductility of concrete bridge columns under seismic loading will be introduced. It was shown in the study that real ductility of bridge columns was greater than the nominal ductility capacity by 1.50 times on average. The coefficient of variation for that relation was observed as 34%. In other words, the bias factor for response modification is 1.50 with coefficient of variation 0.34 and probability distribution as normal.

Lastly, Hwang, Ushiba, and Shinozuka (1988) suggested that mean reduction factor for shear walls as 7.0 with coefficient of variation 0.40. Wall type piers behave as shear walls. For that reason, these values will be used for wall type piers in this study.

5.5 Statistical Parameters for Modeling Errors

The discussions so far described the uncertainties of each parameter that affect the earthquake load on the system that is subjected to it. However, it cannot be affirmed that the uncertainty in earthquake loading is completely defined. In addition to the parameters in load modeling, the uncertainty in the load modeling itself shall be considered. To clarify, the errors in the assumed distribution of equivalent static force over the height of the structure, superposition of modal response, errors of cumulative distribution of peak ground acceleration should be taken into account. Moreover, the effects of translational restraints of slab in its plane, or accuracy of the calculation of tributary mass, uncertainty of soil properties as well as the uncertainty in classification of soil for seismic parameters should be taken into consideration thought, too. On the other hand, single mode lumped mass model constitutes the basis of the calculations in this study. Effect of this has to be taken into account. Lastly, confidence level of suggested earthquake intensity should be considered.

To claim that the reliability analysis is properly established, all parameters which are inherently random variables that affect the reliability of the design should be included. Additional sources of uncertainty as mentioned in the previous paragraph shall be included as modeling uncertainty that is modeling and prediction errors which would be idealizations of real earthquake loads in time and space or incorrect assumptions of probability distribution functions of the parameters.

Ellingwood et al.(1980) also claimed that modeling errors can be estimated for only some occasions and suggested with professional judgment that a value (in this study called λ) with a mean and bias factor of 1.0 and coefficient of variation 0.2 or higher.

In this study, coefficient of variation in modeling errors was not incremented further and used as 0.20 with a mean and bias factor of 1.0 with normal distribution as suggested by Ellingwood et al.(1980).

5.6 Statistical Parameters for Ground Acceleration

Peak ground accelerations used in Turkish design practice are given at bedrock level for five different seismic zones in Turkish Seismic Zoning map (Gulkan et al. 1993). Using these accelerations with relevant site coefficients of corresponding soil classes, design spectrums are developed.

Peak ground accelerations used in Turkish Earthquake Code have probability of exceedance of 10% in 50 years (475 years return period). However, bridges are designed to serve for 75 years life time. For that reason, probability of exceedance of design acceleration in 75 years shall be calculated. Large magnitude earthquakes are physically dependent in nature but they can be assumed to be statistically independent and discrete events in time. They are accepted to have Poisson distribution in time. In other words, if design acceleration has 10% probability of exceedance in 50 years, that can be related to probability of exceedance by following Poisson relation.

Probability of not exceeding the design ground acceleration in 50 years is:

$$P_s(A_{50}) = 1.00 - 0.10 = 0.90 \tag{39}$$

Probability of ground acceleration to be below design value in any of the years is:

$$P_s(A_1) = 0.90^{1/50} = 0.997895 \tag{40}$$

Hence, probability of exceeding the design ground acceleration during the design life time of the bridge is;

$$P_s(A_1) = 1.00 - 0.997895^{75} = 0.146186 \tag{41}$$

In conclusion, probability of observing ground acceleration greater than design acceleration (475 year return period) is 14.62% in 75 years.

Peak ground acceleration required for AASHTO LRFD has probability of exceedance of 5% in 50 years (1000 year return period). Probability of exceedance of design acceleration in 75 years was calculated also.

Probability of not exceeding the design ground acceleration in 50 years is:

$$P_s(A_{50}) = 1.00 - 0.05 = 0.95 \tag{42}$$

Probability of ground acceleration to be below design value in any of the years is:

$$P_s(A_1) = 0.95^{1/50} = 0.99897466 \tag{43}$$

Hence, probability of exceeding the design ground acceleration during the design life time of the bridge is;

$$P_s(A_1) = 1.00 - 0.99897466^{75} = 0.074 \tag{44}$$

In conclusion, probability of observing ground acceleration greater than design acceleration is 7.40% in 75 years.

Peak ground acceleration can be described with extreme value Type II distribution (Ellingwood, 1994). Type II extreme value distribution is also named as Fréchet Distribution. Fréchet Distribution typically has two parameters. α is the shape parameter (α>0) and β is the scale parameter (β>0). Probability distribution function of Fréchet distribution is bounded on the lower side (x>0). Difference of Fréchet Distribution from Type I or Type III extreme value distributions is its heavy upper tail.

$$f_{(x)} = \frac{\alpha}{\beta}\left(\frac{\beta}{x}\right)^{\alpha+1} \mathrm{e}^{-\left(\frac{\beta}{x}\right)^{\alpha}} \tag{45}$$

Peak ground accelerations for different locations in Turkey were given in Table 5-1. 475 and 1000 years accelerations were obtained from Gülkan et al. (1993).

Table 5-1 : Peak Ground Accelerations (Gülkan et al., 1993)

Location	Longitude	Latitude	Seismic Zone	Return Period	
	(degree)	(degree)		475	1000
Ankara	32.853	39.929	4	0.19	0.21
Izmir	27.145	38.433	1	0.51	0.59
Bursa	29.075	40.196	1	0.50	0.58
Antalya	30.709	36.893	2	0.44	0.52
Gaziantep	37.389	37.069	3	0.2	0.24
Samsun	36.331	41.293	2	0.31	0.36
Malatya	38.309	38.355	1	0.41	0.48
Erzincan	39.504	39.74	1	0.59	0.7
Canakkale	26.414	40.155	1	0.57	0.66
Hakkari	43.751	37.568	1	0.56	0.65
Istanbul/Göztepe	29.082	40.98	1	0.42	0.5
Istanbul/Sile	29.628	41.175	2	0.32	0.38

Cumulative distribution function of Fréchet Distribution is given as follows:

$$f_{(x)} = \mathrm{e}^{-\left(\frac{\beta}{x}\right)^{\alpha}} \tag{46}$$

Shape and scale parameters of distribution can be found by solving cumulative distribution function simultaneously for 475 and 1000 year return periods. (Fırat, 2007)

Table 5-2 : Fréchet Distribution Shape and Scale Factors

	Ankara	Izmir	Bursa	Antalya	Gaziantep	Samsun	Malatya	Canakkale	Erzincan	Hakkari	Göztepe/İst.	Şile/İst.
β	0.14	0.31	0.30	0.26	0.12	0.18	0.22	0.37	0.34	0.32	0.24	0.17
α	7.50	4.50	4.50	4.20	4.10	4.20	3.70	3.94	4.00	4.10	4.10	3.60

The design spectrum given in AASTO LRFD requires the short period and long period spectral accelerations at the bedrock level. These accelerations are multiplied with site coefficients to obtain the design spectrum. Accelerations from the zoning map of Gulkan et al. (1993) used in design of highway bridges but only peak ground accelerations were published in that study. Short and long period spectrums for Turkey were published in DLH (2008)[2]. The peak ground accelerations given in DLH (2008) are nearly same as given by Gulkan et al.(1993). Comparison for some sites were given in For that reason peak ground accelerations in Table 5-1 and T=0.20 sec and T=1.0 sec Spectral Accelerations in Table 5-3 were used in this study.

[2] Kıyı ve Liman Yapıları, Demiryolları, Hava Meydanları İnşaatlarına İlişkin Deprem Teknik Yönetmeliği, DLH, 2008

Table 5-3 : T=0.20 Sec and T=1.0 Sec Spectral Accelerations (DLH, 2008)

	Short Period Spectral Acceleration (g) - S_S (T=0.20s)		Long Period Spectral Acceleration (g) – S_1(T=0.20s)	
	Probability of Exceedance in 50 Years		Probability of Exceedance in 50 Years	
	10%	5%	10%	5%
Ankara	0.32	0.40	0.14	0.17
Bursa	1.23	1.45	0.50	0.65
Samsun	0.60	0.70	0.28	0.34
Izmir	1.04	1.25	0.39	0.50
Antalya	0.60	0.72	0.16	0.20
Gaziantep	0.39	0.47	0.15	0.18
Malatya	0.83	0.99	0.30	0.39
Erzincan	1.72	2.02	0.88	1.10
Canakkale	0.79	0.93	0.31	0.39
Hakkari	0.50	0.63	0.14	0.19
Istanbul/ Göztepe	1.19	1.39	0.58	0.73
Istanbul/Sile	0.49	0.57	0.26	0.31

Table 5-4 : Comparison of PGA Values of Gülkan et al. (1993) and DLH (2008)

				Gulkan et al. (1993)		DLH (2008)	
Location	Longitude	Latitude	Seismic	Return Period		Return Period	
	(degree)	(degree)	Zone	475	1000	475	1000
Ankara	32.853	39.929	4	0.19	0.21	0.19	0.20
Izmir	27.145	38.433	1	0.51	0.59	0.52	0.61
Bursa	29.075	40.196	1	0.5	0.58	0.50	0.57
Antalya	30.709	36.893	2	0.44	0.52	0.46	0.51
Gaziantep	37.389	37.069	3	0.2	0.24	0.20	0.23
Samsun	36.331	41.293	2	0.31	0.36	0.30	0.36
Malatya	38.309	38.355	1	0.41	0.48	0.39	0.49
Erzincan	39.504	39.74	1	0.59	0.7	0.60	0.71
Canakkale	26.414	40.155	1	0.57	0.66	0.59	0.63
Hakkari	43.751	37.568	1	0.56	0.65	0.59	0.65
Istanbul/Göztepe	29.082	40.98	1	0.42	0.5	0.42	0.53
Istanbul/Sile	29.628	41.175	2	0.32	0.38	0.32	0.36

CHAPTER 6

ASSESSMENT OF THE TARGET RELIABILITY INDEX FOR SEISMIC DESIGN OF BRIDGES IN TURKEY

Reliability index is an indirect measure of probability of failure of a system with random variables. In this study, system consists of seismic loads and pier columns designed to resist variable load demand. Demand and resistance depend on different random variables which were discussed in previous sections in detail. Table 6-1 and Table 6-2 summarize the statistical inputs required for the analysis of the reliability of column piers subjected to seismic loads.

Table 6-1: Summary of Statistical Parameters of Seismic Load Demand

		Mean Value	Bias Factor	Coefficient of Variation %	Distribution	Reference
Modeling Error	λ_{eq}	1.00	1.00	20	Normal	Ellingwood et al (1980)
Spectrum Modeling Factor	C'	1.00	1.00	Ranges 15 - 60	Normal	Frankel et al. (1997)
Peak Ground Acceleration	A_g	-	-	Table 5-2	Extreme Type II	DLH (2008)
Natural Period	T	-	1.08	-	Normal	Chopra and Goel (2000)
Weight (Cast in Place)	W_c	-	1.03	8	Normal	Ellingwood et al (1980)
Weight (Precast)	W_p	-	1.05	10	Normal	Ellingwood et al (1980)
Response Modification Factor	R	-	1.5	34	Normal	Priestley&Park (1987)

Table 6-2 : Summary of Statistical Parameters of Section Capacity

		Mean Value	Mean Bias Factor	COV %	Distribution	Reference
28-days Cylinder Compressive Strength of concrete	f_c	24.24 Mpa	0.808	10	Normal	Fırat (2007)
Yield Strength of Steel	f_y	451.23 MPa	1.00	8.5	Normal	Fırat (2007)
Rebar Area	A_s	-	1.00	3.0	Normal	Fırat (2007)
Cross Section Height	H	-	1.00	3.8	Normal	Fırat (2007)
Cross Section Width	W	-	1.00	4.4	Normal	Fırat (2007)
Elastic Modulus	E_c	-	0.90	20	Normal	Chopra and Goel (2000)
Effective depth	d	-	1	3.7	Normal	Fırat (2007)

6.1 Description of Bridge Configurations and Structural Properties

Reliability based evaluation of highway bridges according to the Turkish seismic design practice is the main objective of this study. Table 6-3 is obtained from the bridge inventory of "Republic of Turkey General Directorate of Highways (KGM, 2012)".

Table 6-3 : Highway Bridges on State Roads and Provincial Roads Classified According to Types and Properties in Turkish Highway Network (KGM, 2012)

Structure Type	Number of Bridges	Total Length (m)
RC Simple Beam	2401	74141.25
RC Simple Slab	176	2958.10
RC Continuous Beam	43	2838.75
RC Continuous Slab	185	6691.50
RC Cantilever Beam	180	6245.20
RC Cantilever Slab	238	5600.75
RC Gerber Girder	221	19336.70
RC Gerber Slab	40	1558.40
RC Arc	35	2309.65
Pre-stressed Pre-tensioned	2739	182834.95
Pre-stressed Post-tensioned	15	1880.60
RC Box Girder	3	958.00
RC Frame Girder	4	139.00

RC Frame Slab	299	5735.85
Concrete Arc	65	1280.85
Cable Stayed	2	840.00
Stone Arc	47	2989.70
Composite	230	9479.05
Steel Box Section	4	1197.80
Steel Girder RC Slab	5	945.80
Steel Girder Zores Slab	9	221.45
Steel Truss Girder	12	708.85
Other	23	3382.70

It can be observed from Table 6-3 that most of the highway bridges in Turkey have simply supported girders or slabs on single or multiple column piers. Most of these are highway intersection bridges or river bridges that are 2 to 5 spans which have 15 to 35m span lengths with straight plan geometry or very large radius of curvatures and skew angle varying from 0 to 30 degrees. (Avşar, 2009)

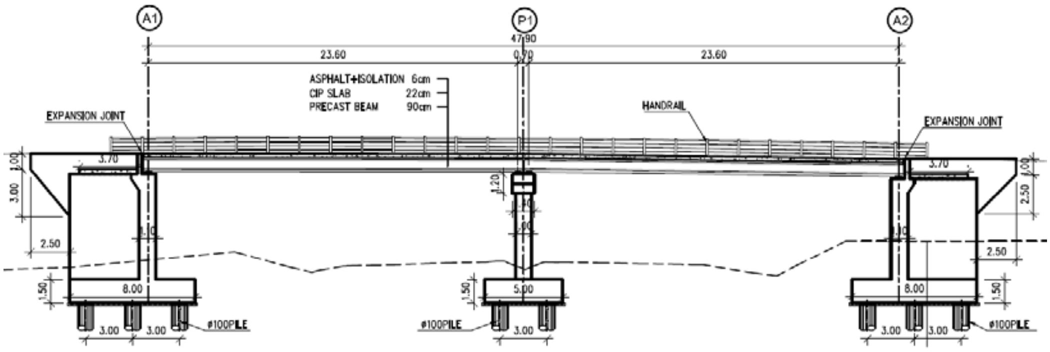

Figure 6-1 : Example 2-Span Bridge Profile

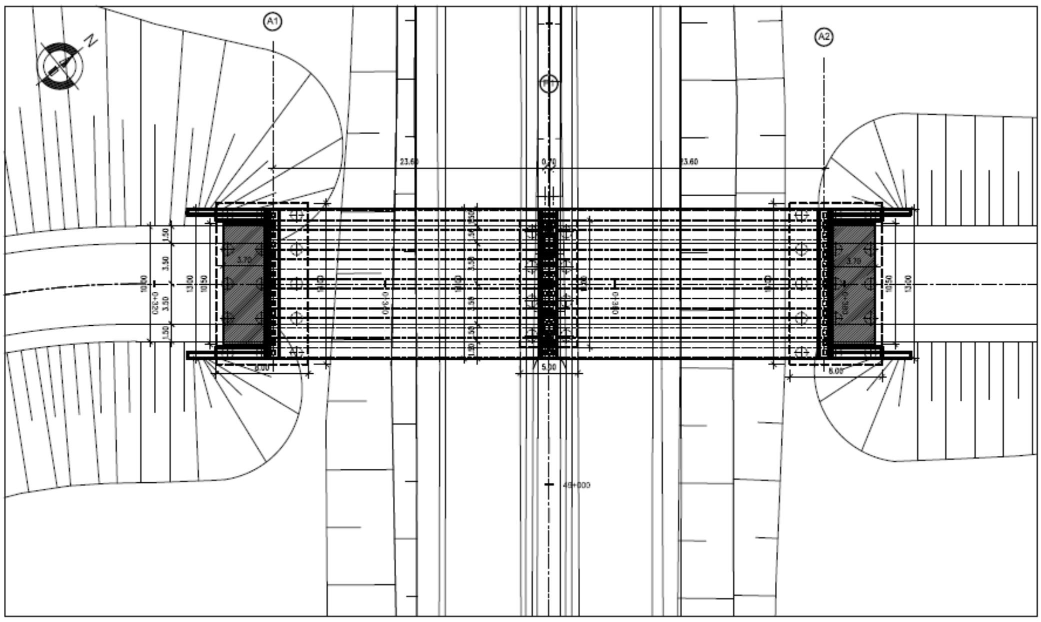

Figure 6-2 : Example 2-Span Bridge Plan

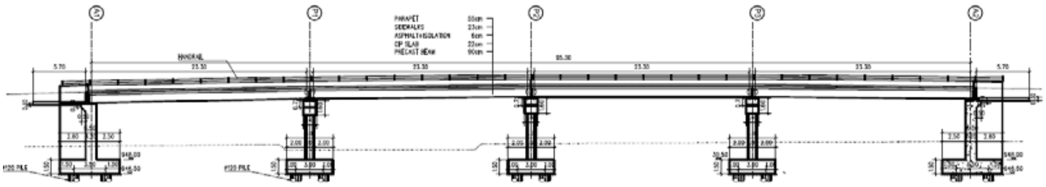

Figure 6-3 : Example 4-Span Bridge Profile

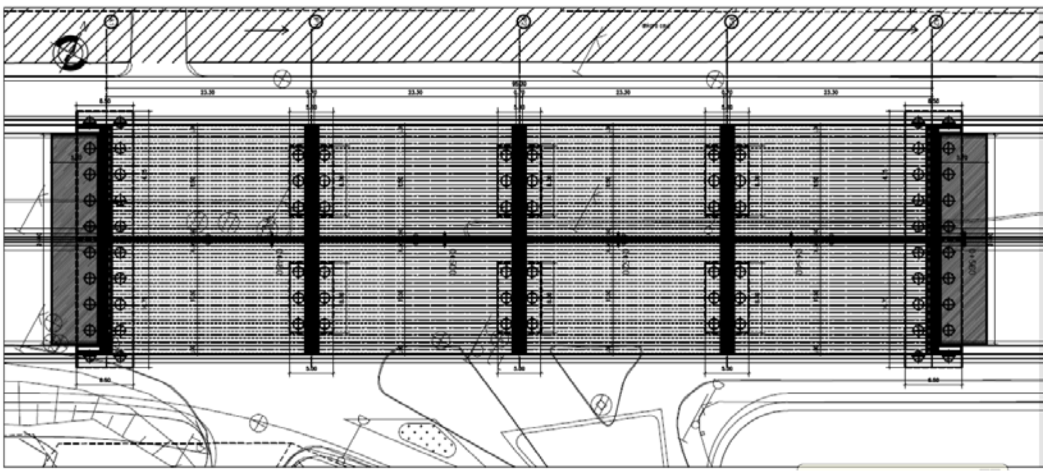

Figure 6-4 : Example 4-Span Bridge Plan

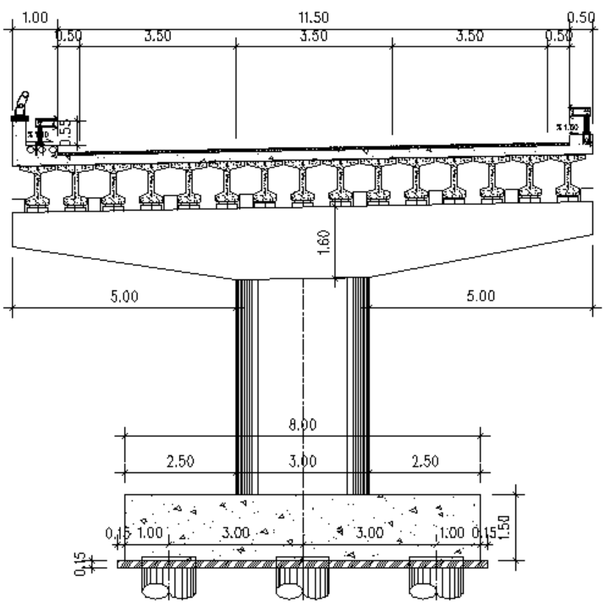

Figure 6-5 : Typical One Column Pier Example

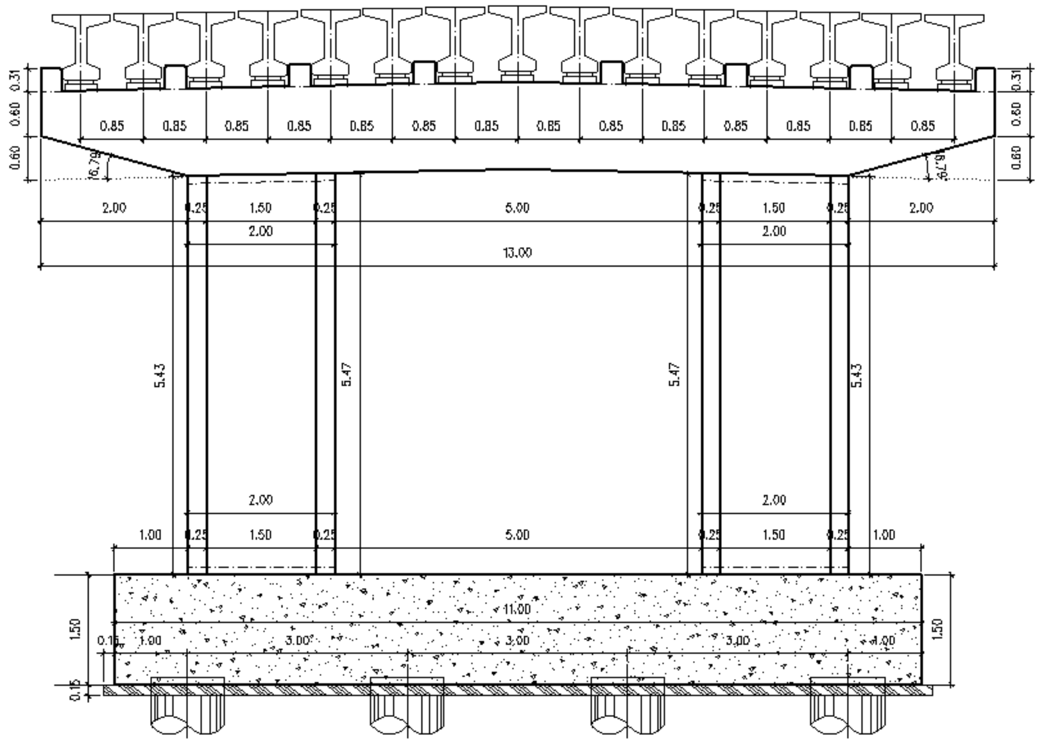

Figure 6-6 : Typical Multiple Column Pier Example

The first option of basic bridge configuration used in this study consists of a two span with a total length of 50 m bridge having the profile shown in Figure 6-1. The geometric properties of the multiple column bridge bent were shown in Figure 6-6. The plan geometry was shown in Figure 6-2.

Pier configuration was formed by two columns each of which has 1.00 m x 2.50 m cross section geometry with concrete strength, $f_c^{'}$, of 30 MPa. The cap beam was 1.20 m deep carrying 15 Turkish Type H90 precast beams and a 0.22m deck slab plus wearing surface. The deck was 13m wide with a 0.75m curb on each side.

Seismic demand on pier column cross-section was determined with SDOF system shown below, where soil structure interaction was not considered (but considered as uncertainty, (Section 5.2), in calculations and column was a cantilever supported on top of the foundation level.

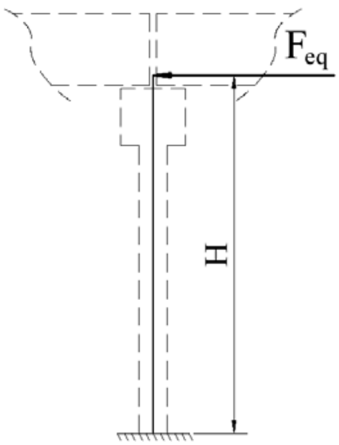

Figure 6-7 : Single Degree Of Freedom Model

A short description of the procedure is provided below for each load and limit state analyzed.

6.1.1 Gravity Loads

The weight applied on each bent was calculated as follows:

• Superstructure weight per span length = 207 kN/m;

• Weight of cap beam = 546 kN;

• Weight of wearing surface and utilities = 25.90 kN/m

• Weight of the columns = 515.60 kN/m

The analysis of the distributed weights produced a dead weight reaction at the top of the pier equal to 8151.50 kN from the superstructure plus 546 kN from the cap and 515.60 kN from the column.

6.1.2 Earthquake Load

The effective weight of the column above the point of fixity was calculated as 515.60 kN. The center of mass was 6.10 m above from the top of the foundation level (point of fixity). Using common practice in bridge engineering and assuming a tributary length of 42 m (50% of the distance to other bent and 70% of distance to abutment), the total weight from the superstructure and wearing surface applied on one bent added up to 8151.50 kN. Thus, the total weight on one bent was equal to 8995.30 kN.

The inertial forces applied on the bent because of the earthquake accelerations were lumped as shown in Figure 6-7.

The natural period of the column bent was calculated from (47). Mass “m” was equal to 916.95 tons. The natural period of the column was found to be T = 0.55 second.

$$T = 2\pi\sqrt{\frac{m}{K}} \tag{47}$$

where;

m = Total seismic mass on column

K = Flexural stiffness of the pier

T = Natural period of pier

The natural period of the system was used in equations in Figure 3-5 to find the spectral accelerations, C_{sm}. The soil was assumed to be of type B in this example. The spectral accelerations calculated for the 1000 year return period as given in Table 6-4. The equivalent lateral force and required moment capacities were determined based on equations given in Section 6.3 of this study.

Table 6-4 : Earthquake Design Requirements for 12 Sites

	C_{sm} (g)	Equivalent Lateral Force F_i (kN)	Required Moment Capacity (kN.m)
Ankara	0.32	2878.50	11705.88
Bursa	1.23	11064.22	44994.49
Samsun	0.60	5397.18	21948.53
Izmir	1.04	9355.11	38044.12
Antalya	0.60	5397.18	21948.53
Gaziantep	0.39	3508.17	14266.55
Malatya	0.83	7466.10	30362.14
Erzincan	1.72	15471.92	62919.13
Canakkale	0.79	7106.29	28898.90
Hakkari	0.50	4497.65	18290.44
Istanbul/ Göztepe	1.19	10704.41	43531.26
Istanbul/Sile	0.49	4407.70	17924.63

It is shown in Table 6-1 that modeling factor was included in the analysis. That modeling factor accounts for selection of SDOF analysis with lumped mass, uncertainty in the predicting the mass applied on single pier, considered height of pier in design and so forth.

Akogul (2007) stated that typical bridges as shown in Figures 6.2 to 6.6 can be analyzed with SDOF models. Comparison of SDOF model results and MDOF models were given in Table 6-5 for two example bridges taken from case study of Akogul (2007).

Table 6-5 : Comparison of SDOF and MDOF Solutions

		Bridge 1		Bridge 2	
		SDOF	MDOF	SDOF	MDOF
T	(sec)	1.36	1.36	1.08	1.08
M	(kNm)	37778	37720	29630	29615
V	(kN)	1712	1709	2675	2673
ΣV	(kN)	7892	7886	8576	8572

where;

M = Maximum moment force on pier

T = Natural period of pier

V = Maximum shear force on pier

ΣV = Total base shear

Bridge 1 in that case study consists of a three span with a total length of 87.40 m bridge having 20 m height single columns at each pier. Pier configuration was formed by single circular columns of 2.80m diameter with concrete strength, f_c', of 30 MPa. Piers were carrying 10 Turkish Type H120 precast beams. Bridge 2 had the same superstructure with 11 m column height. Akogul (2007) showed that the ratio of SDOF results to MDOF results were around 0.96 in average.

Dynamic analysis of the SDOF structure was performed in both orthogonal directions and orthogonal forces were combined with following combinations.

- 1.00 EQ_L + 0.30 EQ_T
- 0.30 EQ_L + 1.00 EQ_T

where;

EQ_L : Seismic load in longitudinal direction of bridge axis

EQ_T : Seismic load in transverse direction of bridge axis

6.2 Reliability Analyses

The aim of this chapter is determine the structural reliability of bridge piers under seismic loads by accounting for the uncertainties encountered. The uncertainties associated with predicting the load carrying capacity of a pier columns were given in Table 6-2. A random variable can take any value described by its probability distribution function. (NCHRP, 2003)

The most important characteristics of a random variable (X) are its mean value ($\overline{X}$) and the standard deviation (σ_x). Coefficient of variation (COV) is a dimensionless measure of uncertainty of the random variable. The coefficient of variation of a random variable X is given as follows;

$$\mathrm{COV} = \frac{\sigma_x}{\bar{X}} \tag{48}$$

Nominal values for the variables described in AASHTO design codes. The ratio of the mean value of variables to the nominal value is described as the mean bias factor (b_x)

$$\mathrm{b}_x = \frac{\bar{X}}{X_n} \tag{49}$$

In reliability analysis, safety was described as the situation in which load carrying capacity (R) exceeds the seismic load demand (S).

$$Z = R - S \tag{50}$$

Using this equation, probability of failure (P_f) can be described as;

$$P_f = \Pr[R \leq S] \tag{51}$$

In this study, both load demand and capacity were plotted on probability papers and it was observed that both are normally distributed. Such a result was expected due to fact that most of the random variable components are normally distributed.

Reliability index, β, corresponds to the divergence of the load demand, S, from the load carrying capacity of the member, R. The larger the value of β means that possibility of the load demand to exceed capacity is less due to chance.

Cumulative standard normal distribution function is given in equation (52).

$$\Phi(z) = P(Z \leq z) = \int_{-\infty}^{z} \frac{1}{\sqrt{2\pi}} e^{\frac{-u^2}{2}} du \tag{52}$$

Equation (52) gives the probability of survival (P_r) that resistance capacity, R, to be greater than load demand, S. Probability of failure can be found by subtracting the result from one.

Reliability analyses were performed with Monte Carlo simulation method. Computation with Monte Carlo numerical method has a simple and straightforward algorithm. That method is preferred to calculate the reliability index, β, for discrete random variables such as ground acceleration which has Type II Extreme Value distribution. Bi-axial bending capacity has a complex calculation and it is hard to solve with analytical methods.

For bridge example solved in Chapter 6.1, the results of Monte Carlo Simulations were illustrated in Table 6-6. In that example, only 15 simulations were performed. Detailed analyses performed with thousands of simulations for accuracy of results.

Table 6-6 : Example of Monte Carlo Simulations

Simulation	Moment Demand	Moment Capacity	Result
	kN.m	kN.m	
1	24746.96	51743.66	OK
2	36243.11	42294.82	OK
3	2699.66	41844.87	OK
4	50843.77	49043.99	FAIL
5	14398.23	44544.54	OK
6	9898.78	46344.32	OK
7	54443.33	40495.04	FAIL
8	23397.13	45444.43	OK
9	28189.20	41844.87	OK
10	19988.98	49043.99	OK
11	28796.47	51743.66	OK
12	32789.86	47694.15	OK
13	43644.65	49943.88	OK
14	31046.19	49043.99	OK
15	29246.41	49043.99	OK
	Number of Survivals		13
	Number of Failures		2

It was shown in Table 6-6 that in 2 of 15 simulations the bridge had failed. The probability of failure was found to be 0.13 (2/15). The probability of survival was calculated as 0.87. In equation (52), left hand side of the formula is 0.87 and reliability index, β can be calculated by taking the inverse of the equation and solving for z. However closed form solution for does not exist and numerical solution is required. For that purpose, numerical solution software can be used or "z-score" tables can be used. The reliability index, β, was calculated as 1.13. However, the accuracy of results is sensitive to number of simulations and 15 simulations were given just as an example.

In addition, reliability indices were calculated with First Order Second Moment (FOSM), First Order Reliability Method (FORM) to compare the results of numerical solution. In this study all random variables were normally distributed except ground accelerations. In numerical simulations Type II Extreme Value distribution was directly implemented in calculations. However, in these analytical solutions, equivalent normal distribution parameters were used for Type II Extreme Value distribution. Statistical parameters for section capacity were taken from Monte Carlo simulations.

First order second moment method (FOSM) used in this study was proposed by Hasofer and Lind (1974) to calculate reliability index. The shortest distance from the origin to the failure surface (in z-coordinates) was defined as the reliability index and that closest point was defined as the design point. Nonlinear failure surfaces require iteration for calculation of reliability index.

In first order reliability method (FORM), the reliability index was described by Rackwitz- Fiessler as the minimum distance from the origin to the failure surface (in standardized normal coordinates). A linear function was proposed to find the distance from the origin point to the design point. Accuracy of FORM decreases as failure function gets nonlinear.

6.2.1 A Brief Description of Monte Carlo Simulation

In calculations, a set of random numbers were generated for each of the random variables in failure function. For example, if the required repetition for successive simulation of column stiffness is 1000, 1000 row of column width, length, height, seismic mass is generated. Generation of samples was done with respect to nominal value, bias factor, coefficient of variation and distribution function of each variable. For each variable, one thousand rows of samples obtained which has the specified statistical parameters obtained from laboratory or other tests. Column stiffness was calculated for each row of parameters and one thousand column stiffness data were obtained. If the calculated set of stiffness values were plotted on a probability paper probability distribution of the column stiffness can be determined. Also mean and standard deviation can be calculated from the data.

If the subject is expanded to seismic design of piers, number of random variables increases. Failure function changes and same procedure is applied. As the number of random variables increases, number of simulations increase. In this study, failure function is simulated 100,000 times for each design to accurately calculate the reliability index. If X random variables are included in the failure function, total $100{,}000^{X}$ samples are generated. If a random variable is included in more than one step in analyses, samples are generated only once since same random variable cannot take different values within a row.

Probability of failure is defined based on the failure function. If a design is simulated 100,000 times, and load demand exceeds the capacity at 1,000 of the simulations, probability of failure is calculated as 0.01 (1,000/100,000). In analysis, the question is whether the system fails due to assumed failure function or not. Sign of the failure function as being lower or greater than zero is to be investigated.

It was mentioned that biaxial bending diagrams were calculated with Greens theorem. Figure 6-8 shows the Monte Carlo simulation results for biaxial bending capacity. It was calculated that load carrying capacity of a column was calculated with 1.15 mean bias and coefficient of variation of 5%. Figure 6-8 also shows that load carrying capacity can be accepted as normally distributed.

Nowak (1999) stated that load carrying capacities of columns were biased by %14. It was mentioned in the study that the distribution is log-normal and coefficient of variation was 13%.

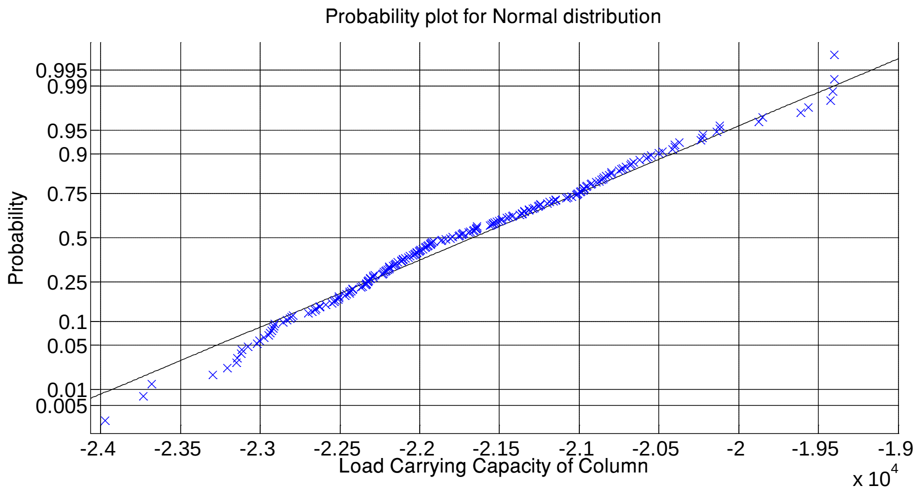

Figure 6-8 : Probability Plot of Load Carrying Capacity Values on Normal Probability Paper

Reliability index of bridge pier columns were calculated for designs per Turkish modification of AASHTO LFD 2002, per AASHTO LRFD 2010, AASHTO LRFD 2007 and modified LRFD 2010 response spectra curve that is described in section 6.4.2.5.

Comparison of these three cases was studied to show the effect of the revisions on demand calculations and capacity provisions. It was mentioned in Chapter 3 that response modification factors were revised from LFD to LRFD. For example, single column earthquake design forces used to be modified with 3 in LFD where response modification factor revised to be 2 for essential bridges in LRFD. Soil classification and spectral acceleration coefficients were revised from LFD to LRFD. A new design spectrum is planned to be proposed within the scope of TUBITAK project number 110G093. Effect of the proposed spectrum on reliability was studied. Resistance factors for columns that are compression controlled revised from 0.50 to 0.75 in LRFD.

In further sections, reliability of the types of bridges summarized above is represented with limit design for three cases described.

6.3 Definition of Failure Function

Random variables of load demand and carrying capacity were discussed and evaluated in Chapter 4 and Chapter 5 to be accounted in the reliability analysis. In this section, uncertainties involved in prediction of load demands and resistance capacity are brought together to define failure function.

Structural safety of a member is defined as the condition that demands on that member as can be force, stress, and displacement etc. not to pass beyond the limit that member can satisfactorily counterpoise. In other words, member fails in case demand exceeds the capacity of the member. Failure in this definition is not necessarily a collapse; it is the failure of the member to function as it was designed to. Design codes are not calibrated to design the capacity of the member to be equal to the demand since there are various uncertainties involved as discussed in previous section. A margin of safety is involved to keep the capacity greater than the demand and amount of this margin is directly related to the reliability of the member as explained as follows.

6.3.1 Failure Function for Bending Moment

For a structural member which is pier column in this section with moment capacity M_{cap} and force demand on the member as M_{demand}, member is satisfactory if;

$$Z = M_{cap} - M_{demand} > 0 \tag{53}$$

where magnitude of the Z describes the amount of safety of section.

As it is discussed in previous sections that, equivalent lateral force on top of the pier is related to natural period of the structure, structure mass, seismic response coefficient, peak ground acceleration and response modification factor. (Ghosn, Moses, & Wang, 2003)

$$F_{eq} = \lambda_{eq} C' C_{sm(T)} A W \quad (54)$$

where;

F_{eq} =Equivalent lateral earthquake force

C' = Spectral Modelling Factor (Section 5.1)

T = Natural Period of The System (Section 5.2)

W = Seismic Weight (Section 5.3)

λ_{eq} = Modeling factor (Section 5.5)

A = Peak Ground Acceleration (Section 5.6)

C_{sm} =Spectral Accelerations

As a result of the nature of the earthquake loading, combined with the various geometrical possibilities in a bridge pier (such as skews of the column local axis, or unsymmetrical geometry due to architectural or other reasons) the piers are mostly subjected to biaxial bending combined with axial load. Due to computational effort for considering biaxial bending problem, designers tend to solve the problem with simplified procedures. In this study an analytically exact solution method is accepted to develop interaction surface for the pier cross-sections. Method is adopted from “Interaction Surfaces of Reinforced – Concrete Sections in Biaxial Bending” (Fafitis, 2001). The method is based on Green’s theorem of integrating arbitrary functions over arbitrary shaped surfaces by Gauss method. Details of algorithm is explained and illustrated in APPENDIX A. This method is preferred in the calculation of interaction surfaces in order not to include any uncertainty due to calculation method. In design codes, up the 3rd degree concrete stress-strain relationships are widely used (rectangular stress block used by ACI-318). For that reason using three node Gauss integration is used to yield to the exact results in resistance calculations. The uncertainty involved in capacity calculations are the uncertainty in material properties such as yield strength of rebar and the cylinder compressive strength of concrete, uncertainty in the geometry of the cross section and the uncertainty in the modeling such as assumptions of material behavior (stress-strain diagrams). Due to the analytically exact solution method with Green's theorem, no uncertainty in the calculation method will be introduced.

As a result, the revised failure function is as follows (Ghosn, Moses, & Wang, 2003).

$$Z_{\mathrm{bending}} = M_{\mathrm{cap}} - \lambda_{\mathrm{eq}} C' C_{sm(T)} \frac{AW}{R_{\mathrm{m}}} H \tag{55}$$

where;

R_m = Response Modification Factor (Section 5.4)

H = Column Height (Figure 6-7)

M_{cap} = Moment Capacity of the Cross Section (Appendix A)

6.3.2 Failure Function for Axial Load

Uncertainty involved in permanent gravity loads and axial capacity of column are relatively less but safety evaluation of axial capacity is required to perform a comprehensive evaluation of columns under seismic loading. It is known that axial failure is not usual mode of failure under seismic loading. Bridge piers designs are governed by bending and shear demands which lead to the cross sections with higher axial capacity than required demand. For that reason an artificial case of loading will be studied to observe the reliability for column axial capacity by ignoring the bending and shear requirements.

Column capacity is calculated with the same program described for bending.

$$Z_{axial} = P_{cap} - \lambda_{eq}W \tag{56}$$

where;

P_{cap} = Axial Load Capacity of the Cross Section (Appendix A)

λ_{eq} = Modeling factor (Section 5.5)

W = Seismic Weight (Section 5.3)

6.3.3 Failure Function for Shear

Uncertainty involved in shear demand is same as the bending demands both of which are related to equivalent lateral force created by seismic action. However, unlike bending, bridge piers are not just designed for seismic forces but also plastic shear forces to avoid shear failure. Both situations are discussed in terms of reliability with the following failure function.

$$Z_{shear1} = V_{conc} + V_{steel} - \lambda_{eq}C'C_{sm(T)}AW \tag{57}$$

$$Z_{shear2} = V_{conc} + V_{steel} - \frac{M_{plastic}}{H} \tag{58}$$

V_{conc} = Shear force carried by core concrete

V_{steel} = Shear force carried by reinforcement

$M_{plastic}$ = Plastic moment capacity of the column

6.4 Results

6.4.1 Axial Failure Mode

Safety for column axial load capacity was presented in Figure 6-9. It is clear by previous discussions on random variables of gravity loads and compressive strength of column that relatively smaller uncertainties were involved. It is calculated that pier columns can satisfy adequate safety (where $\beta > 3.0$) for up to %10 higher axial loads than its nominal axial capacity.

It is a fact that many design codes limits the design axial load to load capacity ratio with 70% (AASHTO LRFD) due to slenderness and eccentricity of loading; hence there is no reliability based problem below that limit according to pure axial compression the results given in Figure 6-9.

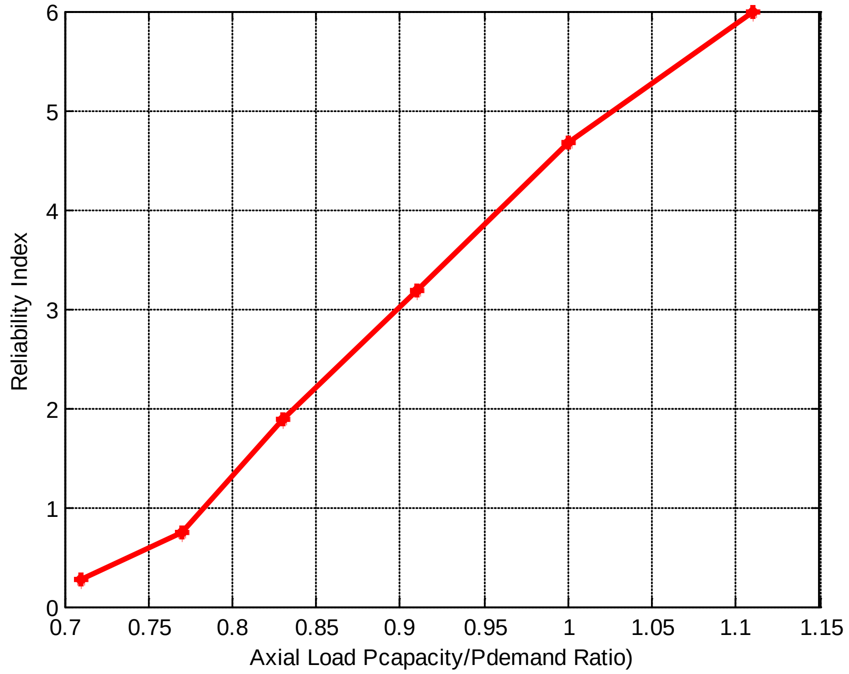

Figure 6-9 : Reliability Index of Column Axial Capacity for DC+DW Loading

6.4.2 Axial Force – Biaxial Bending Failure Mode

6.4.2.1 Reliability Index of Design per AASHTO LRFD 2010

Due to the fact that bending moment demand has the most uncertain components, analysis were performed for different scenarios of different variations.

Peak ground acceleration values were generated with placing pseudo random number generations into inverse cumulative distribution function of Fréchet. The examples of the histograms of acceleration series used in Monte Carlo simulations are shown in Figure 6-10 to Figure 6-12 to present the effect of shape and scale parameters on distribution.

In Turkish design code, design peak ground acceleration has 10% probability of exceedance in 50 years which corresponds to 475 years return period. In AASHTO LRFD 2010 code, design PGA has 1000 years return period. Reliability calculations were also studied with adopting AASHTO LRFD 2010 specifications but using 475 year return period PGA.

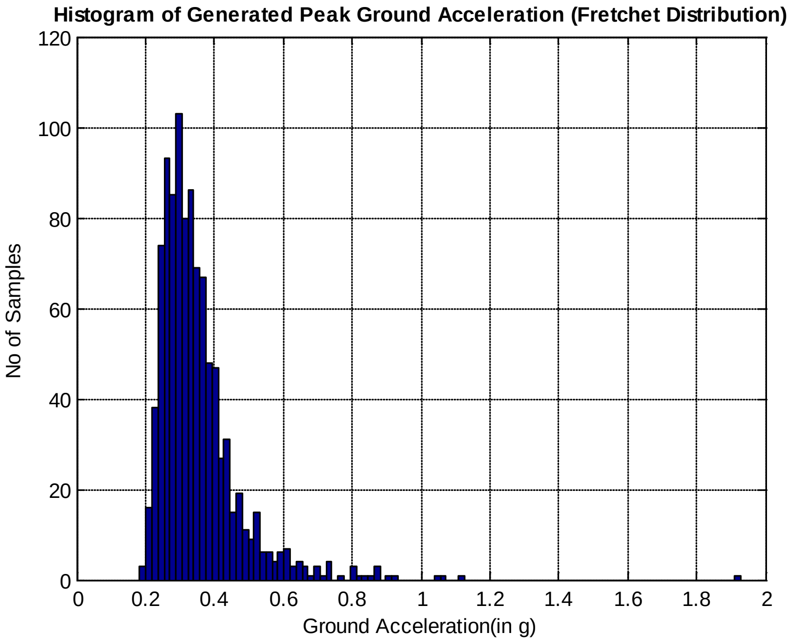

Figure 6-10 : Histogram of Peak Ground Accelerations in Bursa

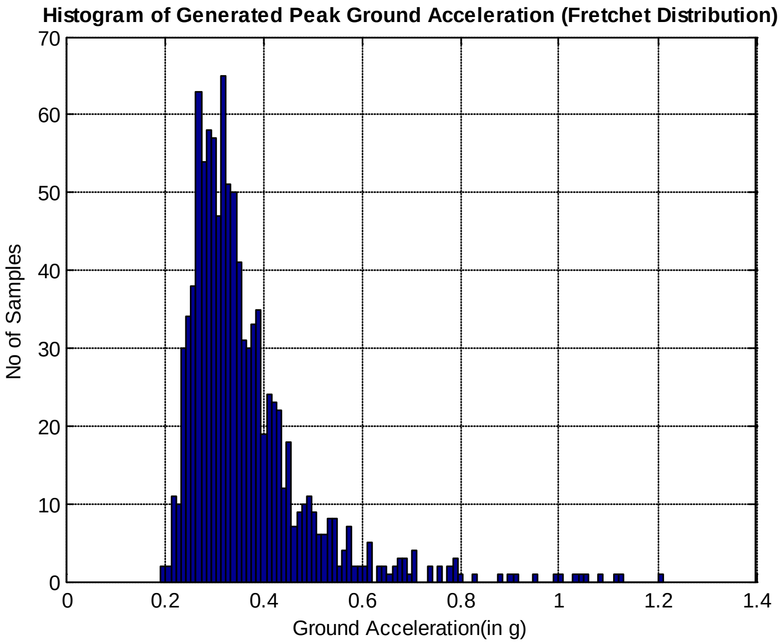

Figure 6-11 : Histogram of Peak Ground Accelerations in İzmir

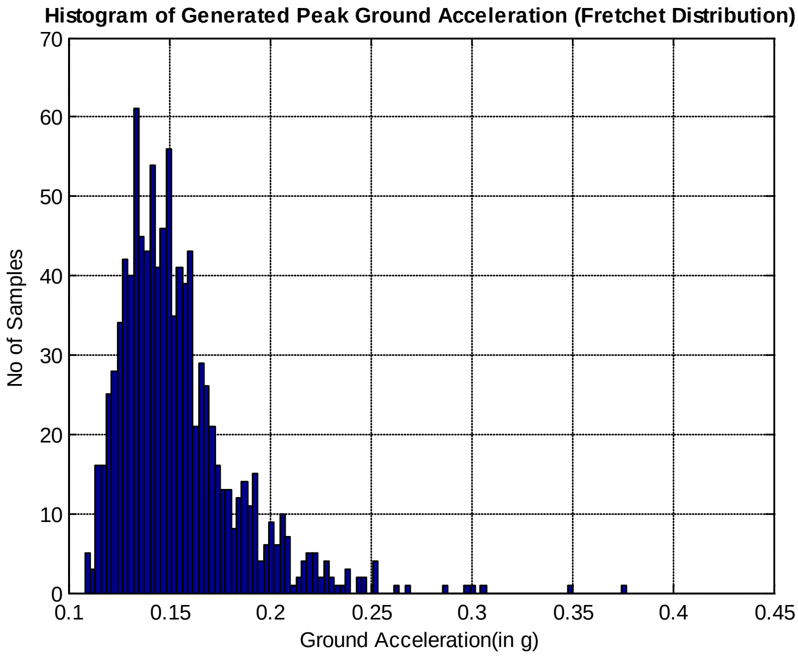

Figure 6-12 : Histogram of Peak Ground Accelerations in Ankara

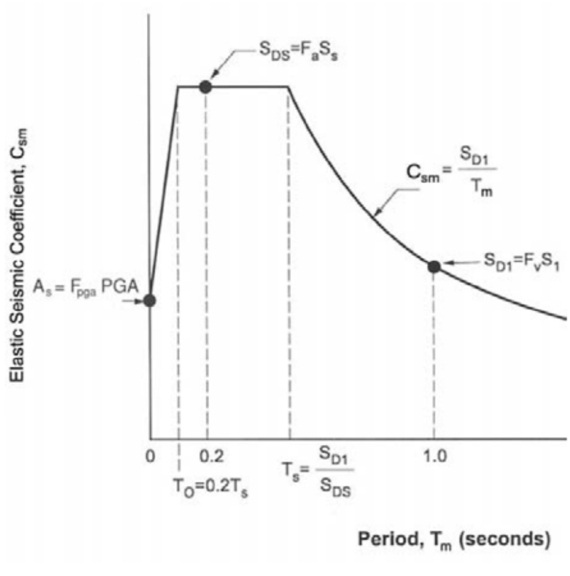

Figure 6-13 : AASHTO LRFD (2010) Design Spectrum

Table 6-7 : Values of Site Factor, F_{pga}, at Zero-Period on Acceleration Spectrum (AASHTO LRFD, 2010)

Fpga					
Pga	0.1	0.2	0.3	0.4	0.5
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.2	1.2	1.1	1	1
D	1.6	1.4	1.2	1.1	1
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

Table 6-8 : Values of Site Factor, F_a, for Short-Period Range of Acceleration Spectrum (AASHTO LRFD, 2010)

Fa					
Ss	0.25	0.5	0.75	1	1.25
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.2	1.2	1.1	1	1
D	1.6	1.4	1.2	1.1	1
E	2.5	1.7	1.2	0.9	0.9

Table 6-9 : Values of Site Factor, F_v, for Long-Period Range of Acceleration Spectrum (AASHTO LRFD, 2010)

F_v					
Sl	0.1	0.2	0.3	0.4	0.5
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*	*	*	*	*

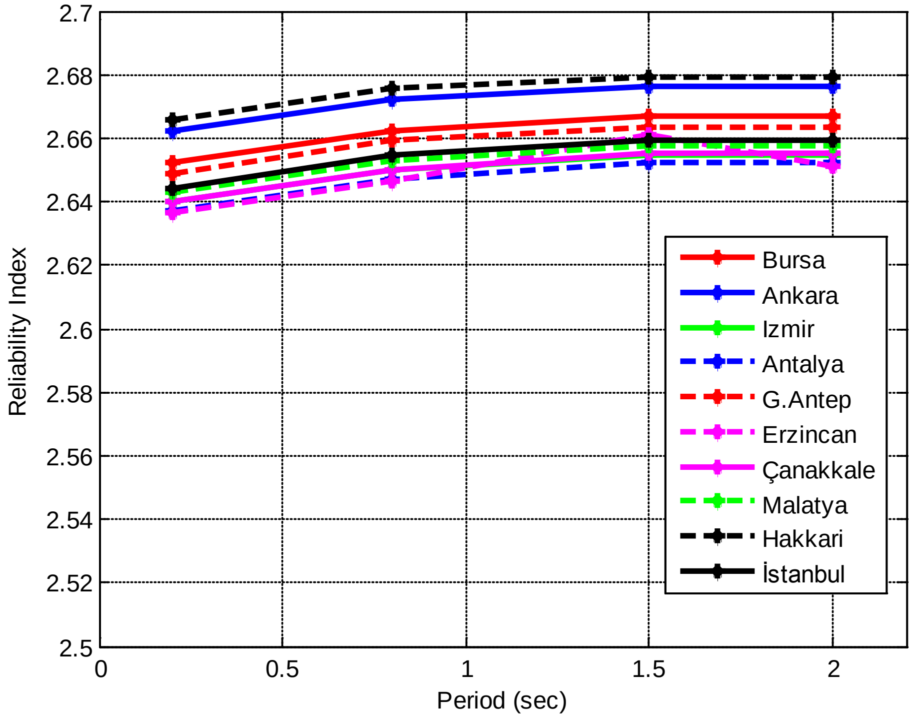

Figure 6-14 : Reliability Index of Biaxial Bending of Pier Columns Designed with using AASHTO LRFD 2010 together with 1000 Years Return Period PGA.

Results of reliability analysis were shown In Figure 6-14. On average, reliability index, β, is 2.66 for columns designed per 1000 years return period earthquakes. The reliability indices were high due to fact that low response modification factor (i.e. 1.50 for single columns) and 1000 year return period ground accelerations were used.

Table 6-10 : Comparison of The Reliability Indices Obtained from Numerical and Analytical Methods for Designs with using AASHTO LRFD 2010 Spectra together with 1000 Years Return Period PGA

	Monte Carlo	FOSM	FORM
Ankara	2.6945	2.6945	2.6945
Bursa	2.7126	2.6857	2.6857
Samsun	2.6953	2.6686	2.6686
Izmir	2.7280	2.6745	2.6745
Antalya	2.6722	2.6722	2.6722
Gaziantep	2.6827	2.6827	2.6827
Malatya	2.6774	2.6774	2.6774
Erzincan	2.6714	2.6714	2.6714
Canakkale	2.7283	2.6748	2.6748
Hakkari	2.6704	2.6974	2.6974
Istanbul/Göztepe	2.6518	2.6786	2.6786
Istanbul/Sile	2.6830	2.6564	2.6564

It is shown in Table 6-10 that numerical simulation results are satisfactorily close to the analytical solution results.

Remember that the reliability indices were also calculated with using of 475 year return period accelerations together with AASHTO LRFD 2010 code to observe the effect of the return period on reliability.

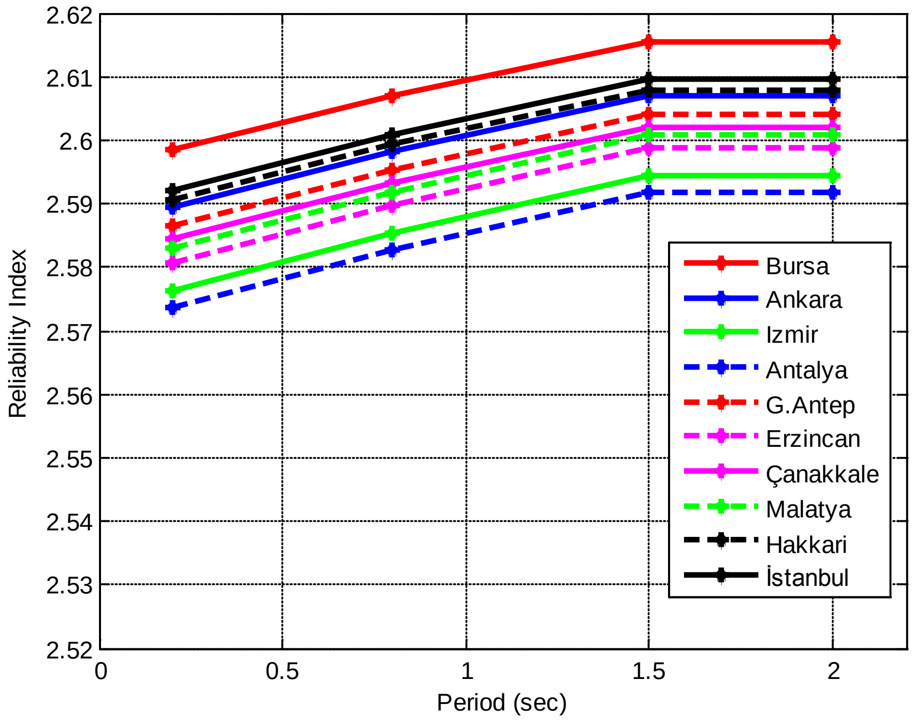

Figure 6-15 : Reliability Index of Biaxial Bending of Pier Columns Designed with using AASHTO LRFD 2010 together with 475 Years Return Period PGA.

It is observed that average reliability index would be 2.59 if 475 years return period accelerations were required by AASHTO LRFD 2010 specifications. Comparison of Figure 6-14 and Figure 6-15 shows that reliability index decreases 3% in average, if 475 years return period PGAs were used instead of 1000 year return period PGAs.

Table 6-11 : Comparison of The Reliability Indices Obtained from Numerical and Analytical Methods for Designs with using AASHTO LRFD 2010 Spectra together with 475 Years Return Period PGA

	Monte Carlo	FOSM	FORM
Ankara	2.5776	2.6302	2.6302
Bursa	2.5853	2.6381	2.6381
Samsun	2.6737	2.6213	2.6213
Izmir	2.6708	2.6184	2.6184
Antalya	2.5638	2.6161	2.6161
Gaziantep	2.6012	2.6275	2.6275
Malatya	2.5982	2.6244	2.6244
Erzincan	2.5962	2.6224	2.6224
Canakkale	2.6781	2.6256	2.6256
Hakkari	2.6838	2.6312	2.6312
Istanbul/Göztepe	2.6589	2.6326	2.6326
Istanbul/Sile	2.5558	2.608	2.608

There are three bridge importance classes described in AASHTO LRFD 2010 and response modification factor increases as the importance factor decreases. The effect of the response modification factor on reliability index, combined with strength reduction factor was shown in Figure 6-16.

In AASHTO LRFD 2010, strength reduction factor Φ=0.90 was given for flexural capacity of columns. Design equation for flexure is as follows;

$$\mathrm{M}_{design} \leq \emptyset \mathrm{M}_{capacity} \qquad (59)$$

From that equation $M_{capacity}/M_{design}$ is minimum 1.11. It is observed that the reliability index can vary from 2.41 to 2.80 as the response modification factor decreases. However, R=1 means that column shall remain elastic during the earthquake.

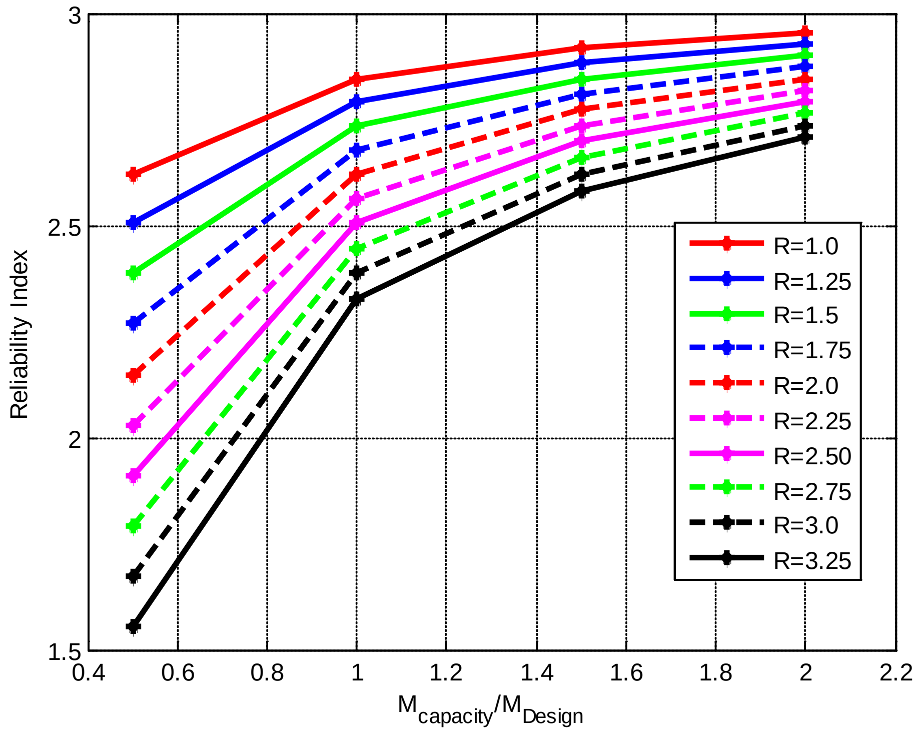

Figure 6-16 : Effect of Response Modification Factor on Reliability Index of Biaxial Bending of Pier Columns Designed with Using AASHTO LRFD 2010 together with 1000 Years Return Period PGA

6.4.2.2 Reliability Index of a Design with Using Turkish Modification of AASHTO LFD 2002

In this section, evaluation of existing designs is focus of the study. For that purpose, it is very important to remember the fundamental differences in seismic design of two codes. First of all, long period portion of the response spectrum curve was revised from $T^{-2/3}$ to T^{-1}. Response modification factors were decreased from 3 to 1.5. Another important difference in two design spectrums is the soil classes and soil factors. In LFD spectrum, soil class was taken into account by a single site coefficient with respect to soil class whereas long period and short period acceleration maps were considered in LRFD design spectrum which led to a variety of spectral acceleration coefficients rather than just one.

In other words, LFD and LRFD design spectrum curves are incomparable with respect to just a soil class parameter. AASHTO LFD (2002) defined the spectral acceleration coefficient as follows:

$$C_{sm} = \frac{1.2\,AS}{T_m^{2/3}} \leq 2.5\,A \tag{60}$$

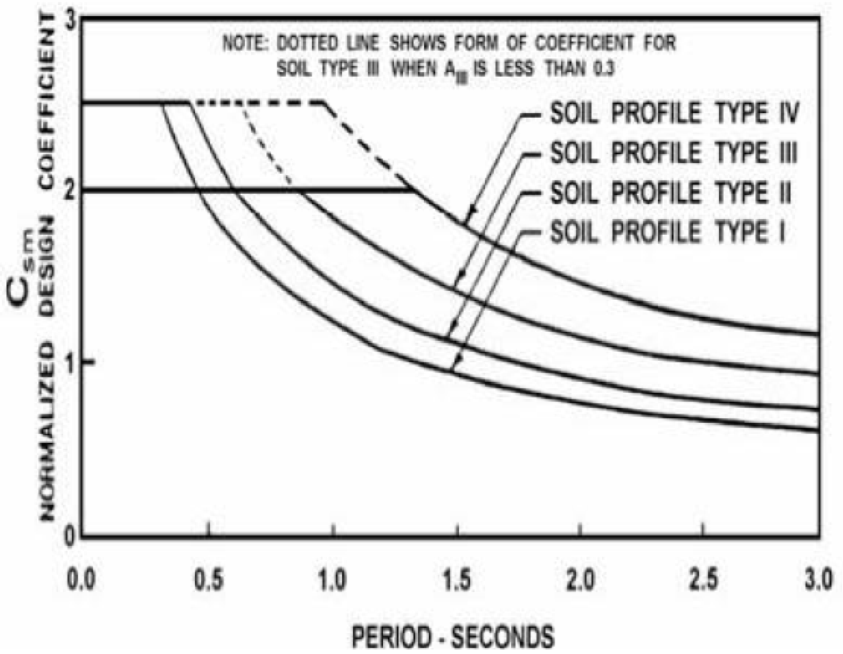

Figure 6-17 : AASHTO LFD 2002 Response Spectrum

The LFD code limited the bending moment capacity of compression controlled sections with 50% of nominal capacity where the limit had increased to 75% in LRFD (2007). In LRFD (2010) reduction factor for columns are 0.90 if p-delta effects do not exist. Gravity load combination factor is increased to 1.30 from 1.00.

In Turkish Earthquake Code (2007), 4 seismic zones were described based on the study of Gulkan et al. (1993).

Table 6-12 : Seismic Zones in TEC 2007

Seismic Zone	PGA (g)
1	>0.4
2	0.3
3	0.2
4	0.1

It is shown in Table 6-12 that Zone I defined for PGA of 0.40g or higher. However, only 0.40g PGA is used in practice. This deficiency may result in under design of bridges where actual PGA is greater than 0.40g. Figure 6-18 revealed that reliability index can be lower than 2.0 for sites with high PGA values such as Bursa, Canakkale, Istanbul, Hakkari etc.

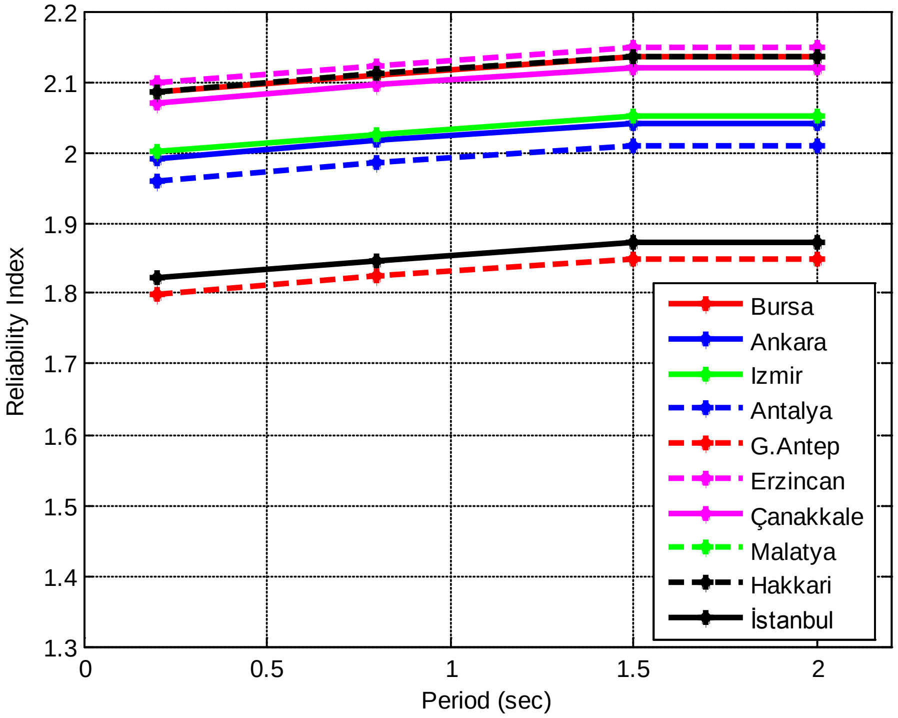

Figure 6-18 : Reliability Index of Biaxial Bending of Pier Columns Designed with Using Modified AASHTO LFD 2002 together with 475 Years Return Period PGA Values Given In Table 6-12.

6.4.2.3 Reliability Index of a Design with Using AASHTO LFD 2002 together with PGA values given by Gulkan et al.(1993).

Reliability index of different sites were studied with direct adaptation of AASHTO LFD 2002 but using the 475 years peak ground accelerations given by Gulkan et al.(1993).

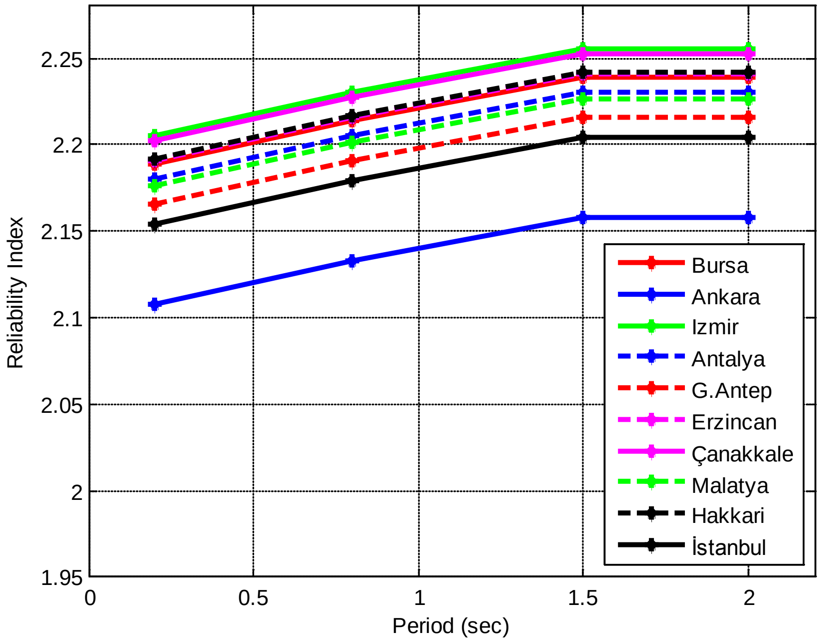

Figure 6-19 : Reliability Index of Biaxial Bending of Pier Columns Designed using AASHTO LFD 2002 together with 475 Years Return Period PGA Values Obtained from Gulkan et al.(1993)

Figure 6-19 showed that average reliability index would be in the order of 2.20 if the peak ground accelerations were taken from Gulkan et al. (1993) in current design practice. In that case, all design PGA values have the same probability of exceedance. As a result, reliability indices shown in Figure 6-19 have less deviation from the average compared to Figure 6-18.

6.4.2.4 Reliability Index of a Design with using AASHTO LRFD 2007 together with PGA Values Given by Gulkan et al.(1993).

In first version of AASHTO LRFD (2007), the design spectrum was same as the AASHTO LFD 2002. However, importance class definitions and response modification factors were as in the latest AASHTO LRFD (2010). Figure 6-20 reflects the effect of the response modification factor which shifted the average reliability index values from 2.20 (LFD 2002) to 2.60.

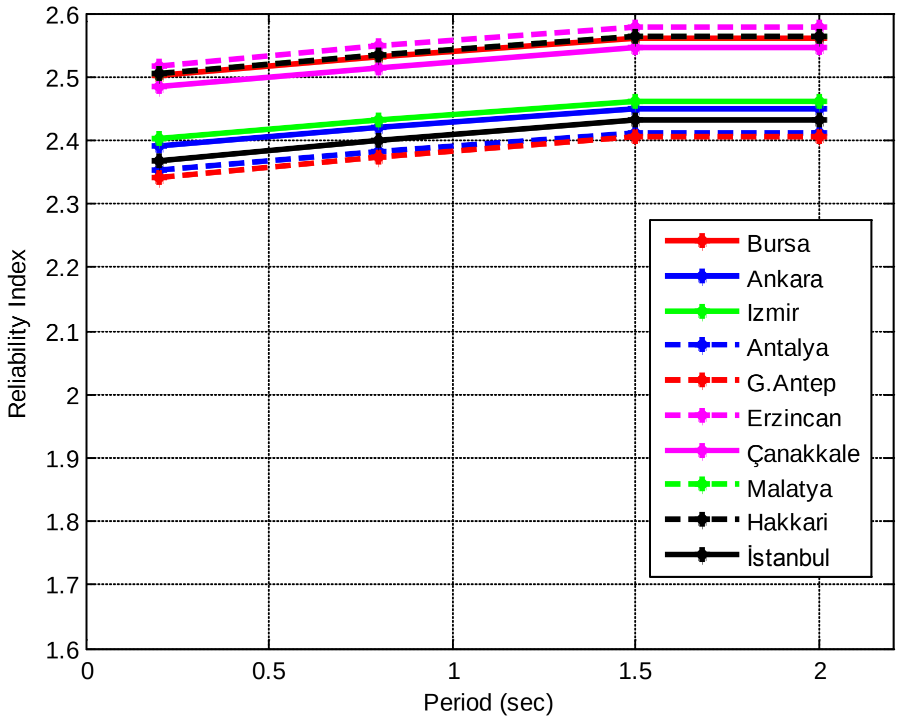

Figure 6-20 : Reliability Index of Biaxial Bending of Pier Columns Designed with using AASHTO LRFD 2007 together with 475 Years Return Period PGA Values Obtained from Gulkan et al.(1993)

Figure 6-20 showed that average reliability index would be scaled 1.20 times since response modification factor decreased from 3.0 to 1.50 for pier columns. The main difference between AASHTO LRFD 2007 and AASTO LFD 2002 resulted from the sensitivity of reliability index to the response modification factor.

6.4.2.5 Reliability Index of Designs per Newly Proposed Design Spectrum

Turkish bridge design authority General Directorate of Highways (KGM) performs a study “110G093 Development of Turkish Bridge Design Engineering and Construction Technologies” associated with Middle East Technical University (METU) and Scientific and Technological Research Council of Turkey (TUBITAK). One of the aims of the project is to develop a load and resistance factor based code for design of new bridges. The design spectrum shown in Figure 6-21 was proposed for the design of new bridges by the mentioned research project.

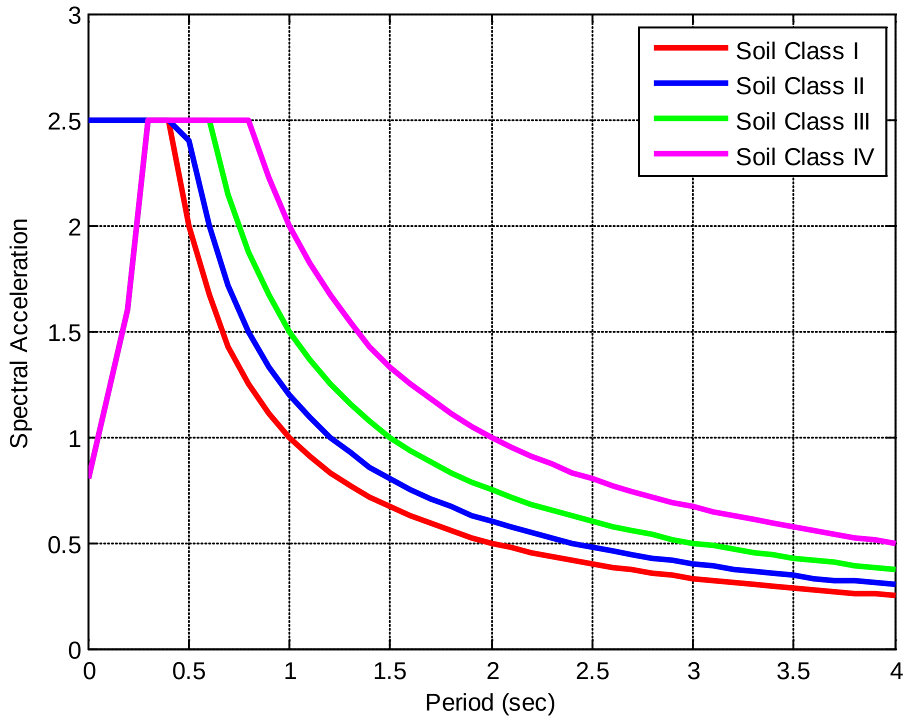

Figure 6-21 : Proposed Design Spectrum by Project 110G093

Calculation of spectral acceleration coefficient is due to following formula:

$$S_a = \frac{A\,S}{T} \tag{61}$$

where;

- Spectral acceleration coefficient , $S_a < 2.50$ A
- Design ground acceleration, A : 10% probability of exceedance in 50 years (475 years return period)
- Site coefficient S,

Table 6-13 : Proposed Site Coefficients

S	S
Soil Class I	1.00
Soil Class II	1.20
Soil Class III	1.50
Soil Class IV	2.20

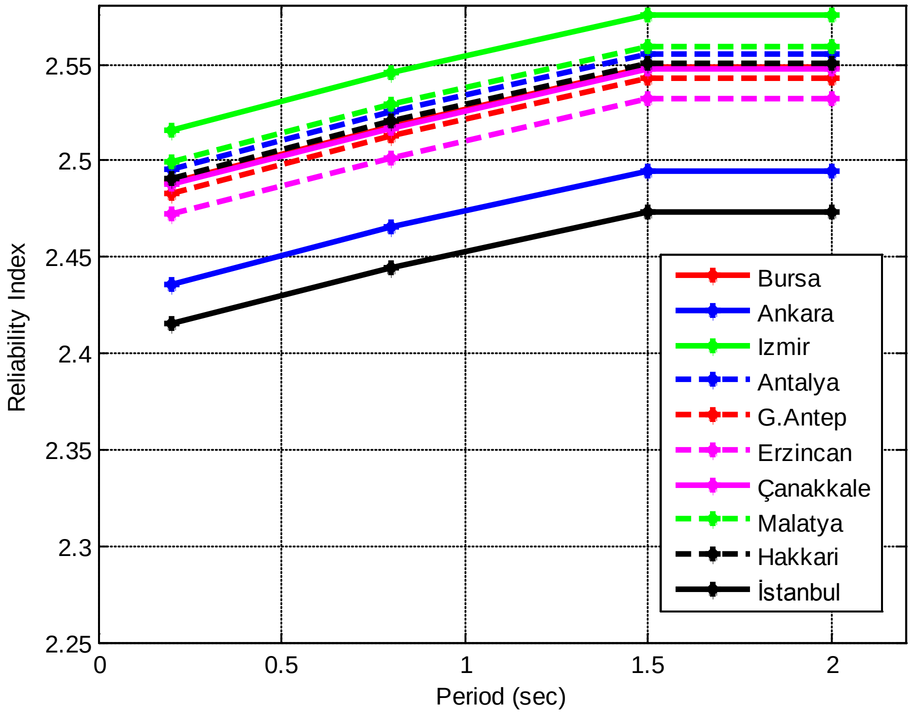

Figure 6-22 : Reliability Index of Biaxial Bending of Pier Columns Designed with using Proposed Design Spectrum together with 475 Years Return Period PGA Values Obtained From Gulkan et al.(1993)

The proposed design specifications have two main revisions on current design practice of AASHTO LFD 2002. First, response modification factor used as R=2.00 for single column bridges. Second, tail portion of design spectra have revised according to T^{-1} different than ASHHTO LFD 2002. In summary, proposed design specifications are similar to the specifications of AASHTO LRFD 2007. For that reason, it can be expected that results given in Figure 6-22 will be in the same trend of Figure 6-20. The reliability indices calculated for proposed design specifications are in the order of 2.50, in average.

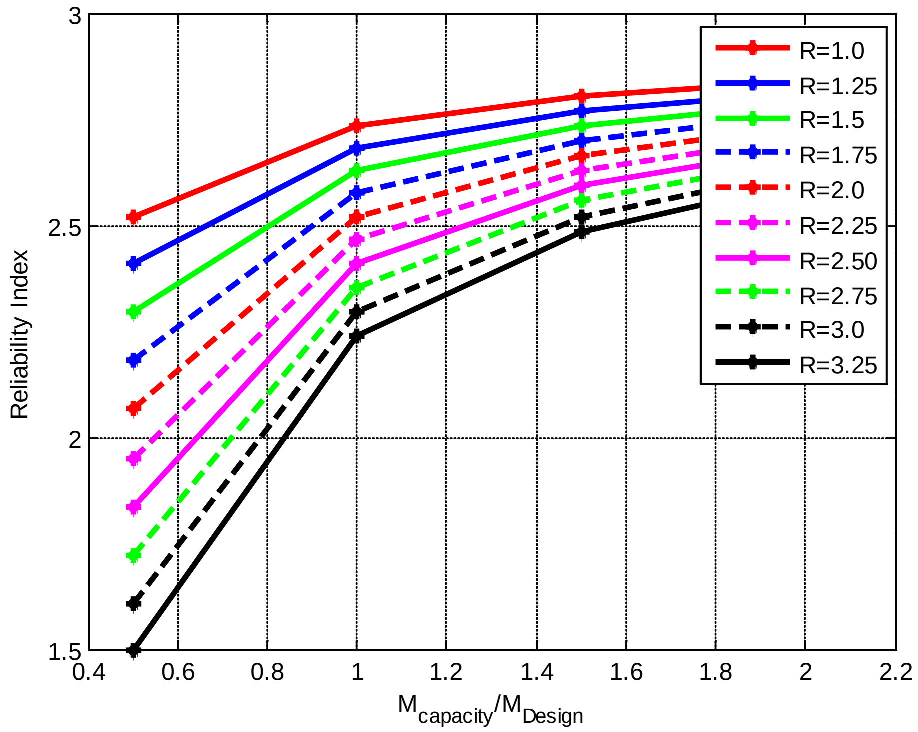

Figure 6-23 : Effect of Response Modification Factor on Reliability Index of Biaxial Bending of Pier Columns Designed with using Proposed Design Spectrum Together with 475 Years Return Period PGA Values Obtained from Gulkan et al.(1993)

Figure 6-23 shows the sensitivity of reliability index to the response modification factors for proposed design specifications. In these specifications, strength reduction factor Φ=0.90 was given for flexural capacity of columns. For that reason, $M_{capacity}/M_{design}$ is equal to minimum 1.11. It is observed that the reliability index can vary from 2.25 to 2.70 as the response modification factor decreases. It can be concluded that, if response modification factor was 3.0, the reliability index would be calculated close to the AASHTO LFD 2002 and if response modification factor was decreased to 1.50, the reliability index would increase to order of 2.60 as it was calculated for AASHTO LRFD 2007. That results show that reliability index is most sensitive to response modification factor among the random variables involved in failure function.

CHAPTER 7

SUMMARY & CONCLUSIONS

Target safety levels are provided in international bridge design codes with load and resistance factor design method. Safety level is determined with reliability theory with respect to the statistical parameters of loads and member capacity.

AASHTO LRFD design codes are calibrated with reliability analysis to achieve target safety. Specifications of AASHTO LFD 2002 are currently used in Turkish design practice. Load and resistance factor design is planned to be used in Turkish bridge designs.

This study aimed to evaluate the safety level of seismic design of Turkish highway bridge pier columns with respect to reliability theory. That aim required the evaluation of statistical characteristics of the loads and capacity in Turkey, in order to determine the reliability indices.

Reliability indices were calculated for bridge piers designed by using

- AASHTO LRFD 2010 together with 1000 years return period PGA values obtained from Gulkan et al. (1993)
- AASHTO LRFD 2010 together with 475 years return period PGA values obtained from Gulkan et al. (1993)

- Modified AASHTO LFD 2002 together with 475 years return period PGA values obtained from TEC (2007)
- AASHTO LFD 2002 together with 475 years return period PGA values obtained from Gulkan et al. (1993)
- AASHTO LRFD 2007 together with 475 years return period PGA values obtained from Gulkan et al. (1993)
- Proposed design specifications of TUBITAK Project 110G093 together with 475 years return period PGA values obtained from Gulkan et al. (1993)

In this study, detailed research for assessment of the statistical parameters was conducted. Failure functions were defined for failure mechanisms of pier columns. Seismic load demand was simulated for billions of scenarios. In addition, analytical methods were introduced to check the simulation results. Member capacities were checked with bi-axial bending – axial compression program. Program was written with algorithm presented in Appendix A.

Results of analysis were presented in Chapter 6. The conclusions are as follows:

1. Pier columns designed with direct adaptation AASHTO LRFD 2010 had reliability index between 2.64 and 2.68.
2. 475 year return period peak ground acceleration is used in Turkish design practice. AASHTO LRFD 2010 requires the use of 1000 year return period peak ground acceleration. If pier columns were designed with the adaptation of AASHTO LRFD 2010 but using 475 year return period map, reliability index is calculated between 2.57 and 2.61. On average 1000 year earthquake design had 3% higher reliability index.
3. Pier columns designed per AASHTO LFD 2002 had reliability index between 2.15 and 2.25.
4. Pier columns designed per modified AASHTO LFD 2002 used in Turkey had reliability index between 1.80 and 2.10. The reliability indices

calculated lower than AASHTO LFD. That is because PGA values obtained Gulkan et al. (1993) can be greater than 0.40g for some sites in high seismic regions. However, in current design practice, highest PGA is used as 0.40g in seismic regions. For that reason, design PGA values have higher probability of exceedance which causes lower reliability index in seismic zones. To avoid that, coordinate based PGA tables should be preferred rather than PGA values used in current practice. (Table 6-12)

5. Pier columns designed per modified AASHTO LRFD 2007 had reliability index between 2.35 and 2.60.
6. Pier columns designed per proposed specifications of TUBITAK Project 110G093 had reliability of 2.50 in average. The target reliability index was reported as β=2.50 or higher in that project.

The bridges designed with current design practice were satisfactory in the recent earthquakes. For that reason existing level of safety can be assumed as a target. It was shown in Chapter 6.4.2.2 that reliability index of bridges designed with using Turkish modification of AASHTO LFD 2002 was calculated to be greater than β=2.0, in average. In order to avoid the reliability indices lower than selected target, PGA values should be obtained from the study of Gulkan et al. (1993).

In conclusion, if PGA values are taken from the Gulkan et al. (1993) all design codes result in bridges that have greater reliability indices from targeted β. It is decision of authorities to increase the target level of safety further, or keeping the existing level according to the requirements and expectations of the public and engineering community.

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APPENDIX A

DEVELOPMENT OF INTERACTION SURFACE FOR BIAXIAL BENDING COMBINED WITH AXIAL LOAD

As a result of the nature of the earthquake loading, combined with the various geometrical possibilities in a bridge pier (such as skew of the column local axis, or unsymmetrical geometry due to architectural or other reasons) the piers are mostly subjected to biaxial bending combined with axial load. Due to computational effort for considering biaxial bending problem designers tend to avoid biaxial bending by either ignoring or the problem is accounted with simplified procedures. In this section the interaction surface for the any arbitrary shaped cross-section is developed. The method used in this study integrates the compressive stress profile over the arbitrary shaped compression surface. Integration is done with numerical methods of Green's theorem which can transform the surface integration to line integration over the intended surface which has to be defined by a closed polygon. In the biaxial bending analysis Bernoulli's beam theory is accepted and tensile strength of the concrete is neglected. Algorithm for development of the interaction surface is adopted from the "Interaction Surfaces of Reinforced – Concrete Sections in Biaxial Bending" (Fafitis, 2001) and assembled with the Monte – Carlo Simulation algorithm of this thesis study.

Following equation defines the concrete stress – strain relation for normal strain ε_{φ} where φ is the distance from the neutral axis.

$$\sigma_{\text{conc}} = \bar{f}(\varepsilon_{\varphi}) \tag{1}$$

If ultimate compressive strain of concrete $\varepsilon_{\text{cu},}$ concrete strain at a distance φ from the neutral axis can be expressed with respect to distance from top compressive fiber to the neutral axis as follows by using Bernoulli's beam assumption.

$$\varepsilon_{\varphi} = \frac{\varepsilon_{\text{cu}}\varphi}{c} \tag{2}$$

With that expression concrete stress can be re-expressed with respect to φ.

$$\sigma_{\text{conc}} = f(\varphi) = g(y) \tag{3}$$

Therefore, stress function described above shall be integrated over the compression surface, A, as follows.

$$N_c = \iint g(y)dA \tag{4}$$

$$M_{xc} \iint y\, g(y)dA \tag{5}$$

$$M_{yc} = -\iint x\, g(y)dA \tag{6}$$

In order to calculate the integrals above a numerical approximation is required. As previously mentioned Green's theorem will be introduced for that purpose. The method transforms the double integrals into line integrals along the perimeter of the compression zone of the cross-section.

This method is preferred in the calculation of interaction surfaces in order not to include any uncertainty due to calculation method. Since up the 2nd degree concretes tress-strain relationships are widely used (first degree rectangular stress block used by ACI-318), using three node Gauss integration will yield to the exact results.

The uncertainty involved in capacity calculations are the uncertainty in material properties such as yield strength of steel and the cylinder compressive strength of concrete, uncertainty in the geometry of the cross section and the uncertainty in the modeling such as assumptions of material behavior (stress-strain diagrams). Due to the analytically exact solution method with Green's theorem, no uncertainty in the calculation method will be introduced.

Double integrals described above are transformed into line integrals along the perimeter of the compression zone. Kaplan (1959) derived the following transformation.

$$\iint \left({}^{\partial \mathrm{Q}}/_{\partial \mathrm{x}} - {}^{\partial \mathrm{P}}/_{\partial \mathrm{y}} \right) \mathrm{dx\,dy} = \oint P dx + \oint Q dy \tag{7}$$

For the following definitions of P and Q which are the functions of x and y, respectively, above formulation is re-arranged.

$$\mathrm{P} = 0 \tag{8}$$

$$\mathrm{Q} = \frac{1}{r+1} \int x^{r+1} y^s g(y) dx \tag{9}$$

$$\iint x^r y^s g(y) dx dy = \frac{1}{r+1} \int x^{r+1} y^s g(y) dx \tag{10}$$

Then the problem is defined as the following line integral.

$$R = \frac{1}{r+1} \int x^{r+1} y^s g(y) dx$$

The resultant, R, depends on the positive integers, r and s, on the right side of the formula as follows.

R	r	s
Nc	0	0
Mxc	0	1
Myc	1	0

In addition, if g(y) is set equal to 1, resultant R gives the following.

R	r	s
A	0	0
Sx	0	1
Sy	1	0
Ix	2	0
Iy	0	2
Ixy	1	1

Steel contribution to the cross section capacity is very straight forward and easy.

$$N_s = \sum A_{sj} f_{sj} \tag{11}$$

$$M_{xs} = \sum y_{sj} A_{sj} f_{sj} \tag{12}$$

$$M_{ys} = -\sum x_{sj} A_{sj} f_{sj} \tag{13}$$

APPENDIX B

ALGORITHM FOR MONTE CARLO SIMULATION METHOD

```
% ARRANGEMENT OF THE CROSS-SECTION

% Transformation of local axis
% Cross-section
xc_tr=xc.*cosd(skew)-yc.*sind(skew)+x0;
yc_tr=xc.*sind(skew)+yc.*cosd(skew)+y0;
% Transformation of Cross-section to Polar Coordinates
% for SORTING
[theta] = cart2pol(xc_tr,yc_tr);
theta=theta.*180/pi;
for i=1:length(node)
    if theta(i)<0
        theta(i)=360+theta(i);
    end
end
% Counterclockwise Sorting Of Coordinates
A= [xc_tr,yc_tr,theta];
A=sortrows(A,3);
while A(1,2)<max(yc_tr)
    A = circshift(A, [1, 0]);
end
A = [A(1:length(node),:); A(1,:)];

% 3POINT - GAUSS-LEGENDRE INTEGRATION
g_index=[0.1127 0.5 0.8873];
g_weight=[0.278 0.444 0.278];
r_s=[0 0; 2 0; 0 2];
    beta=zeros(length(A)-1);
    alpha=zeros(length(A)-1);
for i=1:length(A)-1
    beta(i)=(xc(i+1)-xc(i))/(yc(i+1)-yc(i));
    alpha(i)=xc(i)-beta(i)*yc(i);
end
for a=1:length(r_s)
    r=r_s(a,1);
    s=r_s(a,2);
for i=1:length(A)-1
    for j=1:length(g_index)
        yy(i,j)=(yc(i+1)-yc(i))*g_index(j)+yc(i);
        g_y(i,j)=1;
```

```
        i1=isinf(beta(i));
        if i1==1
            G_y(i,j)=0;
        else

G_y(i,j)=((alpha(i)+beta(i)*yy(i,j))^(r+1))*(yy(i,j)^s)*g_y(i,j);
        end
        S(i,j)=(yc(i+1)-yc(i))*g_weight(j)*G_y(i,j);
    end
end
SS(a)=sum(S(:));
end
A=SS(1);
I_y=SS(2)/3;
I_x=SS(3)/1;


% Column Height L
L_nom=10.5*10^3;
L_bias=1.0;
L_mean=L_bias*L_nom;
L_cov=0.01;
L=L_mean+(L_mean*L_cov).*randn(sample,1);
% Spectral Modeling
C_nom=1.0;
C_bias=1.0;
C_mean=C_nom*C_bias;
C_cov=0.20;%15-40
C=C_mean+(C_mean*C_cov).*randn(sample,1);
% Error
lambda_nom=1.0;
lambda_bias=1.0;
lambda_mean=lambda_nom*lambda_bias;
lambda_cov=0.2;
lambda=lambda_mean+(lambda_mean*lambda_cov).*randn(sample,1);
% Superstructure Weight
Wdc_nom=A*fc_nom*ratio*0.9;
Wdc_bias=0.95;
Wdc_mean=Wdc_nom*Wdc_bias;
Wdc_cov=0.05;
Wdc=Wdc_mean+(Wdc_mean*Wdc_cov).*randn(sample,1);
% Superimposed Weight
Wdw_nom=A*fc_nom*ratio*0.1;
Wdw_bias=0.95;
Wdw_mean=Wdw_nom*Wdw_bias;
Wdw_cov=0.25;
Wdw=Wdw_mean+(Wdw_mean*Wdw_cov).*randn(sample,1);
% Substructure Weight
Wds_nom=A*25*L_nom*10^-6;
Wds_bias=0.95;
Wds_mean=Wds_nom*Wds_bias;
Wds_cov=0.08;
Wds=Wds_mean+(Wds_mean*Wds_cov).*randn(sample,1);
% Total Seismic Mass
mass_nom=(Wdw_nom+Wdc_nom+Wds_nom/2)/9.81/1000;
mass=(Wdw+Wdc+Wds/2)/9.81/1000;
% Moment of Inertia of the Cross Section
k_x_nom=3*E_c*I_x*((L_nom)^-3);
k_x=3*E_c*I_x*((L).^-3);
k_y_nom=3*E_c*I_y*((L_nom)^-3);
```

```
k_y=3*E_c*I_y*((L).^-3);
% Natural Period of the Structure
T_x_nom=2*pi*sqrt(mass_nom*k_x_nom^-1)
T_x_in=2*pi*sqrt(mass.*k_x.^-1);
T_x_bias=1.08;
T_x=T_x_bias*T_x_in;
T_y_nom=2*pi*sqrt(mass_nom*k_y_nom^-1);
T_y_in=2*pi*sqrt(mass.*k_y.^-1);
T_y_bias=1.08;
T_y=T_y_bias*T_y_in;

% Longitudinal
for i=1:sample
if T_x(i)<=T0
    A_s=F_pga*PGA;
    C_sm_x(i)=A_s+(S_ds-A_s)*(T_x(i)/T0);
elseif T_x(i)>T0 && T_x(i)<=T_s
    C_sm_x(i)=S_ds;
else
    C_sm_x(i)=S_dl/T_x(i);
end
end
% Transverse
for i=1:sample
if T_y(i)<=T0
    A_s=F_pga*PGA;
    C_sm_y(i)=A_s+(S_ds-A_s)*(T_y(i)/T0);
elseif T_y(i)>T0 && T_y(i)<=T_s
    C_sm_y(i)=S_ds;
else
    C_sm_y(i)=S_dl/T_y(i);
end
end


% Response Modification Factor 2010
% Longitudinal
Rm_x_2010_nom=1.5;
Rm_x_2010_bias=5.0;
Rm_x_2010_mean=Rm_x_nom*Rm_x_bias;
Rm_x_2010_cov=0.34;
Rm_x_2010=Rm_x_2010_mean+(Rm_x_2010_mean*Rm_x_2010_cov).*(randn(sample,1
));
% Transverse
Rm_y_2010_nom=1.5;
Rm_y_2010_bias=5.0;
Rm_y_2010_mean=Rm_y_nom*Rm_y_bias;
Rm_y_2010_cov=0.34;
Rm_y_2010=Rm_y_2010_mean+(Rm_y_2010_mean*Rm_y_2010_cov).*(randn(sample,1
));

for i=1:sample;
N_eq_2002(i)=C(i)*mass(i)*9.81;
N_eq_2010(i)=C(i)*(Wdw(i)*1.25+Wdc(i)*1.50+Wds(i)*1.25/2);
V_eq_x_2002(i)=C(i)*lambda(i)*mass(i)*9.81*Ao(i)*Sa_x_2002(i);
V_eq_x_2010(i)=C(i)*lambda(i)*mass(i)*9.81*Ao(i)*C_sm_x(i);
V_eq_y_2002(i)=C(i)*lambda(i)*mass(i)*9.81*Ao(i)*Sa_y_2002(i);
V_eq_y_2010(i)=C(i)*lambda(i)*mass(i)*9.81*Ao(i)*C_sm_y(i);
Md_x_2002(i)=V_eq_x_2002(i)*L(i)/Rm_x(i)/1000;
Md_x_2010(i)=V_eq_x_2010(i)*L(i)/Rm_x_2010(i)/1000;
Md_y_2002(i)=V_eq_y_2002(i)*L(i)/Rm_y(i)/1000;
```

```
Md_y_2010(i)=V_eq_y_2010(i)*L(i)/Rm_y_2010(i)/1000;
end

for i=1:sample
    if N_eq_2002(i)/(A*fc_nom/1000)>0.2
        phi=0.5;
    elseif N_eq_2002(i)/(A*fc_nom/1000)<=0.2
        phi=0.90-N_eq_2002(i)/(A*fc_nom/1000)*0.4;
    end

    if N_eq_2010(i)/(A*fc_nom/1000)>0.2
        phi_2010=0.75;
    elseif N_eq_2010(i)/(A*fc_nom/1000)<=0.2
        phi_2010=0.90-N_eq_2010(i)/(A*fc_nom/1000)*0.15;
    end
end

% RESISTANCE CALCULATION

% Material Properties
% Reinforcement Steel
fy_nom=420;
fy_bias=1.075;
fy_mean=fy_nom*fy_bias;
fy_cov=0.09;
fy_std=fy_mean*fy_cov;
sigma=sqrt(log(fy_std/fy_mean^2+1));
mu=log((fy_mean^2)/sqrt(fy_std+fy_mean^2));
fy_s=lognrnd(mu,sigma,sample,1);

area_nom=pi*d^2/4;
area_bias=1.0;
area_mean=area_bias*area_nom;
area_cov=0.03;
area_s=area_mean+(area_mean*area_cov).*randn(sample,1);

% Concrete C30
fc_nom=30;
fc_bias=0.808;
fc_cov=0.009;
fc_mean=fc_nom*fc_bias;
fc_s=fc_mean+(fc_mean*fc_cov).*randn(sample,1);

% ARRANGEMENT OF THE CROSS-SECTION

% Transformation of local axis
% Cross-section
xc_tr=xc.*cosd(teta)-yc.*sind(teta)+x0;
yc_tr=xc.*sind(teta)+yc.*cosd(teta)+y0;
% Reinforcement Bars
xs_tr=xs.*cosd(teta)-ys.*sind(teta)+x0;
ys_tr=xs.*sind(teta)+ys.*cosd(teta)+y0;
% Transformation of Cross-section to Polar Coordinates
% for SORTING
[theta] = cart2pol(xc_tr,yc_tr);
theta=theta.*180/pi;
for i=1:length(node)
    if theta(i)<0
```

```
        theta(i)=360+theta(i);
    end
end
% Counterclockwise Sorting Of Coordinates
A= [xc_tr,yc_tr,theta];
A=sortrows(A,3);
while A(1,2)<max(yc_tr)
    A = circshift(A, [1, 0]);
end
A = [A(1:length(node),:); A(1,:)];



% 3POINT - GAUSS-LEGENDRE INTEGRATION POINTS
g_index=[0.1127 0.5000 0.8873];
g_weight=[0.278 0.444 0.278];
while F_total(tt)<0
ec_bottom=ec_bottom+0.001;
curv(ci)=(ec_top-ec_bottom)/(max(in_yc)-min(in_yc));
c(ci)=ec_top/curv(ci);
if max(in_yc)-c(ci)>=min(in_yc)
    A=in_A;
    xc=A(:,1);
    yc=A(:,2);
    beta=[];
    alpha=[];
for i=1:length(A)-1
    beta(i)=(xc(i+1)-xc(i))/(yc(i+1)-yc(i));
    alpha(i)=xc(i)-beta(i)*yc(i);
end
% Shifting of Neutral Axis
a=2;
for i=1:length(A)-1
    x_int(i)=alpha(i)+beta(i)*(max(in_yc)-c(ci));
        i2=isnan(beta(i));
        i3=isinf(beta(i));
        i1=i2+i3;
    if i1==1

    elseif ((xc(i)>x_int(i)&& x_int(i)>xc(i+1)) || (xc(i)<x_int(i)&&
x_int(i)<xc(i+1))|| (xc(i)==x_int(i))|| (xc(i)==x_int(i)))
       A(length(node)+a,:,:) = [x_int(i) max(in_yc)-c(ci) 0];
       a=a+1;
    end
end
% Deletion of Tension Zone
a=1;
for i=1:length(A)
    if A(a,2)<max(in_yc)-c(ci)
    A(a,:) = [];
    a=a-1;
    end
    a=a+1;
end

% 3 POINT - GAUSS-LEGENDRE INTEGRATION
r_s=[0 0; 0 1; 1 0];
    beta=zeros(length(A)-1);
    alpha=zeros(length(A)-1);
for i=1:length(A)-1
```

```
    beta(i)=(xc(i+1)-xc(i))/(yc(i+1)-yc(i));
    alpha(i)=xc(i)-beta(i)*yc(i);
end
for a=1:length(r_s)
    r=r_s(a,1);
    s=r_s(a,2);
for i=1:length(A)-1
    for j=1:length(g_index)
        yy(i,j)=(yc(i+1)-yc(i))*g_index(j)+yc(i);
        cz=(max(in_yc)-0.85*c(ci));
        if yy(i,j)>=cz
            g_y(i,j)=-0.85*fc;
        else
            g_y(i,j)=0;
        end
G_y(i,j)=((alpha(i)+beta(i)*yy(i,j))^(r+1))*(yy(i,j)^s)*g_y(i,j);
        end
        S(i,j)=(yc(i+1)-yc(i))*g_weight(j)*G_y(i,j);
    end
end
SS(ci,a)=sum(S(:));
S=[];
end
Nc(ci)=sum(SS(ci,1));
Mcx(ci)=sum(SS(ci,2));
Mcy(ci)=-1*sum(SS(ci,3))/2;

% STEEL CONTRIBUTION
for i=1:length(snode)
    es(ci,i)=ec_top+abs((max(in_yc)-ys_tr(i))*curv(ci));

    if es(ci,i)<=-1*fy/Es
        fs(i)=-fy;
    elseif es(ci,i)>=fy/Es
        if es(ci,i)>0.06
        fs(i)=0;
        else
        fs(i)=fy;
        end
    else
        fs(i)=es(ci,i)*Es;
    end
    Fs(i)=fs(i)*area;
    Msx(i)=Fs(i)*ys_tr(i);
    Msy(i)=-1*Fs(i)*xs_tr(i);
end
    F_s(ci)=sum(Fs);
    Mx_s(ci)=sum(Msx);
    My_s(ci)=sum(Msy);
    F_total=F_s+Nc;
    Mx_total=Mx_s+Mcx;
    My_total=My_s+Mcy;
    tt=length(F_total);
end
    for i=1:length(F_total)-1
    F_aa(i,aa) = F_total(i)/1000;
    Mx_aa(i,aa) = Mx_total(i)/1000000;
    My_aa(i,aa) = My_total(i)/1000000;
    angle_rev(aa)=angle(aa);
    end
```