

EFFECTS OF NEAR-FAULT EARTHQUAKES ON SEISMICALLY ISOLATED BRIDGES

by

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Dedicated to, Fisun Yenidogan and Cabbar Yenidogan

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ABSTRACT

EFFECTS OF NEAR-FAULT EARTHQUAKES ON SEISMICALLY ISOLATED BRIDGES

Bridges constitute one of the critical components of the modern transportation systems that need to be functional after major earthquakes. The seismic isolation method has been proposed as an innovative mature performance enhancement strategy to mitigate earthquake risks on bridges. This study aims to review the advances in the seismic isolation technology for bridge applications while assessing the suitability of isolation units in close proximity to active faults. Suite of ground motion sets with pulse and nonpulse contents were compiled from PEER and European Ground Motion Database.

Two multi-span continuous reinforced concrete bridges are adapted to this study by considering distinct nonlinear properties of three isolation systems. The critical appraisal is given to the curved surface spherical friction sliders together with the evaluation of their modelling assumptions in finite element programs. The first bridge example is utilized to perform comprehensive parametric investigations. In contrast, the second bridge example is a part of large scale testing which requires to generate reliable analytic models for the Pseudo Dynamic (PsD) testing.

It has been concluded that the effectiveness of isolation systems in reducing the demand parameters of both bridge models are demonstrated by analytical models. The reliability of implemented analytical models of the Rio-Torto Viaduct were justified through the conducted PsD test. Extensive parametric investigations were indicated that the suite of pulse dominated near source ground motions with coupled vertical components not only led to increase in the demand parameters of isolation systems but also exceeds the code-mandated factors for MCE level earthquakes.

ÖZET

FAYA YAKIN BÖLGELERDE GERÇEKLEŞEN YER SARSINTILARININ DEPREM YALITIMI UYGULANMIŞ KÖPRÜLER ÜZERİNDE YARATACAĞI ETKİLER

Köprüler modern ulaşım sistemlerinin en kritik parçalarından birini oluşturmaktadır ve büyük depremlerden sonra işlevselliklerinin korunması gerekmektedir. Deprem yalıtımı yöntemi, köprülerin deprem risklerinin azaltılmasında olgun ve yenilikçi bir teknoloji olarak önerilmektedir. Bu çalışmada, köprü mühendisliği açısından deprem yalıtım teknolojisinde ortaya çıkan son gelişmeler incelenerek, faya yakın bölgelerde konumlanmış köprülerde deprem yalıtım birimlerinin kullanımının uygunluğunun değerlendirilmesini amaçlamaktadır. Yük pulsu içeren ve içermeyen deprem kayıtlarından oluşan takımlar, PEER ve Avrupa Yer Hareketi Veri Bankalarından derlenmiştir.

Bu kapsamda, çok açıklıklı iki betonarme köprü uyarlanıp, üç deprem yalıtıcısına özgü doğrusal olmayan davranış biçimleri dikkate alınıp incelenmiştir. Değerlendirme sırasında eğri yüzeyli sürtünme yalıtım birimlerine ve bu tip cihazların sonlu elemanlar programlarında modellenmesi için kabul edilen yaklaşımlara özel bir vurgu yapılmıştır. Birinci köprü örneğinden yararlanılarak geniş kapsamlı parametrik incelemeler yürütülmüştür. İkinci köprü örneği ise birinci yaklaşımdan farklı olarak, büyük ölçekli bir deneyin parçası olmasından dolayı Sözdde Dinamik (SD) deney yöntemi için güvenilir analitik modeller oluşturulmasını hedeflemektedir.

Yürütülen çalışma kapsamında, uygulanan deprem yalıtım sisteminin iki köprü modelinde yapısal eleman talep parametrelerinin düşürülmesinde etkin bir rol oynadığı sonucuna varılmıştır. SD deney protokolü dikkate alınarak gerçekleştirilen testlerde, büyük ölçekli köprü test numuneleri için geliştirilen analitik modelin güvenilirliği kanıtlanmıştır. Geniş kapsamlı parametrik araştırmalardan elde edilen sonuçlar, yük pulsu içeren yakın saha deprem kayıtları ile birlikte etkiyen düşey kayıt bileşenlerinin talep parametrelerinde önemli artışlara neden olması yanında yönetmeliklerde En Büyük Deprem için önerilen tasarım katsayısının, en büyük değerinin aşılmasına neden olmuştur.

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LIST OF SYMBOLS / ABBREVIATIONS

cn	Amplitude parameter of FAS
E_c	Tangent modulus of elasticity of the concrete
E_s	Elasticity Modulus of reinforcement
f_{ck}	Characteristic compressive strength
f_{co}	Unconfined concrete strength
f_{ctk}	Characteristic tensile strength
f_{yk}	Minimum yield strength of reinforcement
f_e	Effective lateral confining stress on the concrete
f_{yw}	Yield strength of transverse reinforcement
G	Shear modulus
I_a	Arias intensity
I_c	Characteristic intensity
I_d	Cosenza-Manfredi index
k_h	Horizontal stiffness for elastomers
k_s	Modulus of subgrade reaction
k_v	Vertical stiffness
S_a	Spectral acceleration
S_d	Spectral displacement
S_v	Spectral velocity
V	Shear force
V_r	Shear capacity
V_s	Shear wave velocity
γ	Shear strain
ϕ	Phase angle value of FAS
φ	Zenith angle
θ	Azimuth angle
ω_o	Angular frequency

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASI	Acceleration spectrum intensity
ATC	Applied Technology Council
Caltrans	California Department of Transportation
CAV	Cumulative absolute velocity
CGS	California Geological Survey
COF	Coefficient of friction
COSMOS	Consortium of Organizations for Strong-Motion Observation Systems
CSFS	Curved spherical friction slider
DBD	Displacement-based design
DPC	Italian Civil Protection
DSHA	Deterministic seismic hazard assessment
EC	Eurocode
ED	Energy dissipaters
EDP	Earthquake demand parameters
ELF	Equivalent lateral force
FAS	Fourier amplitude spectrum
FEMA	Federal Emergency Management Agency
FRD	Forward rupture directivity
GMI	Ground motion intensity measures
GMPEs	Ground motion prediction equations
GMSM	Ground motion selection and modification
GMI	Ground motion intensity measures
IBC	International Building Code
IDA	Incremental dynamic analysis
ISESD	European strong motion database
IM	Intensity measure
INGV	National Institute for Geophysics and Volcanology
JRC	Joint Research Center
LRB	Lead rubber bearing
MCSSFS	Multi surface curved spherical friction slider

MOH	Ministry of Health
MSE	Minimum squared error
NGA	Next generation attenuation
NRHA	Nonlinear response history analysis
NTHA	Nonlinear time history analysis
NSF	National Science Foundation
NTC-08	Italian seismic design code
RETRO	Translational access facility of SERIES project
PBEE	Performance based earthquake engineering
PEER	Pacific Earthquake Engineering Research Center
PGA	Peak ground acceleration
PGD	Peak ground displacement
PGMD	PEER ground motion database
PGV	Peak ground velocity
PSA	Pseudo acceleration
PsD	Pseudo-dynamic
PSD	Power spectral density
PSHA	Probabilistic seismic hazard assessment
PSV	Pseudo velocity
RHA	Response history analysis
RMS	Root mean square
SEAOC	Structural engineers association of California
SED	Specific energy density
SDOF	Single degree of freedom
SHA	Seismic hazard assessment
SI	Seismic isolation
TASI	Turkish Seismic Isolation Design Code for Buildings
TS	Turkish Standards
UBC	Uniform Building Code
UHS	Uniform hazard spectrum
ULM	Uniform load method
USGS	U.S. Geological Survey
VSI	Velocity spectrum intensity

1. INTRODUCTION

1.1. Background

Turkey is located in an earthquake prone region and suffered high amount of casualties and loss of property due to earthquakes. Potential impacts of large earthquakes can be mitigated through multi-disciplinary approaches. Bridges constitute the critical part of lifelines and the disruptions in transportation networks might cause significant influences on economy and community on the basis of experienced earthquake damages. It is essential to preserve the functionality of bridges under earthquakes. Current bridge inventories in US and Europe indicate that most of the existing bridges were constructed in 1960s and 1970s and they do not meet the stated performance criteria established in the contemporary seismic design codes. The replacement of all critical bridges that are likely to have inadequate seismic performance during major events is a costly and unrealistic option. Thus, seismic assessment and retrofit activities emerge as a vital need in the earthquake preparedness activities.

The earthquake risk mitigation master plan developed for Istanbul proposes short and long-term strategies and emphasizes the urgent need of retrofitting activities of buildings, important structures and lifelines. Special emphasis was also given to launching of substantial retrofitting programs for critical viaducts and bridges.

After the 1971 San Fernando Earthquake, there has been much discussion about the adequacy of design guidelines of bridges since this major event had a very significant impact on the development of bridge engineering. The large number of bridge collapses during the 1971 San Fernando Earthquake revealed the deficiencies of the 1965 AASHTO and previous bridge design codes in US. Consequently, a series of revisions to the design guidelines is accompanied with comprehensive retrofitting programs for bridges. It is worth noting that safety measures of the conventional seismic design approaches until the end of 1990s is based on single performance level, as life safety at Design Basis Earthquake (DBE) level, rather than preserving its functionality under Maximum Credible Earthquake (MCE) level. This design goal also took place in Section 1626 of UBC-97 by telling that *“The purpose of the*

earthquake provisions herein is primarily safeguard against major structural failures and loss of life, not to limit damage or maintain function". Conventional retrofitting techniques such as steel jacketing of columns and casting of infill walls between columns of bents, strengthening of footings, widening of the pier caps and abutments as well as using restrainer cables are utilized until the late 1980s (Yashinsky, 1998). In other words, the seismic bridge design focused on increasing the lateral strength to resist the inertial *forces* that occurred from ground shaking (Yashinsky and Karshenas, 2003). However, there are some drawbacks associated with this approach in terms of attracting larger seismic forces that might damage other elements of the bridge along the load path while increasing the lateral strength of bents.

The large number of collapsed or heavily damaged bridges during the large magnitude earthquakes accelerated the efforts of improving the state of art in the design and retrofit of bridges. The seismic design of bridges and buildings relies on the dissipation of earthquake induced energy through inelastic response in selected components of the structural elements. Conventional frame type of bridges may be expressed as yielding structures. Peak transient drift is not significantly influenced by changes in the lateral load resisting capacity. This concept is firstly introduced by Veletsos and Newmark (1960) and it is referred as equal displacement rule.

Earthquake response of yielding type of bridge structures is related with structural damage that produces direct economic loss and indirect losses due to possible closure of transportation. Furthermore, it is difficult to estimate deformations and losses accurately through conventional design methods of bridges due to uncertainties associated with assumptions in the design philosophy and earthquake characteristics.

Seismic isolation (SI) is introduced as an alternative technique for the protection of structures under damaging seismic events that satisfies the performance criteria specified in contemporary design guidelines. Application range of SI systems varies from protection of costly equipment to heavy structures.

Simply, earthquakes can be described as a shear dislocation phenomenon which starts to radiate from the focus of the source to the site and amplify or deamplify with respect to site conditions at the site of interest. Damaging effects of the earthquake emerge when the

frequency content of the seismic input motion coincides or in close range with the fundamental frequency of vibration. Seismic isolation application conceptually aims to decouple structures from the horizontal components of earthquake ground motion by shifting the fundamental period of vibration to outside of the damaging range of frequencies and/or to accommodate the displacement demands which might not be provided by the ductility based design concepts. Period shift or accommodation of displacement demands can be provided by installation of elastomeric or curved surface slider bearings to structures where the flexible interface or the sliding layer also limits the transmission of earthquake induced seismic forces to the superstructure or substructure.

In addition to stated goals, seismic isolators are also incorporated to reduce acceleration or displacement response of high rise buildings while allowing designers to minimize structural member sizes, or to control damage and improve their post-earthquake functionality (Higashino and Okamoto, 2006). Effects of a seismic isolation system on damping and the fundamental period of vibration are shown in Figure 1.1.

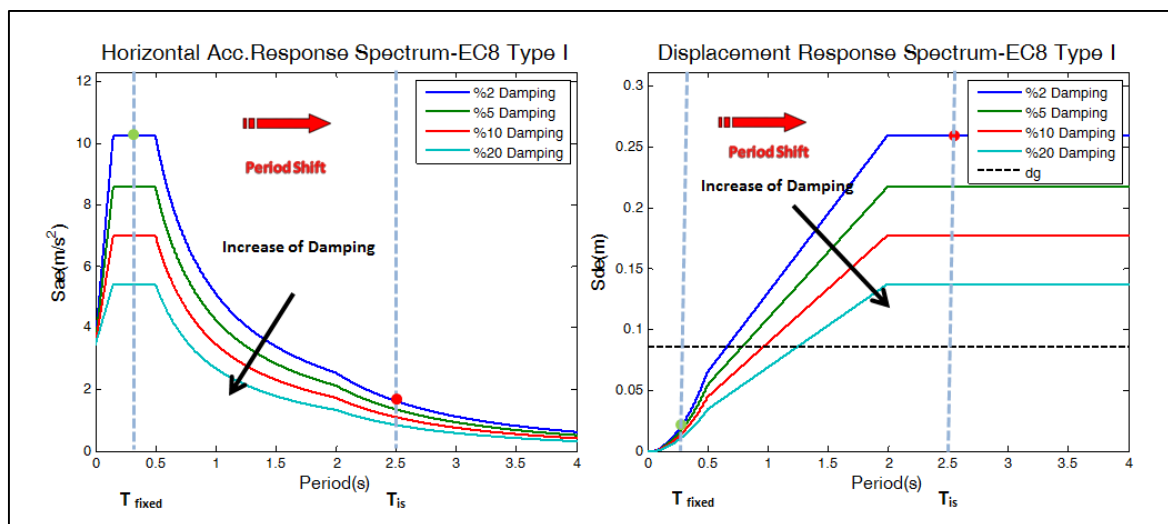


Figure 1.1. Effects of seismic isolation on acceleration and displacement spectra.

1.2. General Information of the Highway Bridge

Seismic design codes and provisions which contain prescriptive rules have been in under development since the catastrophic consequences of major seismic events are well understood by the earthquake engineering community. Improvement in the design guidelines intends to

achieve specific performance by implementing the findings of prominent research studies into practice. Following the conventional seismic design and capacity design philosophy, performance based earthquake engineering (PBEE) concept is introduced by PEER research group (*e.g.* Mackie and Stojadinovic, 2007, Hamburger, 1996). PBEE is widely accepted as a rational approach for the earthquake resistant design of structures. For demonstrative purposes limit states associated with damage in a bridge bent are shown in Figure 1.2 and Figure 1.3.

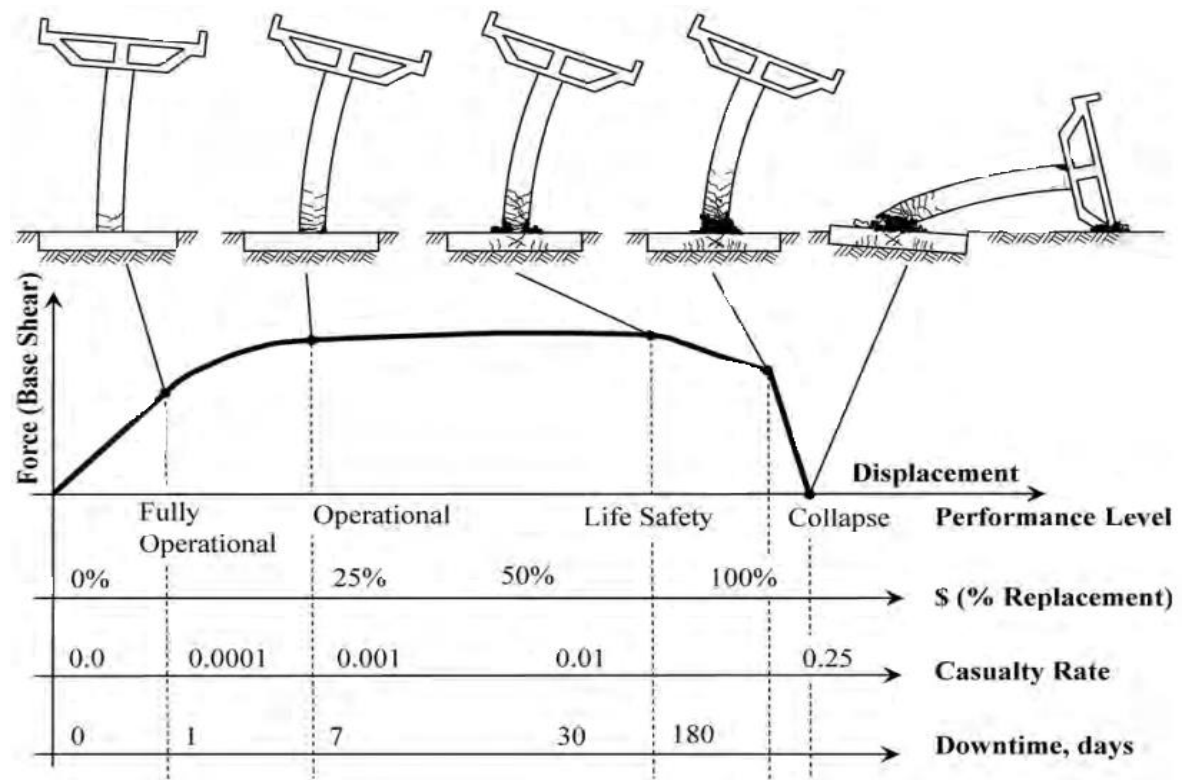


Figure 1.2. Limit states associated with damage in Performance Base Seismic Design (Moehle and Deierlein, 2004).

Performance-oriented goals make the seismic isolation technology a strong candidate for earthquake protection. Bridge seismic isolation is a relatively new concept for engineers when it is compared with conventional retrofit and design methods in practice. It is firstly introduced in AASHTO-1991 under the Seismic Isolation Guide Specification for bridges.

		Earthquake performance level			
		Fully operational	Operational	Life safe	Near collapse
		REPAIRABLE		NON REPAIRABLE	
Earthquake design level	Frequent (40 years)		Unacceptable	Unacceptable	Unacceptable
	Occasional (100 years)		Basic Objective	Unacceptable	Unacceptable
	Rare (550 years)		Essential Objective		Unacceptable
	Very rare (2500 years)		Safety Critical Objective		

Figure 1.3. Performance objective matrix of (SEAOC, 1995).

Seismic isolation and energy dissipation devices provide confidence by satisfying the performance requirements under different design level earthquakes rather than only considering the avoidance of collapse and preserving life safety. Although seismic isolation concept is proposed as an alternative design and retrofit method, erroneous applications in practice may lead to catastrophic consequences. Few of the isolated bridges have experienced major seismic events and Bolu Viaduct is one of real cases where the proposed protective systems failed during the 1999 Düzce Earthquake (Roussis *et al.* 2003). The installed protective devices have failed as depicted in Figure 1.4. The limited number of case studies and past experiences forced researchers to address the advantages, bounding conditions and deficiencies of SI devices using analytical means.

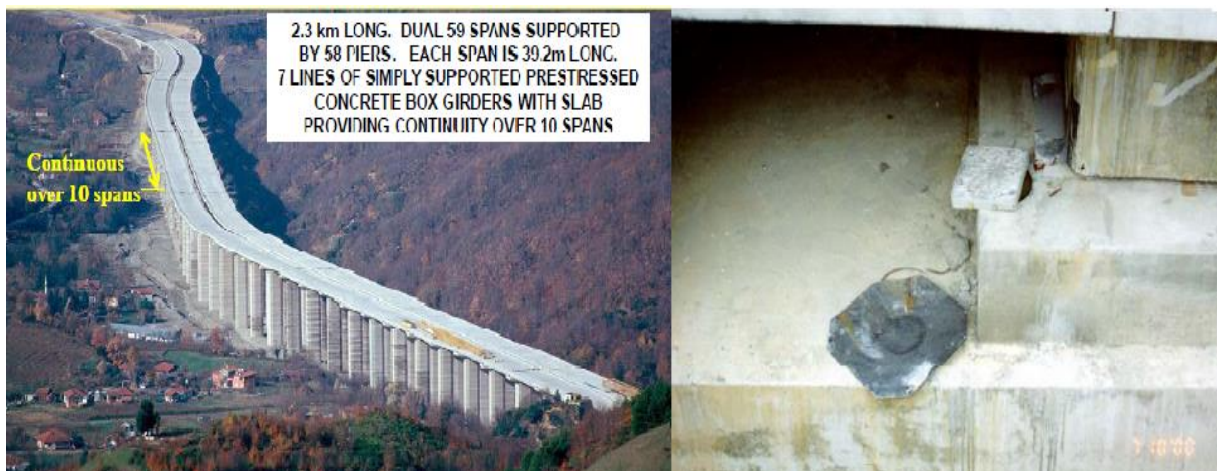


Figure 1.4. Bolu Viaduct and failure of protection devices.

The codes on base isolated buildings and guide specification of seismically isolated bridges requires dynamic response history analysis to verify the design considerations of isolators and the global response of the structures rather than single-mode and multi-mode analysis (IBC-2012, Eurocode 8, AASHTO 2010 Seismic Isolation Guide Specification). The increased use of nonlinear time-history analysis (NTHA) for the determination of structural demands have led to improvement of accuracy in the estimation of structural response. There is a special need to develop guidelines for the selection and scaling of real earthquakes or to simulate ground motion for long period structures such as long span bridges, seismically isolated structures, and liquid storage tanks. Similar effort is performed in the ATC-58 (2011) project for the development of next generation performance-based seismic design procedures for new and existing buildings. Findings of the ATC project are codified in FEMA 445(2006).

Since the 1994 Northridge Earthquake, there is an ongoing discussion about the effectiveness of seismic isolation devices in close proximity (within 20 km to) of faults (Hall *et al.*, 1995, Naeim and Kelly, 1999). Yet, only a few design codes incorporate the near-source effects to represent the demand spectrum (UBC 1994-1997, IBC2000, IBC-2012). Major distinctive characteristics of near-field earthquake ground motions can be listed as directivity, long period pulse content, fling effects and significant vertical ground motion. Directivity of ruptures creates large displacement and velocity pulses in the forward direction. Earthquake records that contains pulse and near-field characteristics are the main design concerns of long period structures. The characterization of long period ground motion is an essential task to introduce the demand for the design of protective units. The sparse coverage of recording stations and the limited number of recorded seismic events constitute the primary obstacles confronted by researchers in the selection and scaling of this type of records with respect to target spectrum.

1.3. Objectives and Scope

The primary objective of this study is to assess the performance of seismically isolated bridges under exposure near-fault effects. Most commonly used isolation devices lead-rubber and curved surface friction slider bearings will be considered. Specific consideration is given to modeling parameters of the isolators, which govern the seismic response of typical bridges through finite element software. This is accomplished by the use of rigorous analytical models

for the isolators. The response of bridges with triple curved surface friction slider bearings is investigated through the analytical models. The intention is to provide support for rigorous dynamic analysis and to provide an insight about the design of seismic isolation of bridges by quantifying the response characteristics. The research tasks to accomplish thesis objectives are as follows:

- Ground motion characterization for near-fault-earthquakes
- Review of current state of art in the selection and scaling in compliance with the seismic hazard assessment
- Efficiency of web-based PEER ground motion database for isolated structures
- Seismic isolation design applications and approaches in practice
- Comparison of modeling assumptions of seismic isolators in the commonly used structural analysis software
- Methods of analysis for seismically isolated bridges
- Analytical and experimental investigations conducted to assess the performance of seismically isolated bridges in near-fault conditions

1.4. Thesis Outline

Chapter 2 provides the basic summary of the ground motion characterization with particular emphasis on the definition of strong ground motion parameters. These parameters are used for the correlation of ground motion intensity (GMI) measures with earthquake demand parameters (EDP).

Chapter 3 presents the distinctive properties of the near-fault earthquake ground motions and their effects on the global behavior of the seismically isolated bridges. Special consideration is given to the tools utilized for long period ground motion characterization.

Chapter 4 addresses to the available selection and scaling procedures in the literature that-are used for the dynamic nonlinear time history analysis.

Chapter 5 provides a literature review about the seismic isolation technology and its specific applications in bridges. Particular emphasis is given to the modeling aspects of the isolation devices, structural sensitivity to different modeling parameters, comparative studies among different isolators and parametric analyses. Code based evaluation procedures and the critical assessment of the current-state-of-the-art are also presented.

Chapter 6 provides a development of the analytical models for different types of seismic isolation devices.

Chapter 7 addresses the investigation of seismic isolation systems through the large scale test of a real bridge located in Italy and developed analytical models.

Chapter 8 focuses on the parametric investigation obtained from the extensive nonlinear time history analysis conducted.

2. ENGINEERING CHARACTERIZATION OF STRONG GROUND MOTIONS

2.1. Introduction

The weak motion seismic activity of earth is more related with the studies of seismologists. In contrast, earthquake engineers are primarily interested to the strong ground motions. Source, path and site effects influence the characteristics of ground motion. All these three effects need to be considered for the proper representation of earthquake ground motion in engineering practice.

Earthquake ground motion time series contain important information that can be directly or indirectly used for the analysis of the response of structures. Recordings of ground motion are important tools for engineers and seismologists to trace the physical phenomena that control earthquakes. Importance of ground motion recordings is certainly emphasized by the following statement of Housner (1970), “The basic data of earthquake engineering are the recordings of ground accelerations during earthquakes. A knowledge of the ground motion is essential to an understanding of the earthquake behavior of structures”. Current state of art in seismic hazard assessment (SHA) and structural analysis require the use of earthquake ground motion recordings as input data.

Seismic design codes and guidelines recommend to use nonlinear response history analysis as the most accurate method of analysis. As a result of the record to record variability, input motions constitute one of the main ingredients that strongly influence the earthquake demand parameters (EDP). Relatively few studies evaluate the correlation of Ground motion intensity measures (GMIs) and EDP (*e.g.* Shome *et al.*, 1998, Cornell *et al.*, 2002, Hancock *et al.*, 2006).

This chapter aims to present the engineering characteristics of the strong ground motion and their usage in earthquake resistant design procedures or in the performance

assessment of structures. Historical perspective of strong-motion recordings and signal processing procedures are beyond the scope of this study. Damage potential of the ground motions will be evaluated by commonly used GMIs for the as recorded components. Horizontal components of earthquake ground motion recordings are included in data set to define the effects of multidirectional loading.

2.2. Strong Ground Motion Parameters

Ground motion intensity measures are the tools incorporated in seismic hazard studies and structural analysis. Engineering characterization of ground motion constitutes the crucial step associated with the correlation of ground motion intensity (GMI) measures with engineering demand parameters (EDP). Increasing number of earthquake records in databases and ease of access to these data make such analysis possible. For engineering purposes, three characteristics of the ground motions that are of primary significance: (1) The amplitude (2) Frequency content (3) Duration of the motion (Kramer, 1996). It is worth to note that only one characteristic of the ground motion might not be sufficient to identify the damage potential of earthquakes. Comprehensive descriptions of strong ground motion parameters can be found in the relevant studies (*e.g.* Kramer, 1996, Stewart, 2001 and Bozorgnia and Bertero, 2004).

Both time domain and frequency domain parameters reveal important information related with the contents of ground motion and its damage potential to a specific structure. Earthquake ground motion at a particular point consists of 3 translational and three rotational components. The common practice only considers the translational components earthquake records (*e.g.* Beyer and Bommer, 2007). The vertical component of ground motions is mostly neglected due to incorrect perception of its effects on structures. However, vertical components might be important in near-source regions where the amplitudes might exceed that of horizontal counterparts.

Earthquake records that are included in PEER ground motion database are classified with respect to magnitude, distance and shear wave velocity averaged over the top 30 m of the soil medium (V_{s30}). On the basis of the stated classification scheme, ground motion bins are created in regular intervals to evaluate the damage potential of input motions. In addition, the

distance parameter is investigated by preparing sets with 30 km intervals from 0 to 150 km with 3 different groups of site characteristics. Total number of earthquake records included in ground motion bins is 6082. Distribution of each GMI versus Joyner-Boore distance is plotted for the considered magnitude bins. In the present section, short description of the ground motion parameters are given under three subcategories within the context of this study. These subcategories reflect the time domain parameters, spectral parameters and energy related parameters. Specified GMI measures are computed for the horizontal components.

Due to vast number of earthquake recordings in the created bins, evaluation of damage potential of each ground motion by performing nonlinear time history analysis for 3D finite element models is not a viable option in terms of the computational time. Consequently, the damage potential of input motions will only be considered under the equivalent lateral force (ELF) analysis procedure of for seismically isolated (SI) structures. Details of the selection and scaling procedures as well as ELF method of analysis in SI structures will be described respectively in the Chapter 4 and 5. Variation of PGA, PGV, PGD, Arias Intensity, sustained maximum acceleration, PGV/PGA, RMS acceleration, RMS velocity, RMS displacement, acceleration spectrum intensity (ASI) and (VSI) velocity spectrum intensity (Von Thun *et al.*, 1988), cumulative absolute velocity (CAV), specific energy, bracketed duration, significant duration, characteristic intensity (I_c), Cosenza-Manfredi index (I_d) and predominant periods, T_{pa} , T_{pv} , T_{pd} of the 6082 ground motions are shown in under three subcategories.

2.2.1. Time Domain Strong Ground Motion Parameters

The most common time domain strong ground motion parameters are the peak ground acceleration (PGA), peak ground velocity (PGV) and peak ground displacement values their absolute maximum values are obtained easily. In pioneering studies of earthquake engineering, PGA has been traditionally used by earthquake engineers to characterize the severity of ground motion. $\ddot{u}_g(t)$, $\dot{u}_g(t)$, $u(t)$ denotes acceleration, velocity and displacement time histories as shown through Equation 2.1 to Equation 2.3.

$$PGA = \max(|\ddot{u}_g(t)|) \quad (2.1)$$

$$PGV = \max(|\dot{u}_g(t)|) \quad (2.2)$$

$$PGD = \max(|u_g(t)|) \quad (2.3)$$

However, the use of these only peak parameters might be misleading for engineers in the estimation of damage level. Apart from these peak ground motion parameters, sustained maximum acceleration introduced by Nuttli (1979), ratio of maximum velocity to maximum acceleration (PGV/PGA) and bracketed duration are also computed as GMI measures in this subcategory. Vm/Am parameter is associated with two peak time domain amplitude parameters which usually occur at different frequency ranges. Therefore this parameter can also be included in the subcategory of spectral parameters. Duration of ground motion parameters might be important for certain conditions where softening and fatigue effects are expected to occur. Two important duration parameters in the literature will be investigated elaborately to determine the influence of duration on SI structures. Bracketed duration is one of the duration parameter related with amplitude. It was defined by Bolt (1969) as the elapsed time between the first and last acceleration greater than a given threshold value (*e.g.*, 0.05g). Distribution of stated parameters with respect to created bins illustrated in Figure 2.1 through Figure 2.6.

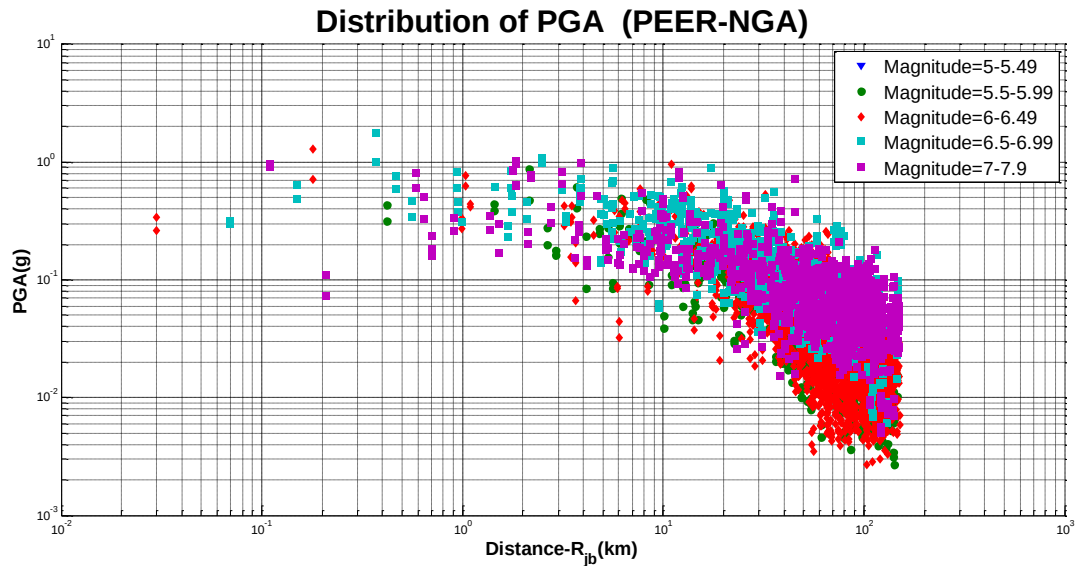


Figure 2.1. Distribution of PGA values.

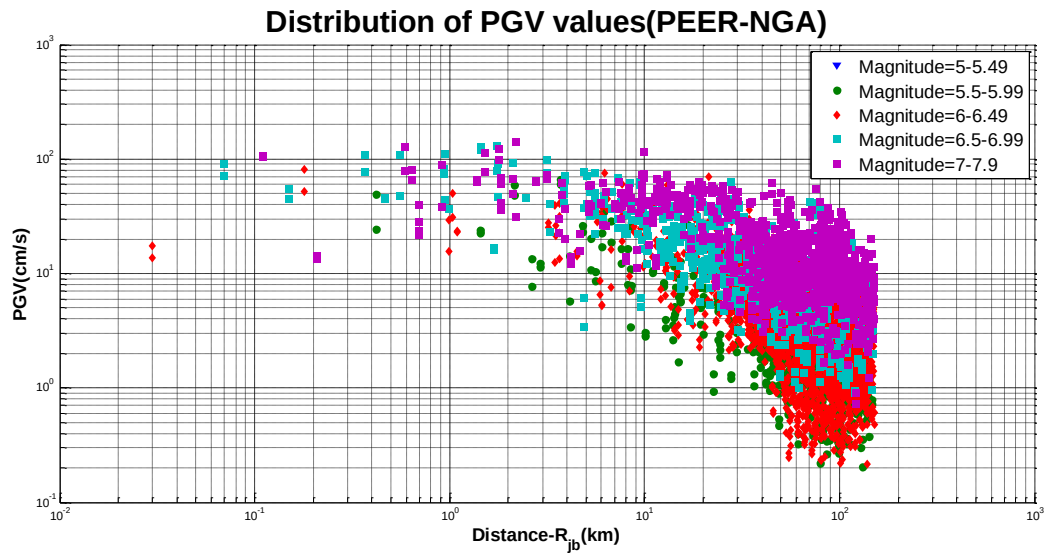


Figure 2.2. Distribution of PGV values.

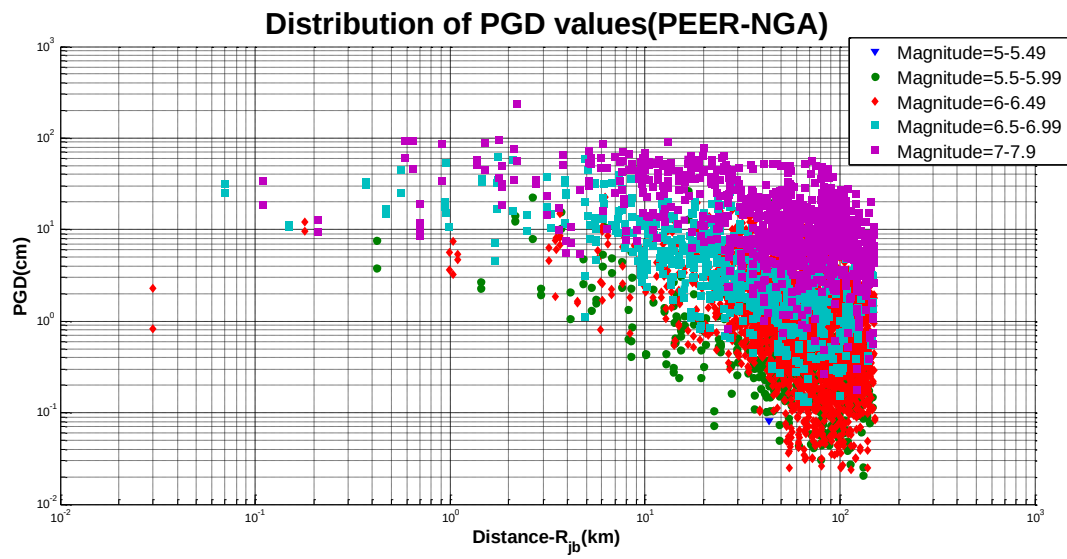


Figure 2.3. Distribution of PGD values.

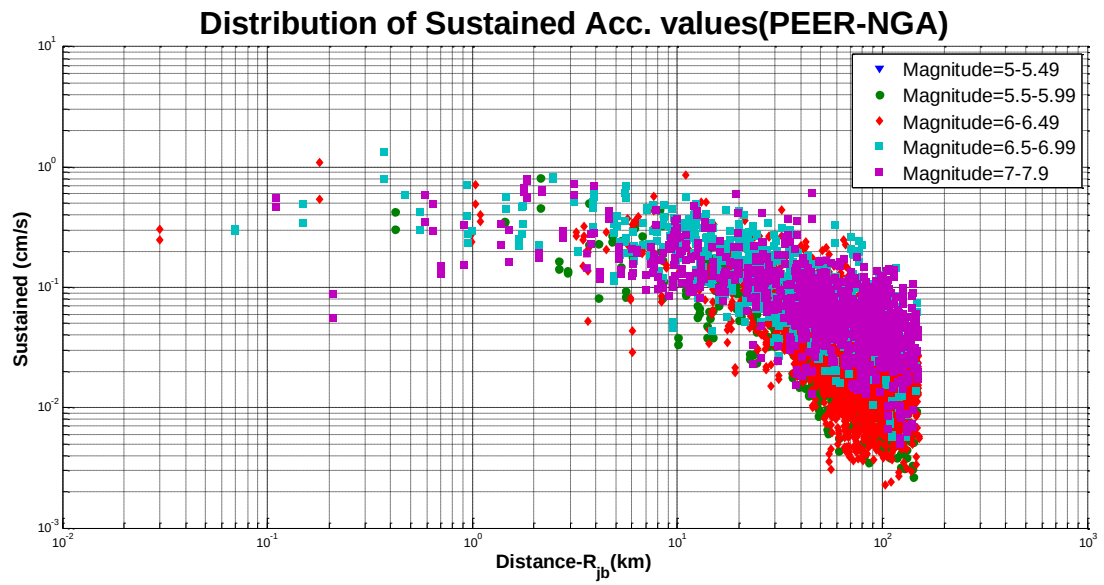


Figure 2.4. Distribution of sustained acceleration values.

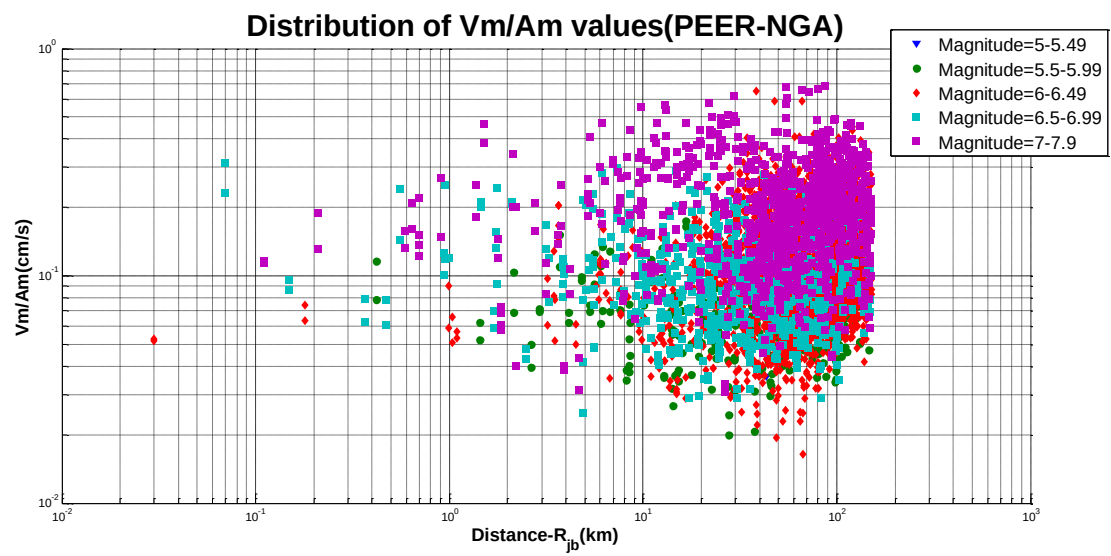


Figure 2.5. Distribution of PGV/PGA values.

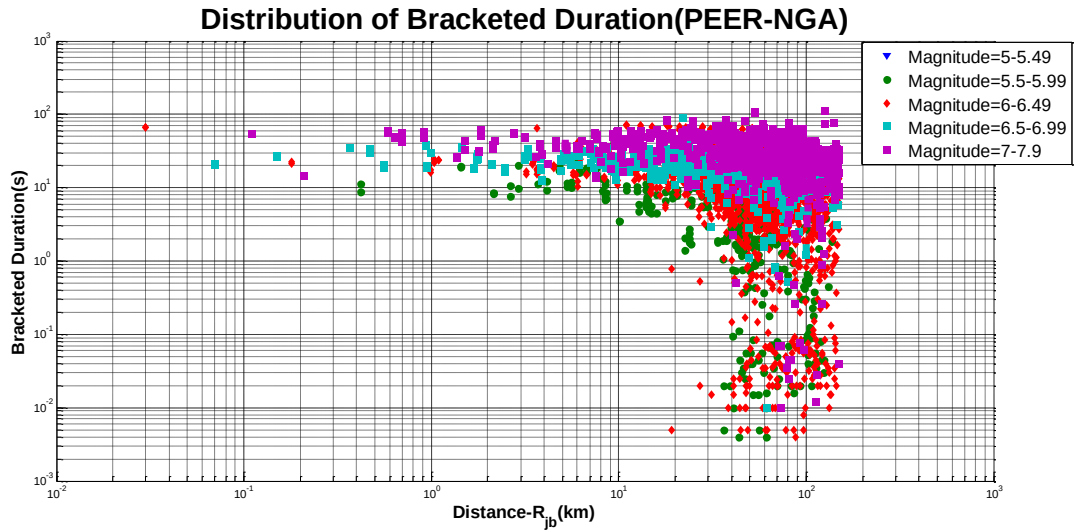


Figure 2.6. Distribution of Bracketed Duration.

2.2.2. Spectral Parameters for the Characterization of Ground Motions

Fourier amplitude and response spectra are the two widely used tools in earthquake engineering. Fourier series represents a summation of simple harmonic components and it can be represented in trigonometric form or by using complex notation. Periodic function can be expressed by Equation 2.4 where c_n and ϕ_n parameters denote the amplitude and phase angle of the n th harmonic motion.

$$f(t) = c_0 + \sum_{n=1}^{\infty} c_n \cos(\omega_n t + \phi_n) \quad (2.4)$$

Plots of Fourier amplitude and phase angle versus frequency are called Fourier amplitude spectrum (FAS) and Fourier phase spectrum. FAS or the closely related power spectral density (PSD), combined with the phase spectrum can completely describe a ground motion (Kramer, 1996). FAS are also employed to quantify the frequency content of ground motions, derivation of predominant frequency and transfer functions.

Likewise FAS, response spectrum concept is an important response quantity which indicates the behavior of a linear elastic SDOF system under a particular ground motion. The concept of response spectrum is firstly introduced by Biot in 1932 as a practical means of

characterizing the ground motions and their effects on structures. G.W. Housner has contributed to the widespread acceptance of the response spectrum concept by the earthquake engineering community (Chopra, 2011). Earthquake excitation, fundamental period of vibration and damping ratio of the structure are used as input parameters to compute the peak response quantity. Different values of damping ratios may also be assigned to cover the encountered behavior of the structure. Response quantities can be expressed in terms of acceleration, velocity or displacement. The maximum spectral values of acceleration, velocity and displacement are referred as spectral acceleration (S_a), spectral velocity (S_v) and spectral displacement (S_d) respectively. For linear, elastic SDOF systems, the spectral displacement, velocity and acceleration are related to each other by following approximate expressions where u and ω_o denote the displacement and angular frequency of the SDOF system.

$$S_d = \max|u| \quad (2.5)$$

$$S_v = \max|\dot{u}| \approx \omega_o S_d = PSV \quad (2.6)$$

$$S_a = \max|\ddot{u}| \approx \omega_o^2 S_d = PSA \quad (2.7)$$

In engineering practice pseudo velocity (PSV) and pseudo acceleration (PSA) values are preferred to be used by practicing engineers as approximate spectral response quantities. PSA and PSV are computed by using spectral displacement value of SDOF system. In most cases, difference between pseudo response values and exact response values are negligible.

Apart from these two well-known earthquake engineering tools, acceleration spectrum intensity (ASI) and velocity spectrum intensity (VSI) are useful parameters to assess the damage potential. ASI and VSI (Von Thun *et al.*, 1988) are respectively defined by formulas in Equation 2.8 and Equation 2.9.

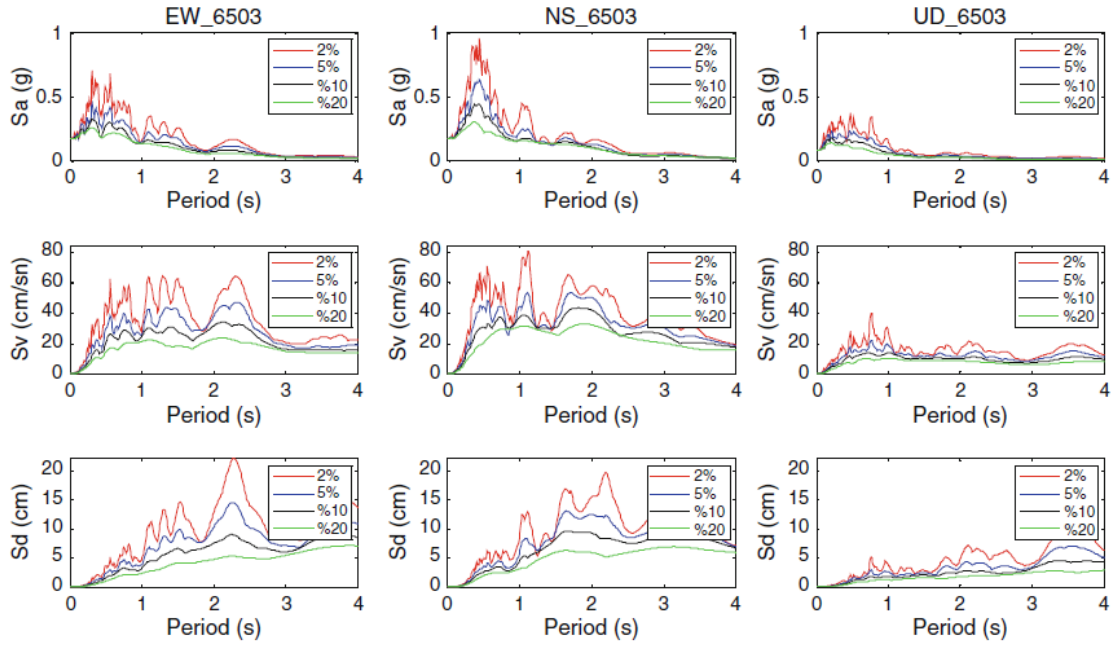


Figure 2.7. Sa, Sv and Sd response spectra of the ground motion recorded at Muradiye Station in Mw 7.1 Van Tabanlı Earthquake (Di Sarno *et al.*, 2013).

$$ASI = \int_{0.1}^{0.5} Sa(ksi = 0.05, T) dT \quad (2.8)$$

$$VSI = \int_{0.1}^{2.5} Sv(ksi = 0.05, T) dT \quad (2.9)$$

Distribution of spectral parameters that are associated with the severity of ground motion such as ASI, VSI and predominant periods of spectral accelerations (Sa), Spectral velocities (Sv) and spectral displacements (Sd) are shown in Figure 2.8 to Figure 2.11.

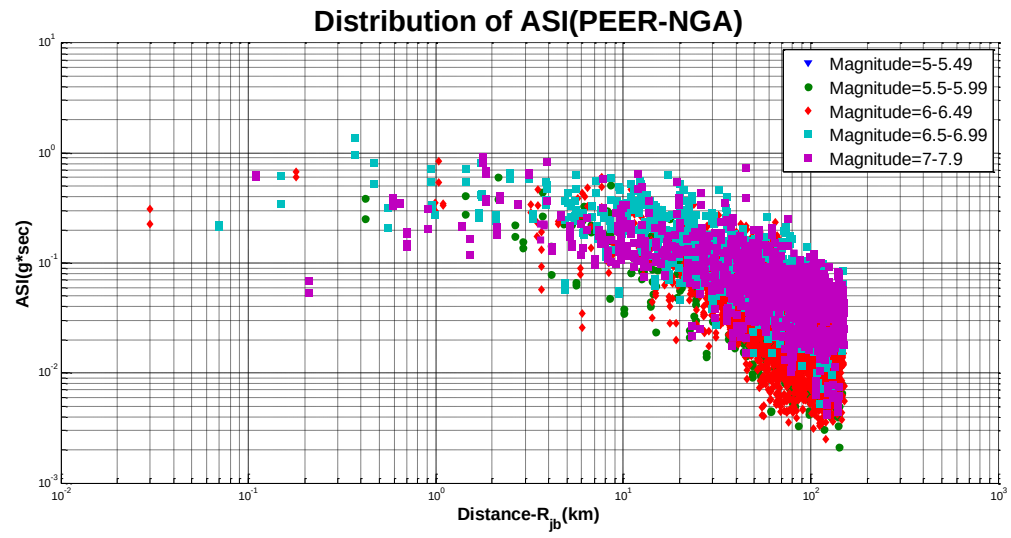


Figure 2.8. Distribution of ASI values.

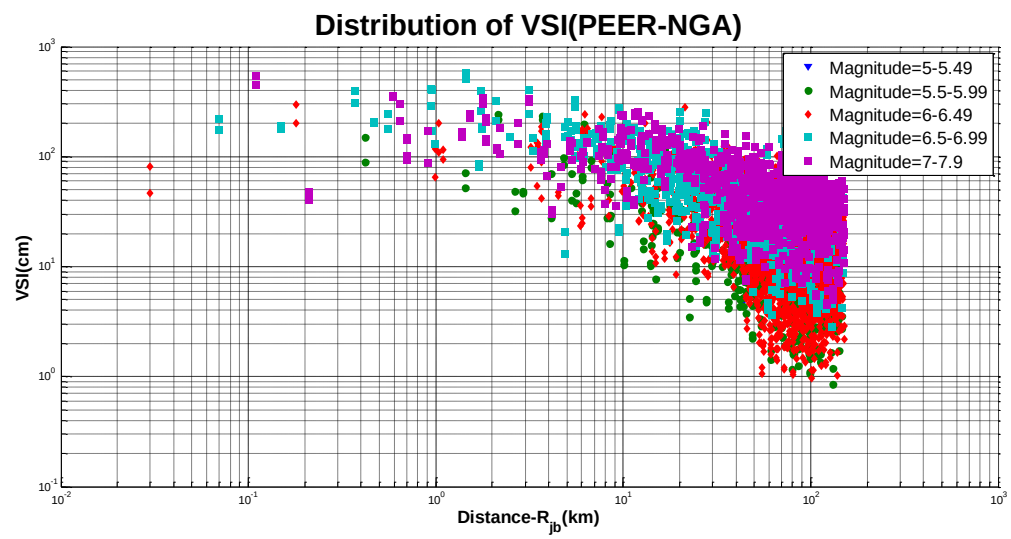


Figure 2.9. Distribution of VSI values.

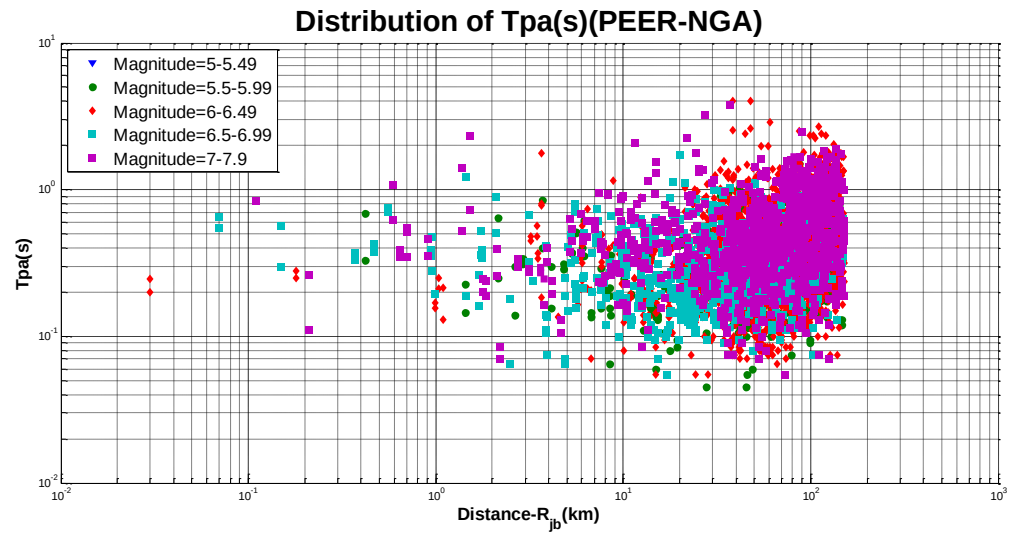


Figure 2.10. Distribution of acceleration pulse period (Tpa) values.

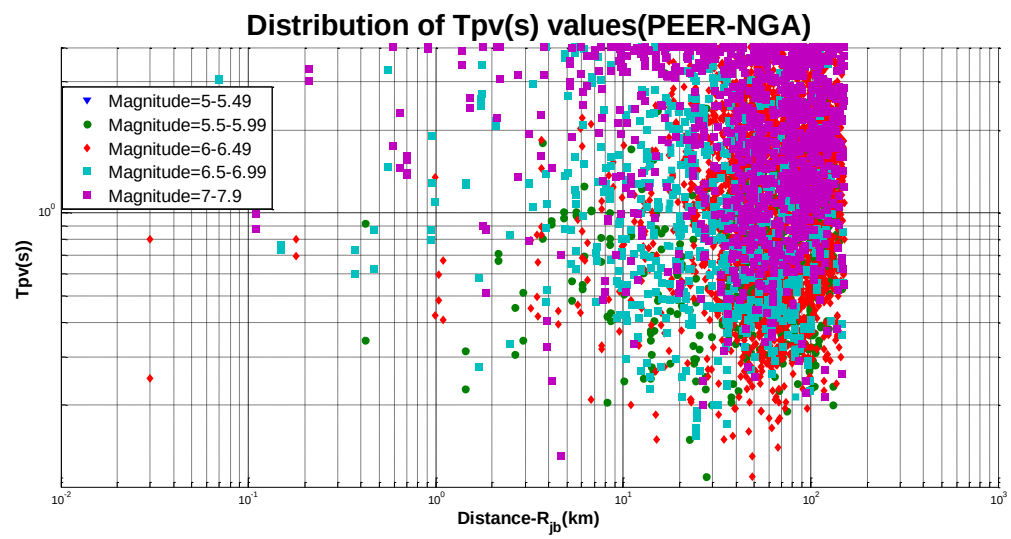


Figure 2.11. Distribution of velocity pulse period (Tpv) values.

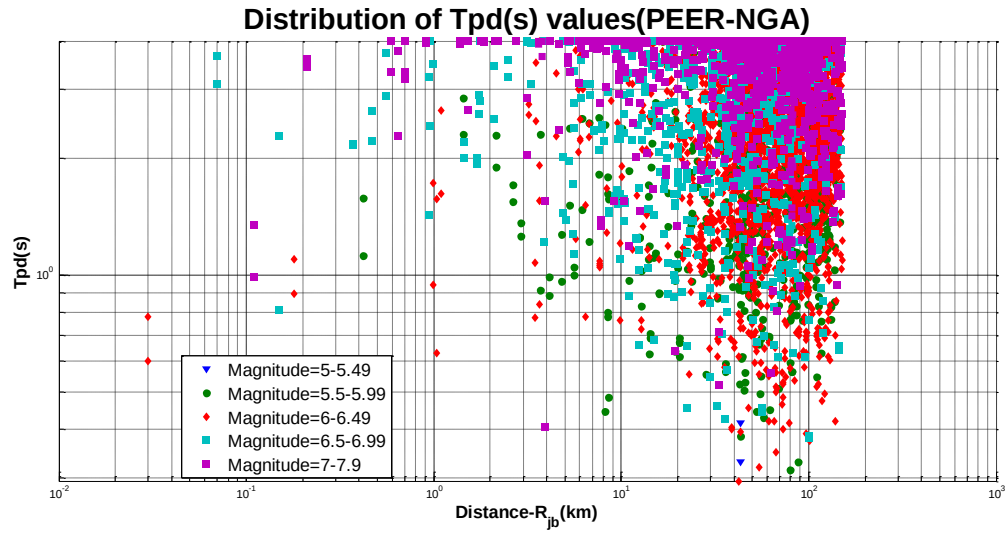


Figure 2.12. Distribution of displacement pulse period (Tpd) values.

2.2.3. Energy Related Parameters for the Characterization of Ground Motions

RMS acceleration, velocity and displacement include both amplitude and frequency content of the excitation as shown below.

$$\begin{aligned}
 a_{RMS} &= \sqrt{\frac{1}{t_{tot}} \int_0^{t_{tot}} [a(t)]^2 dt} \\
 v_{RMS} &= \sqrt{\frac{1}{t_{tot}} \int_0^{t_{tot}} [v(t)]^2 dt} \\
 d_{RMS} &= \sqrt{\frac{1}{t_{tot}} \int_0^{t_{tot}} [d(t)]^2 dt}
 \end{aligned} \tag{2.10}$$

Characteristic intensity is another GMI measure which uses previously described a_{RMS} parameter as shown in Equation 2.11 where t_{tot} in the formula represents the total duration of the ground motion.

$$Ic = a_{RMS}^{\frac{3}{2}} \sqrt{t_{tot}} \tag{2.11}$$

In a similar manner, Arias intensity is related to a_{RMS} parameter. It can be defined as the sum of the total energy per unit weight in a set of undamped elastic SDF systems having frequencies uniformly distributed in the range of zero to infinity (Arias, 1970, Trifunac and Brady, 1975, Bozorgnia, 2004). Arias intensity is calculated by Equation 2.12.

$$Ia = \frac{\pi}{2g} \int_0^{T_{tot}} [a(t)]^2 dt \quad (2.12)$$

Significant duration and Cosenza-Manfredi Index-Id (Cosenza and Manfredi, 2000) also utilizes Arias Intensity parameter while Cosenza-Manfredi Index(Id) also incorporates PGA, PGV values as shown below

$$Id = \frac{Ia}{PGA \ PGV} \quad (2.13)$$

Furthermore, specific energy density (SED) is calculated through the integral of squared velocity time histories and cumulative absolute velocity (CAV) is calculated by the integral of acceleration time histories as indicated in the following Equations.

$$SED = \int_0^{T_{tot}} [v(t)]^2 dt \quad (2.14)$$

$$CAV = \int_0^{T_{tot}} [a(t)] dt \quad (2.15)$$

Distribution of energy related parameters are shown from Figure 2.13 to Figure 2.20.

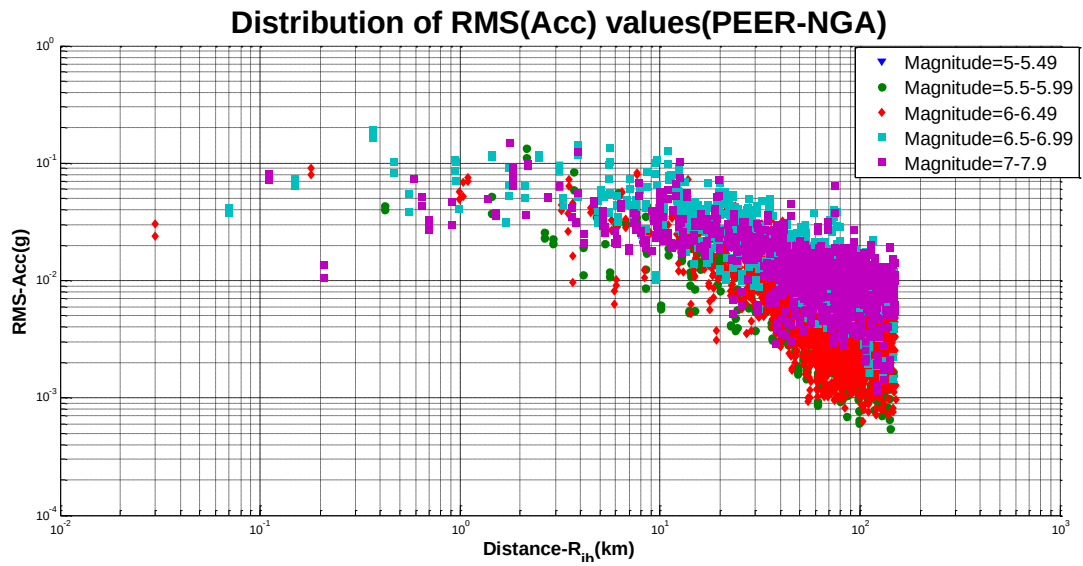


Figure 2.13. Distribution of RMS (Acceleration) values.

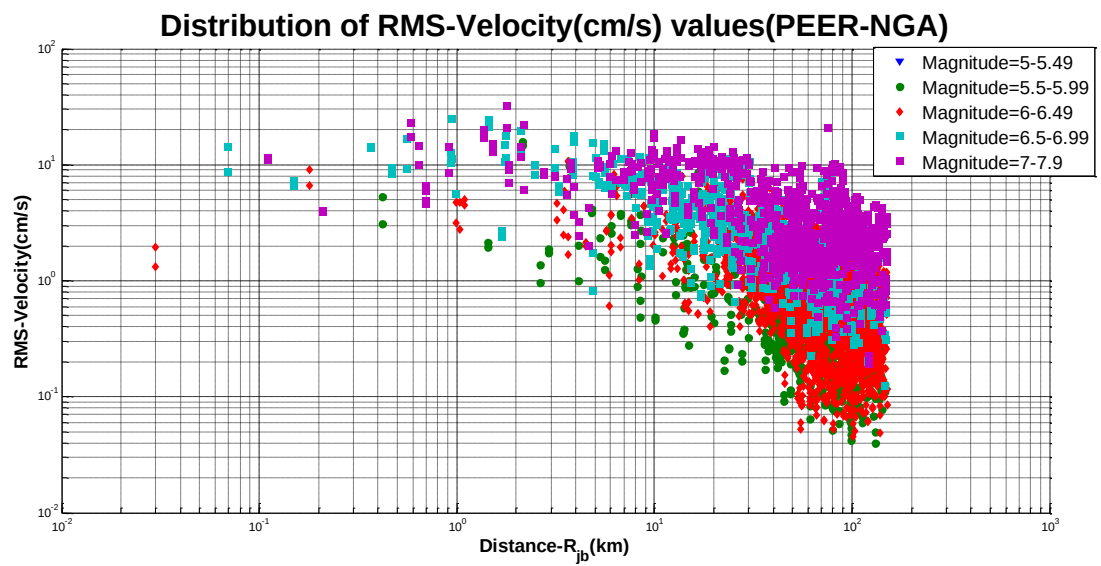


Figure 2.14. Distribution of RMS (Velocity) values.

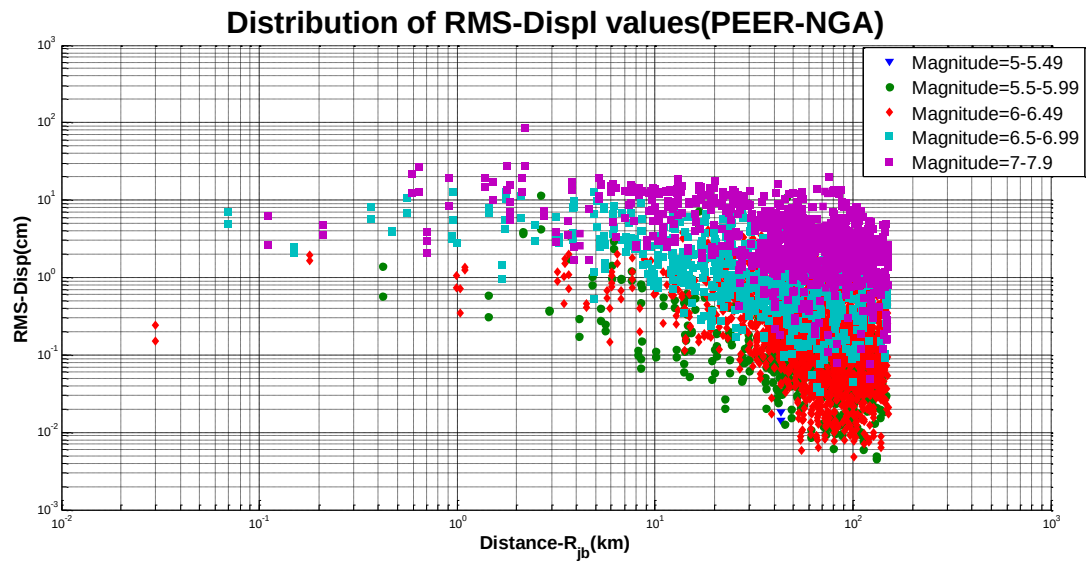


Figure 2.15. Distribution of RMS (Displacement) values.

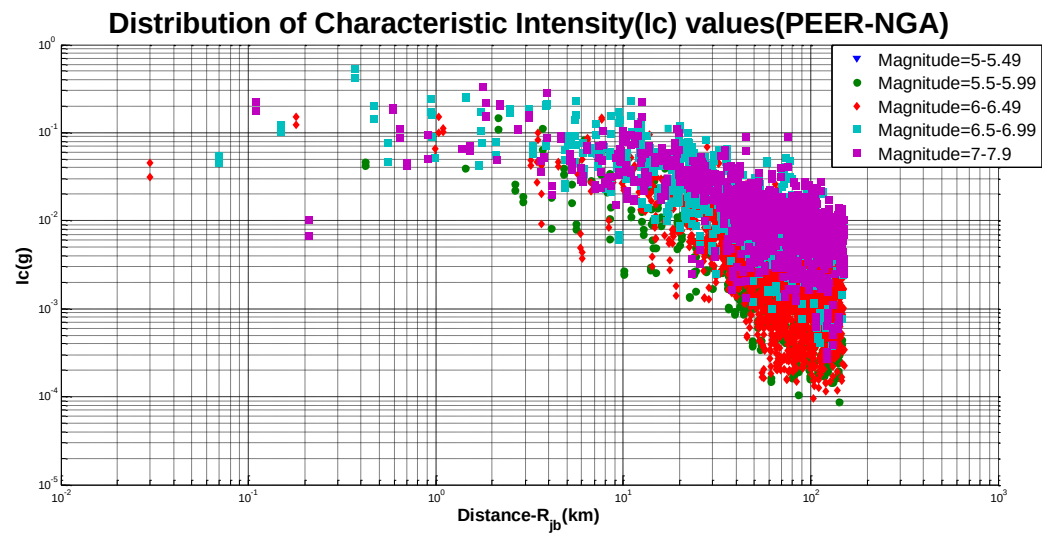


Figure 2.16. Distribution of I_c values.

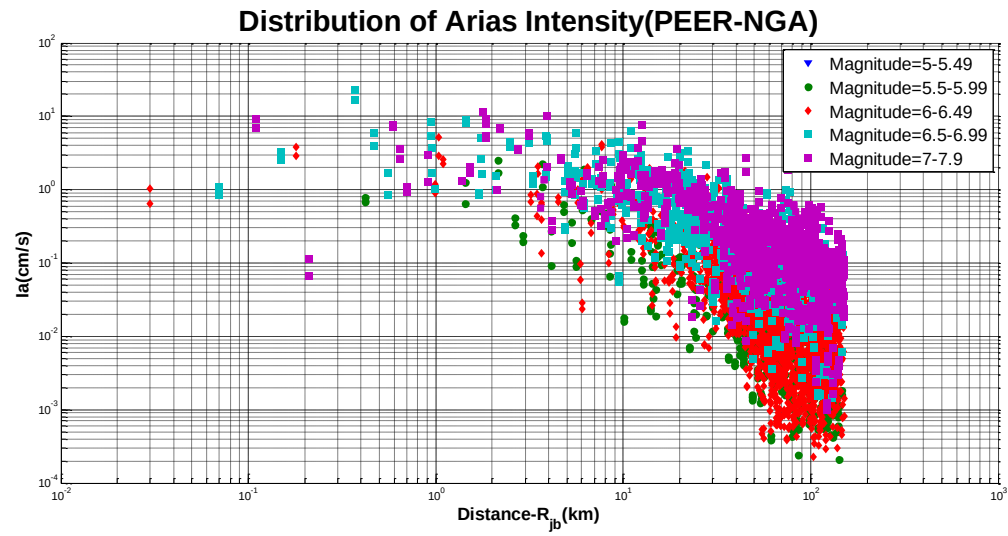


Figure 2.17. Distribution of Arias intensity values.

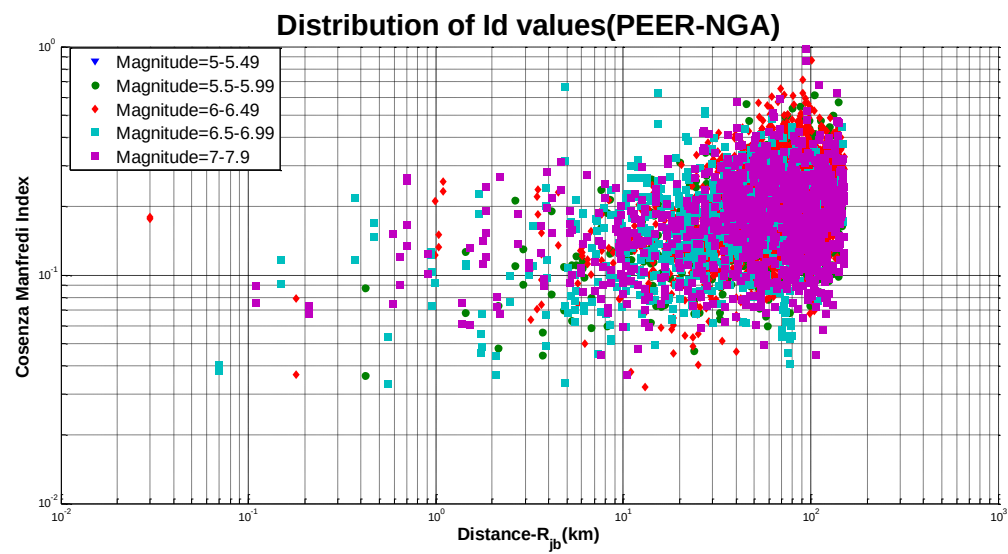


Figure 2.18. Distribution of Id values.

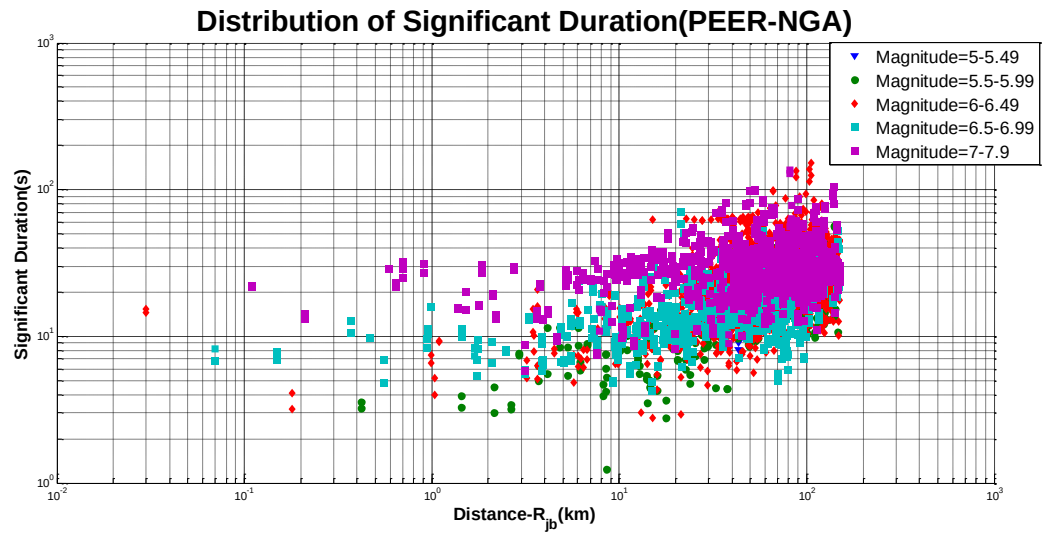


Figure 2.19. Distribution of significant duration values in the created bins.

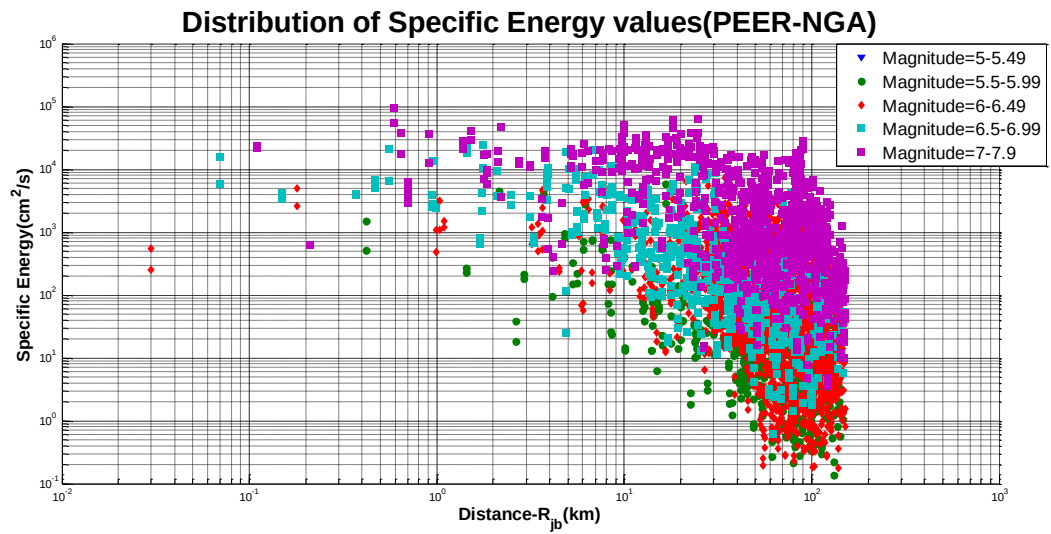


Figure 2.20. Distribution of specific energy density (SED) values.

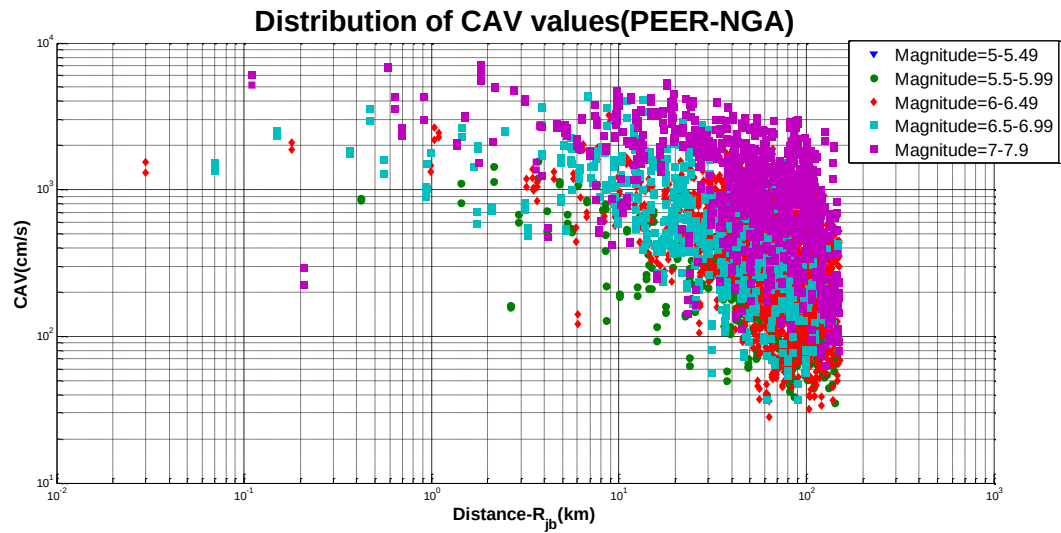


Figure 2.21. Distribution of CAV values.

2.3. Engineering Demand Parameters for the Evaluation of Structural Response

Structural response parameters are incorporated in the quantification of damage induced by strong ground motion. Engineering demand parameter reflects the amount of damage experienced by the structure under certain boundary conditions and GMI measures represent the damage potential of earthquake recordings as the time domain, spectral parameter and the energy related parameters. Similar nomenclature is used by the PEER research group in the scope of performance based engineering concept (*e.g.* Mackie and Stojadinovic, 2003). In this study resultant displacement, Shear force, moment and variation of axial load will be considered as EDP parameters.

Results obtained in the damage potential assessment of earthquake ground motion recordings will be presented in the evaluation of Uniform Load Method (ULM) procedure of seismically isolated structures. ELF and ULM procedure are introduced as simplified method of analysis in seismic design codes (ASCE/SEI 7-10, AASHTO 2010, Eurocode-8). Same procedure is referred as ELF method in the draft version of the Turkish Seismic Isolation Code (TASI, 2009) for Buildings.

3. NEAR-FAULT GROUND MOTIONS

3.1. Background

Design codes and their reference standards describe how to design and detail structures in a prescriptive way. Deficiency of design requirements were realized after post-earthquake observations, and encouraged earthquake engineering community to revise existing design codes and regulations. Distinguishing characteristics of near-fault ground motions were not implemented to design codes until the establishment of the 1990 SEAOC Blue Book. Base isolation Subcommittee of the Northern Section of SEAOC have developed N factors and later these proposed factors are adopted for the 1991 UBC. Following the UBC 1994 and UBC 1997 design factors that account for near fault effects are revised in the ICC 2000 through additional parameters. However, current seismic design practice relies on response spectrum concepts where near-fault characteristics of ground motions might not be provided sufficiently (Sommerville, 1997, Shahi and Baker, 2011).

Distinctive characteristics of near-fault ground motions have been realized through the dense ground motion arrays deployed in the vicinity of active faults. Near-fault ground motions have peculiar characteristics that differ from the rest of the ground motions. The first seismological evidence of the near-field phenomenon dates back to 1950s. Near-fault ground motions contain coherent and orientation-dependent effects at intermediate and long period such as rupture directivity pulses and fling steps. Extensive damage of some structures in near-fault regions are attributed to these particular features. Distinct characteristics of fault-normal and fault parallel component due to near-source effects were firstly reported after the 1971 San Fernando Earthquake (Singh, 1995). Similarly, earthquakes in close proximity to near-fault zones such as in the 1992 Erzincan, 1992 Landers 1994 Northridge California, the 1995 Kobe, Japan, the 1999 Kocaeli and the 1999 Chi Chi Earthquake and the 2011 Christchurch Earthquakes have provided valuable information about the characteristics of this type of ground motions.

Effects of near-source ground motions need to be considered for critical bridges located near an active fault to meet the performance expectations. All of the near-fault

motions do not necessarily contain velocity pulses. Therefore, special emphasis must be given to long period ground motion characterization related with a very rare earthquake shakings. The peak displacement value is one of the critical design parameters of seismically isolated structure where displacement demands might significantly increase when the considered structure is imposed to ground motions with pulse content. One objective of this research work is to better understand the energy demands of near-source ground motion on seismically isolated bridges and make recommendations and question the related stipulations of the currently available code provisions.

3.2. Characteristics of Near-Fault Ground Motions

Post-earthquake observations from major earthquakes indicate that extensive damage of structures are attributed to peculiar characteristics of near-source records within 0-15 km distance to fault rupture. The large amount of data gathered from site surveys and strong ground motion recordings allowed earthquake engineering community to understand the near-field effects. It has been recognized that near-source ground motions are dominated by source characteristics and may vary considerably depending on the characteristics of rupture propagation.

Main distinctive characteristics of near-source earthquakes can be listed as directivity and fling effects which are associated with long period pulse content and large vertical ground motions. Pulse like features of near-source records can be observed in acceleration, velocity and displacement time series. Most of the time, these type of records are characterized by their intense velocity and displacement pulse content. The shear-wave displacements in forward directivity and the static displacements in fling step develop from these strong pulses. From an engineering aspect the velocity pulse is pronounced as a better damage indicator than the acceleration pulse (Hall *et al.*, 1995). Displacement pulses without high amplitude velocity pulses do not necessarily have high damage potential, since it has an enough reaction time. Rupture directivity, fling step and severe vertical component of near-field motions will be explained briefly in the following sections.

3.2.1. Directivity Effects

Damage potential of near-fault ground motions are realized by engineers since the 1971 San Fernando Earthquake. All ground motions that are located in near-fault locations are not subjected to directivity effects. The term directivity generally referred to forward directivity, regarding to occurrence of extensive structural damages under forward directivity. Apart from such common perception, near-fault sites can also be subjected to backward and neutral directivity on the basis of propagation direction, pulse characteristics and seismological features of ground motions. Spatial variation of ground motions and differences between fault-normal and fault parallel components of earthquake records becomes apparent at a period 0.6 and increases in size for larger periods due to rupture directivity effects (Sommerville *et al.*, 1997). A significant portion of the input energy in the rupture process accumulates in a single coherent long-period pulse motion which take place in the beginning of the record. Forward rupture features of generated pulse is shown in Figure 3.1 and Figure 3.2.

The conditions required for directivity effects can be provided by vertical strike-slip and dip slip faulting mechanisms. Near-field locations with dip-slip faulting mechanism additionally can experience hanging wall effects as well. Directivity effects has been studied by numerous researchers as a cause of velocity pulses (*e.g.* Attalla *et al.*, 1998, Hall and Aagaard, 1998, Hall *et al.*, (1995), Somerville and Graves, 1993).

Forward rupture directivity effects occur when the velocity of rupture propagation is close to the shear wave velocity while the rupture propagates forward toward the site, and the direction of slip on the fault is aligned with the site. Ground motions oriented in this forward directivity path may follow certain radiation patterns and generate long-period, short duration, and large-amplitude pulses (Archuleta and Hartzell, 1981, Somerville *et al.*, 1997). Two distinct displacement patterns may be observed as a consequence of rupture process. The first displacement pattern is generally observed in the fault normal component of strike-slip near-source motions. Fault normal oriented component of earthquake shaking is dominated with double-sided velocity pulses associated with such high shear waves in forward directivity effects. The second displacement pattern is linked with the fling step action that accounts for permanent displacement offsets as described in next section.

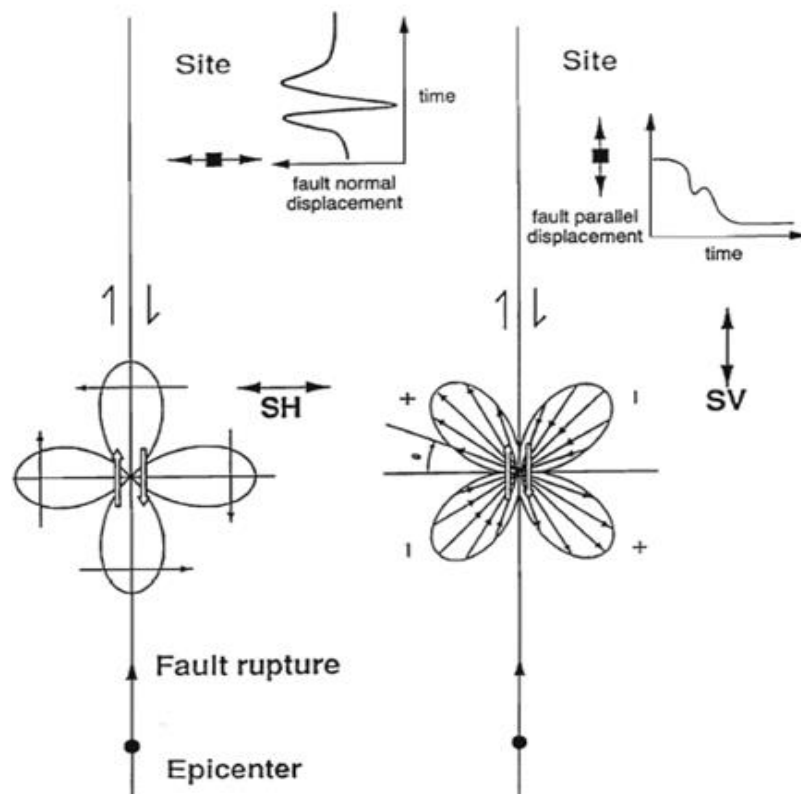


Figure 3.1. Schematic map of the radiation pattern of a vertical strike-slip fault (Sommerville, 1997).

Source rupture characteristics have significant effects on ground motions in near-source zones due to aforementioned reasons. Fault ruptures in large earthquakes, can be significantly different from homogeneous rupture. Two different approaches were developed for the quantification of forward directivity effect in the design. First method utilizes an empirical model of rupture directivity effects which utilizes spectral amplification factors, such as that proposed by Somerville *et al.* (1997). This tool can be used as a guideline for the specification of response spectra and selection of time-histories that are compatible with it.

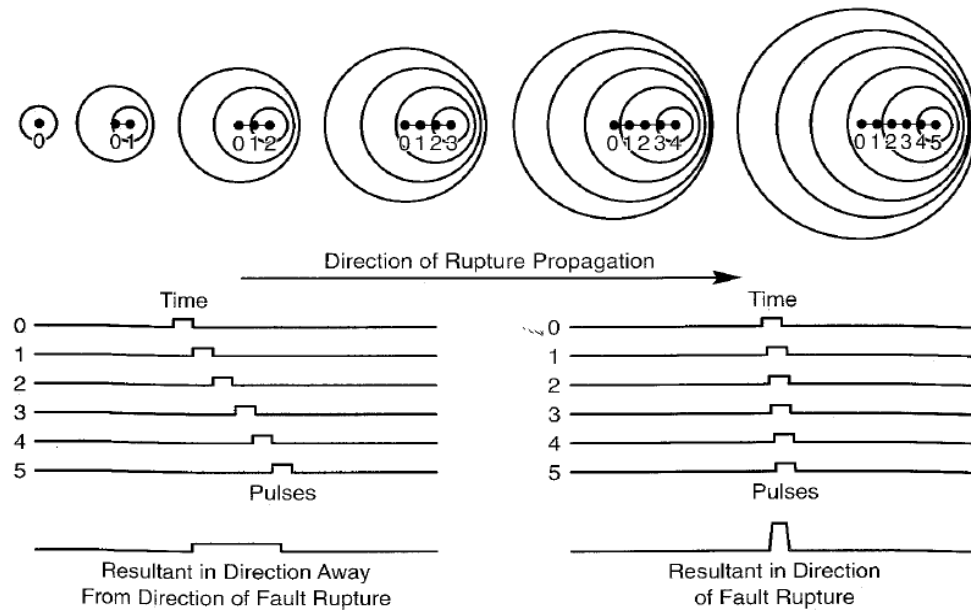


Figure 3.2. Schematic view of directivity effect toward and away from rupture (Singh, 1985).

Modification of ground motion prediction equations needs to be conducted to express duration and strong motion amplitudes in case of rupture directivity. Amount of duration and amplitude of velocity pulses are dependent on the fraction of fault rupture. Somerville's model uses the length ratio(X) and width ratio(Y) for strike-slip and dip-slip faulting mechanisms. The zenith and azimuth angle are denoted respectively by ϕ and θ . Parameters utilized in the proposed model are shown in Figure 3.3.

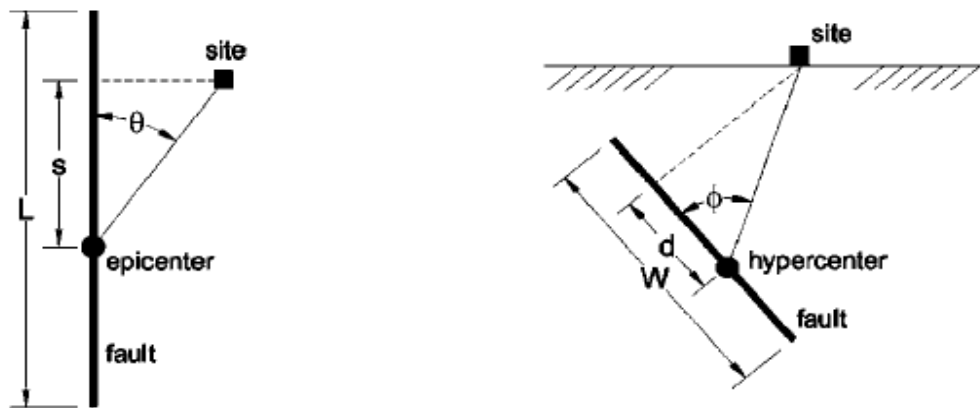


Figure 3.3. Directivity parameters for strike-slip and dip-slip faults defined by Somerville (1997).

Abrahamson and Silva (1997) attenuation relationship have been used in the reference empirical forward directivity model (Sommerville, 1997). The model is independent of site conditions and ground motion intensity. This broadband model was modified by Abrahamson (2000) to incorporate a directivity saturation effect and to taper it at small magnitudes and large distances. As a consequence of such modification notable increase is observed at long periods in probabilistic seismic hazard analysis for rare events. Spudich and Chiou (2008) present correction factors that predict directivity-induced variations of spectral acceleration approximately half of the strike-slip variations predicted by Somerville *et al.* (1997). As a result of such improvements the record to record variability reduces for periods higher than 5s.

Second approach is based on the time-domain representation which characterizes the forward directivity motion through its velocity time history. Forward rupture directivity (FRD) effects generate pulse like motion that differ from other ordinary ground motion pulses. Therefore narrow band representation is required due to natural features of pulse like ground motions. Velocity time history is defined in terms of its peak ground velocity, predominant pulse period and expected number of cycles. Bray and Rodriguez-Marek (2004), Rodriguez-Marek and Bray (2006), and Bray *et al.* (2009) simplify the characterization of near-fault FRD motions by the stated three terms while incorporating the local site conditions. Increasing number of near-source records with rupture directivity effects will contribute to reduction of error term. For near-fault ground motions, different target spectra should be used for the fault-normal and fault-parallel components to preserve the near-fault features in the process of spectral matching or scaling of ground motions.

3.2.2. Fling Step Effect

Fling step effects is a relatively new term for engineers when compared to directivity effects. Fault-parallel components may often experience single-sided velocity pulses that are associated with permanent offset along the fault and it is not strongly coupled with directivity effects. Rate and duration are the two important factors that influences the large displacement demands. The duration of these displacements is closely related to the characteristic slip time of a point on the fault (Heaton, 1990). Recordings of the 1999 Kocaeli

Earthquake in Turkey are exhibited both directivity effects and fling step due to polarization of S-waves (Figure 3.4).

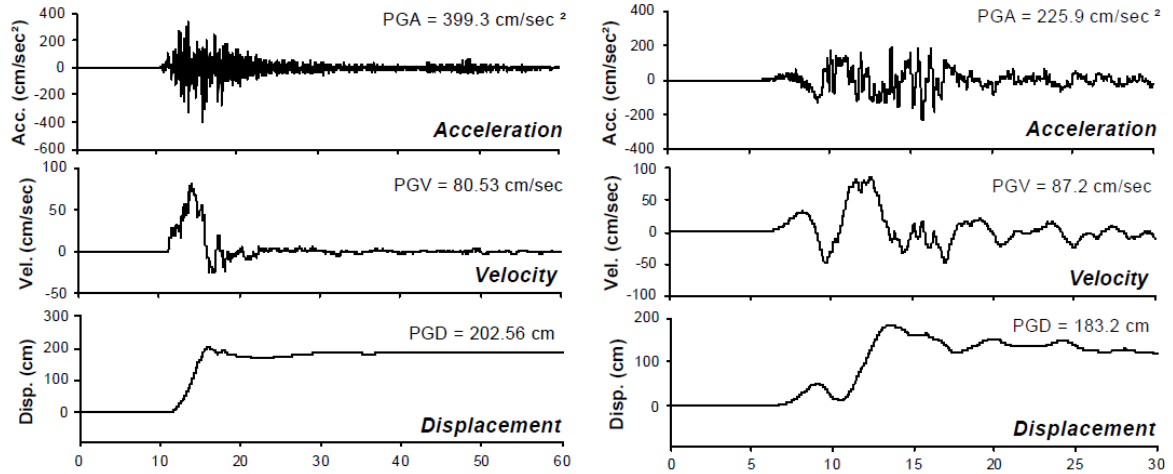


Figure 3.4. Observed fling effects in Sakarya and Yarımca recordings of the 1999 Kocaeli Earthquake.

3.2.3. Vertical Ground Motion

The vertical components of earthquakes are usually neglected as a common design approach in practice. However, field evidences and dynamic response history analysis confirm that structural failures may be attributed to varying tension or compression behavior of elements or its influences on shear and flexural response. Effects of vertical ground motions were included through increased multipliers with an approximated intend in UBC-1997. Some design codes have considered the effects of vertical design spectra simply as 2/3 of its horizontal counterpart. Specific spectral shape of vertical motion were not defined in design codes and standards. Statistical studies on increasing number of records in close proximity to fault rupture gradually changed the common approach and indicates the underestimation by the recommended ratio.

Another important issue about vertical ground motions emerges in the seismic hazard studies for a given annual exceedance frequency in the disaggregation procedure. Vertical and horizontal spectral accelerations are controlled by different earthquake scenarios. Vertical component of ground motions are mostly dominated in the relatively short period

range when compared to horizontal ground motions. In other words, the vertical component of recordings is associated with the arrival of vertically propagating waves, from a seismological perspective.

The peak value of the vertical component of earthquakes may exceed their horizontal counterparts in the vicinity of the active faults. The 2011 Christchurch Earthquake is one of the recent damaging earthquakes which contains severe vertical ground motions as well as rupture directivity effects. PRPC and HVSC stations demonstrate the exceedance of horizontal ground motions values by vertical ground motion. Time series and 5% damped response spectra of PRPC record were illustrated in Figure 3.5 and Figure 3.6, respectively.

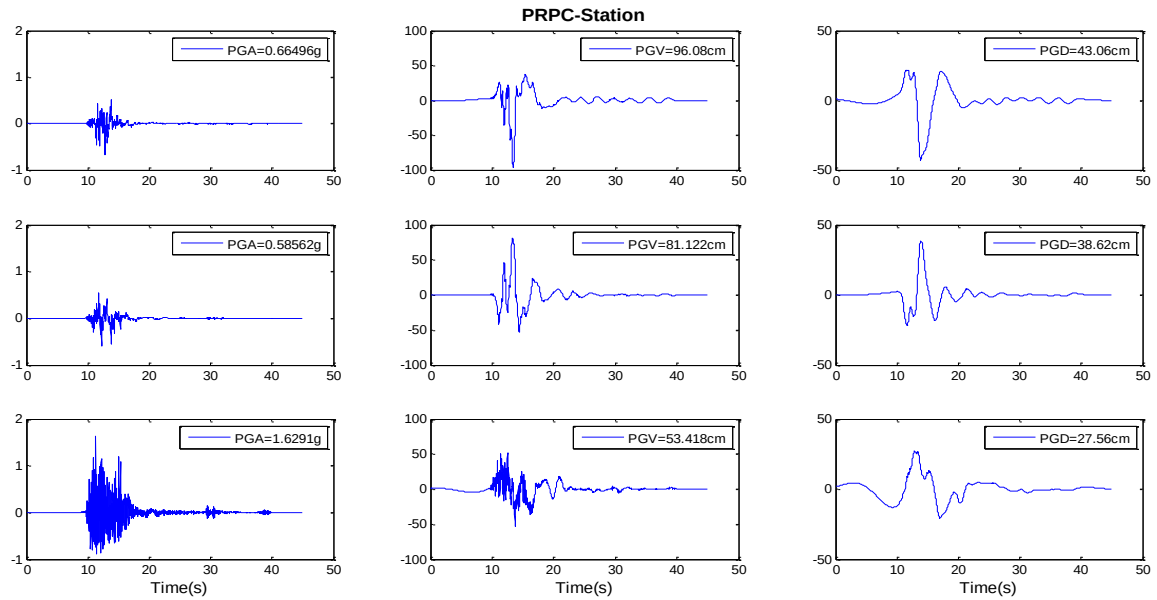


Figure 3.5. Time series of PRPC Station of the 2011 Christchurch Earthquake.

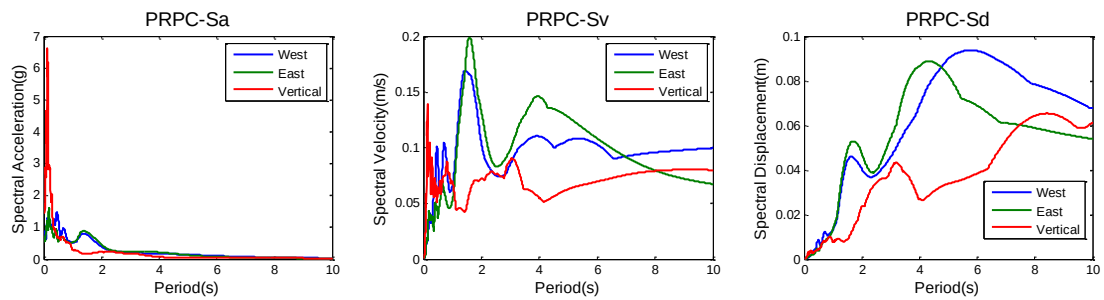


Figure 3.6. Acceleration-velocity-displacement response spectra of PRPC records.

Early studies about the effects of vertical ground motion on conventional bridge response through varying axial loads were conducted by Saadeghvaziri and Foutch (1991). Strong vertical ground motions can have important influences on the isolator response. Behavior of sliding and elastomeric isolators are affected due to tension and compression behavior in axial direction. Rocking motion can be experienced in flat sliding bearings due to large vertical ground motions. Therefore, effects of strong vertical ground motions are also specifically included in this study to determine their effects on the behavior of seismic isolators and associated modelling problems. This requires the use of three component acceleration time-histories for the dynamic response history analyses.

3.3. Characterization of Long Period Strong Ground Motions for Structural Analysis

Proper representation of seismic demand is essential in the design and assessment of critical structures. Seismically isolated structures, high-rise buildings, long-span suspension bridges, cable stayed bridges and liquid storage tanks are classified as long period structures. Current standards and design codes are not adequate for covering the period range of interest for this type of structures with a desirable accuracy. Therefore, the state-of-the-art approach for the design of long period structure is based on the site-specific assessment of the design basis ground motion. Common procedure in this site-specific assessment dictates the disaggregation of the probabilistic hazard to yield magnitude-distance-epsilon values to identify the contributing the hazard scenarios.

Prediction of displacement demands in seismically isolated structures is crucial since it has been identified as better performance indicator. Displacement spectra constitutes the main component of displacement-based design (DBD) procedures as indicated in the recommendations developed by Priestly *et al.* (2007). In some codes the establishment of displacement spectra is based on the assumption of steady-state behavior. European and US standards (IBC, 2012, CEN, 2005) provide characteristic shape of the displacement spectrum on this basis. However, the steady-state assumption may not yield the realistic spectral shape. Corner periods of code-based displacement spectra can be determined on the basis of empirical data or source scaling models. Elastic displacement spectrum of IBC-2012 and Eurocode-8 Part-1 (CEN, 2005) displacement spectrum are shown in Figure 3.7.

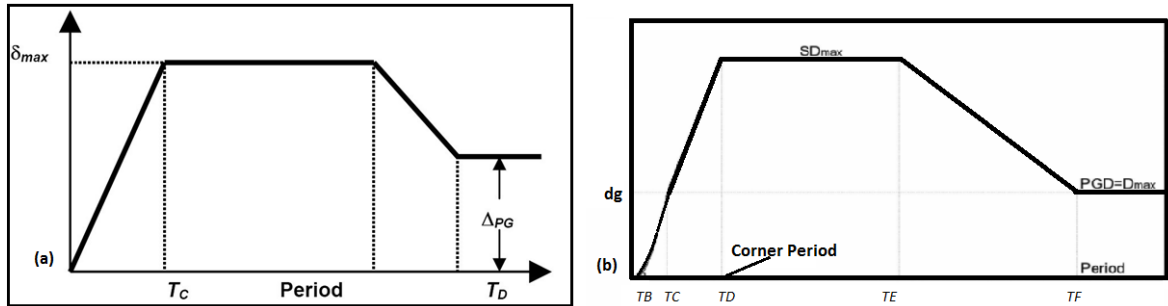


Figure 3.7. (a) Spectral shape of IBC displacement spectrum (b) Spectral shape of Eurocode-8 displacement spectrum.

The determination of damping ratio for structures has always been a challenging and arguable issue for the ground motion characterization in seismic design codes. Modification of earthquake spectra is conducted by applying scale factors to the ordinates of the 5% damped spectra for higher or lower damping values. These damping scale factors can be computed by using either Equation 3.1 for IBC-2006 and ASCE/SEI 7-10 or Equation 3.2 for Eurocode-8, respectively.

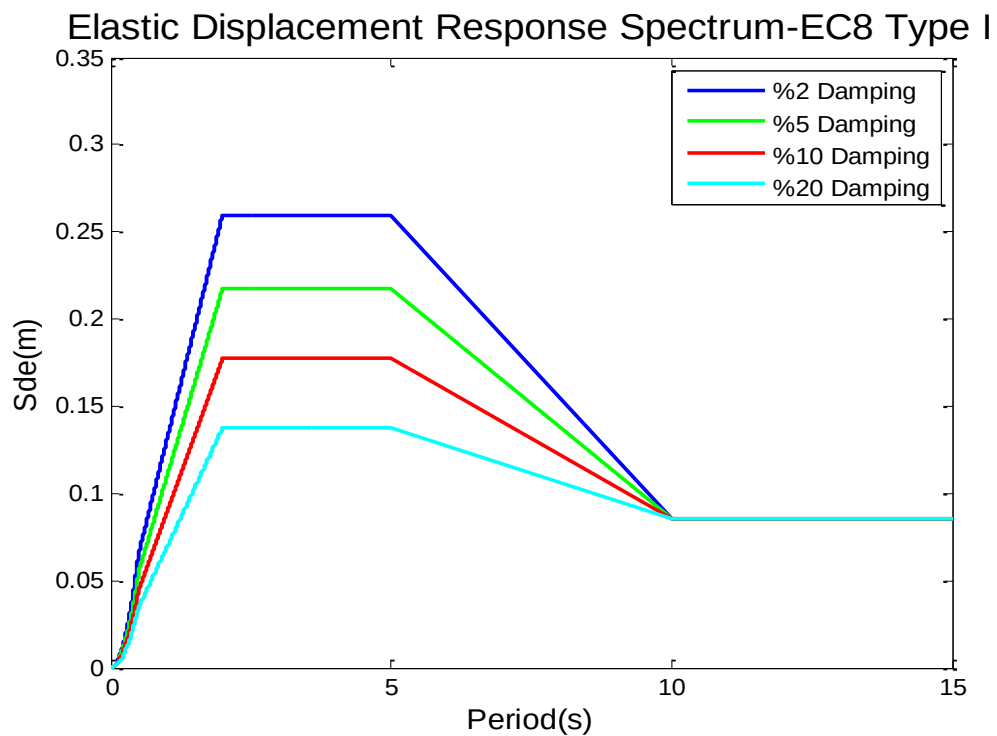


Figure 3.8. Displacement spectra of EC-8 for varying damping values.

$$\eta = \sqrt{\frac{0.07}{0.02 + \xi}} \quad (3.1)$$

$$\eta = \sqrt{\frac{0.10}{0.05 + \xi}} \quad (3.2)$$

Apart from the code-based displacement spectra, ground motion prediction equations can be utilized for the computation of displacement spectra. Bommer and Elnashai (1999), Faccioli *et al.* (2004), Campell and Bozorgnia(2008), and Cauzzi *et al.* (2007) have developed empirical relationships for the prediction of spectral displacement amplitudes based on available strong motion data. Comparison of displacement spectra of Turkish earthquake records with the GMPE of Campell and Bozorgnia (2008) are shown in Figure 3.10.

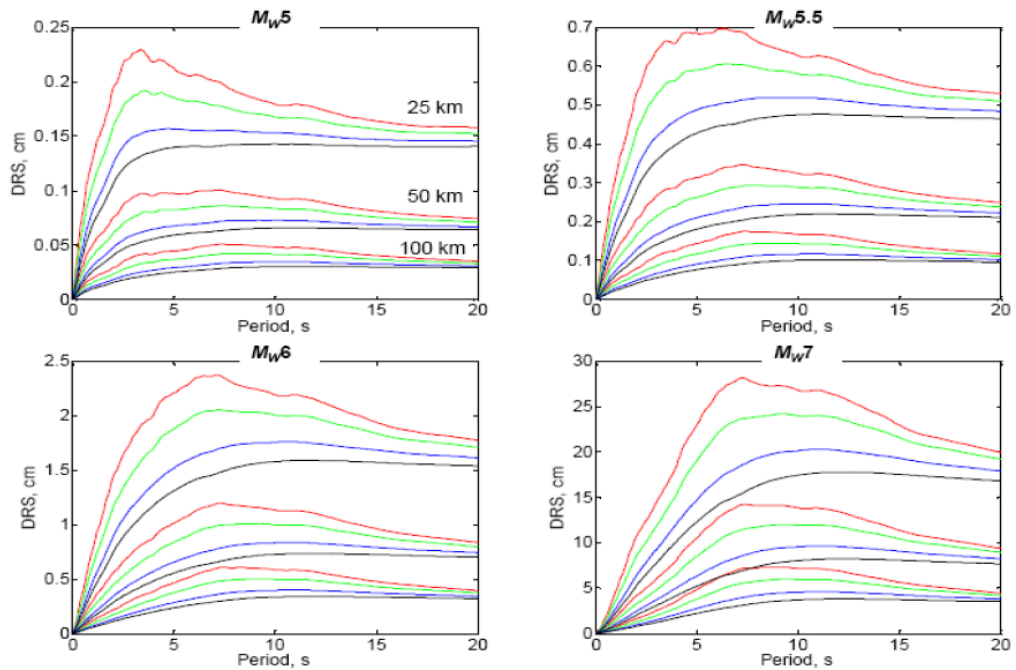


Figure 3.9. Displacement spectra for varying moment magnitude, distance and damping ratios (Cauzzi et al., 2007).

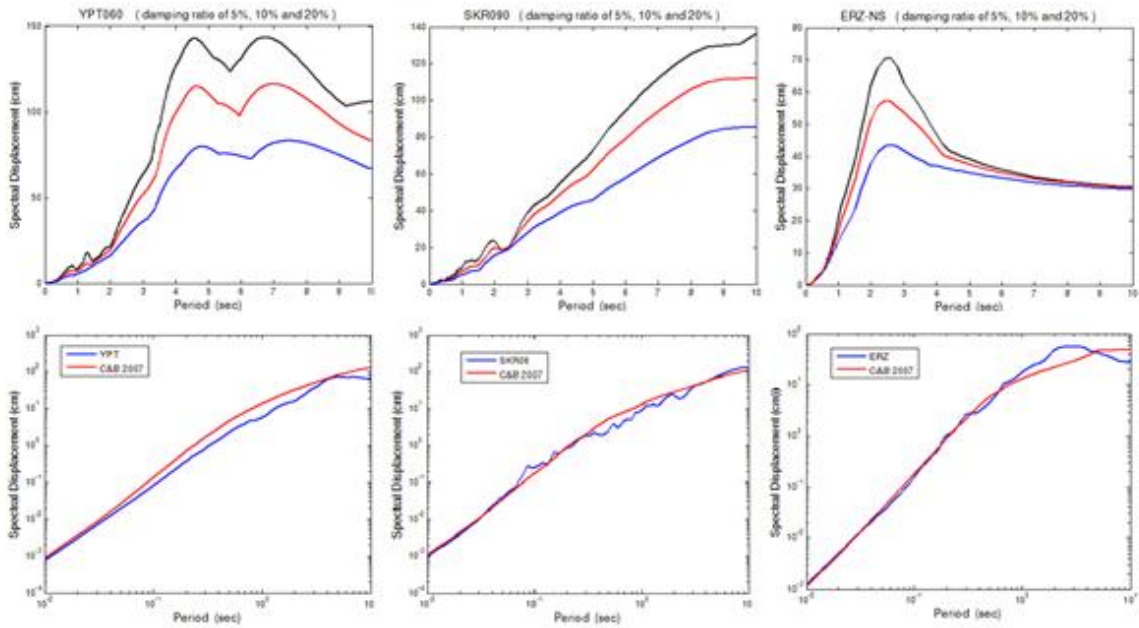


Figure 3.10. (a) Displacement spectra of three near-source earthquake recordings in Turkey with respect to variation of damping (b) Comparison of displacement spectra of three near-source records with Campell and Bozorgnia GMPE (2007).

Since the near-fault ground motions contain long period pulses it is not possible to characterize it through stochastic processes as is the case for other earthquake ground motion. In the current state of knowledge near-fault ground motion pulses can be incorporated into analysis by utilizing two different approaches in the current state of knowledge. First approach relies on the response spectrum method by amplifying the spectral ordinates of the established spectrum concerning near-fault effects for the design of structures (Sommerville, 1997). Therefore, two different spectra should be constructed for fault-normal and fault-parallel component. Empirical spectral amplification factors are determined on the basis of source and distance relationships of the ruptured fault. Recommended amplification factors are independent from site conditions and ground motion intensity. However, there is a growing understanding that response spectrum method is not capable of representing effects of near-fault pulses.

The second approach characterizes the directivity and fling step effects adequately through velocity time series with its peak ground velocity, predominant pulse period and expected number of cycles. Strong motion recordings justify that near-fault pulse is a narrow band pulse and the near fault ground motions containing forward rupture directivity and fling

step effects may be simple enough to be represented by time domain narrow band pulses due to its nature and time-domain velocity time series are better.

The first systematic work on the response of an elasto-plastic single-degree-of freedom system subjected to pulse like ground motion was employed by Veletsos and Newmark (1964). The response spectrum that would result from a half-cycle displacement pulse in their study is shown in Figure 3.11.

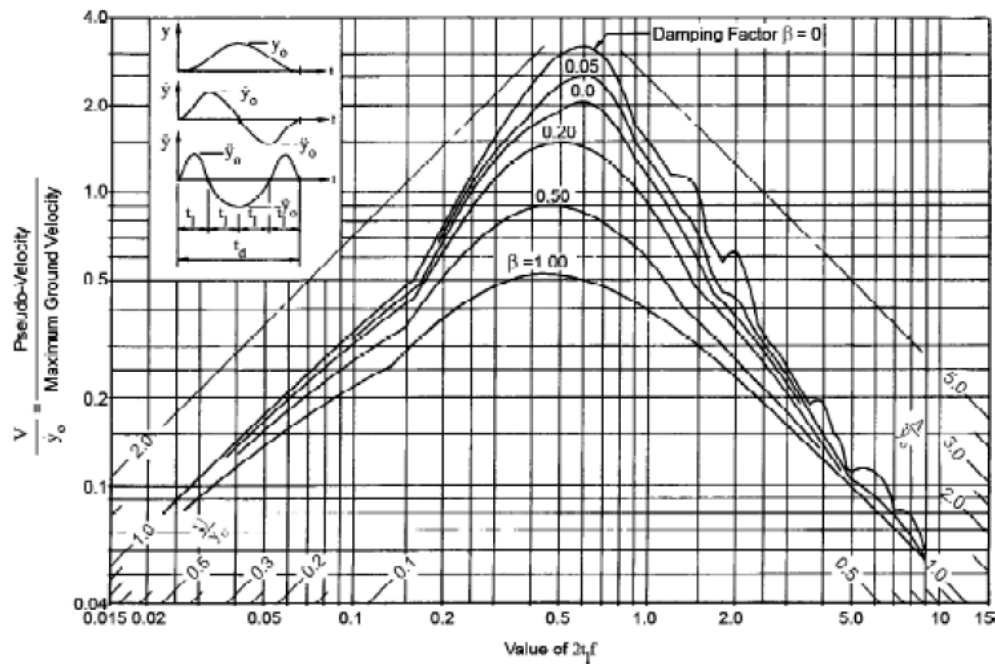


Figure 3.11. Response spectra for a half cycle displacement pulse (Veletsos and Newmark, 1964).

Similarly normalized displacement spectra is provided by derived analytical expressions for fling-step pulse actions (Faccioli *et al.*, 2004). Displacement spectrum can be approximated by simple displacement pulses with $2t_o$ period and peak value of d_{max} . The basic features of the displacement spectrum at long periods are realistic due to shape of the imposed displacement pulse rather than considering acceleration time history. Derived expressions by Faccioli *et al.* (2004) and corresponding displacement spectrum is shown in Figure 3.12.

Somerville has obtained the magnitude scaling relationships for the fault-normal forward directivity pulse period and the peak velocity (PGV) in terms of moment magnitude

Mw and closest distance (Sommerville1997, 2003). Empirical relationships cannot be used for predicting the number of significant cycles due to the rupture mechanism that cannot be known a priori.

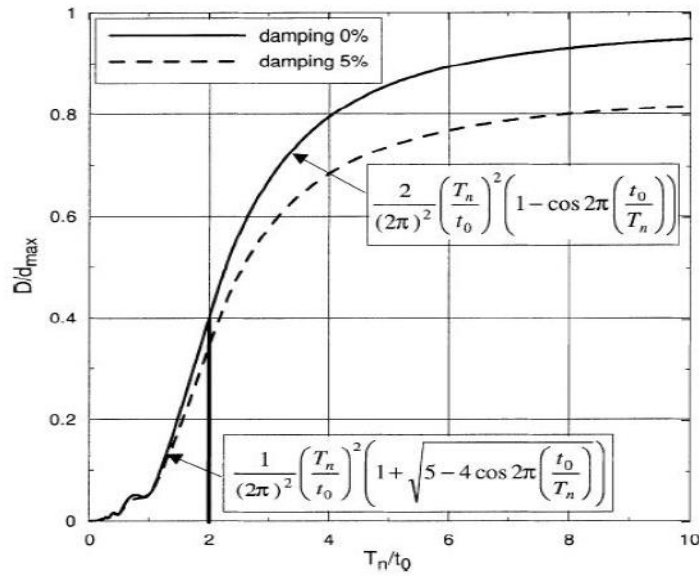


Figure 3.12. Displacement spectra for long period pulse (Faccioli *et al.*, 2004).

Derived empirical relationships from strong-ground motion recordings are expressed by Equation 3.3 and Equation 3.4. Variation of pulse period versus Mw with respect to site conditions and form of time-series with spectral acceleration are shown respectively in Figure 3.13 and Figure 3.14.

$$\log T = -3.1 + 0.5M_w \quad (3.3)$$

$$\log P_{GV} = -1.0 + 0.5M_w - 0.5 \log R \quad (3.4)$$

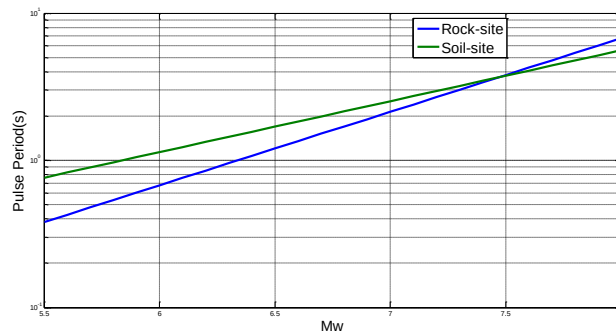


Figure 3.13. Mw scaling of the period of the forward rupture directivity velocity pulse for rock and soil sites (Sommerville, 2003).

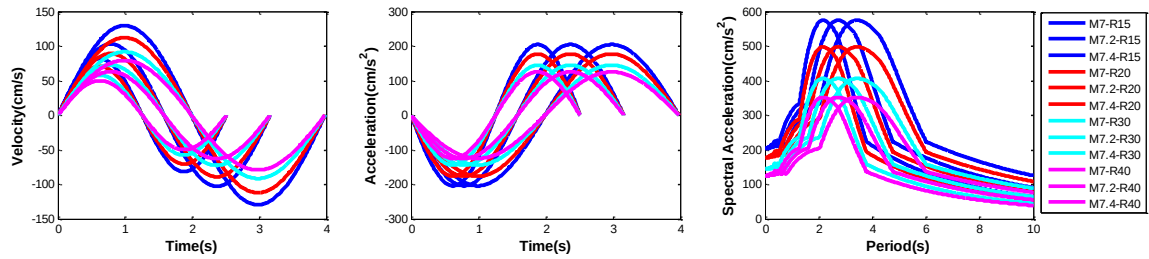


Figure 3.14. The form of velocity-acceleration time-series and corresponding acceleration spectra for FRD.

Bray and Rodriguez-Marek (2004), Rodriguez-Marek and Bray (2006), and Bray *et al.* (2009) simplify the characterization of near-fault forward directivity motions by using only the peak ground velocity approximate period of the dominant pulse, and the number of cycles of pulse motions of the fault-normal component of FRD motions. Effects of local site conditions are taken into consideration by velocity-based ground motion parameters.

Assembly of pulse like ground motions or near-fault ground motion gathered by many researchers to develop a quantitative identification method for ground motions containing strong velocity pulses (Figure 3.15). In this connection Baker (2007), Tothong *et al.* (2007), and Shahi and Baker (2011) have employed a wavelet transform procedure to identify pulse-type signals in a ground motion time history so it can be extracted to classify pulse-type motions in a quantitative manner. Signal processing techniques and source characteristics, site-to source distance and geometry are incorporated to obtain pulse function. This procedure can be used in probabilistic seismic hazard analysis (Shahi and Baker, 2011).

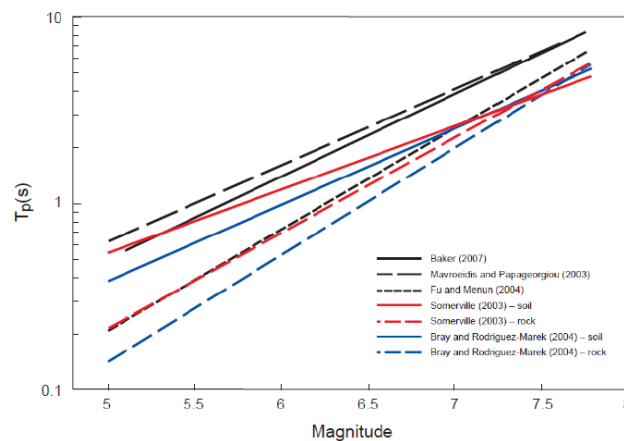


Figure 3.15. Correlations of pulse period with earthquake magnitude.

4. SELECTION AND SCALING OF EARTHQUAKE RECORDS FOR NONLINEAR RESPONSE HISTORY ANALYSIS

4.1. Introduction

The goal of earthquake resistant design is to produce a structure that can withstand a specified level of ground motion with controlled damage levels. Over the last two decades research studies and the advent of computer technology have facilitated the improvement of performance-based design philosophy. Basically, it is focused on meeting the considered performance limit which involves the comparison of demand and capacity related deformation (drift and rotations), where the inelastic deformations are obtained through nonlinear response history analyses of structures for the considered earthquake level. The response history analysis method is the most accurate method of analysis in terms of nonlinear representations of structural elements and earthquake isolation devices. Thus, nonlinear dynamic analyses are strongly required in the design or performance assessment of buildings with high degree of ductility, complex structures with high irregularity in plan or elevation, structures where the higher modes are probable to be excited, liquid storage tanks, seismically isolated bridges, suspension bridges, long-span irregular bridges, tall buildings and critical facilities which could lead to unacceptable direct or indirect losses.

There is a special need to better understand the selection and modification of seed ground motions due to the prevalence of nonlinear response history analysis (RHA) in practice. Proper selection and scaling of ground motion records are two challenging tasks to have an insight on the seismic behavior of critical structures. However, there is no consensus in the engineering community on how to achieve these two tasks. Several studies have shown that collapse capacities can be overestimated/underestimated by significant factors if inappropriate ground motions are utilized for structural analysis (Chandramohan *et al.*, 2013).

Performance assessment and design procedures of critical structures require carrying out source characterization, site characterization, seismic hazard assessment, selecting and

scaling of scenario based earthquake records, performing nonlinear response history analysis and the post processing of results. However, lack of comprehensive guidelines for the selection and scaling of ground motions makes it difficult for practicing engineers considering that the structural analysis results strongly depend on the judgment of engineers. Mostly, this issue has been overcome by expert prepared suite of ground motions that reflects the seismicity of the interested regions and site conditions (*e.g.* SAC Joint Venture Steel Project, 1997, FEMA-P695, 2009). It is widely seen that suites of ground motions of particular project were also used in many different studies without considering the primary objective of the original study or the capability of representation of the seismicity or site conditions of the structure considered. In this respect, special attention needs to be given by the structural engineers. The basic scheme of the selection and scaling procedures in earthquake engineering analysis is given in Figure 4.1.

The number of earthquake ground motions that are required to be used in for response history analysis in design codes relies on the engineering judgment rather than scientific basis (Reyes and Kalkan, 2012). Multiple nonlinear response history analyses (RHA) are often utilized to understand the real behavior of structures under expected seismic excitations. However, performing nonlinear analysis with large number of input motions is computationally expensive. The incorporation of proper selection and scaling methods essentially aims to limit the number of input motions without losing the accuracy of analysis. Accuracy and precision of the proposed methods are the two main concerns of engineers in the prediction of nonlinear response of the structure. Here, the accuracy refer to finding similar median values with the limited number of earthquake records in RHA rather than using a large suite of ground motion sets.

In the scope of this dissertation, the long period ground motion characterization will be associated with the near-field earthquakes and special emphasis will be given to ground motions that contain forward directivity, pulse content and large vertical ground motions within 0-15 km away from fault rupture. This chapter primarily overviews the development of contemporary ground motion databases, basics of ground motion selection and modification (GMSM) methods for real recordings, that would be needed for unidirectional and bi-directional analysis of seismically isolated bridges. Additionally, incorporation of

different definitions of spectral components will be provided to address the selection and scaling procedures under unidirectional or bidirectional earthquake loading.

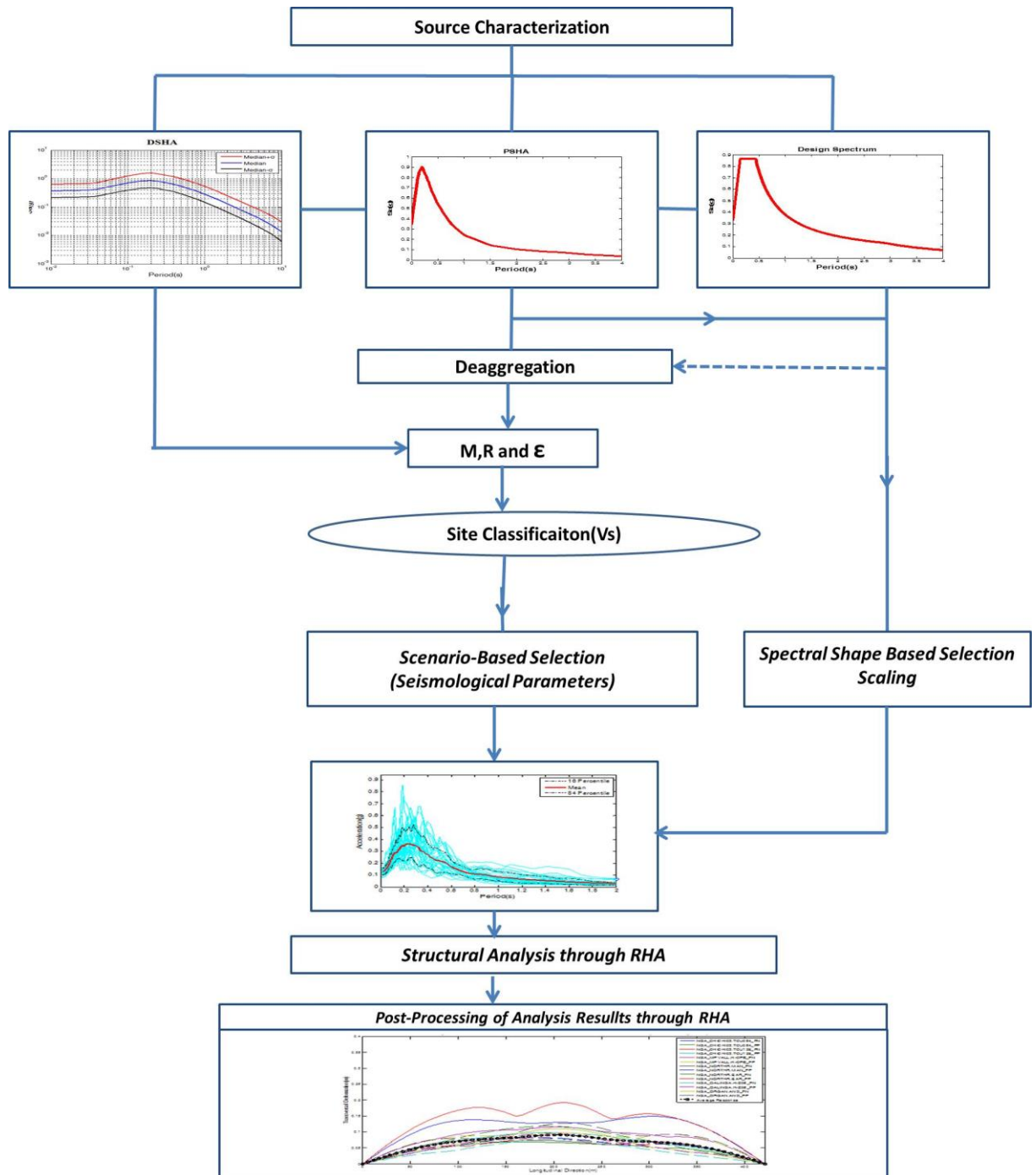


Figure 4.1. Flow chart of applied procedure in selection and scaling of accelerograms for the selected bridges (Yenidogan and Erdik, 2013).

It is worth emphasizing that GSM methods referred here only consists of linear amplitude scaling. Methods employed for the adjustment of spectrum compatible ground motions in frequency domain or simulation of ground motions based on numerous seismological source parameters are not included in this study. PEER ground motion database (PGMD) is used for selecting input motions to conduct nonlinear time history analysis of seismically isolated bridges. Details of the web-based tool will be briefly explained in the next sub-section.

4.2. Overview of the Historical Evolution of Ground Motion Databases

Ground motion databases are essential tools for developing ground motion prediction equations (GMPEs), earthquake hazard assessment and performing nonlinear time history analysis. Most of the ground motion databases contain metadata such as magnitude, various site to source distance measures, style of faulting, local site conditions at the recording station and the relevant strong ground motion parameters that can be used as an engineering quantifications. Ease of access to the ground motion databases and increasing number of earthquake recordings led to the development of vulnerability analysis, new design and assessment approaches in performance based design.

Data dissemination through internet websites was a landmark study which made it possible to search and download digital form of earthquake recordings (Wald, 1997). Generally the most advanced internet websites were launched in Japan, Europe and US according to high seismicity and vast number of recorded motions. Japanese strong ground motion data are provided by K-Net (<http://www.k-net.bosaz.go.jp/k-net/zndex-en.shtml>). Similar efforts were performed in Europe through European Strong motion database (ISESD, <http://www.isesd.hi.is/>) which was launched in 2002 (Ambraseys *et al.*, 2003). The first efforts of compiling strong motion data that are recorded by Turkish national strong motion data compiled and processed systematically by considering the geophysical measurements and site conditions at stations (Akkar *et al.*, 2010). A total number of 4607 strong motions are included in the database.

In addition to stated European and Japanese databases, two important websites named as The Consortium of Organizations for Strong-Motion Observation Systems (COSMOS, <http://strongmotioncenter.org/vdc/scripts/default.plx>) Virtual Data Center and PEER ground motion database (PGMD) were launched in US. COSMOS contains records from US and 14 contributing countries and it is funded by the California Geological Survey (CGS), the U.S. Geological Survey (USGS) and the National Science Foundation (NSF). One of the distinct properties of COSMOS database is the availability of raw data where the utilization of different signal processing procedures can be carried out by users to eliminate errors of raw data and reduce long period noises while preserving characteristics of recordings. In contrast, ISESD and PEER ground motion databases provide processed earthquake records which might not be sufficient to characterize long period ground motions regarding to lose of important data.

PEER ground motion database can be considered as the most sophisticated amongst the listed databases regarding to implemented tools for selecting and scaling of ground motions and being the principal resource for the development of the updated attenuation relationships in the NGA research project coordinated by PEER-Lifelines Program (Chiou *et al.*, 2006, 2008; Power *et al.*, 2008). The PEER NGA database includes 3551 three-component recordings from 173 earthquakes and 1456 recording stations.

Response spectra, peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), significant duration, assessments of the lowest usable frequency (longest usable period) for response spectra, and presence and periods of strong velocity pulses are quantified in the PEER ground motion database. The recommended lowest usable frequency is an important parameter particularly in the long period ground motion characterization for removing the low frequency noise. Influence of long-period filter cut-off on elastic spectral displacements are investigated by Akkar and Bommer (2006). Results indicated that the maximum period at which spectral displacements can be calculated depend strongly on the site class, magnitude and filter order and the reliable long period range for digital accelerograms where it can be approximately twice of analog accelerograms. Thus, preservation of long period characteristics of ground motions must be provided for seismically isolated structures, tall buildings, liquid storage tanks and long-span bridges.

Theoretical background of the latest web-based tool of PEER ground motion database relies on the recently proposed methods where the target spectrum is represented by conditional mean spectrum (Baker, 2011) and the efficient ground motion selection algorithm is employed for matching a target response spectrum mean and variance (Jayaram *et al.*, 2011). Although ground motion components are determined by the orientation of the recording instrument an exceptional work has been conducted for NGA database to obtain rotated fault normal and fault parallel components (PEER, 2010).

4.3. Objectives of GSM

Different ground motion selection procedures are proposed in the literature. It is essential to compile and group the proposed methods with respect to their objectives for a consistent comparative analysis. Similar efforts were performed in the PEER lifeline project for buildings (Haselton *et al.*, 2009, Goulet *et al.*, 2008). According to Haselton *et al.* (2009) selection and scaling methods can be grouped on the following two main categories.

- First general category is based on scaling to a Uniform Hazard Spectrum (UHS) with the fixed target spectral acceleration value that addresses the fundamental mode of the considered structure, $S_a(T_1)$. This approach is applied in the former ASCE standard (ASCE/SEI 7-05, 2005).
- The second main category reflect the record characteristics which might affect the nonlinear response of the structure. Selected ground motions do not only considers the spectral shape but also account the damage potential of inherent characteristics of earthquake recordings.

In this aspect, some GSM methods aim to estimate median response as well as the variability in structural response. Seismic hazard assessment is a crucial step that needs to be performed in the representation of target spectrum for critical structures. Target spectrum may be established from a probabilistic seismic hazard assessment (PSHA) as an end result of deaggregation of uniform hazard spectrum (UHS) for a certain annual probability of

exceedance or can be obtained from deterministic seismic hazard assessment (DSHA). Individual seismic scenarios are developed for each relevant seismic source and a specified ground motion probability level is selected for DSHA analysis. In contrast, all possible and relevant deterministic earthquake scenarios and all possible ground motion probability levels are considered in PSHA. Both approaches have been used over 40 years. Both epistemic and aleatory uncertainties are associated in these two approaches.

4.4. Criteria for Selecting Ground Motions for Structural Analysis

Nonlinear dynamic analysis is becoming widely accepted as the most accurate method of analysis. Seismic provisions in the current design codes and standards contain the analysis of seismically isolated structures more than two decades. According to seismic design codes or guidelines most of the conventionally designed structures will experience inelastic deformations under major earthquakes. Ideal suite of earthquake records needs to have record to record variability and provide a better median prediction (relative to the point of comparison) with a smaller dispersion.

The procedure used in choosing records is dictated by the information available to the engineers regarding to the seismic hazard or by ground motions available for the site of interest. Guidance about the selection of ground motions for nonlinear response history analysis of critical structures is somehow limited in the literature. Generally, the following four options are proposed to the engineers for the selection of acceleration time-series as input motions for dynamic RHA.

1. Artificial accelerograms
2. Synthetic accelerograms
3. Modified real recordings in the frequency domain to obtain spectrum compatible records
4. Linear amplitude scaled earthquake recordings

Artificial spectrum compatible records can be generated by using power spectral density function from smoothed spectrum. Even though the generated ground motions will be completely compatible with the design spectrum, energy content and the number of cycles of

these types of input motions will not represent the expected ground motions. Moreover, analysis results will give higher earthquake demand parameters. Similar approach that uses power spectral density function has been originally implemented in SIMQUAKE by Gasparini and Vanmarcke (1976).

The second option is the use of synthetic ground motions where the source models are represented either by point source or through extended source models. Synthetic accelerograms are generated from the assigned seismological source model and accounts for path and site effects. Main drawback of this alternative is the assignment of vast number of seismological parameters and the required expert opinion in the selection of appropriate parameters. It should be kept in mind that obtained earthquake records will be strongly dependent on the regional seismicity as well as the seismotectonic of the considered site. In such cases the values of source parameters are not directly accessible through measurements from recordings and it influences the quality of earthquake simulations. The 2011 Van Earthquake can provide an example to this particular problem as it is almost impossible to generate highly correlated synthetic ground motion history for the recording stations located on the hanging and foot wall site, due to complexity of the thrust fault mechanism (Di Sarno, *et al.*, 2013).

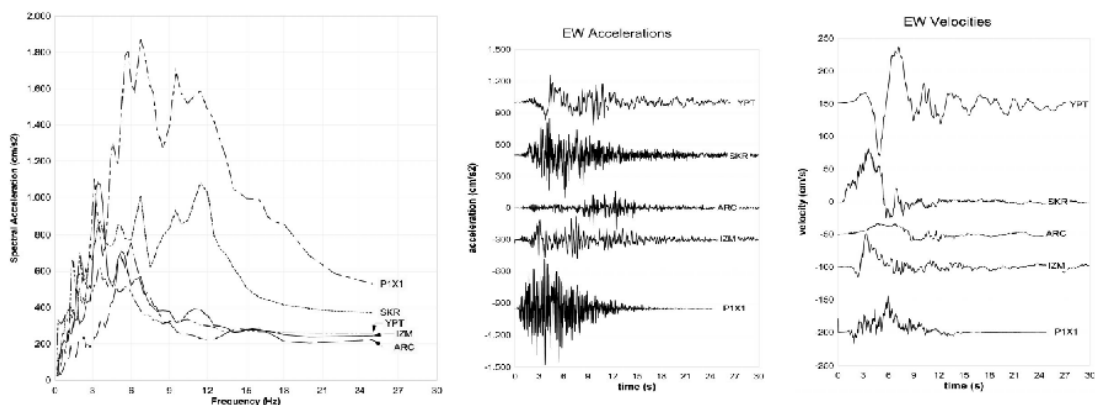


Figure 4.2. Comparison of spectral accelerations of the simulated and as-recorded data (Durukal and Erdik, 2001).

Simulation of near-field earthquakes can be achieved by hybrid simulation procedures by addressing the low and high frequency component of the ground motion separately and then combining these generated two motions. Step by step hybrid simulation procedure for

long period earthquakes is proposed by Durukal and Erdik (2001). Comparison of simulated record with real earthquake records are shown in Figure 4.2. On the basis of complexity of simulation procedures engineers who do not have solid knowledge about engineering seismology and seismotectonic of the interested region should not use this option to generate synthetic earthquake recordings.

The third option consists of ground motion modification methods that take place in frequency domain. Wavelet analysis may be utilized to modify the seed motions. The wavelet adjustment ensures the preservation of the long period non-stationary phase of the original time history. Careful selection procedure of the seed motions is crucial step to minimize the required wavelet adjustment. The simplest way might be the incorporation of root mean square (RMS) difference in normalized record and target spectral as implemented in the web-based European Strong Motion Database (Ambraseys *et al.*, 2004).

There are numbers of software available which performs time domain modification of acceleration time histories that are compatible with the user specified spectrum such as RSPMATCH 2005 (Hancock *et al.*, 2006). The methodology of RSPMATCH 2005 is based on that proposed procedure of Lilhanand and Tseng (1987, 1988). The adjusted time series always need to be checked for avoiding the excessive adjustment in terms of energy. Modification of ground motions aims to match the spectral shape to target spectrum rather than considering seismological parameters.

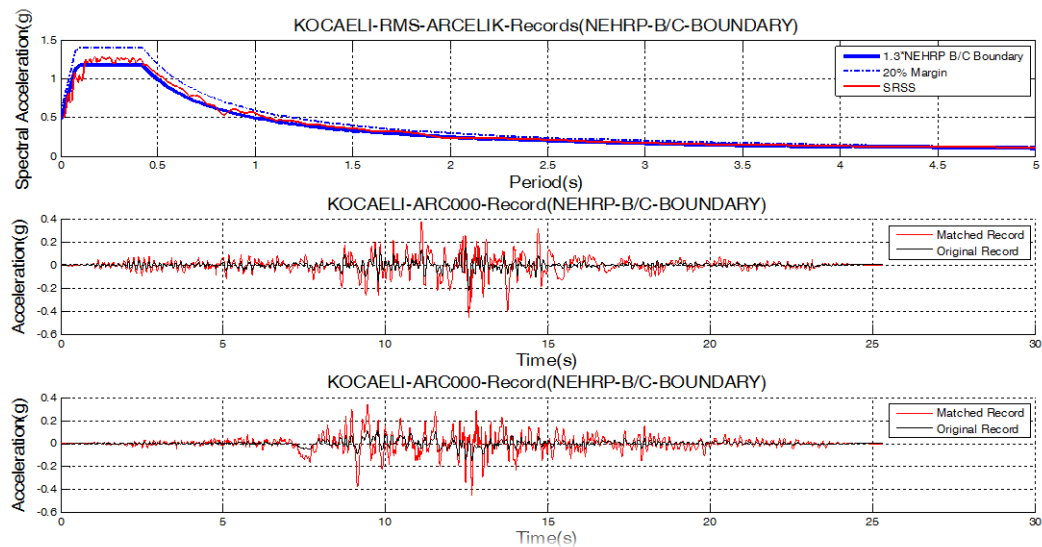


Figure 4.3. Adjusted seed motions by conducting wavelet analysis.

Linear amplitude scaling of real earthquake records is the most commonly used method amongst scaling procedures due to the unaltered frequency content of the records. Only real records and linear scale factors considered for minimum manipulation. Proper suite of ground motions aims to represent the expected level of ground shaking at the considered site. Linear amplitude scaling of geometric mean of the horizontal components is shown in Figure 4.4.

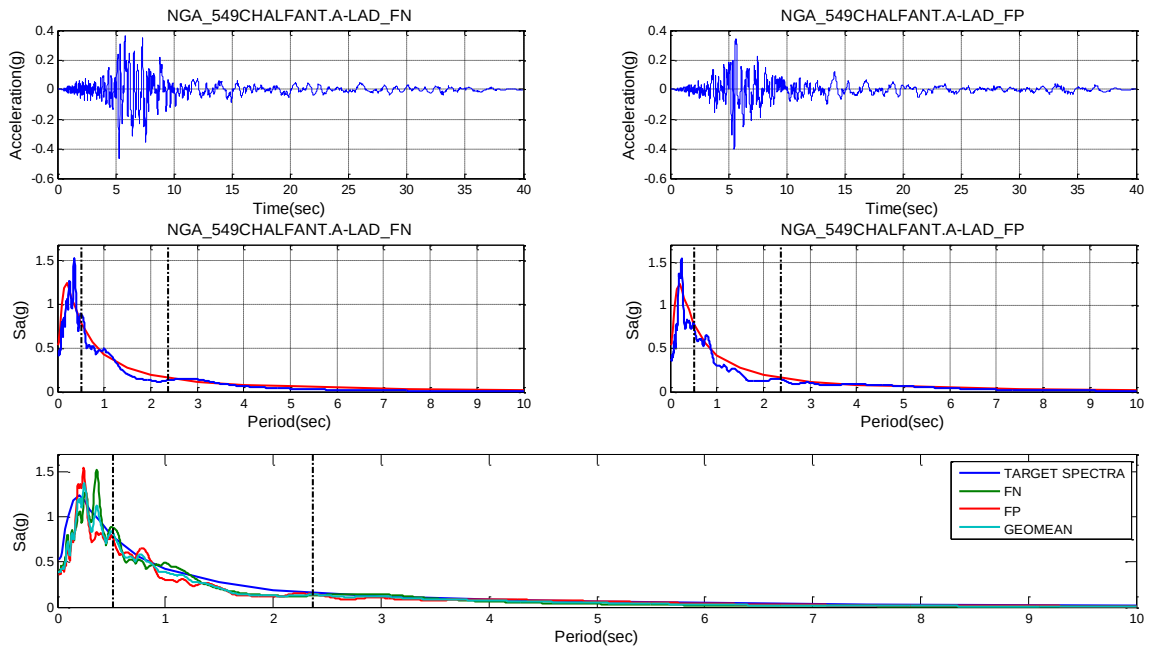


Figure 4.4. Linear amplitude scaling of horizontal components of and geometric mean component.

4.4.1. Selection Strategies for Earthquake Ground Motion

Dynamic response history analysis of structures and also the testing of large scale structures require the proper selection and scaling techniques of earthquake records. Careful ground motion selection can achieve the reduction in bias and variance of structural response while allowing the user to correlate the damage by using simple measures of intensity such as elastic spectral acceleration (Baker and Cornell, 2006). Different ground motion selection and modification (GMSM) methods currently available for their use in bridge structures as well as seismically isolated structures. The key steps herein are to quantify and correlate the damage on the basis of Earthquake Demand Parameters (EDP) from time-history analyses. Most of the time, the displacement capacity and the shear force transmission are the two main concerns in

seismically isolated structures. Therefore, long period ground motion characterization is essential for the design of seismic isolation for preliminary calculations prior to the nonlinear RHA.

When selection of real records for seismic design of structures is concerned, the current state of best practice on the uniform hazard spectrum (UHS), which is an elastic spectrum defined by means of the seismic hazard at the site (Baker and Cornell, 2004, 2005). In this study, results of both deterministic earthquake scenario and probabilistic seismic hazard study are incorporated for the performance evaluation of analytical models of seismically isolated bridges. The strength of ground motions are often quantified by an Intensity Measure (IM) at particular period, wherein IMs is used to predict the structural response. Probabilistic seismic-hazard deaggregation involves determining earthquake variables, principally magnitude, distance and values of other random variables defining seismic events that contribute to a selected seismic-hazard level (McGuire, 1995; Bazzurro and Cornell, 1999). Current practice and tendency for record selection in PSHA stand in need to deaggregate the hazard at the first mode period of the structure (T_1), determine the magnitude (M) and site-to-source distance (r) pair, and select recorded ground motions corresponding to the modal $[M, r]$ pair and local site conditions. Epsilon (ϵ) defines the standard deviation of the observed data from the mean logarithmic spectral acceleration of the GMPEs. Epsilon parameter is implemented in the deaggregation studies for the indication of spectral shape (Baker and Cornell, 2005). It is computed by subtracting the mean predicted value, $\ln Sa(T_1)$ from the record's $\ln Sa(T_1)$, and dividing by the logarithmic standard deviation. An example of hazard curve and deaggregation results for seismically isolated bridges at $Sa(T)$ value is provided in Figure 4.5.

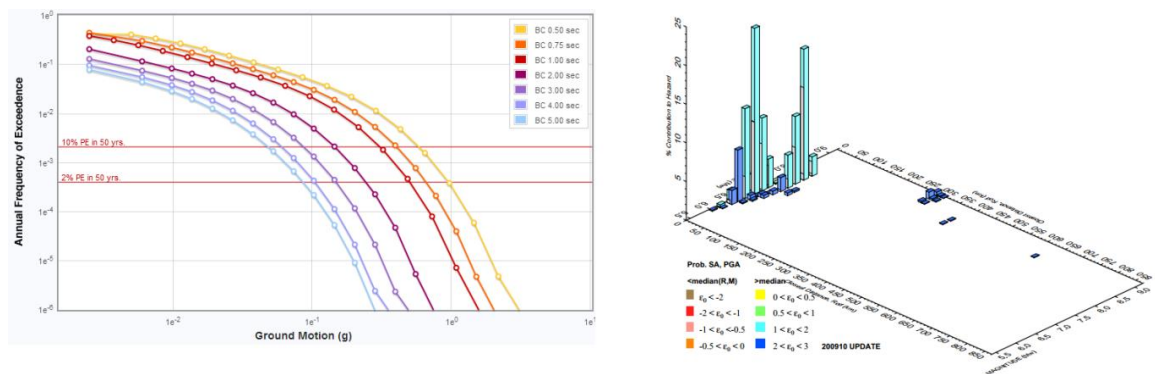


Figure 4.5. Hazard curves and deaggregation results of the selected site for $Sa(3s)$.

In order to achieve the amplitude and/or frequency scaling of ground motions to match a uniform hazard spectrum, two well-known approaches have been used in practice. As discussed, the first one considers the perfect match to a target spectrum by amplitude or frequency scaling of single-component ground motion. The second method utilizes linear amplitude scaling of pairs of ground motions to get the average value of the square root of the sum of the squares (SRSS) of the 5% damped spectral ordinates. Obtained spectral values shall not be less than 10 % of the 1.3 times of the 5% damped target spectrum over the period range of $0.2 T_F$ to $1.5 T_F$ (FEMA 450, ASCE/SEI 7-05, 2006). This suggestion in design codes was originally developed for seismically isolated structures.

Different selecting and scaling strategies are available for matching the seed motions to the target spectrum best after scaling or without scaling. As a preliminary selection methodology minimum deviation of spectral values are aimed in the period range of interest. In this sense, new search engine is developed for the European Strong Motion Database (Ambraseys *et al.*, 2004) which checks the most suitable records that have close spectral shapes to the target spectrum shape. The tool provided by Ambraseys and his colleagues uses Equation (4.1) that accounts for the root mean-square difference. This equation gives the average root-mean-square difference of records that are anchored to PGA value (Beyer and Bommer, 2007).

$$D_{RMS} = \frac{1}{N} \sqrt{\sum_{i=1}^N \left(\frac{Sa_R(T_i)}{PGA_R} - \frac{Sa_T(T_i)}{PGA_T} \right)^2} \quad (4.1)$$

$Sa_R(T_i)$: The spectral acceleration of the record at the specified period value, T_i

$Sa_T(T_i)$: The spectral acceleration of the target spectrum at the specified period value, T_i

PGA_R : The peak ground acceleration of the record

PGA_T : The target peak ground acceleration

N : The number of periods

However, most of the engineering structures are investigated through broader period range of interest and the response spectral shape over a period range of significance to structural response has been found to be closely correlated to inelastic structural response in a

number of studies (*e.g.* Shome *et al.*, 1998; Cordova *et al.*, 2001, Luco and Cornell, 2007, Jalayer, 2003, Baker and Cornell 2004, Luco and Bazurro, 2004, Baker and Cornell 2005, 2006, 2008). Most of the seismic design guidelines recommend the period range of interest to be between $0.2 T_F$ and $1.5 T_F$. T_F represents the fundamental vibration period of the structure and upper bound value of period range of interest can be extended due to derived ductility parameter. Therefore, the proposed method in the tool of European ground motion database was modified to search bi-directional records with respect to SA values with a scale factor, α to get minimum root-mean square difference, as shown in Equation 4.2. Horizontal components of each earthquake ground motion are scaled by the same scale factor, α . However, it is worth noting that suggested period range of interest for isolated structures is between $0.5T_D$ – $1.25T_M$ where T_D corresponds to period for design based earthquake level and T_M corresponds to maximum credible earthquake.

$$D_{rms} = \sqrt{\frac{1}{k-j+1} \sum_{i=j}^k (\alpha S a_R(T_i) - S a_T(T_i))^2} \quad (4.2)$$

Minimum root mean square in Equation 4.2 built on the exact amplitude values at specified period range. In order to eliminate the influence of variation by the amplitude parameter normalization of error, the target value at each period must be obtained as shown in Equation 4.3. This equation is also utilized, for selecting the best suitable sets of ground motion to prepare pool of ground motions, in the computer software developed by Iervolino *et al.*, (2009). It considers the European ground motions from three different databases.

$$D_{rms} = \sqrt{\frac{1}{k-j+1} \sum_{i=j}^k \left(\frac{\alpha S a_R(T_i) - S a_T(T_i)}{S a_T(T_i)} \right)^2} \quad (4.3)$$

4.5. Web-Based Selection and Scaling Tool of PGMD

Selection and scaling of suite of ground motions are performed through utilization of web-based tool of PEER ground motion database (PEER, 2010). The PEER Ground Motion Database (PGMD) – Beta Version is limited to recorded time series from shallow crustal earthquakes. Compilation of shallow crustal earthquakes from all over the world makes the utilization of web-based tool possible for other countries with similar seismotectonic conditions like Turkey. The procedure of the utilized web-based tool can be summarized in the following three basic steps as follows

1. Developing the target spectrum
2. Specifying criteria and limits for the spectral shape based selection
3. Selection of earthquake records from the pool of ground motion by the specified criteria and limits

4.5.1. Developing the Target Spectrum for Web-Based PGMD Tool

Three options are offered to user to construct the target spectrum for the earthquake record selection and scaling as follows

- 1) Code-based design spectrum
- 2) User-defined Spectrum
- 3) Target spectrum based on the next generation Ground Motion Prediction Equations (GMPEs)

First choice is based on the code-based target spectrum where the user needs to assign the requested site-class-adjusted 0.2 second spectral acceleration, $S_{0.2}$; site-class-adjusted 1.0-second spectral acceleration, S_1 ; and the transition period T_L as defined in NEHRP Provisions (Figure 4.6), ASCE Standard ASCE/SEI 7-05 (ASCE, 2006) and the International Building Code, (ICC, 2006). Site class adjustment is performed with respect to the average shear wave velocity of the upper 30 m (V_{s30}) to classify the site classes regarding to NEHRP provision (2003). Generic soil amplification factors are provided in US and European Codes for V_{s30} parameter (Eurocode-8, 2004, ASCE Standard ASCE/SEI 7-05, ASCE/SEI 7-10).

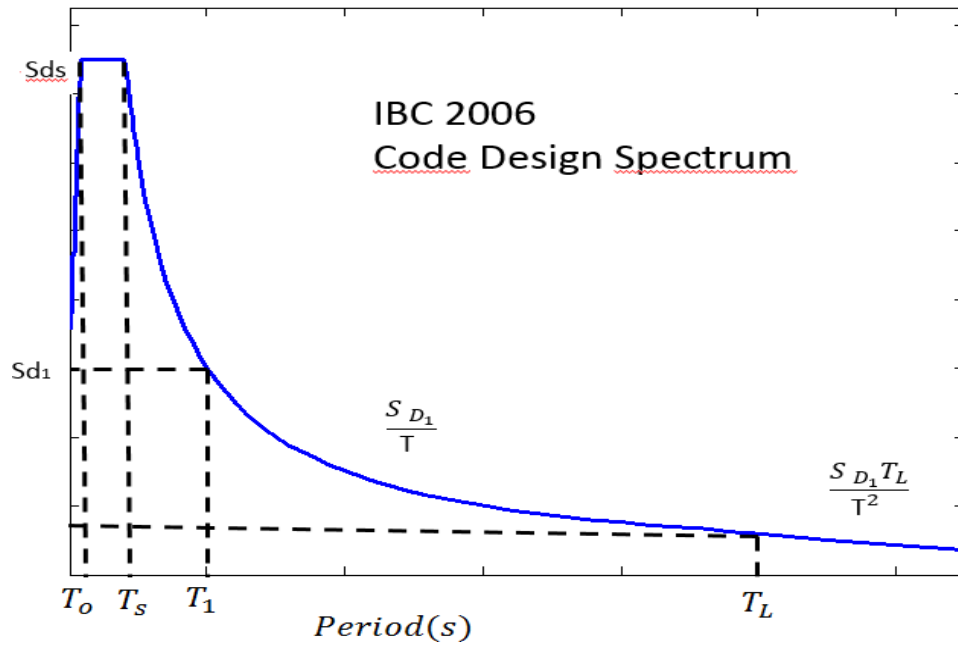


Figure 4.6. Code-based target spectrum (ICC, 2006).

Second option has the flexibility of assigning previously calculated target spectrum. Key issue of this option is the format of the data with periods and response spectral accelerations values. Target spectrum might be either obtained from probabilistic studies (UHS) or from deterministic scenario earthquake. This option is incorporated to assign deterministic scenario earthquake for 5 different source-to-site distance (Figure 4.7) and UHS spectra of earthquake with 2475 return period deaggregated from PSHA study in Italy for the assessment of seismically isolated bridge (Yenidogan et al., 2013).

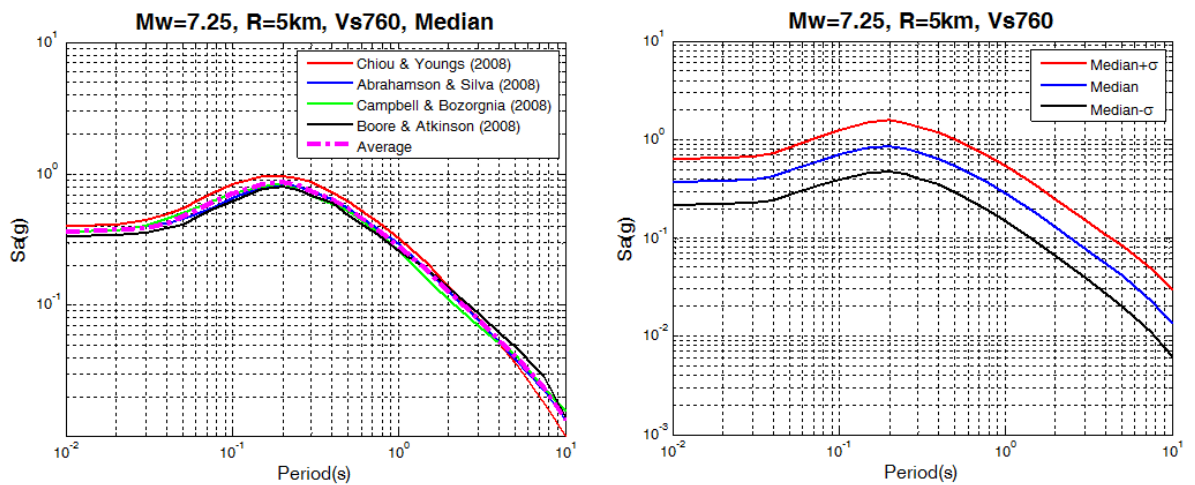


Figure 4.7. Deterministic seismic hazard assessment for scenario earthquake for SI Bridges

The last option is a special case of user-defined spectrum. Target spectrum is constructed through incorporation of set of ground motion prediction equations that are developed for shallow earthquakes in the NGA project (Abrahamson and Silva 2008, Boore and Atkinson 2008; Campbell and Bozorgnia 2008, Chiou and Youngs, 2008, and Idriss 2008).

One of the key parameter to construct the target spectrum by using this option is the epsilon parameter which represents the number of standard deviation (Figure 4.8). It is an important parameter that is used to establish earthquake levels for performance based design while reflecting the record to record variability. Construction of the target spectrum by developing the conditional mean spectrum is a special case of UHS as denoted in Figure 4.9.

Primary objective of conditional mean spectrum is to select and modify earthquake records such that each of them meets the target averaged value at $S_a(T_1)$ value as well as providing a suite of ground motions that reflects the variability of target spectrum. Baker and Cornell (2005) propose that the residual ε of the ground motion should be considered when selecting the records and the method used for the correlation model is proposed by Baker and Jayaram (2008).

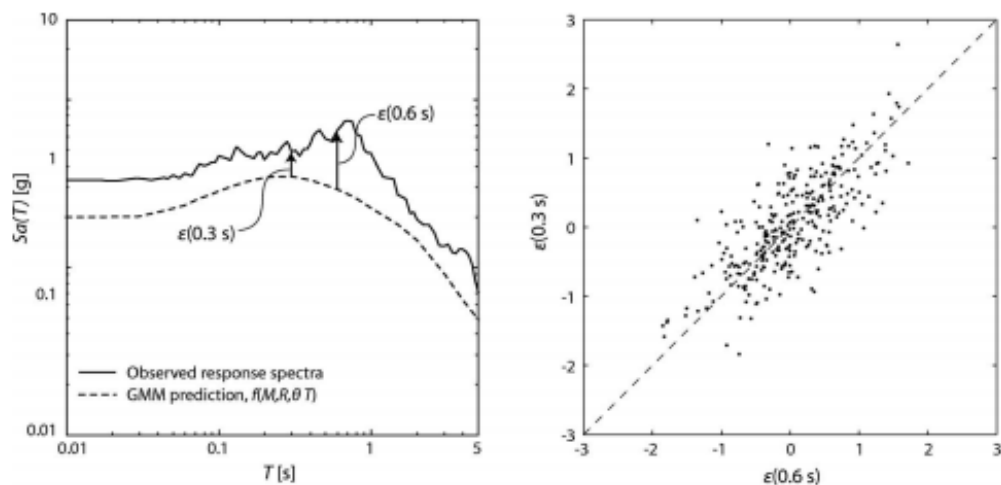


Figure 4.8. Calculation of epsilon (ε) value as shape indicator with the help of GMPEs and earthquake records (Baker and Jayaram, 2008).

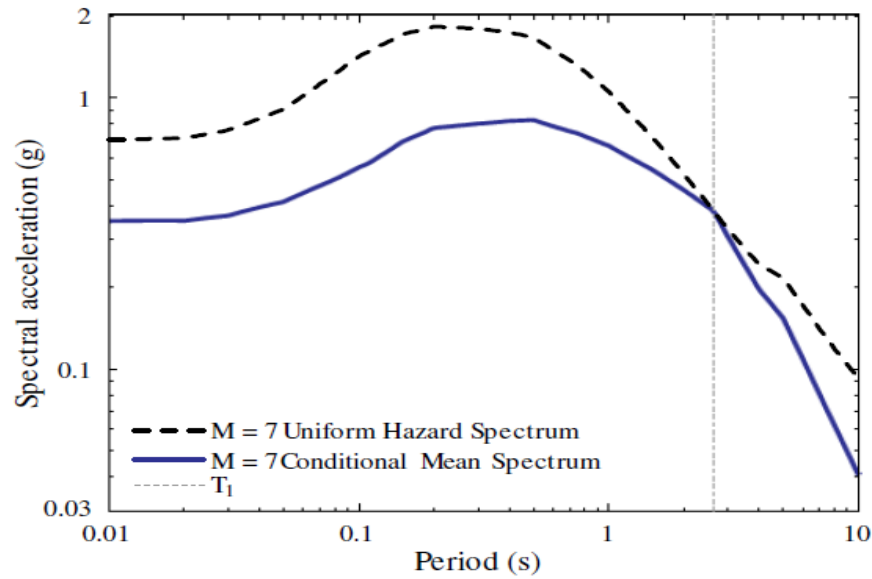


Figure 4.9. Conditional mean and uniform hazard spectra for M7 and $\epsilon=2$ scenario for building type of structure (Goulet *et al.*, 2008).

4.5.2. Specifying Criteria and Limits of Search Options for Selection and Scaling of Ground Motion Records

The PGMD web-based tool offers two options for the selection of earthquake records. The first option serves to seek particular records from NGA numbers. The second option searches the suitable set of records in terms of general acceptance criteria where the fault-mechanism, inclusion of records with pulse content, limits of magnitude range, source to site distance and V_{s30} parameter are defined. Pulse-like nature of a ground motions in PEER NGA database was identified using the method developed by Baker (2007). General criteria in defining the pulse content of records can be summarized as follows

- The pulse is relative larger than the residual features of the ground motion
- Early arrival of pulse that is associated with rupture directivity effects
- Large absolute velocity amplitudes that are higher than 30 cm/sec

Ideal suite of records needs to represent the seismicity of the interested site and estimate the median response of the structure. Basic criterion of the provided tool in PGMD is to select ground motions that have spectral shape close to the target spectrum over the specified period range of interest. Measurement of good fit to the target spectrum is defined by minimum

squared error (MSE) between computed logarithm of spectral acceleration values and target spectrum (PEER, 2010).

Three different scaling options are given to users in terms of linear amplitude scaling with minimum manipulation to improve their match to the target spectrum. The provided tool only considers linear amplitude scaling for the best spectral match because of the processed records. In other words, the first option considers the scaling of selected records in a way that the spectral acceleration at a specific period matches to the target spectral acceleration at that period. Scale factor is calculated by using Equation 4.4. According to Shome *et al.* (1998) and Iervolino and Cornell (2005) magnitude and distance do not influence the median response if the records are scaled to match the design spectrum at the fundamental period. Thus this option can be assessed to check the first mode dominated structures.

$$f = \frac{SA^{target}(T_s)}{SA^{record}(T_s)} \quad (4.4)$$

The second option relies on applying a scale factor that minimizes MSE value over the specified period range of interest. As a consequence of this approach, average of selected records will have similar spectral shape to the target over the period range of interest. (PEER, 2010). The weighting factor is utilized for the calculation of MSE which allows applying scale factors in narrow period range of interest while maintaining a reasonable match over a broad period range. Implementation of scale factor for the calculation of MSE is shown in Equation 4.5 and 4.6. The period range of significance may include periods shorter than the fundamental structure period because of higher-mode effects and periods longer than the fundamental structure period because of structure softening during inelastic actions.

$$MSE = \frac{\sum_i w(T_i) \{ \ln[SA^{target}(T_i)] - \ln[f * SA^{record}(T_i)] \}^2}{\sum_i w(T_i)} \quad (4.5)$$

$$\ln f = \frac{\sum_i w(T_i) \ln \left(\frac{SA^{target}(T_i)}{SA^{record}(T_i)} \right)}{\sum_i w(T_i)} \quad (4.6)$$

The last option is the use of processed records in PGMD without applying any scale factors. In other words, MSE ranking is performed for scale factor of one.

4.6. Different Definitions of Spectral Ordinates for Selection and Scaling of Real Records

Ground motion at a point consists of three translational and three rotational components. There are few studies which consider the rotational component of ground motions (Basu *et al.*, 2012, Kalkan and Grazier, 2007). However, intensity and frequency content of rotational components are not measured while their inclusion may increase the translational and torsional response. Current state of art in structural design only considers the translational components for analysis.

Analysis of critical structures requires consideration of multi-component excitations to see the interaction of structural system in translational directions. Classification of critical bridges is determined on the basis of importance and complexity as well as their economic influences after a moderate or severe earthquake. Although bridges can be classified as simple structures, the low redundancy of global system may lead to catastrophic consequences (Priestly *et al.*, 1996). In this aspect, dynamic analysis of seismically isolated bridges is essential for grasping the coupling effect of isolation units.

Different ground motion selection and modification (GMSM) methods are evaluated to provide some guideline for structures. PEER GSM Working Group has employed such effort for buildings that are exposed to unidirectional excitations (Haselton *et al.*, 2009). However, bridges have two principle directions named as longitudinal and transverse axes in which evaluation of performance assessment and /or design of bridges need to be performed under multi-directional excitations. Weak and strong axes of bridges can be derived by using nonlinear static procedures whereas for seismically isolated structures it won't be feasible to assess it due to installation of isolation layer which decouples the superstructure and substructure. In fact most of the response analyses of seismically isolated bridges were carried out under unidirectional excitation.

Due to aforementioned reasons bridges need to be considered under multi-directional excitations for the satisfactory prediction of results. These concerns lead to the selection and scaling of horizontal pairs by use of new definitions for the representation of the seismic demand. Different definitions of spectral ordinates are used to express horizontal pairs by a single value. The most common definitions of horizontal components can be listed as the geometric mean (GM) spectrum, arithmetic mean, envelope and square root of the sum of squares (SRSS) spectrum. Amongst these definitions, the geometric mean of as-recorded horizontal components in x and y direction is the most commonly used quantity for the orthogonal components and it can be calculated by using Equation 4.7. Large set of GMPEs are based on GM parameter due to small variations of results in terms of ground motion axes and the direct comparison of spectra of target and ground motion intensity measure. This parameter was also used in studies of Malhotra (2003) and Baker and Cornell (2006) to identify concerns about bi-directional input motions.

$$Sa_{GM_{xy}}(T) = \sqrt{Sa_x(T) \cdot Sa_y(T)} \quad (4.7)$$

Likewise in GM, SRSS spectrum is incorporated for presenting the coupled effect of bidirectional excitations as a design provision (ASCE/SEI 7-10). SRSS spectrum of horizontal pairs can be calculated by using Equation 4.8.

$$Sa_{SRSS_{xy}}(T) = \sqrt{Sa_x^2(T) + Sa_y^2(T)} \quad (4.8)$$

The spectral ordinate values depend on the orientation of the ground-motion axes. The PGMD web-based selection and scaling tool utilizes the geometric mean value of fault-normal (FN) and fault-parallel (FP) oriented horizontal components where the single component earthquake records were excluded. Regarding to random orientation of ground motion axes GMRotD50 and GMRotI50 parameters are proposed by Boore *et al.* (2006). These two terms were aimed in reducing the variance of ground motion measure. The former parameter calculates the median value of geometric mean from possible orientations of ground motion at each period. The latter parameter is an approximation of GMRotD50 with constant axis orientation for all periods. It is worth mentioning that regarding to the orientation of ground

motion axes a new term so-called maximum direction spectral acceleration is defined by means of the response spectrum along the strongest axis of shaking.

4.7. Review of Code Provisions for Selecting and Scaling of Real Records

Apart from site specific studies of critical structures, seismic design codes and standards serve as main guidance in response history analysis and on the selection and scaling of ground motions. Additionally, selected response histories that are compatible with specified criteria are used to define and adjust the response modification factor, R and the displacement amplification factor C_d for specific building typologies in ASCE 7/SEI 7-10 (ASCE, 2010).

Seismic design codes from 1980s and 1990s explicitly considered only one limit state and one level of earthquake loading which may be associated with the life safety (UBC-97). New Zealand and Japanese codes can be considered as exceptional in terms of employing two different levels of earthquake actions in that era (Uang, 1993). General review of ground motion specification in codes for obtaining a proper suite of ground motions were investigated by many researchers (*e.g.* Sextos *et al.*, 2011, Iervolino *et al.*, 2009, Bommer and Ruggeri, 2002).

Results of nonlinear RHA provide insight to engineers about the performance of structures. Therefore, determination of goals in the response history analysis is crucial before choosing procedures to select and scale ground motions. For instance, when the mean drift responses or isolator displacement capacity is the main concern for a given maximum credible earthquake, incorporation of ground motions that have similar spectral shape to a target spectrum may be a proper approach due to reduction of variability in spectral acceleration of individual records among the suite of ground motions. In contrast, when the probability of shear failure is considered for a force controlled shear-wall, both prediction of mean value of demand and inherent record to record variability become two important parameters where the spectrum matching may not fit to the goals due to suppression of record to record variability. In case of performance assessment, the most suitable method for selecting and scaling ground motions will significantly depend on the type of assessment being performed.

Three different approaches for performance assessment can be categorized as intensity-based assessments, scenario-based assessments and risk-based assessments. The last option is the most comprehensive assessment tool as a result of its broad content.

Due to aforementioned reasons, a number of codes and standards are prepared to serve different purposes. Building and bridge codes define how new structures are designed or how the performance assessed and retrofitting activities were conducted through consideration of the target minimum seismic performance. The most commonly used codes and guidelines for structures can be listed as follows: Design of buildings and bridges in Eurocode 8 Part 1-2 (CEN, 2005), NEHRP recommended provisions for seismic regulations for new buildings and other structures, FEMA 368 (FEMA, 2001), Minimum design loads for buildings and other structures, ASCE/SEI Standards 7-05/10 (ASCE, 2006, ASCE, 2010), Seismic analysis of safety-related nuclear structures, ASCE 4-98 (ASCE, 2000), New Zealand Standard 1170.5 (NZS, 2004), Italian Seismic Code NTC-08 (2008), AASHTO LRFD Bridge Design Specification (2010) and DLH-Design Specification for Railway Bridges (2007). All of the specified codes and design guidelines have section concerned with the ground motion selection and scaling criteria. However, most codes do not distinguish between record selection for unidirectional analysis and multi-directional analysis (Beyer and Bommer, 2007). Because of this reason, special attention needs to be given for selecting and scaling of earthquake records for multi-directional input motion as well as direction of loading and post-processing of results.

Eurocode-8 Part 2 (CEN, 2005) and AASHTO LRFD Bridge Design Specification (2010) include comments about the use of dynamic response history analysis for either elastic or inelastic analysis. Performance expectations of bridges vary with the importance classification of the bridge. Significant damage might be acceptable for rare earthquake shaking for ordinary bridges. Classification of bridges is also given by the California Department of Transportation in terms of geometric configuration, number of spans, structural elements and susceptibility of site-specific risk.

Although ASCE 7-05/SEI and ASCE 2010/SEI were mainly prepared for buildings and other structures. Rules for the calculation of component demands in seismically isolated bridges are essentially identical to ASCE 7/SEI 7-05 and ASCE 7/SEI 7-10. Therefore brief summary of ASCE 7/SEI 7-05 and ASCE/SEI 7-10 are also given in here. According to the

International Building Code (ICBO, 2009) which refers to ASCE/SEI-7 Section 16.2 (ASCE, 2005, 2010) earthquake records of response history analysis should be selected from events of magnitudes, fault, distance and source mechanisms that are consistent with MCE level of shaking. Distinction of two dimensional and three dimensional analyses were given in Section 16.1.3.1 and Section 16.1.3.2 in ASCE 7/SEI 7-05 (2005), respectively. SRSS spectra of appropriate horizontal pairs are computed for 5% damped spectra which must be in compliant with the defined seismic hazard as well as meeting the minimum acceptance criteria. Real earthquake records are preferred for RHA where simulated records are only allowed when the required number of ground motions is not available for the interested region. Period range of interest for ground motion selection and scaling is expressed between $0.2T_F$ - $1.5T_F$.

ASCE/SEI 7-05 and ASCE/SEI 7-10 differ by the definition of target spectrum. In ASCE/SEI 7-05 (2005) target spectrum is expressed by geometric mean while it is defined by means of maximum rotated components (Huang et al., 2008). Adjusted short period value can be taken as 1.1 and 1.3 at S_1 spectral value. In addition, each pair of ground motions at sites within 5 km to the controlling active fault shall be rotated to fault-normal and fault parallel directions of the dominating source. Average of suite of records should not be less than the target spectrum of MCE level of shaking.

Acceptance criteria for component checking and selecting and scaling of earthquake records are applied to AASHTO LRFD Bridge Design Specifications as well as AASHTO Guide Specifications for Seismic Isolation Design (2010). Additional information about tectonic settings, earthquake magnitude, type of faulting, source-to-site distance, local soil conditions, spectral shape and near-fault characteristics are provided in commentary to Article 4.7.4.3.4b. The AASHTO Guide Specifications for Seismic Isolation Design (AASHTO, 2010) requirements for response history analysis is given in Article 7.4. The rules in calculation of component demands of isolation units are identical in ASCE/SEI 7-05 and ASCE/SEI 7-10.

Three significant differences between AASHTO Specifications and ASCE/SEI 7 on the topic of selecting and scaling of ground motions are in terms of response spectrum matching, consideration of vertical ground motion and on the site-to-source distance for rotating horizontal pairs oriented to fault normal and fault parallel directions. One of the important

emphasis in AASHTO is about near-field condition and transformation of as recorded horizontal ground motions into principal components before obtaining spectrum compatible earthquake records within 0-10 km (6 miles) source to site distance. The major principal component should be applied in fault-normal direction while applying minor component in fault-parallel direction. Design based earthquake (DBE) in AASHTO (2010) is defined by ground motions having 7% probability of exceedance in 75 years. At least three response-spectrum compatible time histories shall be used to represent the DBE level earthquake. All translational components of earthquake required to be applied simultaneously during a nonlinear response history analysis and their maximum response should be taken in design actions. If a minimum of seven time histories are used for each component of motion, the design actions can be calculated on the basis of mean response in each principal direction.

Eurocode-8 Part 2 (CEN, 2005) is prepared for the seismic design actions of bridges and it uses same spectral shape of elastic design spectrum of Part-1 for buildings for RHA. However, it presents different procedure for the selection of records in Section 3.2.3 due to two principal directions of bridge. At least three pairs of record shall be used and selection of these pairs needs to be consistent with the determined magnitude, source-to-site distance and fault mechanism of the seismic event. When the required number of pairs is not available properly adjusted or simulated ground motions can be used in RHA. SRSS of pairs shall be used as spectral parameter for scaling of 5% damped spectra of each component. Ensemble average of SRSS spectra is used in design and performance assessment actions for the incorporation of seven or more pair of ground motions. Otherwise, maximum response value will be utilized for design actions. Period range of interest is defined between $0.2T_1$ and $1.5T_1$ like ASCE/SEI 7-05 and lower bound of SRSS spectra must be larger than 1.3 times of the established elastic design spectrum. T_1 represent the fundamental period of conventional bridge types while this value is taken as effective period value for bridges in the same range. It is allowed to adjust ground motions if large variations are observed between the ensemble average of selected records and elastic design spectrum. When three component ground-motion time-history recordings are used for nonlinear time-history analysis, scaling of the horizontal pairs of components may be carried out in accordance with previously explained rules which are independent from the scaling of the vertical components. The ensemble average of the relevant vertical spectra shall not be than 10% of the 5% damped vertical elastic design spectrum in the period range between $0.2T_v$ and $1.5T_v$, where T_v is the period of the

lowest mode where the response to the vertical component prevails over the response to the horizontal components. An important assumption of EC-8 Part-2 is the allowance of independent scaling of the pairs of horizontal recordings and of the vertical recordings.

The two main issues faced in record selecting and scaling for time history analysis are number of earthquake records and the limit of scale factors. Required number of earthquake records in design codes and specifications is a controversial topic. Statistical examination of number of ground motions in ASCE/SEI-7 (2005, 2010) were assessed on the basis of elastic-perfectly plastic and bilinear single degree of freedom systems (SDOF) by Reyes and Kalkan (2012). It is worth noting that specified number of ground motions is based on engineering experience rather than a comprehensive evaluation. Analysis results indicate that the number of earthquake records not to be less than seven to avoid inefficient and inconsistent results. Upper and lower limits of scale factors are not specified in design codes and design engineers are hesitant to define these two limits. Scale factors closer to unity are preferred which corresponds to unscaled records. Recommended upper limit of scale factor varies from 2 to 4 (Bommer and Acevedo, 2004). Main concerns of applying scale limits may be associated with the variation of geophysical parameters to generate such scaled records.

5. THE CURRENT STATE OF ART IN SEISMIC ISOLATION TECHNOLOGY

5.1. Introduction

Conventional aseismic design of structures relies on fulfilling the life safety criterion by allowing inelastic deformations in specially detailed regions of structural members to dissipate the transmitted earthquake energy. Code compliant performance criteria in conventional seismic design approach can only be achieved through the occurrence of inelastic deformations under design basis earthquake (DBE). Recent advances in earthquake engineering lead to the development of a rational basis in performance based earthquake engineering (PBEE) for the desired behavior of structures at each defined earthquake level. However, accurate estimation of deformations and calculation of likelihood of direct and/or indirect losses are not possible by performing traditional aseismic design methods due to uncertainties of the material properties and the modeling assumptions used for structural analysis. In this sense, seismic isolation (SI) or more commonly referred as base isolation (BI) technique is considered as an alternative design method to conventional design philosophy. Distinct beneficial features of seismic isolation technique cannot be provided in the concept of conventional aseismic design.

Recent advances in structural control concept lead to development of new seismic isolation and energy dissipation devices. Primary objective of seismic isolation can be expressed as decoupling of the structure from the damaging effects of earthquake shaking through the elongation of fundamental period or concentration of excessive damages at isolator locations for long period structures. Shifting of the fundamental frequency to a lower value may result as excessive deformation at isolation level. Inherent damping characteristics are necessary in limiting the probable excessive deformations that may be detrimental for bridges at expansion joints and abutment locations. Wider expansion joints and increased seat lengths may be required to accommodate these displacements. In case of insufficient clearance, pounding effects might be experienced in seismically isolated bridges due to the large displacement demands. A proper seismic isolation bearing should provide both horizontal flexibility and energy dissipation characteristics while supporting the superstructure under service loads. Contemporary seismic isolation system needs to include these three functions in

a single unit. Moreover, rigidity under wind and vehicle braking loads must be provided to avoid the unacceptable movement of the superstructure because of the horizontal flexibility. Effects of the horizontal flexibility and the inherent damping are depicted in Figure 5.1.

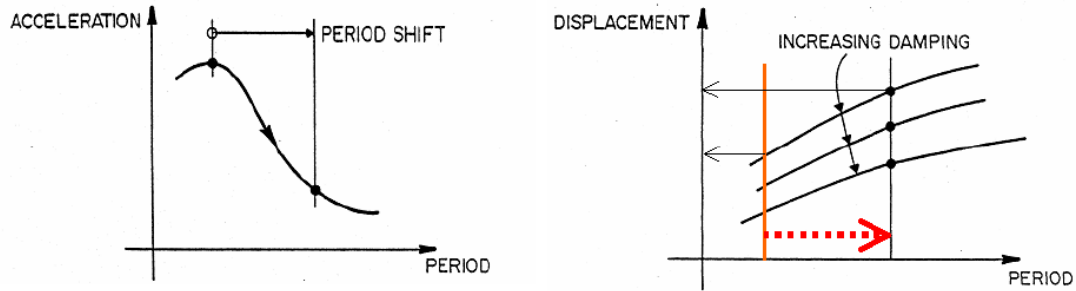


Figure 5.1. Effects of horizontal flexibility and inherent damping increase.

Earthquake protection systems are classified into three main groups as active, semi-active, passive and hybrid control systems. All of the isolation units used in this study are classified as passive protection systems. This chapter mainly addresses the brief history and basic concepts of seismic isolation technology with particular emphasis on mechanical characteristics of the two commonly used isolators in seismically isolated bridges. In the scope of this study both experimental and analytical studies are covered to identify the response characteristics of seismic isolators in close proximity to active faults. Analytical research study focuses on the investigation of effectiveness of seismically isolated bridges in near fault regions by assessing the performance of two common types of isolators: the lead rubber bearing (LRB) and curved surface friction slider bearings. In addition, modeling assumptions of triple curved surface friction bearing in are reviewed and implemented for their performance assessment.

5.2. Historical Development of Seismic Isolation Technology

Early seismic protective systems in the history date back to approximately 500 BC. Parthenon, Athens (447-436BC) with sliding-rocking of columns, Kunya-Urgench, Turkmenistan Kutlug Timur Minaret with sliding foundations (1320AD) can be listed as the first examples of earthquake protective systems in the history. First patent of the seismic isolation system is reported by US Patent Agency to Joules Touaillon with Patent number of 99973 on February 15, 1870. English scientist, John Milne who is known as the “Father of

Seismology” investigated the performance of buildings in seismically prone areas and made recommendations for earthquake resistant design during his studies in Tokyo between 1876 – and 1895. His experimental studies about seismically isolated structure is one of the pioneering works published in a report to the British Association for the Advancement of Science in 1885 (Naeim and Kelly,1999). Following these first patents and early works of SI new patents were given to J. Bechtold from Germany in 1907 and to Dr. Calantarientis in 1909 for new foundation systems.

Although patents for seismic isolation methods were obtained more than 140 years ago, the first modern seismically isolated building was designed by Russian engineers in Ashkhabad, Turkmenistan in 1955 by employing suspended cables. Elastomeric bearings are most commonly used contemporary isolation units in practice. The first application of such system was designed for the Pestolozzi School in Skopje, Yugoslavia in 1969. Then, the shortcomings of this system were reported as having the same stiffness value for horizontal and vertical response (Naeim and Kelly, 1999). Note that, currently, the rubber isolators that were used in the elementary school were replaced with its contemporary versions regarding to improper vertical response characteristics as shown in Figure 5.2.



Figure 5.2. Replacement of the first example of elastomeric type of isolators with their current contemporary versions in Pestolozzi School.

Since 1970 new approaches were directed towards developing rational and useful methods for the enhancement of earthquake resistance of structures and its contents. As an

alternative to ductility based design in 1970s researchers from New Zealand were developed economical and practical energy dissipating devices to control the displacement demand (*e.g.* Skinner *et al.*, 1975, Robinson and Greenbank, 1976 and Robinson, 1982). The first contemporary seismically isolated building in New Zealand is situated in Wellington and the construction of William Clayton building is completed in 1982 by using lead rubber bearings (Figure 5.3). Similarly, the first contemporary seismic isolation application in US is applied by The California Department of Transportation (Caltrans) on a bridge at the Sierra Point Overpass, San Francisco in 1985 (Taylor and Igusa, 2004).

The first common use of contemporary seismic isolation systems in Japan is applied to achieve floor isolation for the protection of costly equipment's prior to isolation of buildings in the early 1980s.

Recent advances in this relatively new technology were increased the number of applications. The detailed review of earlier and recent works on base isolation systems and applications had been reported by Christopolous and Filiatrault (2007), Buckle and Mayes (1990) and Jangid and Datta (1995).

5.3. Seismic Isolation and Energy Dissipation Applications in Turkey

Number of seismic isolation applications in Turkey is significantly increased and became quite prevalent after the 1999 Kocaeli (Mw 7.4) Earthquake. The 2011 Van Earthquake reminded the importance of continued functionality of critical structures like hospitals and directed decision maker's attention to isolation devices. Number of seismically isolated hospital buildings has been increasing in regions with high seismicity to provide continued functionality after a major earthquake. Hospital buildings need to be operational for the resilience of cities. Therefore, it is important to preserve the functionality of hospitals and protection of costly medical equipment immediately after a major earthquake (Yenidogan and Erdik, 2014).

Number of seismically isolated bridges has increased after the 1999 Kocaeli and the 1999 Düzce Earthquake. In the 1999 Düzce Earthquake, Bolu Viaducts turn into be one of the important case studies in the world where the deficiency in principle design rules are attributed

to the underestimation of seismic demand. The Bolu Viaduct is a 2.3 km long dual 59 spans supported by 58 piers. Each span is 39.2m long with two parallel bridges with span length of 39.2m and pier height of maximum 49m was nearly completed when it was struck by the 1999 Düzce Earthquake. Continuity of 7 lines of simply supported prestressed concrete box girders with slab were provided over 10 spans. Complete failure of the seismic isolation system due to excessive deformations and transmission of shear forces are shown in Figure 5.3. Evaluation of the seismic isolation system's design indicated that requirements of the AASHTO Guide Specifications for Seismic Isolation Design were not fulfilled for the existing SI and ED system before Düzce Earthquake (Roussis *et al.*, 2003, Yenidogan and Erdik, 2014).



Figure 5.3. Complete failure of existing protection systems of Bolu Viaducts.

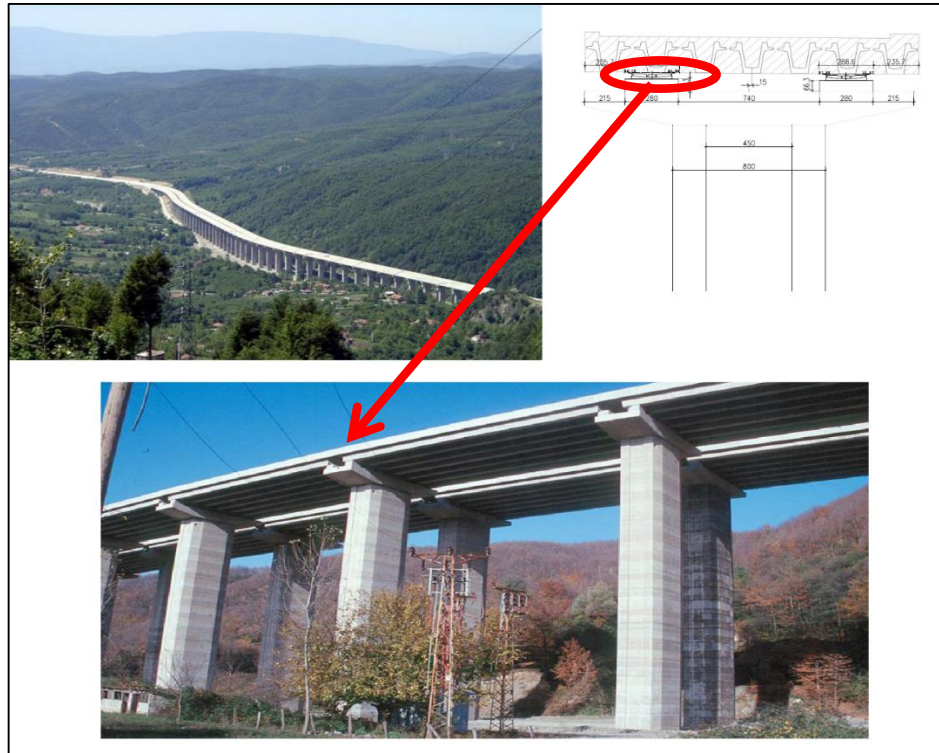


Figure 5.4. General view of Bolu Viaduct and location of SI units.

The development of urban earthquake risk mitigation master plans of Istanbul proposed short- and long-term strategies and emphasized the need of urgent retrofitting applications for lifelines. Mecidiyeköy Viaduct is accepted as one of the vital transportation links which needs to give service without any disruption after a major earthquake. One of the challenging issues in the rehabilitation process of Mecidiyeköy Viaduct was being in service during the retrofit project. SI and conventional methods such as steel jacketing, rehabilitation of foundation system were used together for seismic upgrading (Figure 5.5).

Another important SI application for bridges is carried out on Bozüyük-Mekece state highway. Seismic upgrading of the state highway was required due to heavy traffic and accidents. Total length of the project is about 85 km and the project includes number of tunnels, bridges and overpasses. Sakarya-II Viaduct is one of the most important sections where SSI effects are as well associated (Figure 5.6).



Figure 5.5. Location of Mecidiyeköy Viaduct and retrofit activities (Yenidogan and Erdik, 2014).

Distribution of passive control applications in Turkey is depicted below. In addition to stated numbers in Figure 5.7 more than 46 projects related with retrofit and design applications are either in the construction tender or design tender phases (Yenidogan and Erdik, 2014). Among available isolation units elastomeric and sliding type of isolation devices are two most commonly used isolators in Turkey likewise in the world.



Figure 5.6. Seismic isolation project of Sakarya II-project (By courtesy of Limak construction co. Ankara).

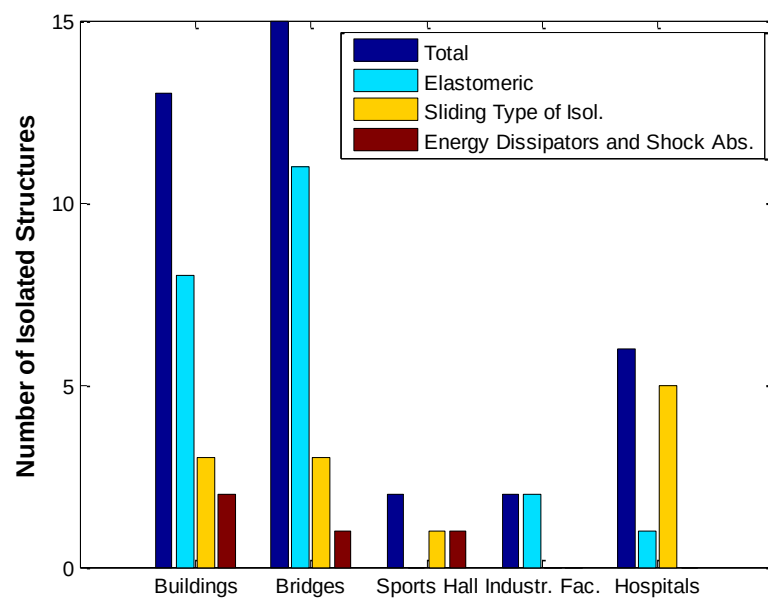


Figure 5.7. Distribution of SI and ED devices in completed constructions (Yenidogan and Erdik, 2014).

5.4. Overview of Design Codes for Seismically Isolated Structures in Turkey

Proper and common use of response modification technique such as in SI and/or ED system requires prescriptive design and testing provisions for practicing engineers. The latest version of Turkish Seismic Design Code was published in 2007 (SBBSZ, 2007) and four different performance limits were included for the assessment and design of conventional buildings. However, design principles and testing of anti-seismic devices were not considered in the current Turkish Seismic Design Code. Draft version of Turkish Seismic Isolation Design Code for Buildings (TASI, 2008) was firstly published in 2008 parallel to the advent of seismic isolation technology and increased number of SI applications in Turkey. Seismic isolation guideline for buildings is planned to take a place in the revised version of the current Turkish Seismic Code as a new section which will be attributed to buildings equipped with passive control systems. Recently, the Ministry of Health (MoH) of Turkish Republic has prepared a draft technical specification for the seismic isolation of new and existing hospital buildings over 100 bed capacity located in Seismic Zones 1 and 2.

In contrast, regarding to the lack of seismic design codes and guidelines for bridges in Turkey appropriate approaches of Eurocode 8-Part 2 (CEN,2005), AASHTO LRFD Bridge Design Specifications (2012) and AASHTO Guide Specifications for Seismic Isolation Design (2010) have been incorporated to check the design applications by General Directorate of Highways. Critical projects are also peer-reviewed by board of experts to check the design and control results of prototype tests.

As per the draft Turkish Seismic Isolation Design Code-TSIDC (TASI, 2008), the materials used in the manufacturing process of elastomeric isolation units shall conform to the requirements of the Chapter 8.2.2 of the European Standard (EN 15129-2006) and the design and control of the isolation units will be based on European Standard EN 1337-3:2005: Structural Bearings - Elastomeric Bearings. Typical sketch of these bearings with their force displacement hysteresis curves are shown in Figure 5.8 and Figure 5.9 (TASI, 2008). Four different method of analysis are defined with respect to building, isolation system and the

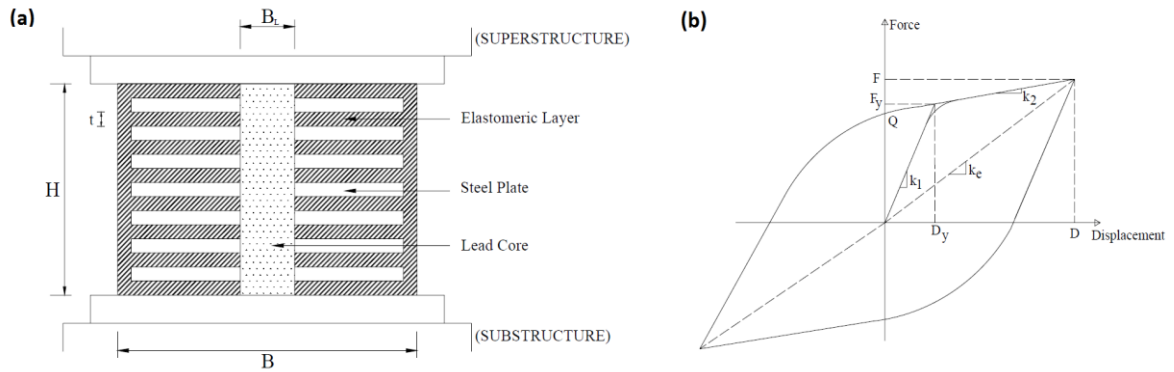


Figure 5.8. (a) Typical section of a lead core elastomeric isolation unit (b) force-deformation (hysteresis) curve for lead core elastomeric isolation unit (TASI, 2008).

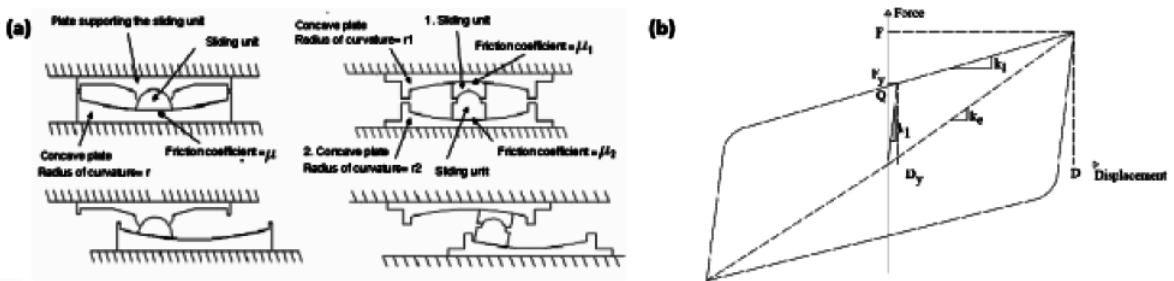


Figure 5.9. (a) Simple sections of the single and double curved surface slider isolation unit (b) Typical force - displacement hysteresis of curved surface slider type isolation units (TASI, 2008).

The latest version of the well-known curved friction slider type of isolation unit (also known as the triple friction pendulum (TFP) bearing) was used in several important projects in Turkey. Generally refer to proposed representation of the overall behavior, verified by experimental and analytical results (Fenz. and Constantinou, 2008, Morgan, 2007, Dao *et al.*, 2013).

For likelihood of large displacement demands at isolation interface, energy dissipaters (ED) might be used in conjunction with isolators for limiting the excessive displacements and transmitted shear force. Number of structures equipped with ED is low when compared to SI systems. It is worth noting that prototype tests are necessary for the verification of design parameters of SI systems as well as reduction of uncertainty during the lifetime of isolators. Due to lack of capable facilities in Turkey required prototype testing sequences have been performed in different laboratories. Moreover, shaking table tests of isolators might be useful

to have a better insight about representative testing specimens under earthquake excitation. Shaking table tests of SI systems were only performed in the laboratory of Kandilli Observatory and Earthquake Research Institute (KOERI) in Turkey for research purposes (*e.g.* Yenidogan *et al.*, 2013, Yenidogan *et al.*, 2007, Kesti, 2009, Uckan *et al.*, 2004).

5.5. Isolation Hardware and Mechanical Characteristics

There are two common types of seismic isolation units used for passive control of bridges, elastomeric bearings (*e.g.* low- and high-damping rubber; lead rubber) and sliding bearings (*e.g.* spherical sliding or Friction Pendulum bearings; flat sliding or EradiQuake bearings). Guide specifications are focused on these two main classes of isolators. Mechanical properties and analytical models of elastomeric and sliding bearings are defined briefly in following sections.

5.5.1. Multilayer Elastomeric Bearings

Elastomeric bearings consist of natural or synthetic rubber layers, thin steel shims and upper and lower end plates. Multi-layered rubbers are bonded to steel shims through vulcanization process. Hardness of elastomers can be easily measured by durometer. Protection of elastomeric bearing from environmental conditions is achieved through rubber cover. Environmental conditions such as corrosion and ozone attack may be detrimental for the desired mechanical characteristics of bearing. Isolators may deflect the energy through the dynamic of the system rather than absorbing the earthquake energy by isolation interface. However certain level of damping is beneficial to suppress possible resonance of the system (Konstantinidis and Kelly, 2011). Damping characteristic can be provided by the rubber compound or additional elements such as lead plug or steel elements. Elastomeric bearings can be categorized with respect to their inherent damping properties as follows

1. Low-damping rubber bearing ;
2. High-damping rubber bearing;

Low damping rubber bearings generally exhibit linear behavior up to 100-150% shear strain, beyond this value stiffening occurs (Figure 5.10). The calculation of shear strains is carried out with respect to lateral load, compression and rotation effects. Typical shear modulus of low damping rubber bearings ranges from 0.65 to 0.9 MPa while the equivalent damping ratio takes values between 2-3% at 100 % shear strain. Supplementary energy dissipation devices like viscous fluid dampers, frictional dampers, yielding hysteretic steel dampers might be required to limit displacement demands.

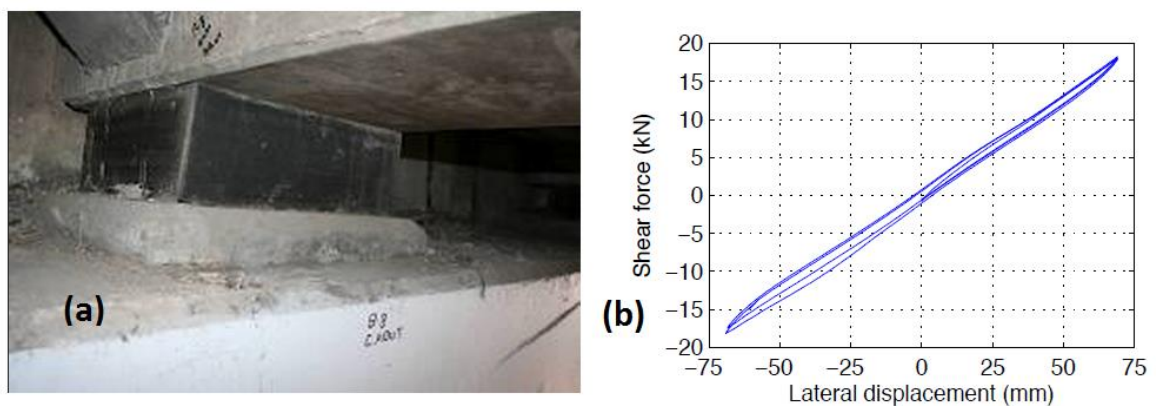


Figure 5.10. (a) Low-damping rubber bearing for bridges (b) Force-displacement hysteresis of low damping rubber bearing.

HDR bearing provide a higher level of damping through mixing of chemical additives to the raw rubber in the fabrication process of bearings (Figure 5.11). An intrinsic energy dissipation of HDR bearings reaches up to 10–20% for 100% shear strain (Naeim and Kelly 1999).

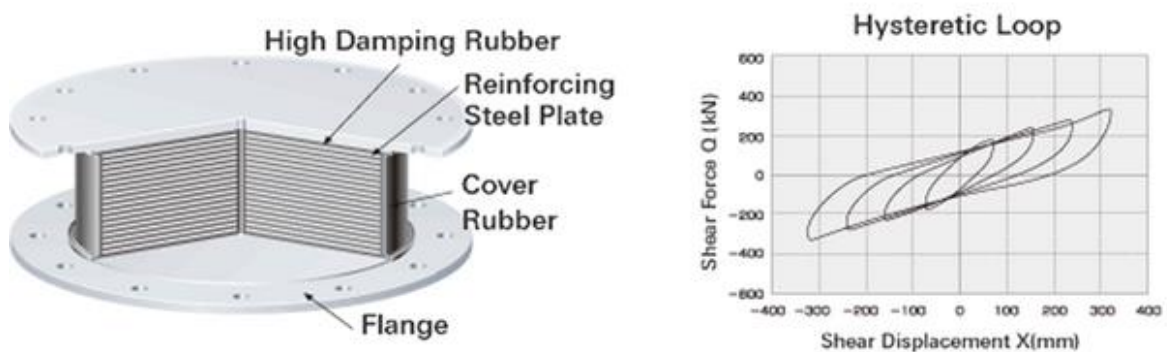


Figure 5.11. Geometric configuration of HDR bearings and its hysteresis loop.

Horizontal stiffness of the elastomeric bearing is calculated by Equation 5.1 where shear modulus, bonded area are denoted by G and A_b , respectively. Total rubber thickness is represented by T_r . Certain amount of rubber cover also can be included in the calculation of horizontal stiffness. Vertical stiffness of elastomeric bearings is calculated in a similar manner by considering the compression modulus of the rubber-steel composite, E_c and area of steel shim rather than shear modulus and bonded rubber bearing. Shape factor, S determined as a ratio of loaded area to force-free area of rubber. Typical bridge bearings have lower S factor in comparison with isolators.

$$K_h = \frac{GA_b}{T_r} \quad (5.1)$$

$$E_c = 6GS^2 \quad (5.2)$$

For bearings with very large shape factor, S compressibility of rubber affects the value of the compression modulus of the rubber-steel composite, E_c . In such cases a more accurate estimate of E_c may be obtained by considering bulk modulus, K . Elastomeric seismic isolators have high shape factor which lead to mitigation of rocking motion and enhancement of stability. It is essential to perform adequacy checks for both elastomeric bridge bearings as well as elastomeric isolators.

5.5.2. Lead Rubber Bearing (LRB)

Lead-rubber bearing (LRB) was introduced in New Zealand in the late 1970s and LRB has been utilized in seismic isolation applications (Robinson, 1977, 1982, Skinner *et al.*, 1993). Fabrication process of LRB is identical to low damping rubber bearing except the implementation of lead core which yields in low level of stresses at normal temperatures and have capability of recrystallization for a stable dynamic behavior under multiple cycles. In other words, the elastomer provides the low horizontal stiffness component and the lead core provides the energy dissipation feature. The maximum shear-strain varies in the range of 125-200 %. A cut-away view of the bearing illustrates the internal construction of lead-rubber bearing in Figure 5.12.

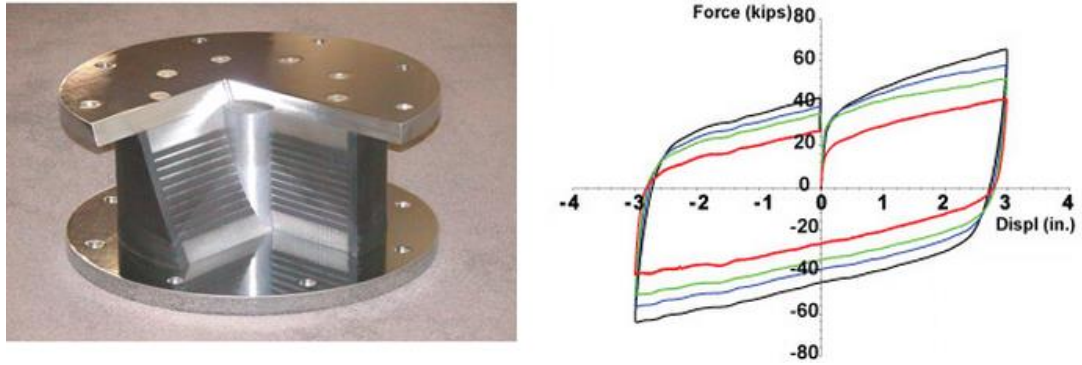


Figure 5.12. Construction of LRB b) Typical force-deformation loop for LRB (Courtesy of DIS).

The horizontal force-deformation relationship of a lead-rubber bearing is idealized by rate independent linearization where the typical hysteresis loop is governed by characteristic strength (Q_d), post-elastic stiffness (K_d) and yield displacement (D_y). Characteristic strength can be calculated by using the properties of lead plug. Yield strength and cross-sectional area of the lead core are denoted by σ_L and A_L respectively. Rate independent idealization is used as an effective tool in AASHTO Guide Specification (AASHTO, 2010). Ideal bilinear representation can be afforded by Bouc-Wen or coupled plasticity model. Coupled plasticity with circular yield surfaces are used to realistically simulate the bearing motion. If this behavior is omitted where it may lead to underestimation of displacement demands.

$$Q_d = \sigma_L A_L \quad (5.3)$$

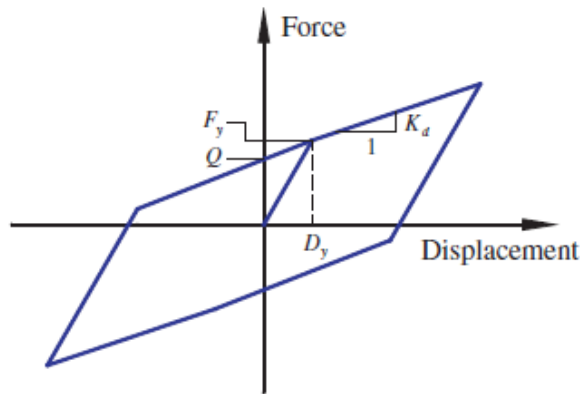


Figure 5.13. Typical idealized bilinear hysteretic force-deformation loop (AASHTO, 2010).

Two directional smooth bilinear hysteretic model of isolators in programs like SAP2000, 3D-BASIS-ME was originally established by using the approach of Park *et al.* (1986) and acceptable results are obtained on the behavior of isolators in bi-directional motion (*e.g.* Nagarajaiah *et al.*, 1991, Mokha *et al.*, 1993). Isotropic formulation is an appropriate idealization for describing the behavior Lead Rubber Bearings. The forces along the orthogonal directions that are mobilized during earthquake excitation are described by Equation 5.4 and Equation 5.5.

$$F_x = \alpha \frac{F^y}{Y} U_x + (1 - \alpha) F^y Z_x \quad (5.4)$$

$$F_y = \alpha \frac{F^y}{Y} U_y + (1 - \alpha) F^y Z_y \quad (5.5)$$

α is the post-yield stiffness ratio, F^y is the yield force and Y is the yield displacement. Z_x and Z_y are dimensionless variables governed by following system of differential equations proposed by Park (1986). A , γ and β are dimensionless parameters that controls the shape of the hysteresis loops. Proposed model can be utilized for all types of elastomeric bearings. Special attention needs to be given for high damping rubber bearing due to strain dependent stiffening behavior.

$$\begin{Bmatrix} \dot{Z}_x \\ \dot{Z}_y \end{Bmatrix} = \begin{Bmatrix} A \dot{U}_x \\ A \dot{U}_y \end{Bmatrix} - \begin{pmatrix} Z_x^2 (\gamma Sgn(\dot{U}_x Z_x) + \beta) & Z_x Z_y (\gamma Sgn(\dot{U}_y Z_y) + \beta) \\ Z_x Z_y (\gamma Sgn(\dot{U}_x Z_x) + \beta) & Z_y^2 (\gamma Sgn(\dot{U}_y Z_y) + \beta) \end{pmatrix} \begin{Bmatrix} \dot{U}_x \\ \dot{U}_y \end{Bmatrix} \quad (5.6)$$

It is worth noting that effects of heating due to consecutive reversed cycles result as strength degradation which cannot be captured by proposed coupled plasticity modeling assumptions. Theoretical models that account for effects of heating in lead core have been developed and verified by Kalpakidis and Constantinou (2009) through experimental testing.

5.5.3. Sliding Type of Isolation Systems

Initial design of contemporary sliding isolators are developed by using a similar approach proposed by Johannes Avetian Calantarients in 1909. In contrast, contemporary sliding isolation systems consist of filled or unfilled Polytetrafluorethylene (PTFE) material and stainless steel rather than separation layer of Talc. Weight of the structure can be supported

by utilizing a sliding interface with a low coefficient of friction, which limits the resistance to horizontal forces in the limit of displacement capacity. A major advantage of these systems is their insensitivity to the frequency content of excitation, an apparent consequence of their frictional behavior. The frictional characteristics of sliding system are dependent on temperature, velocity, degree of wear and cleanliness of the surface.

5.5.3.1. PTFE Sliding Bearings. PTFE spherical bearings can be used either as large displacement capacity expansion bridge bearings (flat sliding bearings) or as fixed bridge bearings with proper design principles and material characteristics. General use of PTFE bearing aims to accommodate thermal and travel effects. One of the shortcomings of flat and spherical PTFE sliding bearing is the lack of restoring force and to be in need of supplemental devices for avoiding permanent offsets. In this sense, PTFE sliding bearings can be used in combination with elastomeric bearings in high seismicity regions. Such combination have been used to assess the effectiveness of a $\frac{1}{4}$ scaled steel structure by Yenidogan *et al.* (2013). Spherical bearings and their one-directional versions of cylindrical bearings have large capacity to accommodate rotation with very low resistance capacity to the superimposed moment. Geometrical configuration of PTFE spherical bearings is shown in Figure 5.14.

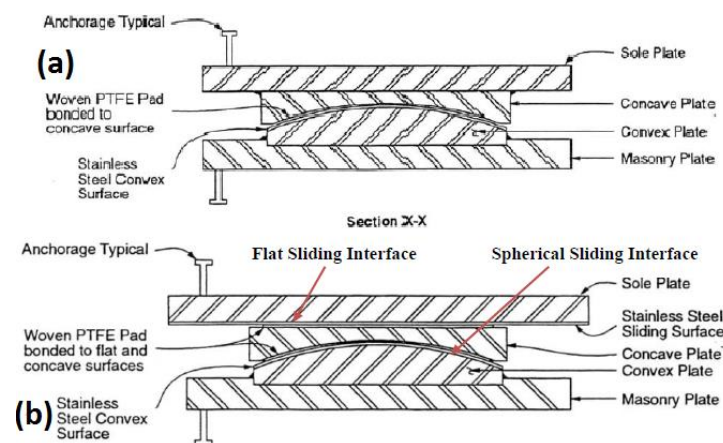


Figure 5.14. (a) Fixed Spherical Bearing b) Multidirectional PTFE Sliding Spherical Bearing (Caltrans, 1994).

Mokha *et al.* (1988) have conducted a series of tests on sheet-type Teflon sliding bearings, and obtained measurements of the static and sliding kinetic coefficients of friction under varying bearing pressures while calibrating the mathematical model of friction for PTFE

sliding bearings. Critical findings are the dependency of the frictional force on the velocity of sliding, on bearing pressure and condition of interface. By using Equation 5.7 dependency of coefficient of friction can be obtained as shown in Figure 5.15. Maximum coefficient of friction, minimum coefficient of friction are denoted by f_{max} , f_{min} where a represent rate parameter and velocity.

$$\mu = f_{max} - (f_{max} - f_{min}) \exp(-a|\dot{u}|) \quad (5.7)$$

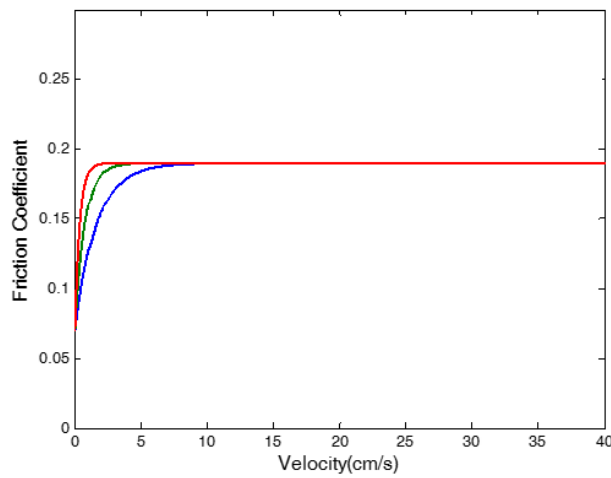


Figure 5.15. Variation of coefficient of friction for PTFE sliding interface for different rate parameters.

For flat sliding bearing, the mobilized orthogonal forces under biaxial earthquake loading can be determined by using the same stated system of differential equations (Equation 5.6) for LRB bearings (Constantinou, 1990, Mokha *et al.*, 1993). Main difference will be the determination of friction forces regarding to acted normal force. Similar theoretical and experimental research are also performed for curved surface friction sliders. The AASHTO Guide Specifications provide a simplified method to estimate the seismic demands for bidirectional input motions.

5.5.3.2. Curved Surface Friction Slider Bearings. Curved surface friction slider bearings is a form of sliding bearing consisting of an articulated slider on a spherical surface which was patented as Friction Pendulum System(FPS) in 1986 by Earthquake Protection Systems, Inc. (Zayas *et al.* 1987). The first application of FPS in the industry was used to retrofit an

apartment building in California in 1989 (Naeim and Kelly 1996). Since then, the conventional curved surface friction sliders have been widely used to isolate buildings, bridges and storage tanks and it has been incorporated into the design codes (AASHTO 1999, AASHTO 2010, International Building Code, 2000). Geometric configuration and free body diagram of a curved surface friction slider is illustrated in Figure 5.16.

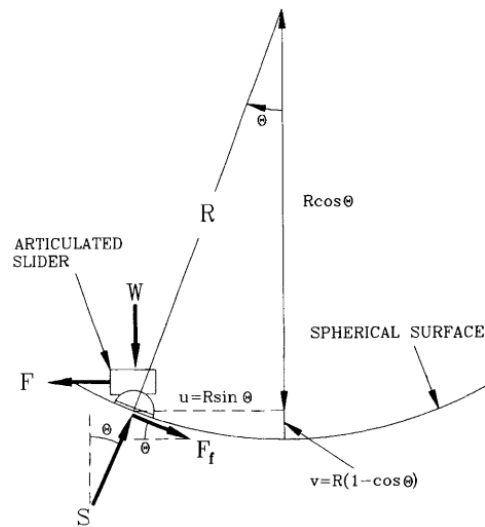


Figure 5.16. Free body diagram of conventional single concave curved surface friction slider.

The two important characteristics of the bidirectional sliding of curved surface friction slider motion can be summarized in terms of isotropic force-deformation and stick-slip condition (Constantinou *et al.*, 1990). Stick-slip condition of curved-surface friction slider is significantly observed in the characterization test of recommended device by the manufacturer. A simplified approach for modeling the frictional response considers two independent unidirectional elements according to the Coulomb's model in the orthogonal directions. Force-deformation calculation can be carried out by using Equation 5.8 where N is the normal force acting on the isolator, R is the radius of curvature, u and $\dot{u}$ are the sliding displacement and the sliding velocity, respectively.

$$F = N\mu sgn(\dot{u}) + \frac{N}{R}u \quad (5.8)$$

Nonlinear time history analysis of an isolated bridge by using coupled plasticity models were performed by various researchers (Warn and Whittaker, 2004, Andersson and Mahin, 2004) to assess the accuracy of uniform load pattern method in AASHTO Guide Specification by accounting bidirectional earthquake excitation. Typical derived force-deformation relationship of a conventional curved surface friction slider by using Equation 5.8 is shown in Figure 5.17.

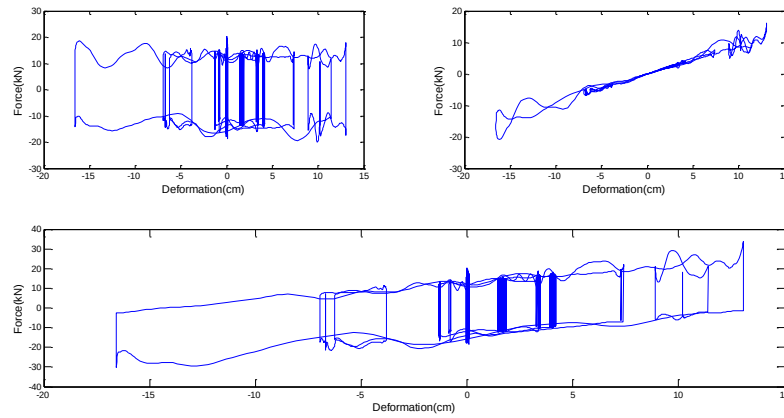


Figure 5.17. The intrinsic response components and force-deformation relationship of a conventional curved surface friction slider.

The AASHTO Guide Specifications (1999, 2010) Section 7 on analysis procedures states that: “To simplify the nonlinear behavior of the isolator unit, a bilinear simplification may be used” as illustrated in Figure 5.13.

$$f\mu = \begin{bmatrix} f\mu_x \\ f\mu_y \end{bmatrix} = N\mu \begin{bmatrix} \text{sgn}(\dot{u}_x) \\ \text{sgn}(\dot{u}_y) \end{bmatrix} \quad (5.9)$$

Following the single curved spherical bearing geometric modifications are applied to enhance the earthquake protection potential while reducing the cost issues. Three different version of spherical sliding bearings are in use for seismic isolation applications. Correspondingly double concave curved surface friction sliders and triple curved surface friction sliders are offered to earthquake engineering community. Double curved surface friction sliders consists of two concave stainless steel surface while the triple curved surface sliders have four concave stainless steel sliders. By using the derived theoretical force–displacement relationship of the double curved surface friction sliders, characterization testing was performed to confirm the theoretical predictions Fenz and Constantinou (2006).

Hysteresis loops and mathematical models of double curved surface friction slider and triple curved surface friction slider are shown in Figure 5.18.

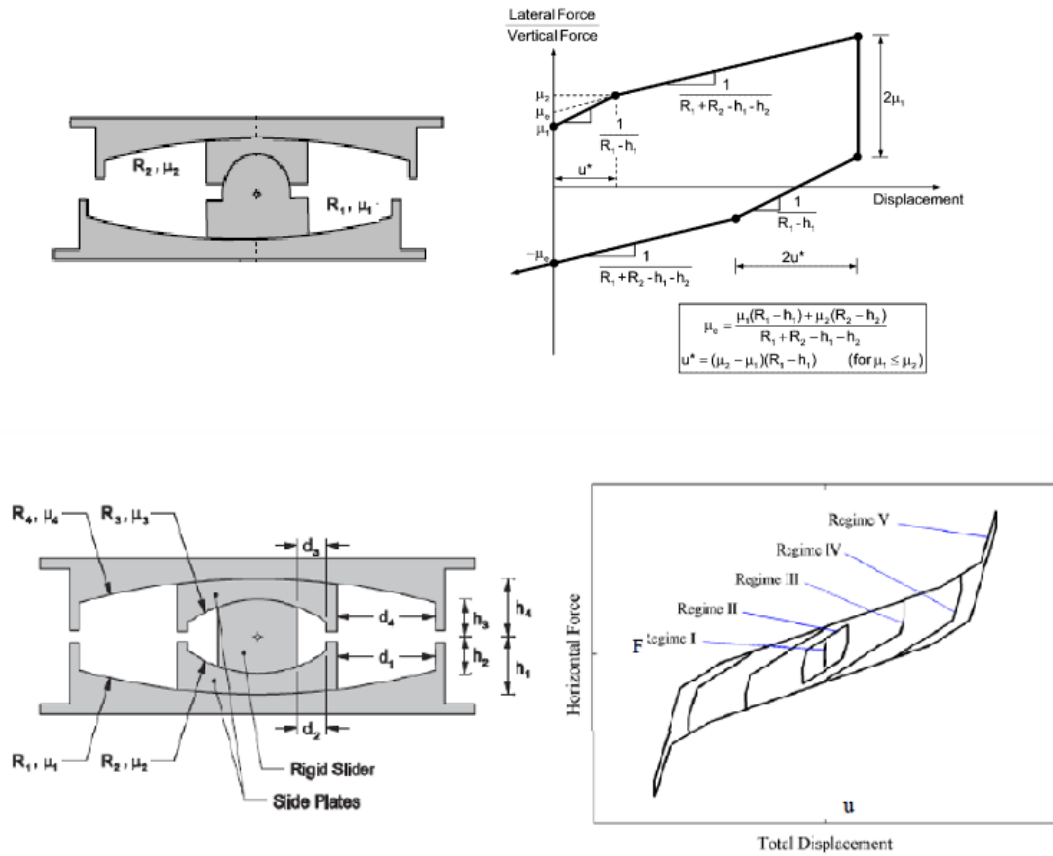


Figure 5.18. Geometric configuration of double (top) and triple (bottom) curved surface friction sliders and their sliding regimes (Fenz and Constantinou, 2006, 2008).

The latest version of curved surface friction sliders are patented as triple friction pendulum system and it is one of the sophisticated passive control system in the practice. When configured properly the Triple Friction Pendulum (FP) bearing exhibits desirable changes in stiffness and damping with increasing amplitude of displacement. The Triple Friction Pendulum (FP) isolator exhibits multiple changes in stiffness and strength with increasing amplitude of displacement. Details on the behavior of the isolator, including test data, shake table testing and comparison of analytical and experimental data may be found in Morgan (2007) and Fenz and Constantinou (2008).

Internal construction and five sliding regimes of the TFP are shown in Figure 5.18 (Fenz and Constantinou, 2008). The overall behavior and equations governing the force-displacement relationship in each stage are summarized by various researchers (Fenz and Constantinou, 2008, Morgan and Mahin, 2010, Dao *et al.*, 2013). The overall behavior of TFP can be modeled by series connection (Fenz and Constantinou, 2008) which incorporates three single concave curved surface slider or by parallel connection of flat sliding bearing and curved surface friction slider (Sarlis and Constantinou, 2010) in computer programs. Two different analytical tools of the TFP are shown in Figure 5.19.

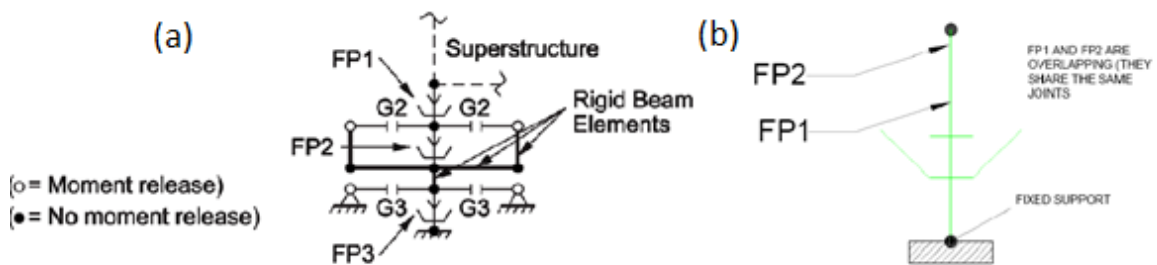


Figure 5.19. Series and parallel model of the TFP.

5.6. Seismic Isolation Technology for Highway Bridges

Bridges are one of the critical components of transportation network which might be exposed to high seismic risk regarding to their low redundancy. Vulnerability of bridges were experienced and well documented by site survey reports following large earthquakes (Basöz and Kiremidjen, 1998, Buckle, 1994, Kawashima *et al.*, 2009, Kawashima and Matsuzaki, 2012). Since the 1971 San Fernando Earthquake, bridge engineering is evolved and after each major earthquake new findings relevant with the post-earthquake observations are implemented into seismic design codes, guidelines and specifications.

Earthquake protective systems are utilized for the reduction of transmitted inertial forces from the deck to piers and foundations. Nowadays, several types of isolation systems (IS) are in use and new protective systems are proposed to improve the performance of bridges. Enhancement of earthquake resistance of bridges is relatively easier than buildings by replacing conventional bridge bearings with properly designed seismic isolation systems seismic isolators. This procedure might be applied to bridges in an easy and rapid manner.

Seismic isolators are typically installed between piers, abutments, and the deck of bridges as shown in Figure 5.20.

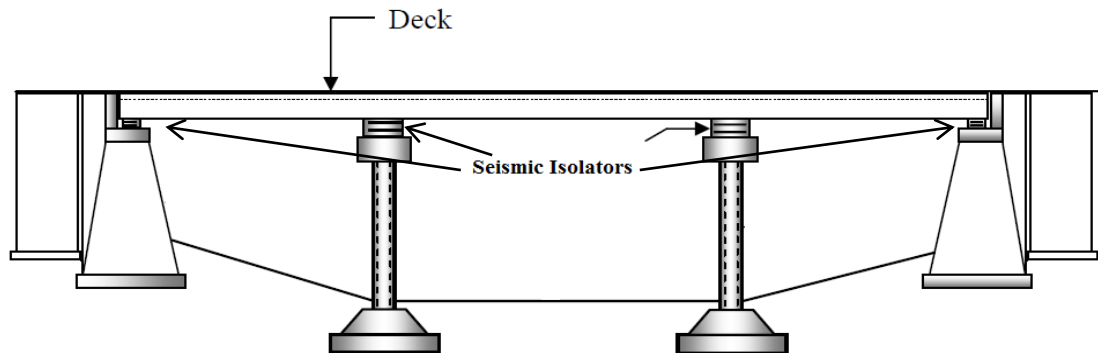


Figure 5.20. Typical location of seismic isolators in bridge applications.

Installation of seismic isolators does not only provide regular response but also reduce transmission of large force and moments. Alternative configurations such as installation of isolation units at the bottom of bents, partial isolations of bridges with highly flexible piers and rocking mechanisms can be also be considered for appropriate bridge applications.

Taylor and Igusa (2004) identified the following distinguishing properties of bridges from buildings in terms of structural response

1. Spatial variations of ground motions may become critical also in the longitudinal direction of long span bridges. Assessment of seismically isolated skewed bridges can be considered under this title for unrestricted movements.
2. Bridges are classified as simple structures and the superstructure is supported by flexible bents. Low redundancy of this type of structures might lead to catastrophic consequences.
3. Due to variations in geographical terrain, irregular configuration of substructures in height may affect the flexibility of the substructure significantly.

4. Long span bridges are typically more flexible in the vertical direction when they are compared with buildings. Thus, vertical ground motions are important for bridges in close proximity to active fault.
5. The main concern in the isolation of building type of structures is to limit the transmission of forces to the superstructure while the primary concern in bridges is vice versa.

Lead-rubber bearing and friction pendulum bearing are the two most commonly used isolators in bridges due to their reliable and well defined modeling assumptions, ease of installation, practical use and relatively cost-effective characteristics in manufacturing process. Design procedures for these two types of isolators are well-integrated to the current code provisions in US, Japan, Europe and New Zealand (IBC 2012; IBC 2003; NEHRP 2003, AASHTO LRFD Bridge Design Specification, FEMA 356, Eurocode 8 Part-1, 2). Applications of seismic isolation in regions with high earthquake hazard require bearings with large displacement capacity. Therefore the displacement demand is a key parameter for the design of isolation units and the long period ground motion characterization would be crucial to determine this demand as emphasized in Chapter 3. The main goal of response analysis is to estimate the displacement demand and the amount of shear force that can be safely transmitted to the substructure, while re-centering requirements of the isolation system are fulfilled. Re-centering capability of isolation devices can be checked with respect to given equations in seismic design codes (CEN, 2005, AASHTO LRFD Bridge Specification, 2010). Devices with good re-centering capability are preferred in the design process.

5.7. Methods of Analysis for Seismically Isolated Bridges

Method of analyses that are offered for seismically isolated structures in design codes and specifications (AASHTO, 2012, CEN, 2005) are named as follows

- 1) Uniform Load Method
- 2) Single Mode Spectral Method
- 3) Multimode Spectral Method
- 4) Time-History Method

The Uniform Load Method approximates the earthquake in terms of an equivalent static force. The uniform load method is a single mode method of analysis. The multi-mode method of analysis can be considered as a simplified procedure which incorporates linear elastic element and linear viscous element to represent the nonlinear behavior of seismic isolation system. Single mode-method of analysis is an iterative procedure that continues until the assumed and calculated values of isolator displacement are equal. Rigid substructure assumption is used in the beginning of the applied procedure. However, this assumption is not valid for all cases and effect of flexible substructure needs to be checked.

Multimode method of analysis is essentially a response spectrum analysis. Representation of higher damping values in the response spectrum are utilized by means of the damping modification value, B . The spectral ordinates are only modified for the isolated modes of the bridge whereas the spectral values at higher modes remain as established. Analysis should be performed separately in two principal directions and then combined by 30% rule. Principal directions of curved bridges are determined with respect to its chord.

Incorporation of nonlinear representation of isolation units can be achieved by performing nonlinear time history analysis. Explicit characteristics of each isolator need to be expressed by computer programs that are capable to conduct such analysis. 3D-Basis (Tsopelas *et al.*, 2005) is a public domain software to conduct analysis for buildings and liquid storage tanks like its commercially available counterparts such as SAP2000 (CSI, 2010), OpenSees (McKenna, 2007), ANSYS (Swanson Analysis Systems, 1996) and ABAQUS (Hibbitt, Karlsson and Sorensen, 2004).

6. MODELING AND ANALYSIS OF SEISMICALLY ISOLATED BRIDGES

6.1. Introduction

Bridges constitute the critical part of lifelines and the maintenance of functionality of the bridges after large earthquakes is an important problem. Research related with bridges were mainly focused on the mitigation of earthquake risk as a consequence of damage. Significant number of research studies have investigated the performance of fixed base straight, skewed and curved bridges. It is worth mentioning that the results obtained depended strongly on modeling assumptions and the capability of analysis techniques used. Estimation of deformations and the calculation of the direct and/or indirect losses in traditional bridges are associated with uncertainties. In contrast, proper use of seismic isolation method minimizes these uncertainties by concentrating the nonlinear behavior at the isolation units. In this chapter effectiveness of seismic isolation for bridges will be evaluated analytically for two different bridge applications under near-field earthquakes.

The first bridge application (Bridge-A) is elaborately selected to exhibit the effectiveness of seismic isolation for bridges in near-fault regions with varying skew angles. The detrimental effects of skew angles and orientation of ground motion axes were shown in numerous studies (*e.g.* Debasis *et al.*, 2012, Abdel-Mohti and Pekcan, 2008). The selected bridge was previously presented as one of the example bridges in the Federal Highway Administration Seismic Design Course, Design Example No.4, prepared by Berger/Abam Engineers, Sep. 1996 (NTIS, document no. PB97-142111). The bridge has a box girder superstructure and the substructure is monolithically connected to the deck. The same bridge model is modified for the seismic isolation application. Main purpose of this study is to evaluate the effectiveness of seismic isolation systems by utilizing this bridge example in close proximity to an active fault. Three different types of isolation devices are considered to evaluate and compare their performance in reduction of the deformation demands and in transmission of inertial loads. Modeling assumptions for triple friction pendulum system, proposed by various researchers (*e. g.* Fenz and Constantinou, 2008, Morgan and Mahin, 2010), are applied for this comparative analysis. Design principles of isolation devices are beyond the scope of this study,

interested readers can refer to the code-based provisions (AASHTO, 2012 and Eurocode-8, Part 2, 2005). Implementation of isolation devices to the analytical bridge models were done with respect to AASHTO LRFD design procedures.

The second bridge example (Bridge-B) is a part of an experimental campaign conducted in ELSA laboratories to assess the effectiveness of conventional curved surface friction sliders for the Rio-Torto Viaduct in Italy (Yenidogan *et al.*, 2013). KOERI is one of the partners of the pseudo-dynamic testing that took place in ELSA laboratories. Rio-Torto Viaduct is defined as an important connection link between Florence and Bologna and its functionality after moderate to severe earthquakes are required to be investigated. Nonlinear behavior of the as-built model and seismically isolated counterpart are investigated under the financial support of RETRO project which was coordinated by Fabrizio Paolacci from University of Roma Tre (Paolacci *et al.*, 2013, De Risi *et al.*, 2011, Yenidogan *et al.*, 2013). In order to measure the effectiveness of selected isolator units, as-built and isolated bridge model are generated for the Rio-Torto Viaduct. As an outreach of the Transnational Access of SERIES project author performed additional parametric studies to evaluate the performance of the proposed curved surface friction slider that are exposed to near-fault earthquakes (Yenidogan and Erdik, 2013).

6.2. Description of Bridge Models

6.2.1. Box Girder Multi-Span Continuous Three Span RC Highway Bridge

The original FHWA#4 bridge is a 30 degree skewed continuous, three-span, cast-in-place concrete box girder structure with a monolithic connection of deck and substructure. The original bridge design was prepared by Berger/Abam Engineers in 1996, adequately sustains the dead loads and displacement demands. For the implementation of seismic isolation technique additional diaphragm is adopted by Constantinou *et al.* (2011). The two intermediate bents consist of two circular columns with a cap beam on top. The main geometry of the bridge, section properties and foundation properties are remained the same as in the original bridge. Only minor changes in the bridge geometry were implemented in order to facilitate seismic isolation (i.e., use of larger expansion joints, use of separate cross beam in bents, instead of a single integral with the box girder, and columns that are fixed at the footings). Performance of the bridge is evaluated by response history analysis where the input motions were selected and scaled with respect to deterministic seismic scenario earthquake.

Figures 6.1, and Figure 6.2 show, respectively, the plan together with the elevation and a section of the bridge at an intermediate bent. In Figure 6.2 the bent is shown at the skew angle of 30 degree, whereas for the box girder section is perpendicular to the longitudinal axis. The actual distance between the column centerlines is 7.93 m (see Figure 6.1). The bridge is isolated with two isolators at each abutment and pier locations with a total number of 8 isolators. The isolator units are installed between the superstructure and substructure.

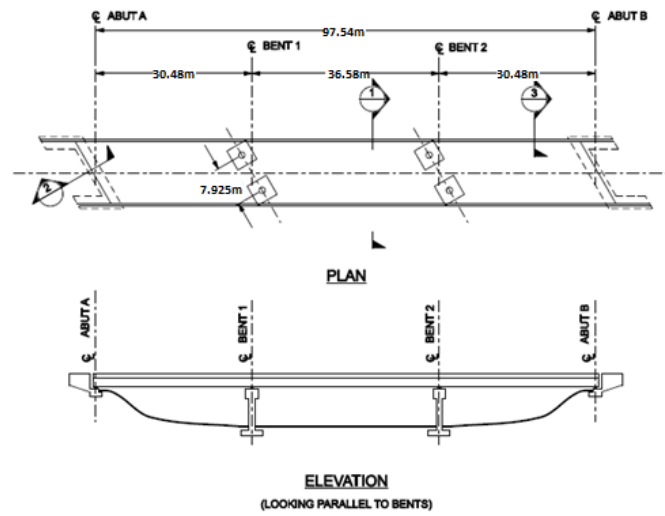


Figure 6.1. Plan and elevation of the FHWA#4 Bridge with 30 Degree skew angle.

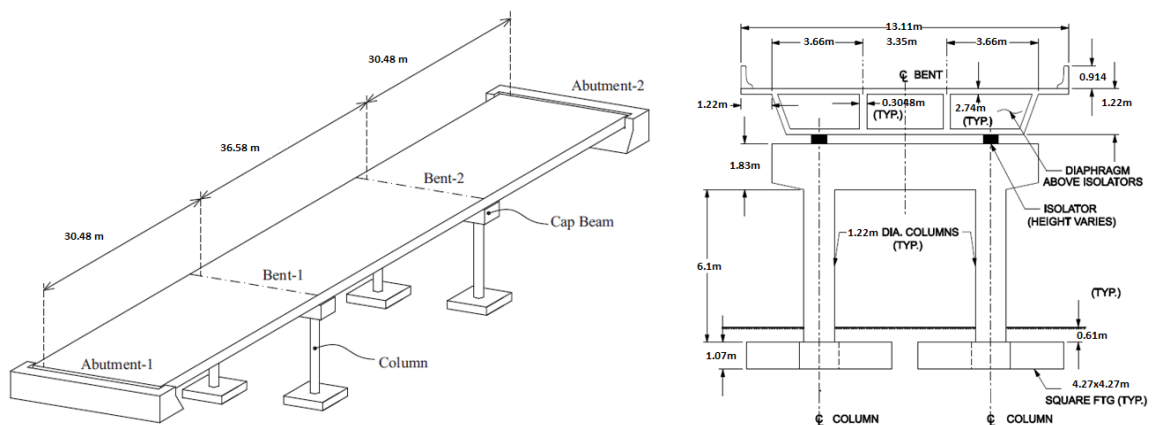


Figure 6.2. Dimensions of the considered FHWA#4 bridge for seismic isolation application.

The detailed analysis of bridges are facilitated with the advent of modelling options in commercial programs. These programs contain different type of superstructure and substructure elements in their library with varying geometries. Practicing engineers rarely use 3D solid elements for bridges regarding to high level of modelling efforts and the required computational time in dynamic analysis. In fact, complex geometric configurations of the superstructure is only considered for assessing the effects of service loads and self-weight on the response of the bridge deck. Transverse and longitudinal stiffness of the deck are often stiffer than the other structural components of bridges (*e.g.* piers, conventional bearings, isolators and restrainers). Since the detailed modeling of the deck is not essential for the performance assessment of seismic isolators. Cross sectional properties of structural elements are given in Table 6.1. The total weight of the isolated bridge superstructure is 22650 kN and the distance between the centerline of columns is 7.925 m for 30 degree skew angle. Effect of skew angle on the seismic isolators are investigated for three different variations of skew angle as shown in Figure 6.3.

Table 6.1. Properties of structural elements.

Element/ property	Box Girder	Bent Cap Beam	Column
Area(m ²)	6.76	2.23	1.17
Shear Area(m ²)	2.25	2.23	1.17
Shear Area(m ²)	5.3	2.23	1.17
Moment of Inertia	0.08	0.28	0.08
Moment of Inertia	3.46	0.62	0.08
Torsional Constant	15.28	0.65	0.22

The flexibility at pier foundations are considered in terms of soil-springs and change in the response of seismic isolators due to soil-structure interaction are assessed. Parameters of soil-spring are given in Table 6.2.

Table 6.2. Parameters of soil-springs for SSI action.

Constant	$K_x(\text{kN/m})$	$K_y(\text{kN/m})$	$K_z(\text{kN/m})$	$K_{rX}(\text{kN-m/rad})$	$K_{rY}(\text{kN-m/rad})$	$K_{rX}(\text{kN-m/rad})$
Description	Vertical Stiffness	Transverse Stiffness	Longitudinal Stiffness	Torsional Stiffness	Rocking Stiffness	Rocking Stiffness
Value	1377664.6	1503172.1	1503172.1	15591908	9653425	9653425

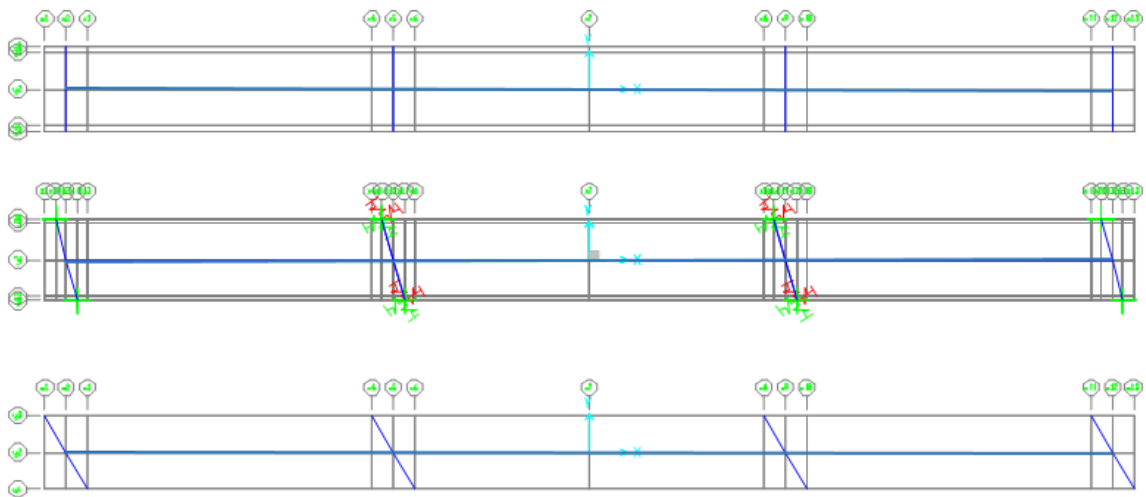


Figure 6.3. Variation of skew angle (0° - 15° - 30°).

6.3. Seismic Demand Representation

The seismic hazard analysis is aimed to provide strong-motion parameters at a selected site for the purpose of earthquake resistant design. There are generally two well-known approaches for the quantification of seismic hazard. One of them is the probabilistic seismic hazard analysis (PSHA), which accounts for all possible earthquake scenarios that could affect the site and results in hazard represented by ground motions parameters at reference ground conditions, such as peak ground acceleration and spectral accelerations. The other approach is the deterministic earthquake hazard assessment. In PSHA, deaggregation involves the determining earthquake variables, principally magnitude, distance and values of other random

variables defining seismic events that contribute to a selected seismic-hazard level (McGuire, 1995; Bazzurro and Cornell, 1999).

In terms of deterministic approach, the strong-motion parameters are estimated for the maximum credible earthquake, design-basis earthquake and operational level, assumed to occur at the closest possible distance from the site of interest, without considering the likelihood of its occurrence during a specified exposure period. Deterministic seismic hazard scenarios are based on varying site-to-source distance for strike-slip fault mechanism on rock site with $M_w=7.25$.

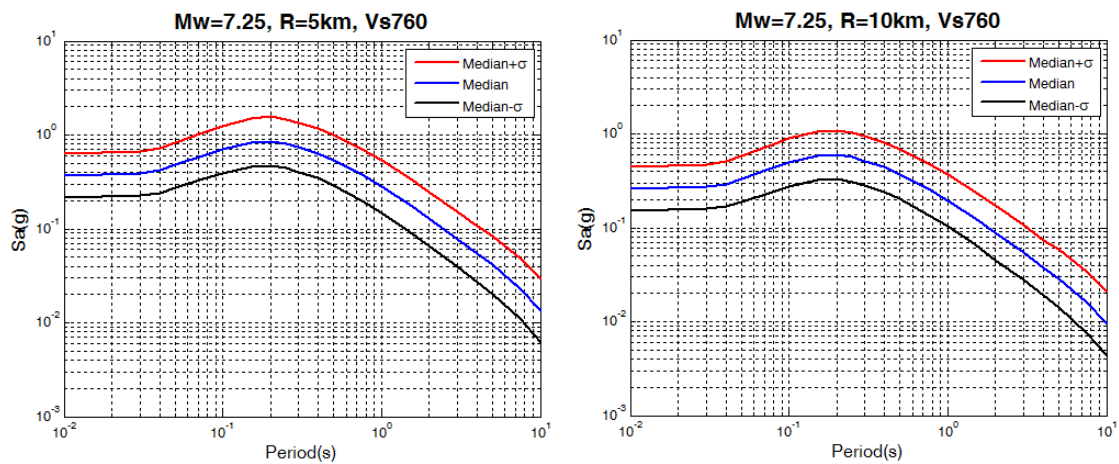


Figure 6.4. Target spectra for the seismically isolated bridge.

6.4. Selection and Scaling of Earthquake Ground motions for Parametric Studies

Nonlinear response history analysis is the most accurate method of analysis for the performance-based assessment and design of bridges. It stands for the inelastic response characteristics and dynamic actions in the structure. Selection and scaling of ground motion have been one of the most challenging and arguable issues for the proper representation of the seismicity. Since the ground motions serve as the main ingredient of structural analysis, seven pairs of ground motions are prepared for each earthquake level. Suite of ground motions do not have more than 2 pairs of recordings from the same earthquake to provide record-to-record variability while limiting the dominance of a particular event.

Careful ground motion selection can achieve the reduction in bias and variance of structural response as is gained by more advanced measures of ground motion intensity, while allowing the user to process the records using simple measures of intensity such as elastic spectral acceleration (Baker and Cornell, 2006). Web-based tool of PEER ground motion database is utilized for the selection and scaling of ground motions. A basic criterion used by the PGMD to select a representative acceleration time series is that the spectrum of the time series provides a “good match” to the user’s target spectrum over the spectral period range of interest. Period range of interest is related with the fundamental vibration frequency of the seismically isolated structure. Moreover, response spectral shape over a period range of interest has been found to be closely correlated to inelastic structural response in a number of studies (*e.g.* Shome *et al.* 1998; Cordova *et al.* 2001, Luco and Cornell 2007, Jalayer 2003, Baker and Cornell, 2004, Luco and Bazurro, 2004, Baker and Cornell 2005, 2006, 2008).

Deterministic seismic hazard scenario is calculated for varying source to site distances. Effect of pulse content in near-fault regions is assessed through the prepared sets of ground motions (Figure 6.4 and Figure 6.5). Selected ground motions for earthquake level with a 475 return period is shown from Table 6.3 to Table 6.4. Accordingly, the same procedure is applied for earthquake scenario with a 2475 year return period. Period range of interest for the scaling of the ground motion is limited between periods 1.5s to 3.75s, as long as the limits of scale factors are over the range 0.5 to 2.

The moment magnitudes for the seed ground motions are selected to be between 6.19 and 7.28; the site-to-source distances (Joyner and Boore distance) are between 0 and 19.1 km, and all the records are associated with Site Classes C and D per the 2010 AASHTO Specifications. Pairs of earthquake records, in Fault-normal and Fault-Parallel orientations, are applied to the bridge along the longitudinal and transverse directions, respectively. Then, the orientation of same ground motions are flipped to check if any significant change occurs. Contemporary design codes requires to check the effects of directionality on the structures that are located in the vicinity of active faults. Thus, the same suite of ground motions are oriented in the principal directions for the evaluation of directionality effects on the seismic isolators.

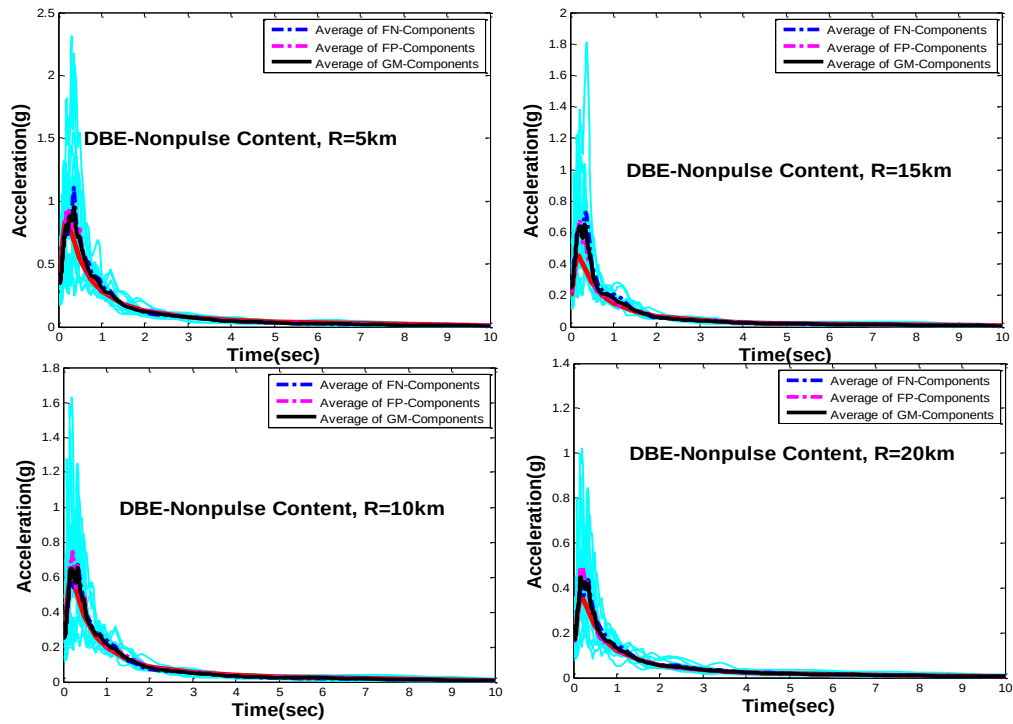


Figure 6.5. Scaled suite of ground motions with nonpulse content for DBE level earthquake.

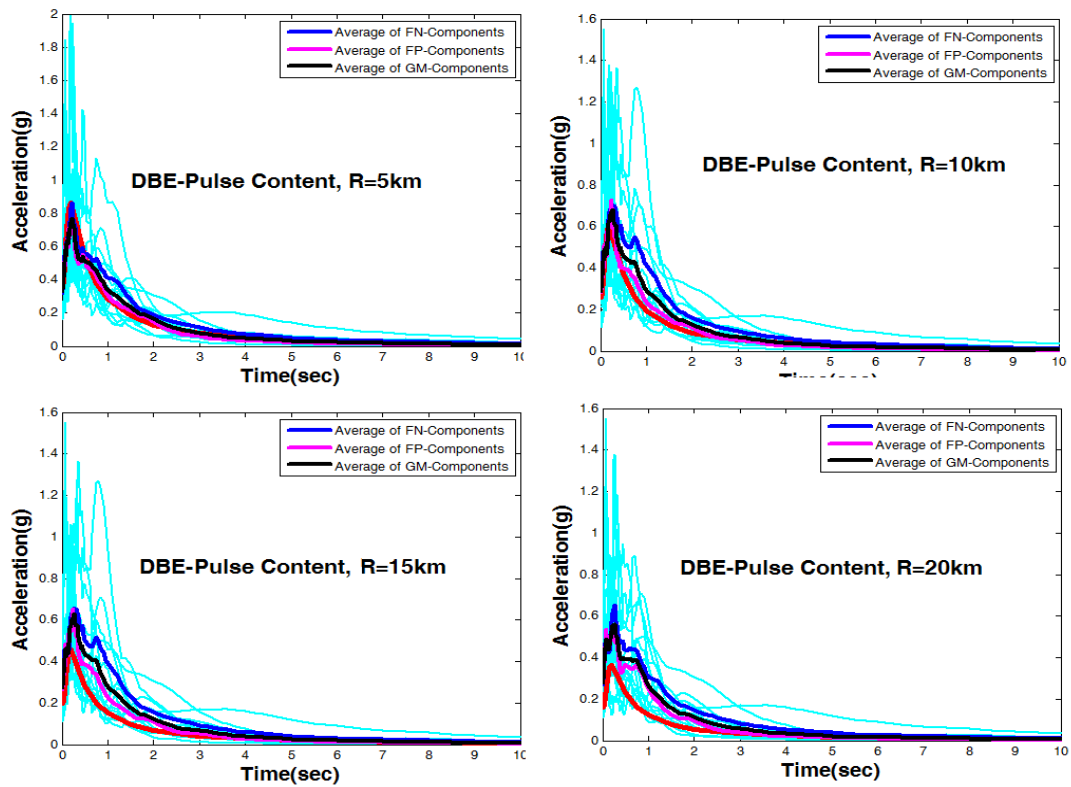


Figure 6.6. Scaled suite of ground motions with pulse content for DBE level earthquake.

Table 6.3. Selected earthquake records with nonpulse content for UHS with a 475 return period (R=0, 5 and 10 km).

SET	MSE	Pulse	Event	Year	Station	Mag	Mech.	Rjb (km)	Rrup (km)	Vs30 (m/s)
MR0	0.0209	0 0	Chalfant Valley-02	1986	Bishop - LADWP South St	6.19	SS	14.4	17.2	271.4
	0.0349	0 0	Superstition Hills-02	1987	Westmorland Fire Stat.	6.54	SS	13	13	193.7
	0.0466	0 0	Imperial Valley-06	1979	Calexico Fire Station	6.53	SS	10.4	10.4	231.2
	0.0503	0 0	Managua Nicaragua01	1972	Managua- ESSO	6.24	SS	3.5	4.1	288.8
	0.0531	0 0	Kobe- Japan	1995	Shin-Osaka	6.9	SS	19.1	19.1	256
	0.0731	0 0	Victoria- Mexico	1980	Chihuahua	6.33	SS	18.5	19	274.5
	0.0745	0 0	Chi-Chi- Taiwan-04	1999	CHY074	6.2	SS	6	6.2	553.4
MR5	0.0116		Imperial Valley-06	1979	Calexico Fire Station	6.53	SS	10.4	10.4	231.2
	0.0196	0 0	Chalfant Valley-02	1986	Bishop - LADWP South St	6.19	SS	14.4	17.2	271.4
	0.0348	0 0	Super. Hills-02	1987	Westmorland Fire St.	6.54	SS	13	13	193.7
	0.0519	0 0	Man.-Nicaragua01	1972	Managua- ESSO	6.24	SS	3.5	4.1	288.8
	0.0548	0 0	Kobe- Japan	1995	Shin-Osaka	6.9	SS	19.1	19.1	256
	0.064	0 0	Parkfield	1966	Cholame Shandon Array #5	6.19	SS	9.6	9.6	289.6
	0.0672	0 0	Chi-Chi- Taiwan-04	1999	CHY080	6.2	SS	12.4	12.5	680
MR10	0.0118	0 0	Imperial Valley-06	1979	Calexico Fire Station	6.53	SS	10.4	10.4	231.2
	0.0197	0 0	Chalfant Valley-02	1986	Bishop - LADWP South St.	6.19	SS	14.4	17.2	271.4
	0.0255	0 0	Parkfield	1966	Cholame Shandon Array #8	6.19	SS	12.9	12.9	256.8
	0.0347	0 0	Superstition Hills-02	1987	Westmorland Fire Stat.	6.54	SS	13	13	193.7
	0.0523	0 0	Managua-Nicaragua01	1972	Managua- ESSO	6.24	SS	3.5	4.1	288.8
	0.0553	0 0	Kobe- Japan	1995	Shin-Osaka	6.9	SS	19.1	19.1	256
	0.0569	0 0	Düzce- Turkey	1999	Lamont 1061	7.14	SS	11.5	11.5	481

Table 6.4. Selected earthquake records with nonpulse content for UHS with a 475 return period (R=15 km, R=20 km).

SET	MSE	Pulse	Event	Year	Station	Mag	Mech.	Rjb (km)	Rrup (km)	Vs30 (m/s)
MR15	0.012	0 0	Imperial Valley-06	1979	Calexico Fire Station	6.53	SS	10.4	10.4	231.2
	0.0199	0 0	Chalfant Valley-02	1986	Bishop - LADWP South St	6.19	SS	14.4	17.2	271.4
	0.026	0 0	Parkfield	1966	Cholame-ShandonArray #8	6.19	SS	12.9	12.9	256.8
	0.051	0 0	Düzce- Turkey	1999	Lamont 1061	7.14	SS	11.5	11.5	481
	0.053	0 0	Superstition Hills-02	1987	Westmorland Fire St.	6.54	SS	13	13	193.7
	0.0531	0 0	Managua-Nicaragua-1	1972	Managua- ESSO	6.24	SS	3.5	4.1	288.8
	0.0553	0 0	Parkfield	1966	Temblor pre-1969	6.19	SS	16	16	527.9
MR20	0.0123	0 0	Imperial Valley-06	1979	Calexico Fire Station	6.53	SS	10.4	10.4	231.2
	0.0201	0 0	Chalfant Valley-02	1986	Bishop - LADWP South St	6.19	SS	14.4	17.2	271.4
	0.0266	0 0	Parkfield	1966	Cholame-ShandonArray#8	6.19	SS	12.9	12.9	256.8
	0.051	0 0	Düzce- Turkey	1999	Lamont 1061	7.14	SS	11.5	11.5	481
	0.0531	0 0	Superstition Hills-01	1987	Wildlife Liquef. Array	6.22	SS	17.6	17.6	207.5
	0.0625	0 0	Chi-Chi- Taiwan-04	1999	CHY024	6.2	SS	19.7	19.7	427.7
	0.0631	0 0	Managua-Nicaragua-1	1972	Managua- ESSO	6.24	SS	3.5	4.1	288.8

Table 6.5. Selected earthquake records with pulse content for UHS with a 475 return period (R=0, 5 and 10 km).

SET	MSE	Pulse	Tp	Event	Year	Station	Mag	Mech.	Rjb (km)	Rrup (km)	Vs30 (m/s)
MR0	0.0101	1 1	5.9 2	Imperial Valley-06	1979	El Centro Differential Array	6.53	SS	5.1	5.1	202.3
	0.0408	1 0	2.4 --	Superstition Hills-02	1987	El Centro Imp. Co. Cent	6.54	SS	18.2	18.2	192.1
	0.043	0 1	-- 5.6	Düzce- Turkey	1999	Düzce	7.14	SS	0	6.6	276
	0.0683	1 0	5.1 --	Landers	1992	Lucerne	7.28	SS	2.2	2.2	684.9
	0.0743	1 1	2.7 2.2	Erzincan-Turkey	1992	Erzincan	6.69	SS	0	4.4	274.5
	0.1094	1 0	1.4 --	Kobe- Japan	1995	Takarazuka	6.9	SS	0	0.3	312
	0.4253	1 1	0.95 1.1	Morgan Hill	1984	Coyote Lake Dam	6.19	SS	0.2	0.5	597.1
MR5	0.0097	1 1	5.9 2	Imperial Valley-06	1979	El Centro Differential Array	6.53	SS	5.1	5.1	202.3
	0.0408	1 0	2.4 --	Superstition Hills-02	1987	El Centro Imp. Co. Cent	6.54	SS	18.2	18.2	192.1
	0.0664	1 0	5.1 --	Landers	1992	Lucerne	7.28	SS	2.2	2.2	684.9
	0.0863	0 1	-- 0.91	Düzce- Turkey	1999	Bolu	7.14	SS	12	12	326
	0.1244	1 1	2.7 2.2	Erzincan- Turkey	1992	Erzincan	6.69	SS	0	4.4	274.5
	0.1943	1 0	1.4 --	Kobe- Japan	1995	Takarazuka	6.9	SS	0	0.3	312
	0.4225	1 0	1.2 --	Morgan Hill	1984	Gilroy Array #6	6.19	SS	9.8	9.9	663.3
MR10	0.0252	1 0	7.4 --	Imperial Valley-06	1979	El Centro Array #11	6.53	SS	12.4	12.4	196.2
	0.0406	1 0	2.4 --	Superstition Hills-02	1987	El Centro Imp. Co. Cent	6.54	SS	18.2	18.2	192.1
	0.1035	1 0	5.1 --	Landers	1992	Lucerne	7.28	SS	2.2	2.2	684.9
	0.2393	0 1	-- 0.91	Düzce- Turkey	1999	Bolu	7.14	SS	12	12	326
	0.4237	1 0	1.2 --	Morgan Hill	1984	Gilroy Array #6	6.19	SS	9.8	9.9	663.3
	0.424	1 1	2.7 2.2	Erzincan- Turkey	1992	Erzincan	6.69	SS	0	4.4	274.5
	0.4778	1 0	1.0 --	Kobe- Japan	1995	KJMA	6.9	SS	0.9	1	312

Table 6.6. Selected earthquake records with pulse content for UHS with a 475 return period (R=15 and 20 km).

SET	MSE	Pulse	Tp	Event	Year	Station	Mag	Mech.	Rjb (km)	Rrup (km)	Vs30 (m/s)
MR15	0.0319	1 0	7.4 --	Imperial Valley-06	1979	El Centro Array #11	6.53	SS	12.4	12.4	196.2
	0.1076	1 0	2.4 --	Superstition Hills-02	1987	El Centro Imp. Co. Cent	6.54	SS	18.2	18.2	192.1
	0.2757	1 0	5.1 --	Landers	1992	Lucerne	7.28	SS	2.2	2.2	684.9
	0.4261	1 0	1.2 --	Morgan Hill	1984	Gilroy Array #6	6.19	SS	9.8	9.9	663.3
	0.5183	0 1	-- 0.91	Düzce- Turkey	1999	Bolu	7.14	SS	12	12	326
	0.8077	1 1	2.7 2.2	Erzincan-Turkey	1992	Erzincan	6.69	SS	0	4.4	274.5
	0.8413	1 0	1.0 --	Kobe- Japan	1995	KJMA	6.9	SS	0.9	1	312
MR20	0.0741	1 0	2.3 --	Imperial Valley-06	1979	Agrarias	6.53	SS	0	0.7	274.5
	0.2533	1 0	2.4 --	Superstition Hills-02	1987	El Centro Imp. Co. Cent	6.54	SS	18.2	18.2	192.1
	0.4287	1 0	1.2 --	Morgan Hill	1984	Gilroy Array #6	6.19	SS	9.8	9.9	663.3
	0.5013	1 0	5.1 --	Landers	1992	Lucerne	7.28	SS	2.2	2.2	684.9
	0.7517	1 1	0.95 1.1	Morgan Hill	1984	Coyote Lake Dam (SW Abut)	6.19	SS	0.2	0.5	597.1
	0.8261	0 1	-- 0.91	Düzce- Turkey	1999	Bolu	7.14	SS	12	12	326
	1.1949	1 1	2.7 2.2	Erzincan- Turkey	1992	Erzincan	6.69	SS	0	4.4	274.5

6.5. Seismic Isolators

Seismic isolation technique has been widely used as a design and rehabilitation method for bridges. Curved surface friction sliders and lead rubber bearings are two commonly used isolators for the enhancement of the earthquake resistance of bridges. Critical appraisal is given to curved surface friction sliders regarding to the recent advances and innovations in the industry. The latest version of the sliding system is patented as Triple friction pendulum (TFP) by EPS (Zayas and Low, 2006) and the overall behavior of this device have been captured by several mathematical models (e.g. Fenz and Constantinou, 2008, Sarlis and Constantinou 2011, Becker and Mahin, 2012). Properly configured bearings with multiple concave curved surfaces have a distinct force-displacement relationship when compared with the conventional type of curved surface friction slider. In other words, transition of force-displacement hysteresis of TFP is adaptive in which exhibits desirable changes in stiffness and strength with increasing amplitude of displacement. Shake table test results are in good agreement with mathematical models that stand for the adaptive force-displacements of the multi-spherical sliding system (Morgan, 2007 and Fenz and Constantinou, 2008). However, most of the commercial programs are not capable of covering this complex force-displacement relationship with the currently available elements.

Two approaches can be employed for modelling the overall behavior of multi-concave friction sliders while tracing the peculiar force-displacement hysteresis. The first one is the implementation of the new device with a complex force-hysteresis rule to trace the overall behavior (e.g. Fenz and Constantinou, 2008, Morgan and Mahin, 2010). This goal can be achieved by developing user-defined elements as is in OpenSees (e.g. Becker and Mahin, 2012, Dao et al., 2013). In contrast, the second approach relies on the properly configured existing elements for representing the overall force-displacement loops. Analyses are carried out by several researchers in terms of series and parallel connections of existing link elements particularly in SAP2000 (Fenz and Constantinou 2008, Sarlis and Constantinou, 2010). It is worth mentioning that, the assembly of conventional curved surface sliding bearings, gap elements, rigid link elements, and releases in series model may take longer computational time than device specific solutions. The latter approach were adopted to model TFP bearing while the conventional curved surface friction slider and lead rubber bearings are modeled by the provided existing rubber and friction link elements.

SAP2000 already has a built-in conventional curved surface friction sliders that exhibit rigid linear behavior. Three curved surface friction slider are connected in series while the gap elements have parallel connection with the two of the curved surface friction sliders as illustrated in Section 5(Figure 5.19). Recommendations are made for series modeling of TFP bearing in SAP2000 (Fenz and Constantinou, 2008, Tsai, 2014). Comparative analyses of the device specific model and the assembly of link elements in SAP2000 are illustrated through analysis of a simple seismically isolated frame structure.

Performance of lead rubber bearing and sliding type of bearings in near-fault locations will be assessed by performing single mode method of analysis and nonlinear response history analysis. Although the normal load acting on the bearings at pier and abutment locations could be different, identical bearing dimensions are selected for the simplicity. Conventional curved surface friction slider and triple friction pendulum bearing were used as the sliding type of isolators. The geometric configuration of curved surface friction sliders that are considered provided in Figure 6.7.

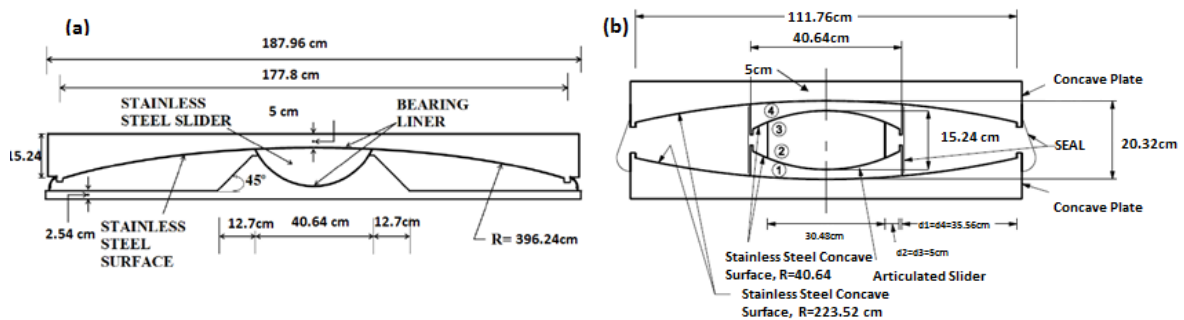


Figure 6.7. Sliding type of bearings that are incorporated in the parametric studies.

Conventional curved surface friction slider has a single sliding regime while the TFP may exhibit 5 different sliding regimes as shown in Figure 6.8. Beneficial features of TFP may be used in high seismicity regions to reduce the cost issue.

By assuming that the series model is the most accurate mathematical model for the TFP (Sarlis and Constantinou, 2010), has shown that the parallel connection of conventional curved surface friction sliders together with the flat sliding bearings may also cover the overall behavior with a faster computational time until the restraining displacement is reached.

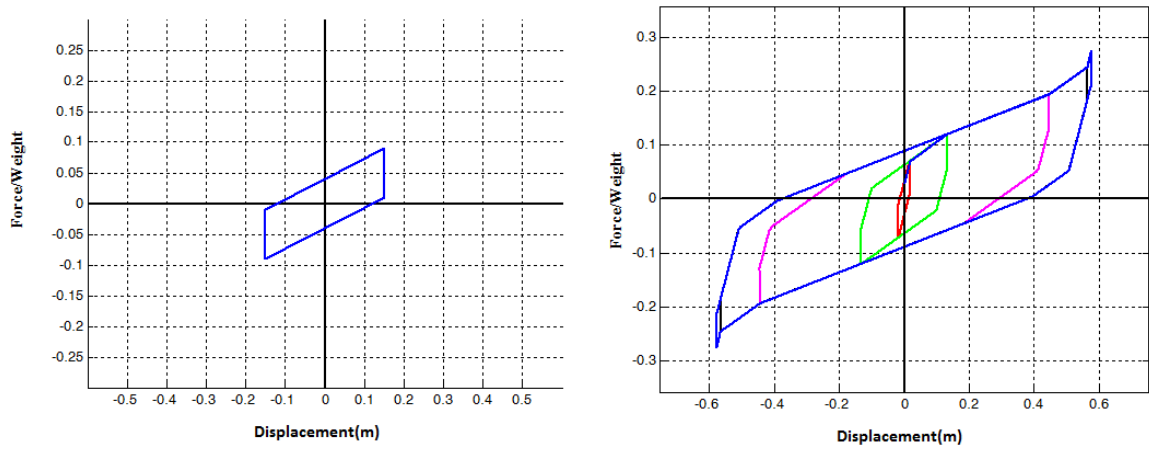


Figure 6.8. Comparison of sliding regimes of conventional curved friction slider and the TFP bearing with series connection.

Considering the vast number of nonlinear response history studies, parallel model of the TFP is implemented to the seismically isolated bridge model. Idealized force-displacement hysteresis loop of the implemented parallel model is shown in Figure 6.9.

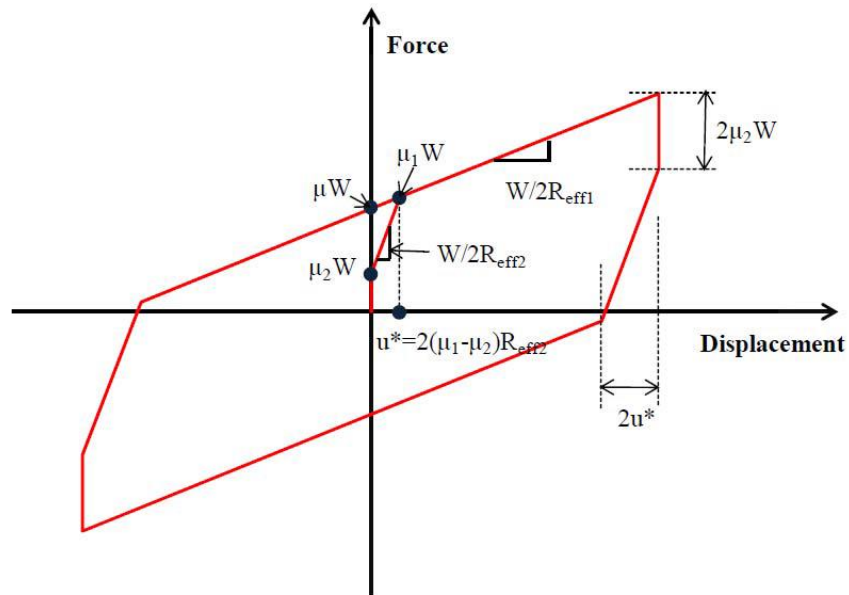


Figure 6.9. Force displacement hysteresis of specially configured multi-curved surface friction slider in this study (Fenz and Constantinou, 2008).

Table 6.7 Friction parameters of the selected multi-spherical sliding bearing

Property	Abutment Bearing	Pier Bearing
$\mu_{1L}=\mu_{4L}$	0.09	0.06
$\mu_{2L}=\mu_{3L}$	0.087	0.035
$\mu_{1U}=\mu_{4U}$	0.15	0.06
$\mu_{2U}=\mu_{3U}$	0.144	0.035

Geometric configuration of LRB used in analysis and mechanical characteristics are respectively, shown in Figure 6.9.

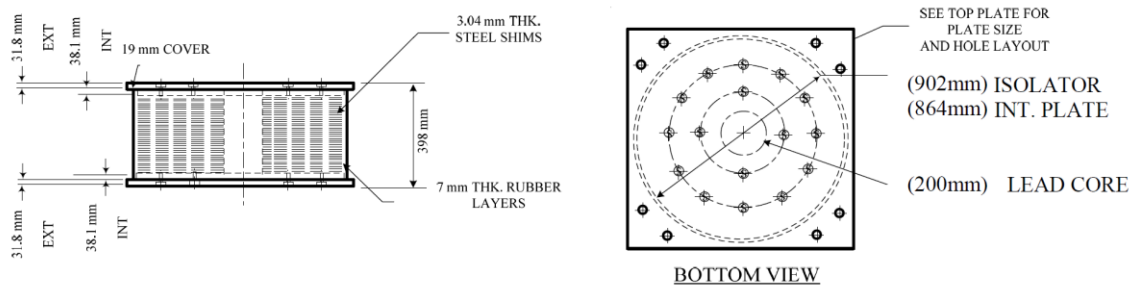


Figure 6.10. Geometric configuration of the utilized LRB bearing.

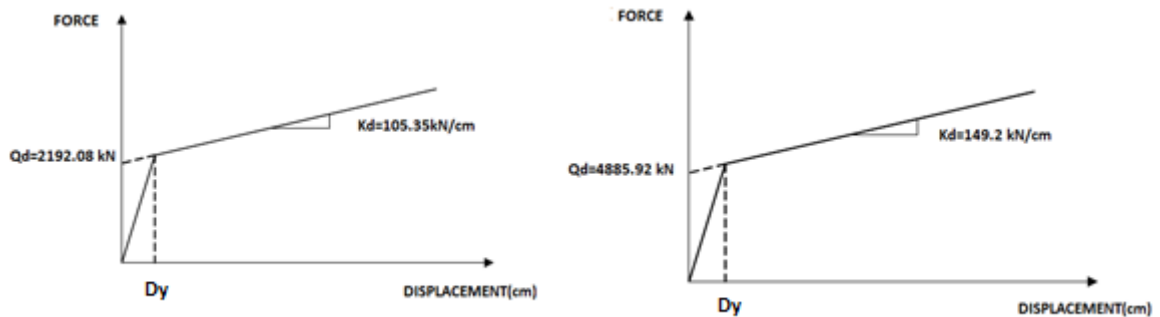


Figure 6.11. Idealized bilinear force-displacement relationship for the single mode analysis of LRB.

6.6. Dynamic Response History Analysis

The analysis was performed with the programs SAP2000, Version 14.2.2 (CSI, 2010) and OpenSees (McKenna et al., 2007). The Fast Nonlinear Analysis (FNA) method with a large number of Ritz vectors is incorporated for the dynamic response history analysis in SAP2000, to obtain results that are essentially exact. Concentration of nonlinearity at isolation interface was provided by the properly assigned assembly of nonlinear link elements. The global damping matrix is constructed by specifying 2% critical modal damping in each mode of vibration.

The results presented in Section 8 consist of the isolator displacement results and the longitudinal and the transverse shear forces at the pier and abutment locations. The peak displacement response is the maximum out of all 4 abutment isolators and all 4 pier isolators. The forces given are the maximum for individual bearings at the abutment and pier locations. Nonlinear response history analyses are performed for bridge models used in single mode method of analysis and for the 3D finite element bridge models to check the variation of results. It should be noted that the analysis does not include the effects of accidental torsion.

7. EVALUATION OF SEISMIC ISOLATION APPLICATION FOR AN EXISTING REINFORCED CONCRETE HIGHWAY BRIDGE THROUGH PSEUDO-DYNAMIC TEST

7.1. Introduction

Most of the bridges in Italy were constructed in the late 60s and early 70s without adequate seismic design considerations. Accordingly, Italian Civil Protection (DPC) has launched a broad-spectrum nationwide seismic risk mitigation scheme with the seismological and engineering support of the National Institute for Geophysics and Volcanology (INGV), ReLuis and EU Centere in Italy (Dolce, 2012). Similar efforts were also put into practice in a comprehensive research program by Italian ReLuis consortium (Pinto and Mancini, 2009) which was aimed to formulate pre-normative European guidelines for the performance assessment of existing bridges. On the basis of previous and currently ongoing studies Rio-Torto Viaduct can be considered as one of the representative of this type of structure in Europe due to its material characteristics and design assumptions that are in compliant with the contemporary seismic design codes. In this sense RETRO project is proposed as a Translational Access (TA) facility of the Seismic Engineering Research Infrastructures for European Synergies (SERIES) project under the Seventh Framework Program of the European Commission.

The RETRO project aims to contribute to the risk mitigation studies by performing large scale tests and extended analytical studies. Seismically isolated and as-built model of the Rio-Torto Viaduct are considered to evaluate the effectiveness of the utilized earthquake protection system. Main concern in large scale or full scale testing is to identify the progressive collapse mechanism which might not be identified by small-scale tests. KOERI is one of the contributing partners of this project in terms of selection and scaling of ground motions and performance assessment of seismic isolators. Outcome of the research study will address the efficiency of the proposed retrofitting methods and give an insight about nonlinear behavior of structural elements through Pseudo-Dynamic (PsD) testing protocol. Two piers of the considered bridge were constructed and tested by means of PsD substructuring technique.

Details of the test protocol and applied methodology can be found in various research studies (*e.g.* Pinto *et al.*, 2004, Paolacci *et al.*, 2013, Bursi *et al.*, 2013).

7.2. Description of the Bridge

The Rio-Torto Viaduct is an old RC bridge located in Emilia-Romagna region as a connection link between Florence and Bologna has a total length of 421.1m (Figure 7.1). It consists of 13 spans with two independent roadways supported by 12 pairs of portal frame bents. Piers of the bridge, composed of circular solid and hollow sections with varying diameters, are connected to cap beams at the top of each bent. The height of piers are in the range of 13.8 m to 41 m as illustrated in Figure 7.2. Rio-Torto Viaduct is considered as a critical lifeline with class of IV with a nominal life of 100 years with respect to the seismic design code of Italy (NTC-08). Testing of Rio-Torto Viaduct took place in ELSA Laboratory of JRC (Ispra, Italy) and the two piers were built (scale 1:2.5) and tested by means of PsD substructuring technique.



Figure 7.1. (a) Location of Rio-Torto Viaduct (b) General view of piers (c) Deck geometry.

The recent 2012 Emilia Earthquake (M_w , 5.9) struck the Emilia-Romagna region which is approximately located 36 km north of the city Bologna. Main shock and after shock sequences of the 2012 Emilia Earthquake certainly supported the urgent need of assessing the performance of critical lifelines. Thus, particular emphasis is given to 2012 Emilia Earthquake where the 20th and 29th May 2012 records were elaborately used as input ground motions for the PsD test.

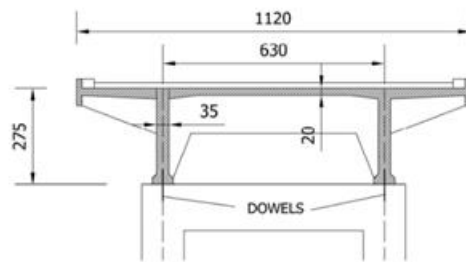
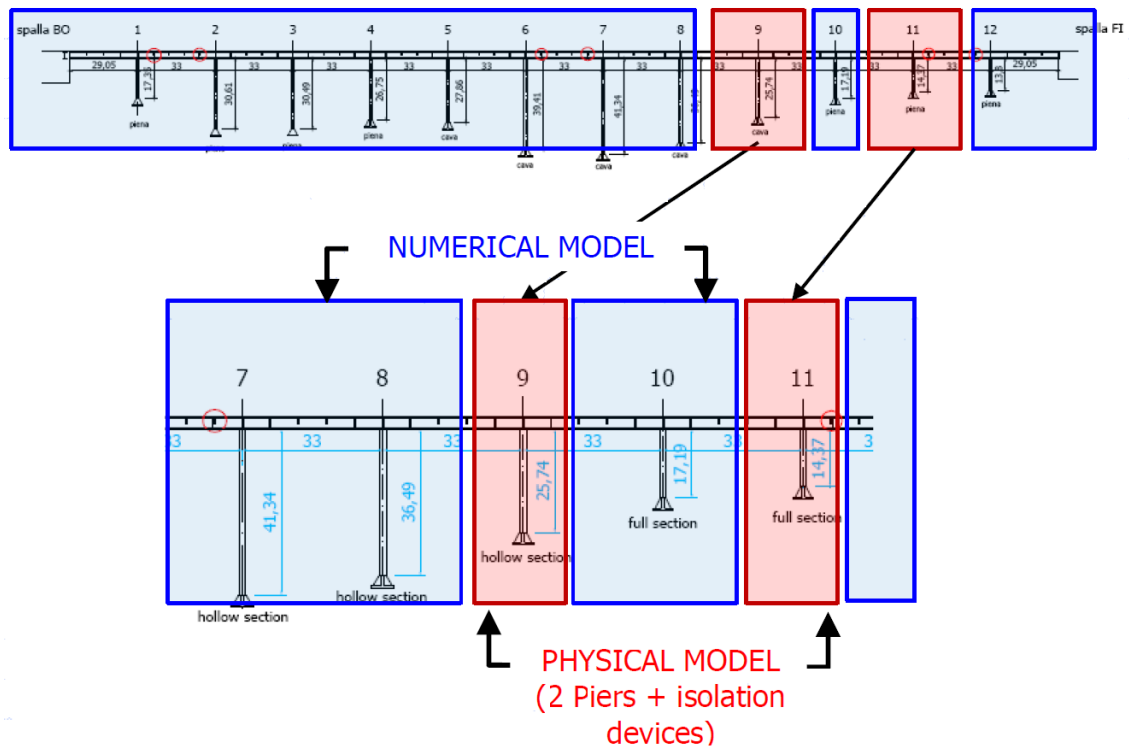


Figure 7.3. a) Cross section of the deck b) Gerber Saddles.

Structural properties of the deck are given in Table 7.1.

Table 7.1. Properties of the superstructure.

Superstructure	Calculated
Axial Area (m^2)	4.6621
Torsional Inertia (m^4)	9.5635
Bending Inertia about Axis 2 (m^4)	51.87
Bending Inertia about Axis 3 (m^4)	3.466

Since the available material properties of the structural element is limited, the nominal characteristic strength of concrete and steel (AQ42) are assumed to be 30 MPa and 350 MPa, respectively. AQ42 was one of the most commonly used plain steel bars for the construction

of RC bridges in that era. Test specimens are prepared with respect to these stated values. Details of the longitudinal steel bars for piers 9 and 11 are illustrated in Figure 7.4. For the Pier 9, external and internal diameter of the hollow section are equal to 160 cm and 100 cm. Diameter of the solid section equals to 120cm for the Pier 11. Total weight acting on Pier 9 and Pier 11 is calculated as 5600 kN. In-situ tests are required to verify assigned material properties for the structural elements.

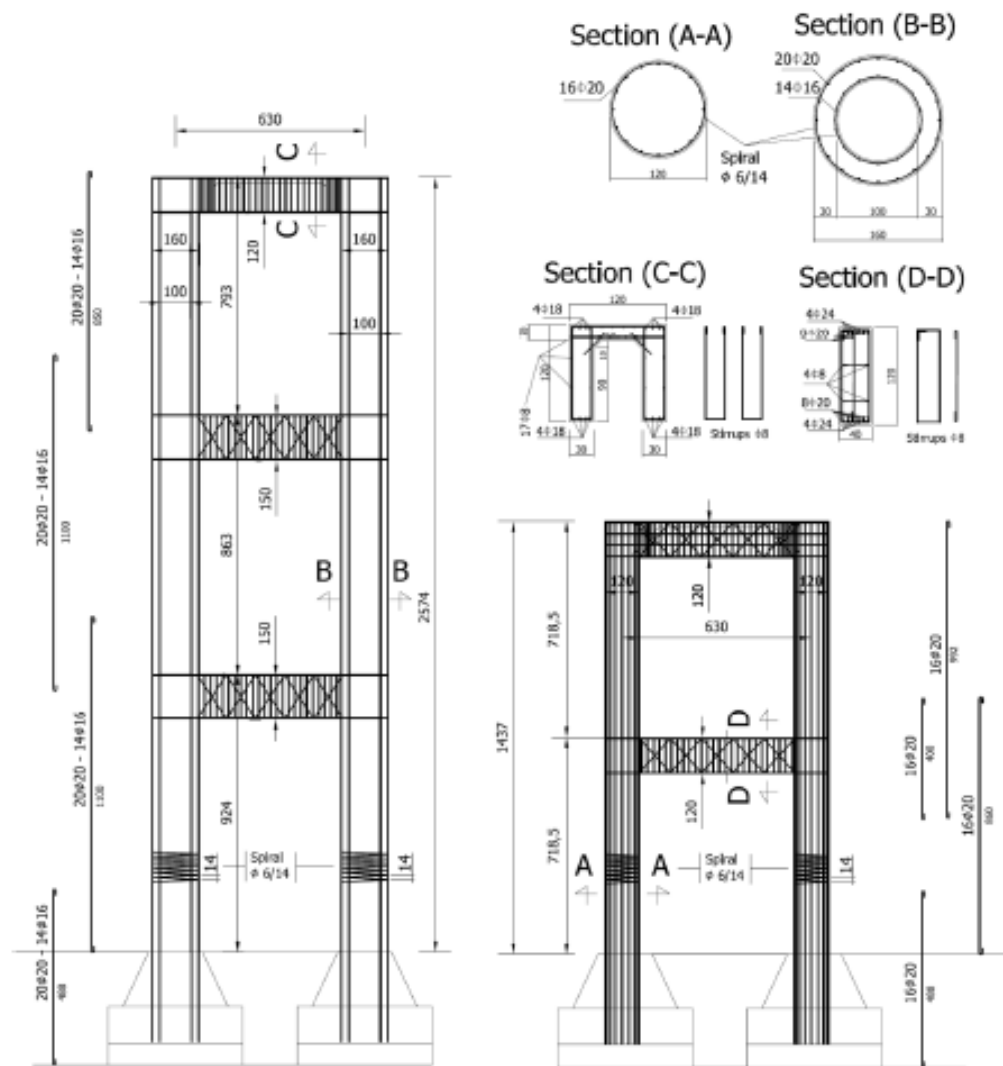


Figure 7.4. Reinforcement details of Pier 9 and Pier 11.

7.3. Representation of the Seismic Demand and Review of Code Provision in Italy

Dynamic response history analysis and tests of large scale structures require the proper selection and scaling techniques. Ground motion selection procedures are generally associated

with a target response spectrum which depends on the identification of the seismic hazard. Current practice for the record selection rely on the deaggregation of the uniform hazard spectrum at the fundamental vibration period of the structure (T_1), to yield the modal magnitude (M) and site-to-source distance (r) pairs, the ground motions are then selected corresponding to the modal $[M, r]$ pair and local site conditions. The historical seismicity of zone 913 indicate that the major past earthquakes had magnitudes ranging from 6.13 to 6.6 and that there is no any active fault in the region.

The PSHA study in Italy, conducted by INGV, are utilized in the Italian seismic design code for 9 return periods (NTC-08, 2008). Current version of seismogenic zones in Italian territory, ZS9 (INGV) and earthquake catalogue of CPTI04 (INGV) are incorporated together with appropriate ground motion prediction equations (Ambraseys *et al.*, 2003, Sabetta and Pugliese, 1996) for the PSHA study (Figure 7.5). NTC-08 introduces four design spectra named as SLO, SLD, SLV and SLC that correspond to the serviceability and ultimate limit states.

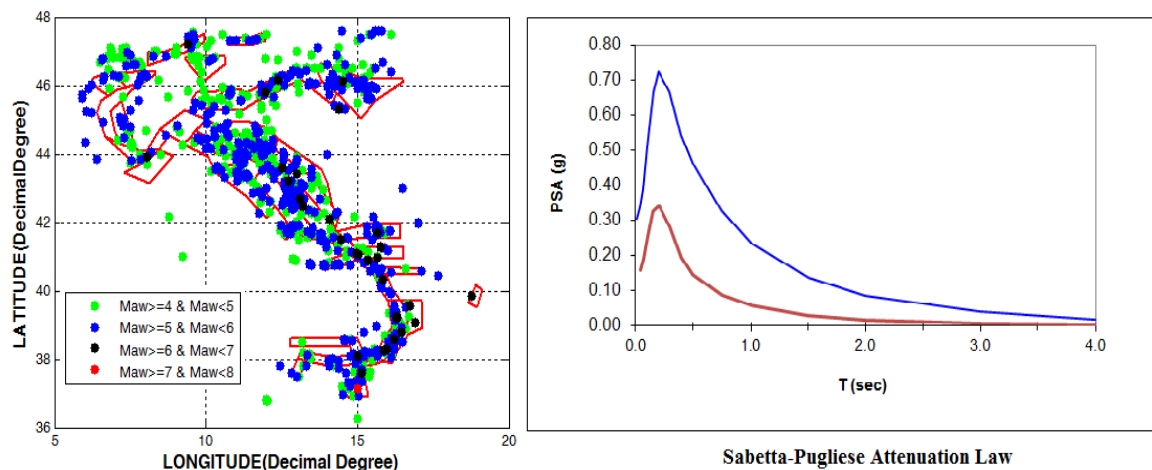


Figure 7.5. (a) ZS9, Seismogenic zones and seismicity of Italian territory with respect to CPTI04 catalogue (b) Prediction of ground motion characteristics for magnitude range=5-6 and $R_{epi}=5$ km by performing Sabetta and Pugliese attenuation law(1996).

Italian seismic design code requires PGA value as a reference quantity for the establishment of the design spectra. Therefore, PGA value is computed at each node which is regularly spaced with 5 km intervals. The Annex B of NTC-08 provides a_g (PGA) values for the entire national territory. A new tool is developed for the ease of extraction of required

parameters in the vicinity of the interested structure. In particular, PGAs of 0.25g (Life safety limit state - SLV) and 0.33g (Collapse limit state - SLC) are obtained, respectively for probabilities of occurrence of 5% and 2% over a 50 year period. PGA value and established uniform hazard spectra of the Rio-Torto Viaduct in Emilia-Romagna region are shown in Figure 7.6 and Figure 7.7.

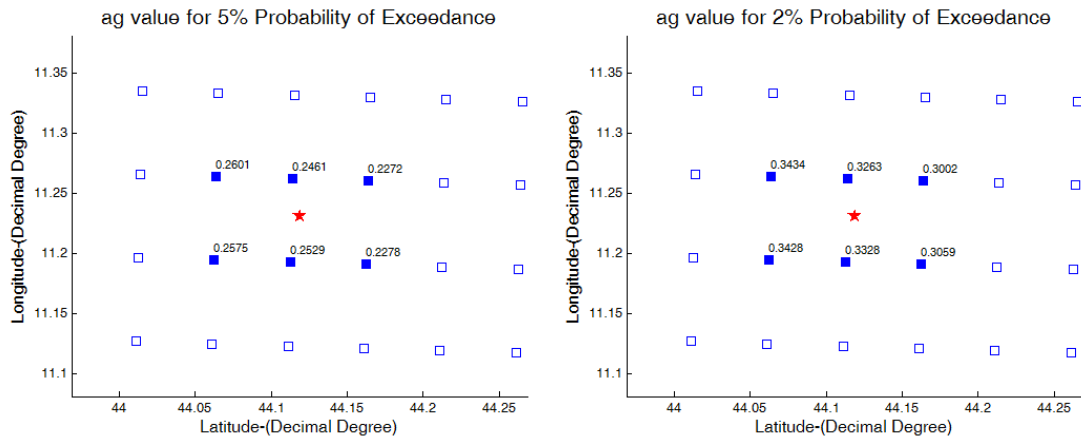


Figure 7.6. Location of Rio-Torto Viaduct and expected PGA values at nearby grids (NTC-08).

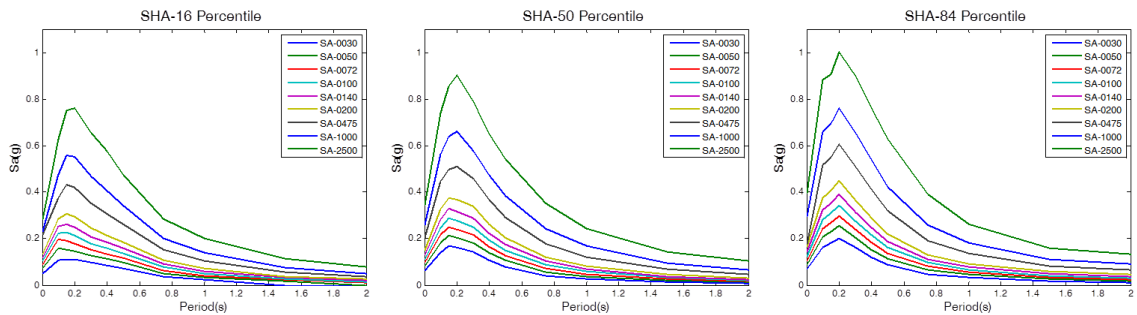


Figure 7.7. UHS (Uniform Hazard Spectrum) for 16-50-84 percentile in the vicinity of the Rio-Torto Viaduct.

PGA values of UHS spectra are in good agreement with the results of an in-house seismic hazard program, MathHazard (Gianini, 2000). This software utilizes Gutenberg-Richter recurrence relationship and Sabetta-Pugliese ground motion prediction equation for Italy. Results of MathHazard program for Rio-Torto Viaduct are illustrated in Figure 7.8. The national reference seismic hazard model MPS04 (INGV) is characterized by the expected PGA values with a 10% probability of exceedance in 50 years that ranges between 0.10 g and 0.15 g which is a medium level of seismic hazard in Italy.

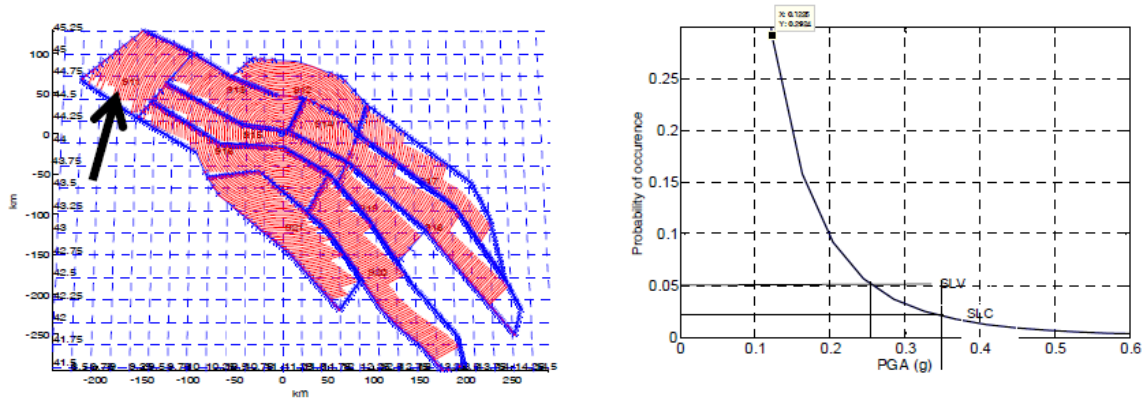


Figure 7.8. Location of Rio-Torto and probability of exceedance values for PGA.

7.4. Selection and Scaling of Appropriate Ground Motions for NRHA

Two-well known approaches have been commonly used in practice for matching the scaled ground motions to UHS for nonlinear response history analysis (NRHA). First one considers the amplitude or frequency scaling of single-component ground motions to exactly match to a target spectrum. The second procedure uses the 5% damped average value of the square root of the sum of squares of pairs of ground motions which is not less than 1.3 times of the UHS over the periods from $0.2 T_1$ to $1.5 T_1$ (ASCE/SEI 7-10). The latter is originally developed for seismically isolated structures. Linear amplitude scaling procedure is employed in this study to have the minimum manipulation on the selected suite of ground motions.

PGMD is sought to find representative input motions that fit to UHS spectra with a 2% probability of exceedance in Emilia-Romagna region. The PGMD have a web-based tool that searches records in the database with the spectral shapes similar to the target spectrum. Most of the engineering structures are investigated through a broader period range of interest rather than only considering the fundamental period of vibration (T_F) of the structure. The seismic design guidelines recommend the period range of interest to be between $0.2 T_F$ and $1.5 T_F$. Upper bound values of period range of interest can be extended on the basis of ductility of the structure. As a preliminary selection methodology, minimum deviation of spectral values from UHS is the main concern over the period range of interest. Amount of deviation in terms of spectral values can be calculated by using Equation 7.1. In addition to well-known procedures,

methods that stand for weight factors at particular periods might be used to obtain suitable sets of records, too (*e.g.*, PEER, 2010).

$$\delta = \sqrt{\frac{1}{N} \sum \left(\frac{Sa_{med}(Ti) - Sa_s(Ti)}{Sa_s(Ti)} \right)^2} \quad (7.1)$$

N denotes the number of considered spectral values between the period ranges of interest, $Sa_s(Ti)$ is the spectral value of the target spectrum and the $Sa_{med}(Ti)$ parameter is the mean spectral value of the evaluated ground motions. For limiting the number of ground motions in nonlinear dynamic response history analysis, a proxy SDOF model is constructed to determine the damage potential of selected ground motions. The nonlinear response of the SDOF model is represented by bilinear force-deformation relationships as shown in Figure 7.12.

The Emilia Earthquake of May 20, 2012 (M_L 5.9, INGV; M_W 6.11, and May 29, 2012 (M_L 5.8, INGV; M_W 5.96) struck the Emilia-Romagna region. Earthquake sequence of Emilia-Romagna region which is in close proximity to the location of the bridge made our attention to focus on the seismicity of the region. Main concern was the amount of damage accumulation on the Rio-Torto Viaduct. Main shocks and aftershock sequences between dates May 20th 2012 to June 18 2012 are shown in Figure 7.9. Total number of recorded events is 246 during this 1 month period while 6 of them have magnitudes higher than 5. Because of these reasons it was essential to use records from Emilia events in the Pseudo-Dynamic (PsD) test of the Rio-Torto Viaduct.

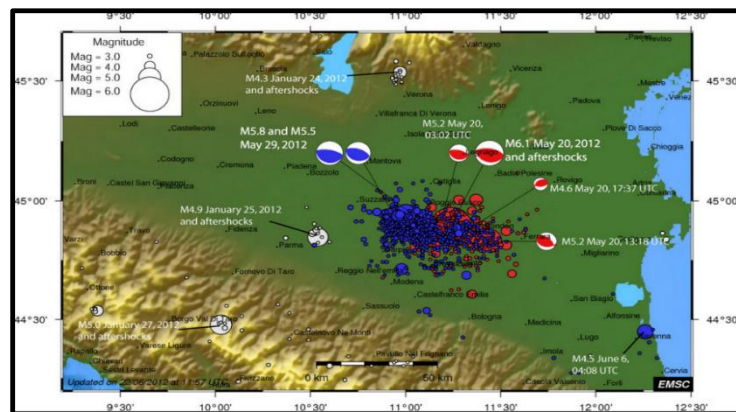


Figure 7.9. Distribution of earthquake sequences in close proximity to Rio-Torto Viaduct (<http://www.emsc-csem.org/>).

7.5. Retrofitting Scheme of the Rio-Torto Viaduct

Seismic isolation method is one of the proposed retrofitting schemes for Rio-Torto Viaduct which is superior to its conventional counterparts in terms of maintaining the substructure in the elastic range. The curved surface friction slider bearing is adapted for the seismic isolation of Rio-Torto Viaduct. Large displacement demands on the expansion joints at abutments is one of the problems in the longitudinal direction of bridges with seismic isolation. The sliding type of isolator aims to accomplish following three essential objectives:

1. Preserving the piers in the elastic range
2. Reducing the displacement demand at expansion joints
3. Obtaining a regular deck deformation pattern in comparison with the as-built model

The final design of isolators were implemented into the 3 dimensional model of the Rio-Torto Viaduct for the evaluation of adequacy of provided isolators by means of nonlinear response history analysis.

Since Rio-Torto Viaduct is located in a moderate seismic zone, conventional curved surface friction slider is the most suitable bearing among currently available sliding bearings in practice due to ease of adaptation to PsD test technique. Bilinear force-displacement relationship can be calculated by using Equation 5.7 under constant vertical load. Needless to say that larger force-displacement demands in high seismicity regions can certainly be accommodated by using double concave or triple concave friction pendulum. However, triple concave friction pendulum bearing has more complex force-displacement hysteresis with a varying strength and stiffness properties in conjunction with increasing amplitude of displacement.

Curved surface friction sliders and the associated design parameters were provided by Alga (Sartori, 2011). Geometric configuration and force-deformation hysteresis of the single concave sliding bearing is shown in Figure 7.10.

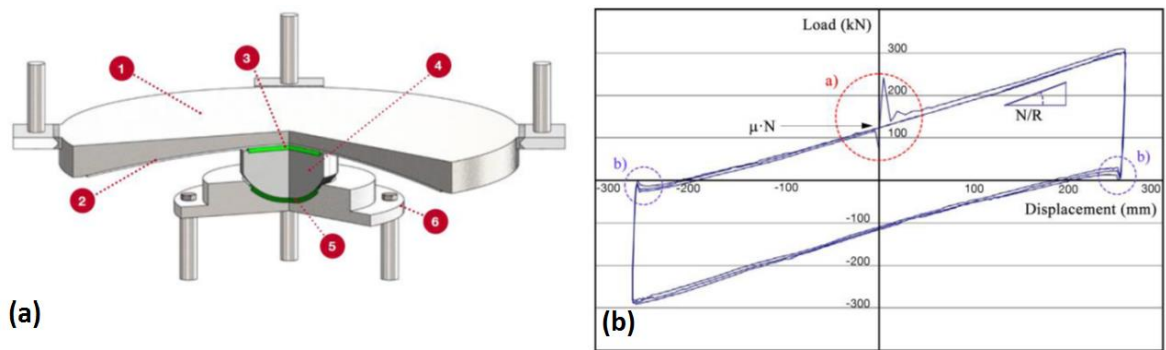


Figure 7.10. (a) Geometric configuration of curved surface sliding bearing (b) Force-displacement hysteresis of selected isolator (Courtesy of Alga).

The curved surface friction slider used in tests is designed by considering single mode method of analysis. Moreover, another check was performed by Della Corte *et al.* (2013) according to proposed displacement-based design method (Priestly, 2007). It is worth noting that applied methodology can be easily extended by applying minor modifications for different earthquake-protection devices that may exhibit an idealized bilinear hysteretic behavior. The single curved friction slider used has a 3m radius of curvature with a 4 % coefficient of friction. Initial yield displacement is assumed to be 0.5mm. Adequacy of the mechanical characteristics and the stated parameters of curved surface friction slider is checked through analytical studies. Two records of 2012 Emilia Earthquake are used as input motions for the nonlinear response history analyses.

7.6. Analytical Models of the Rio Torto Viaduct

Piers of the Rio-Torto Viaduct consist of moment-resisting portal frames with transverse beams at each bent. All bridge piers have plain steel bars and further investigation is conducted in the task of RETRO project. Results of previous quasi-static experimental tests and numerical analyses were exhibited the susceptibility of transverse beams to shear failures as shown in Figure 7.11 (Giannini and Paolacci, 2009).

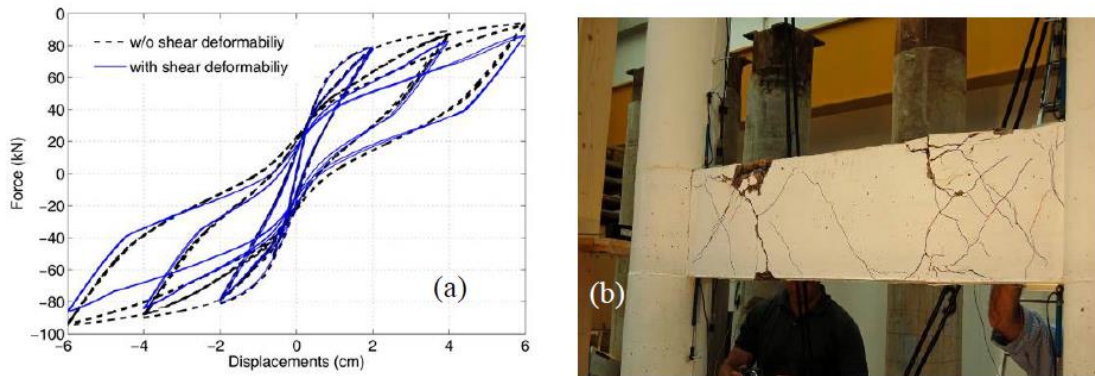


Figure 7.11. (a) Force deformation cycles of Pier 12 (b) Observed damages at transverse beams in Pier 12 under quasi-static tests (Giannini and Paolacci, 2009).

Seismic isolation methods aim to avoid such undesirable damages at structural elements. Previous quasi-static test results of Pier 12 are implemented into the developed analytical models of PsD test. In the preliminary analysis phase, SDOF bilinear model that accounts for strain hardening behavior, simplified linear and nonlinear MDOF OpenSees models are used to evaluate the damage potential of records. Simplified and refined 3D bridge models are generated in OpenSees (McKenna *et al.*, 2007). Results of bilinear SDOF model and simplified MDOF analytical models of the as-built configuration are utilized to evaluate the damage potential of the selected sets of earthquake records (Figure 7.12). Since the rate-independent representation is commonly used for seismically isolated bridges, the inelastic SDOF model works better with this model where the superstructure and substructure is assumed as rigid body.

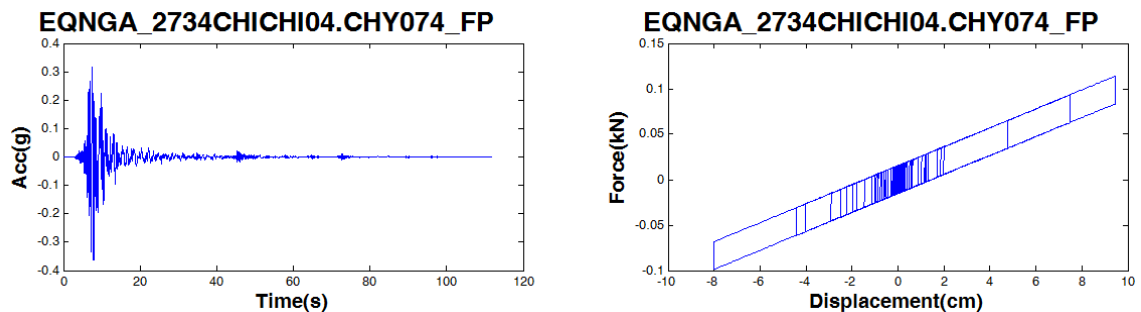


Figure 7.12. Inelastic SDOF structure.

Four limit states are included for the performance assessment of Rio Torto Viaduct and suitable sets of earthquake ground motion recordings are scaled. Modifications on the model

were done on the basis of findings from nonlinear response history analysis and experimental results.

The simplified proxy bridge model is generated to represent the behavior of 3D-Fiber model of the Rio-Torto Viaduct. This model consists of nonlinear springs for expressing the hysteresis loops of piers and Gerber saddle elements which is considered to act like expansion joints as shown in Figure 7.14. Hysteretic and pinching elements are utilized to define the nonlinear behavior of piers in the simplified bridge model. The force-deformation behavior of these two elements are shown in Figure 7.14.

The degree of accuracy of the simplified proxy structure is in an acceptable margin for the evaluation of maximum force-deformation relationship. However, small amount of permanent offsets are observed in the simplified nonlinear proxy structure due to assigned force-deformation path of the nonlinear spring (Figure 7.13).

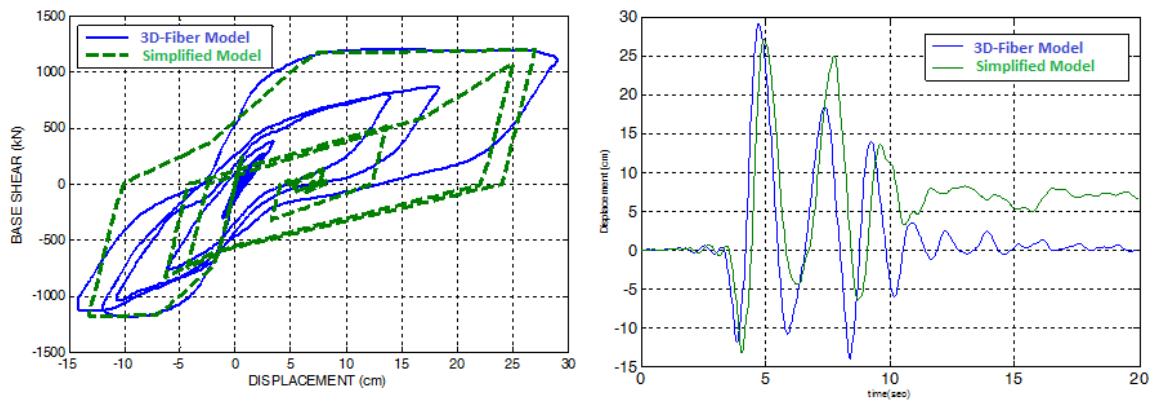


Figure 7.13. Comparison of force-deformation hysteresis and displacement response of Pier 11 for the 3D-fiber model and the simplified proxy model.

Results of the as-built model are expressed in terms of displacement response histories of piers and envelope displacement profile of the deck were given in Figure 7.15. Force-displacement relationship of each pier with respect to assigned nonlinear properties is shown in Figure 7.16.

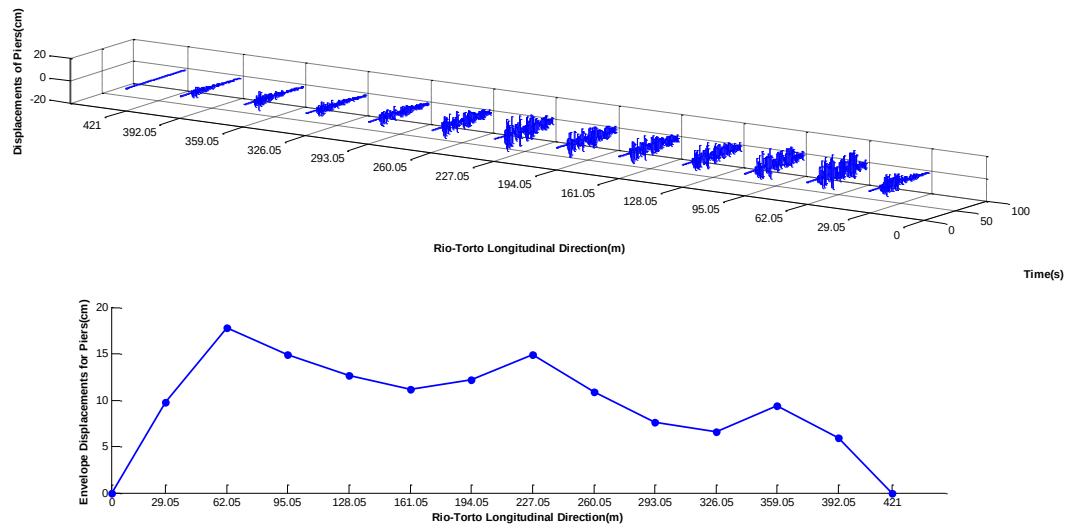


Figure 7.15. (a) Displacement time histories at pier locations (b) Envelope displacements of the deck.

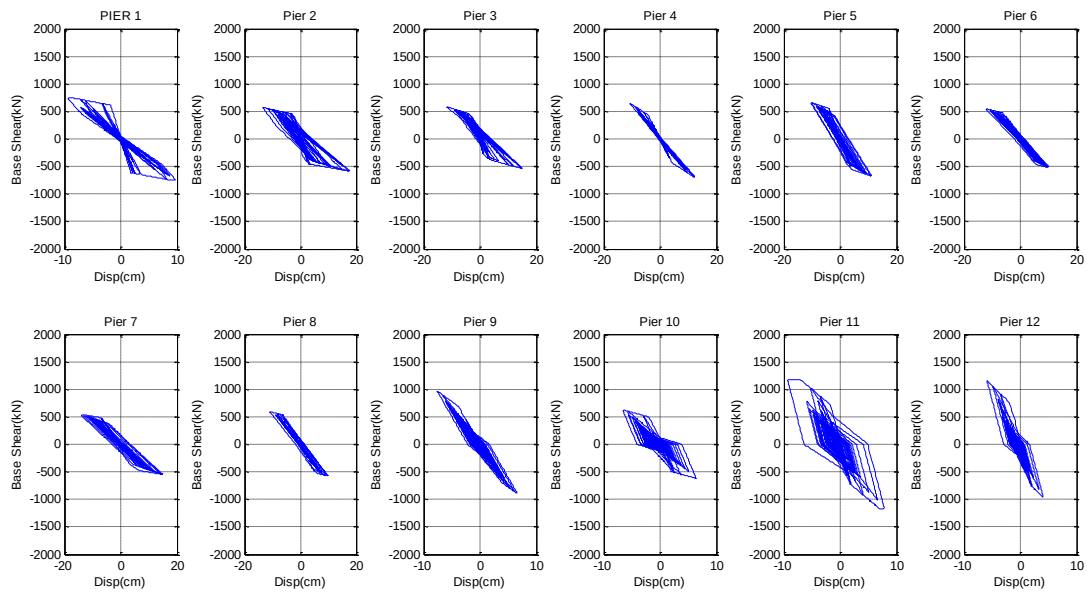


Figure 7.16. Force-Displacement hysteresis of piers for fixed base condition under a selected ground motion.

Likewise the as-built bridge model, the new simplified MDOF proxy structure is generated for seismically isolated bridge. Zero-length elements are employed to model the characteristics of the curved surface friction slider by using the design parameters provided in Section 7.5. Force-displacement hysteresis loop of idealized curved surface friction sliders at each bent is illustrated in Figure 7.17.

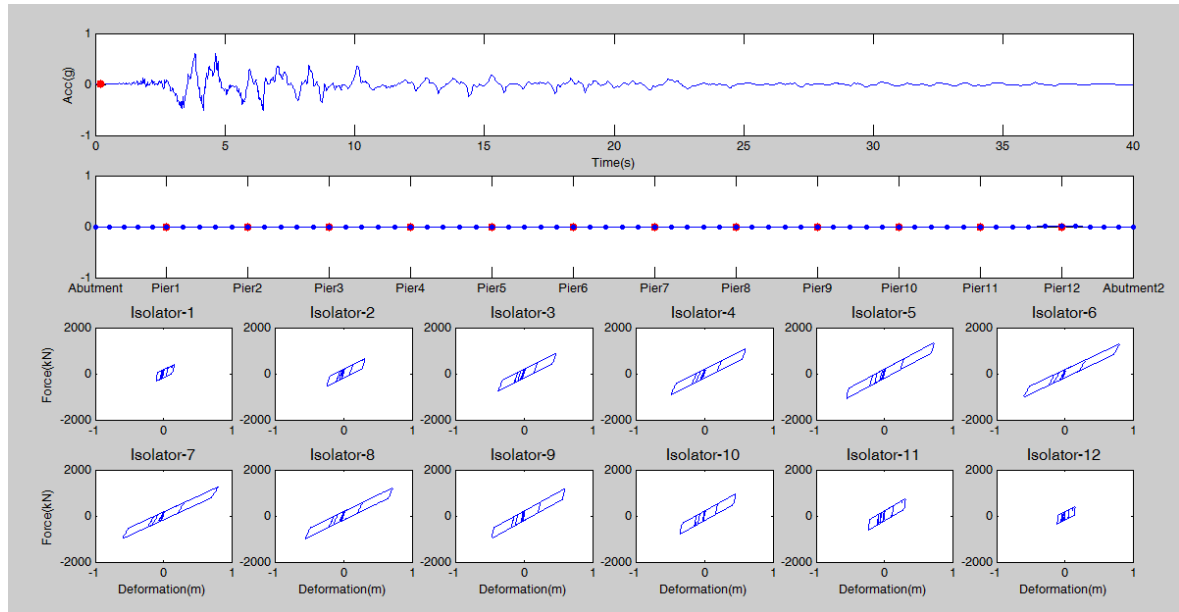


Figure 7.17. Simplified bridge model of the Rio-Torto Bridge for the retrofitted case.

Finally, a refined three-dimensional nonlinear finite-element model with fiber elements was developed in OpenSees (McKenna *et al.*, 2007) for the Rio-Torto Viaduct. Several 3D-Bridge models are generated to assess the phenomenological issues associated with the as-built model by the contributing partners of RETRO Project. Our main concern was to assess the efficiency of seismic isolators to reduce the amount of damage in comparison with the as-built model. Results of the PsD tests were used as reference point for the verification of the generated analytical models. The 3D fiber as-built model is illustrated in Figure 7.18.

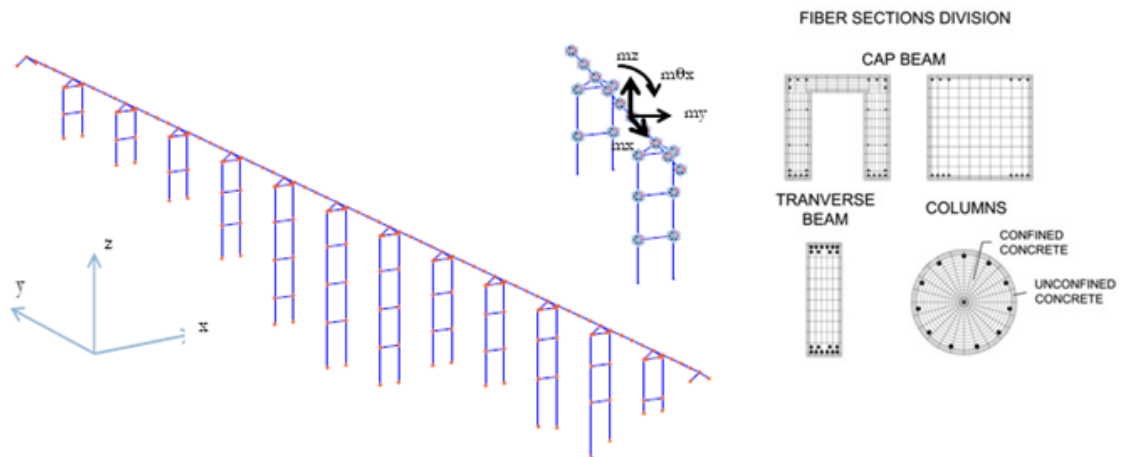


Figure 7.18. 3D-fiber model of the Rio-Torto Viaduct.

Dynamic characteristics of the 3D as-built model is given in Table 7.2. Fiber elements are employed to assess the nonlinearity in the structural components. Fundamental vibration period in the transverse direction is found to be 1.58s for the as-built bridge model.

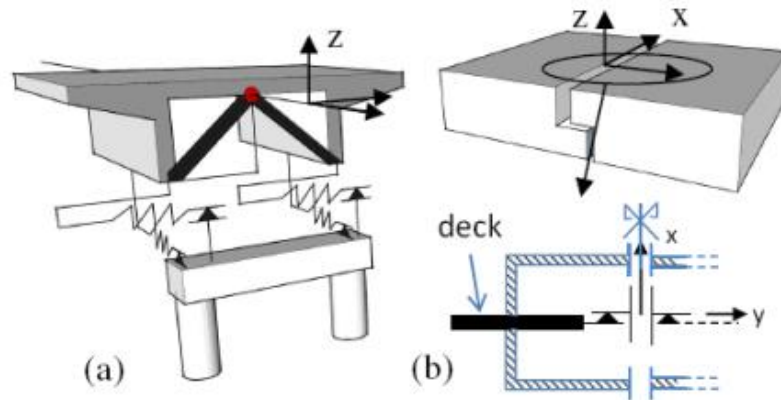


Figure 7.19. (a) Implemented superstructure model (b) Gerber saddles.

Table 7.2. Periods of vibration and modal participating mass for the as-built model.

Mode	T(sec)	MPM(x)	MPM(y)
1	3.23	0	92.41
2	1.58	27.35	0
3	1.45	2.94	0
4	1.4	24.89	0
5	0.94	7.14	0
6	0.81	22.51	0

Seismically isolated bridge model differs from the as-built model in terms of modeling assumptions and properties of the structural elements. In the refined numerical model the bridge piers are represented as non-linear fiber beam-column elements. Isolators are modelled as non-linear elements taking into account the hysteretic behavior through a Coulomb friction model, considering the “hardening” effect due to the radius of curvature.

3D-model of the seismically isolated bridge consists of linear elastic beam-column elements rather than fiber counterparts regarding to applied rehabilitation method. The primary goal of this procedure relies on keeping the substructure and superstructure elements in the elastic range. Effectiveness of the isolation system is enhanced by the replacement of Gerber saddles with the transverse pre-stressed structural component. This modification will make it possible to have a multi-span continuous deck system. The initial design of the isolators is based on the response of piers in transversal direction in which nonlinear –static procedures are conducted to calculate the required parameters of piers. An alternative design of the curved surface friction slider is done according to the displacement-based design (Della Corte *et al.*, 2011, Yenidogan and Erdik, 2013). The former study relies on the direct displacement based procedure proposed by Priestley *et al.* (2007). Since the transmission of inertial forces is crucial, the strength capacity of each pier must be certainly checked under the breakaway motion. All of the stated design criteria are applied to both simplified nonlinear and the refined 3D model of the isolated bridge.

7.7. PsD Test of the Rio Torto Bridge

The Pseudo-Dynamic (PsD) test combines the numerical method that has a properly condensed number of Degrees-of-Freedom (DOF) and the physical part that takes place in the laboratory. As a result of the applied input motion by actuators, reaction forces are measured. This feature of the test is taken into account when the substructuring technique is introduced (Dermizakis and Mahin, 1985) to obtain the response of the structure by constructing the limited portion of the structure in the laboratory. Selected portion of the structure is usually the most vulnerable component of the structure.

Loading protocol of the PsD test was based on the previously conducted analytical models, selected input signals and similitude rules for the appropriate representation of the full scale structure. The PsD test utilizes both physical testing and the sub-structuring technique which simulates the behavior of the bridge by performing numerical analysis for the selected components of the bridge (deck + 10 piers). By this way, full 3D model of the bridge would entail a high computational effort to exhibit the nonlinear behavior of piers that are expected to occur during the PsD test. Therefore, the generation of simplified models of the Rio-Torto Viaduct is critical to check the test results and control the unprecedented problems associated with the PsD test.

The Mirandola records (MRN station) were utilized because of the ground motion characteristics (*i.e.* PGAs, significant duration). The record of May 29th East-West was used for the SLD limit stated and the North-South component of the same earthquake was used to assess the seismic performance at the ULS limit state. The four limit states of the NTC-08 is shown in the Figure 7.20(a).

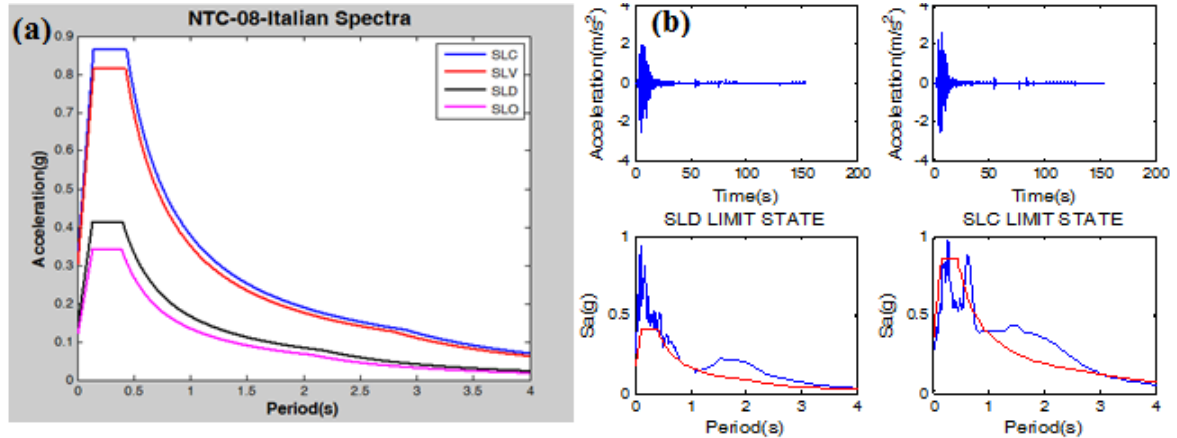


Figure 7.20. (a) NTC-08 design spectra at four limit states (b) Input signals of PsD test for the serviceability and ultimate limit states.

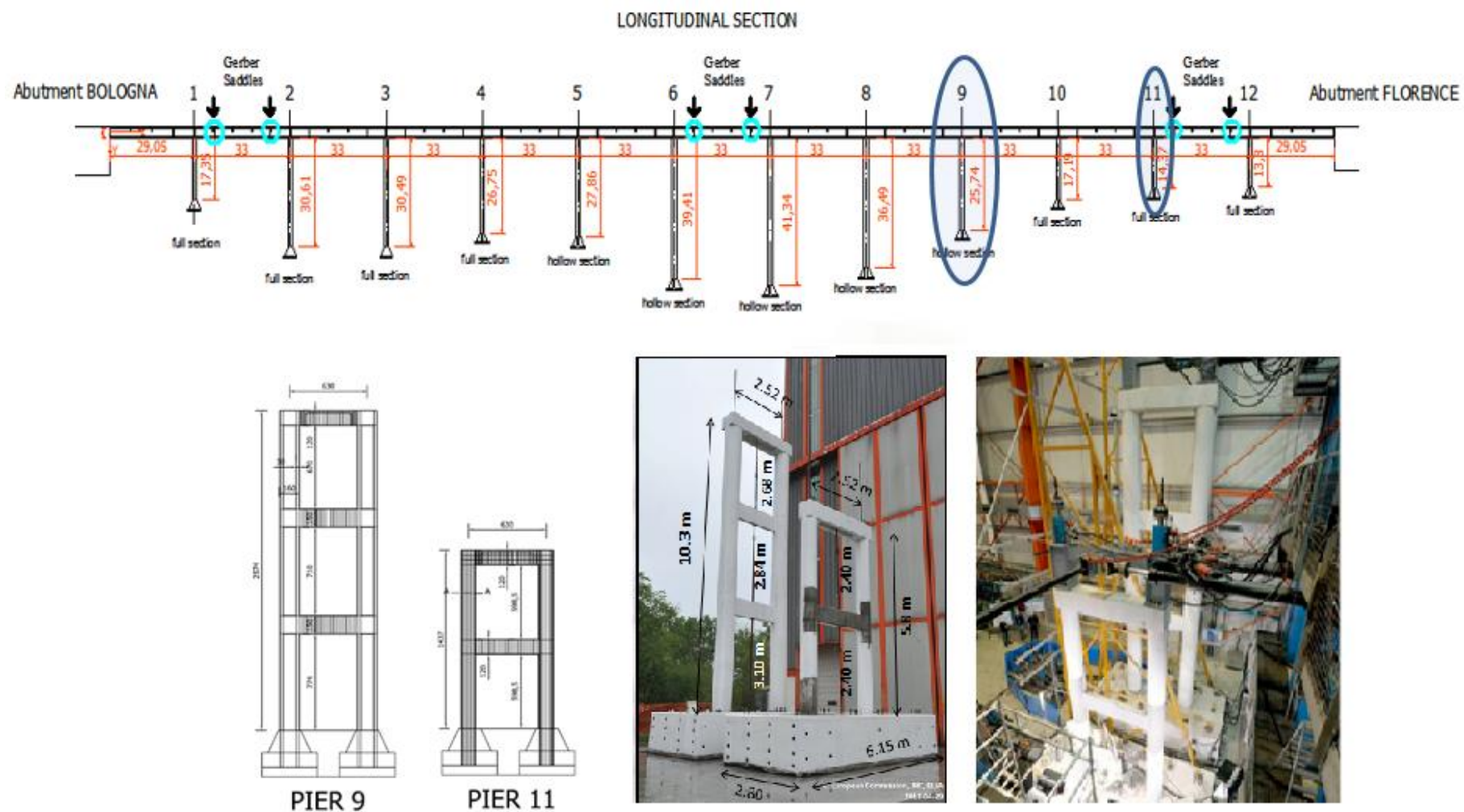


Figure 7.21. Mock-ups of the PsD test.

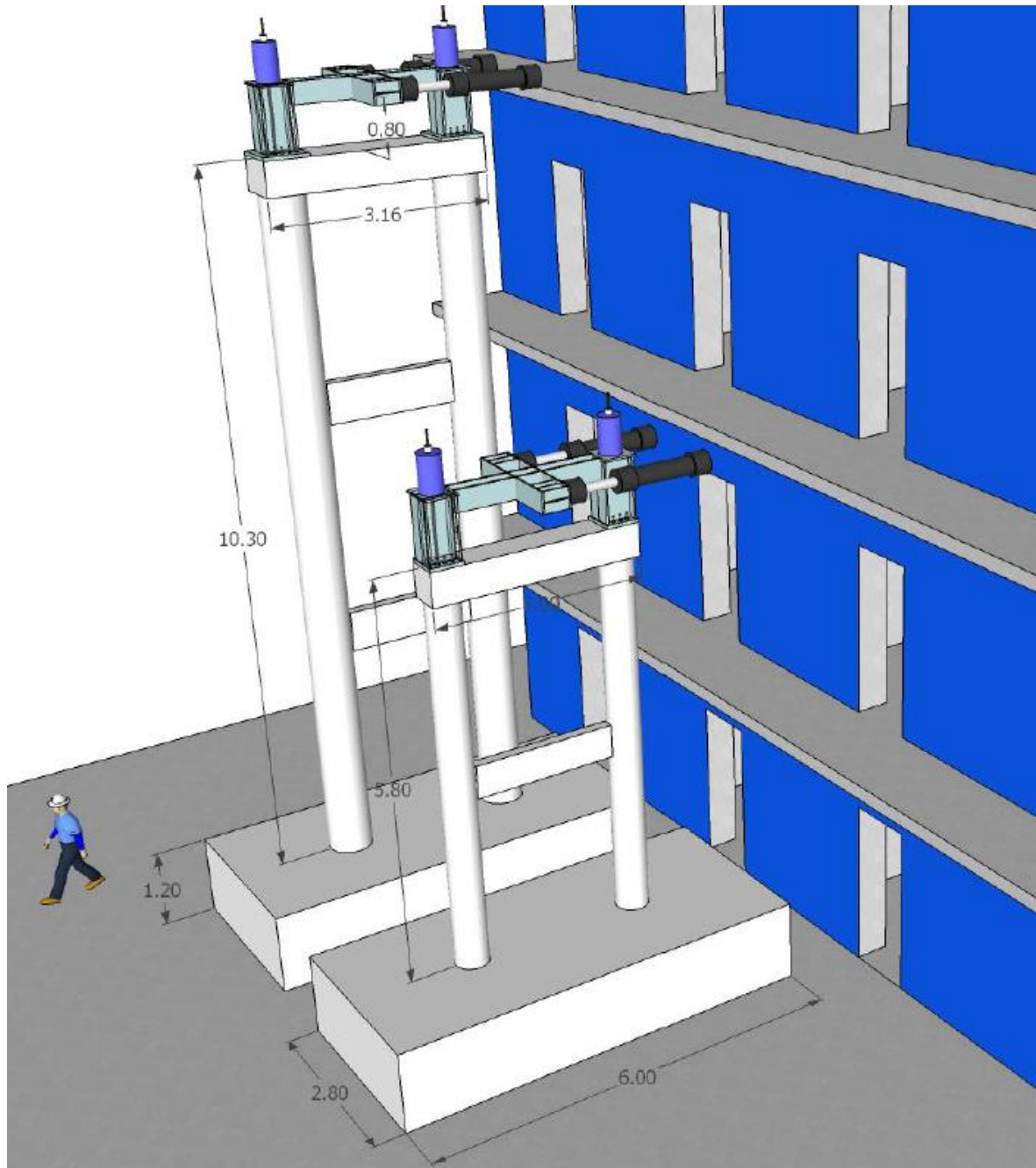


Figure 7.22. Dimension of test specimen and location of actuators (The RETRO Project, 2013).

8. PARAMETRIC INVESTIGATIONS

8.1. Introduction

Seismic isolation units aim to protect the substructure and the superstructure of bridges by maintaining the damage of the structural components in the elastic range while accommodating the nonlinearity at isolation interface. This chapter elaborates and provides a parametric study on the response of highway bridges that are isolated with sliding or elastomeric type of earthquake protection systems. Effects of near-fault earthquake ground motions on long period structures have been one of the challenging topics in terms of high seismic demands on seismic isolators and velocity sensitive friction force for sliding type of bearings. Performance assessment of seismically isolated bridges requires conducting comprehensive analytical and experimental studies. Therefore, there is a special need to assess the suitability of isolation systems for near-fault earthquakes and compare the distinct characteristics of isolators to provide reliable and feasible engineering solutions.

The lack of seismic design codes and guide specifications of highway bridges in Turkey have been forcing design engineers and experts to adopt the well-known available design specifications in US or Europe (*e.g.* AASHTO, 2010, EC-8 Part-2). However, there is no consensus on which code will be used by engineers in Turkey for the seismically isolated bridges and how would the near-fault actions take into consideration. Moreover, the relevant standard code section of these specifications differs from each other principally in terms of establishment of design spectrum, code-requirements and test protocols.

In this research study following bridge models are considered for the nonlinear response history analysis,

1. A continuous, three-span, cast-in-place concrete box girder structure(Bridge-A)
 - a. 30-degree skew
 - b. 15-degree skew
 - c. Straight bridge

2. Thirteen-span reinforced concrete highway bridge with varying pier heights and section properties (Bridge-B)
 - a. As-Built configuration of fixed base bridge
 - b. Seismically isolated bridge

The first and second bridge example are hereafter referred as Bridge-A and Bridge-B, respectively. Two different implementation methods of seismic isolation systems are used in these two example bridges. The first method is based on the installation of seismic isolators at piers and abutments for the Bridge-A. In contrast, partial isolation of the Bridge-B is used as a performance enhancement strategy for the existing bridge where the seismic isolators are only located at the top of each bent. OpenSees and SAP2000 programs are preferred for the structural analysis of seismically isolated bridges due to their large set of link elements in their library and common use in the industry and scientific studies.

LRB and single curved surface friction sliders have been used commonly by engineering community. In addition, the proposed modeling assumptions of advanced model of multi curved surface friction sliders are assumed to give acceptable overall behavior which were already justified by experimental results (Morgan, 2007, Fenz and Constantinou, 2008, Dao *et al.*, 2013). Thus, sliding type of isolators, lead rubber bearings are adopted to this parametric study with properly assigned mechanical properties which may accommodate the estimated large seismic demands in near-fault regions. Conventional type of curved surface friction slider, multi-surface curved spherical friction slider and LRB are utilized in the Bridge-A for the response modification purposes and limiting the amount of damage in the vicinity of earthquake source.

Apart from the parametric study of Bridge-A, effectiveness of conventional curved surface friction sliders are evaluated by the Pseudo-Dynamic (PsD) test method for Bridge-B. Bridge-B is an existing highway bridge located in Italy and tested through Pseudo-Dynamic experiment as a part of Translational Access (TA) facility in Series Project. Large scale specimens of Rio-Torto Viaduct is constructed in ELSA laboratory of Joint Research Center (JRC) for the pseudo-dynamic experiment where the conventional type of isolators are incorporated for the response modification of the Rio-Torto Viaduct. Three different

representative bridge models were generated to account for the as-built configuration and isolated case of the Rio-Torto Viaduct. Among these bridge models, simplified bridge models represent the structural characteristics in terms of effective properties or through nonlinear springs which is compatible with the obtained previous test results of piers (Giannini and Paolacci, 2009). The nonlinear simplified bridge model is critical for checking the ongoing test which relies on the developed 3D-refined fiber model of the Bridge-B. In other words, it has been used as a reliable proxy of the refined three dimensional fiber models for the pseudo-dynamic testing in order to trace the amount of damage instantaneously.

This section will address the results of NRHA of the 3D refined models of the two bridge examples which stand for the spatially distributed isolators.

8.2. Modeling Assumptions of Seismic Isolation systems and Analysis Results of Bridge-A

The results presented in this section consists of performed parametric studies for the Bridge-A and Bridge B. Analysis procedures of AASHTO LRFD and Guide Specification for Seismically Isolated Bridges (2010) are examined by using the simplified models and three dimensional model of the Bridge-A. A total number of 8 isolation units are used in each isolation system to evaluate the performance of the entire system under near-fault conditions. Effectiveness of the seismic isolation as a response modification method is assessed through the nonlinear response history analysis. Earthquake ground acceleration records have been appropriately selected and scaled to be compatible with the performed seismic hazard study.

There are generally two well-known approaches for the quantification of earthquake hazard. One of them is the probabilistic seismic hazard assessment which accounts for all possible earthquake scenarios that could affect the site. The other one is the deterministic hazard assessment which involves the determination of the scenario earthquake, identification of proper attenuation relationships and appropriate site response quantification. The latter one is employed for the Bridge-A by utilizing Chiou and Youngs (2008), Abrahamson and Silva (2008), Campbell and Bozorgnia (2008), Boore and Atkinson (2008) NGA attenuation relationships for three different level of earthquakes. Average of the stated ground motion prediction equations are established for earthquakes with strike-slip fault mechanism that can

generate $M_w=7.25$ on engineering bedrock at varying source-to-site distances. Although three level scenario earthquakes established, the seismic demand representation in this study is only presented for the design basis earthquake (DBE) with 475 years of return period and the maximum credible earthquake (MCE) levels which corresponds to a return period of 2475 years. It is a common approach to use median $+\sigma$ level to be representative of MCE level while the median value corresponds to DBE level. The DBE and MCE shakings are represented by a 5%-damped acceleration response spectrum.

The suite of ground motions have been properly selected and scaled with respect to the target spectrum. Two sets of ground motions are prepared for each scenario earthquake to investigate the effect of pulse content on the behavior of isolation units in near-fault regions. Selection and scaling of ground motions were performed by using the linear amplitude scaling procedure of the web-based tool of PEER ground motion database. This procedure ensure to preserve the frequency content of seed ground motions (Figure 8.1).

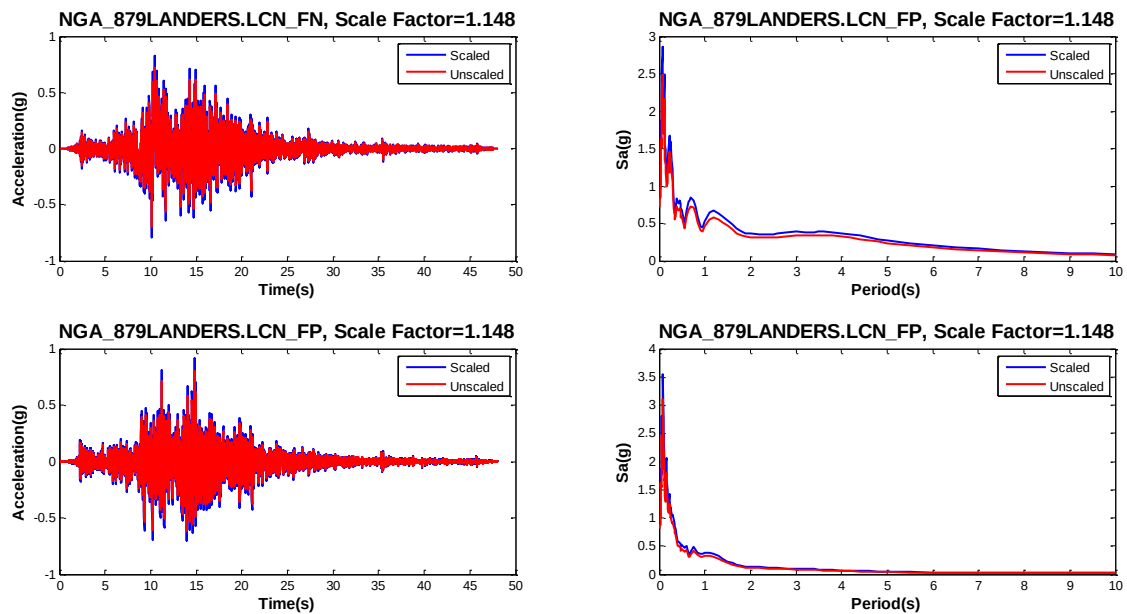


Figure 8.1. Amplitude scaled records of Landers Lucerne Fire Station for MCE level earthquake scenario.

It is worth mentioning that specific search criteria are applied to the pool of the selected and scaled ground motions. Refined sets of ground motions are intended to have three component seed motions for simultaneous multi-directional earthquake loading and limited

number of earthquake ground motions from the same event is incorporated to avoid the dominance of a particular event. Earthquake ground motion records are sorted by considering the computed minimum mean squared error for the assigned target spectrum which aims to list the ground motions with respect to their spectral shape. The ground motions in the PEER database had been already rotated to fault-normal (FN) and fault-parallel (FP) components. Scaled ground motions are acted with respect to the longitudinal and transversal direction of the Bridge-A as shown in Figure 8.2(a). Then the same pairs of earthquake ground motions are flipped (Figure 8.2b) to check the occurrence of maximum demands under these two loading cases.

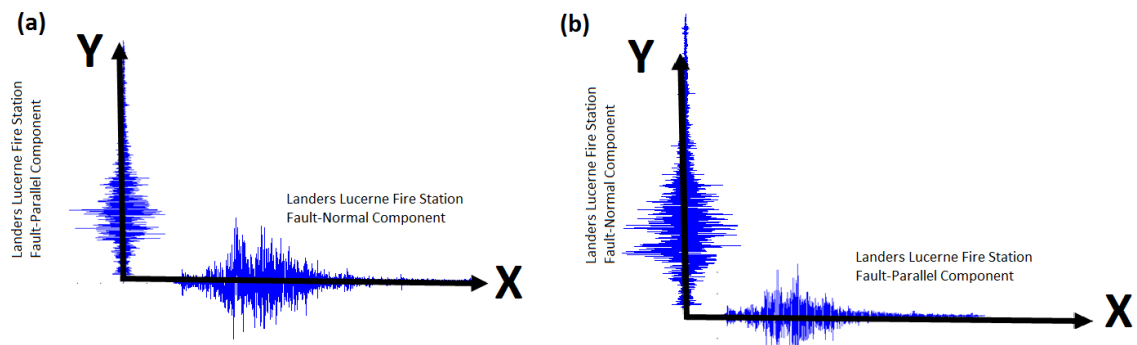


Figure 8.2. Acting directions of Landers-Lucerne Fire Station records.

Spectral shape of nonpulse and pulse like suite of ground motions for nonlinear response history analysis of the Bridge-A at DBE and MCE level are shown from Figure 8.3 to Figure 8.6.

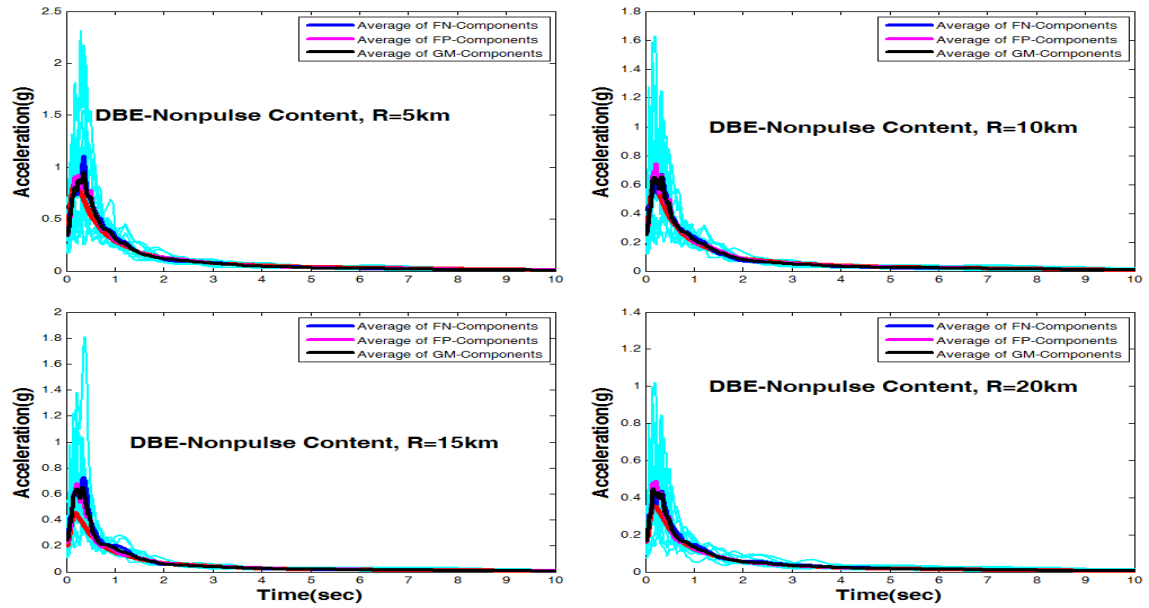


Figure 8.3. Spectral shape of selected ground motions with nonpulse content at DBE level earthquake.

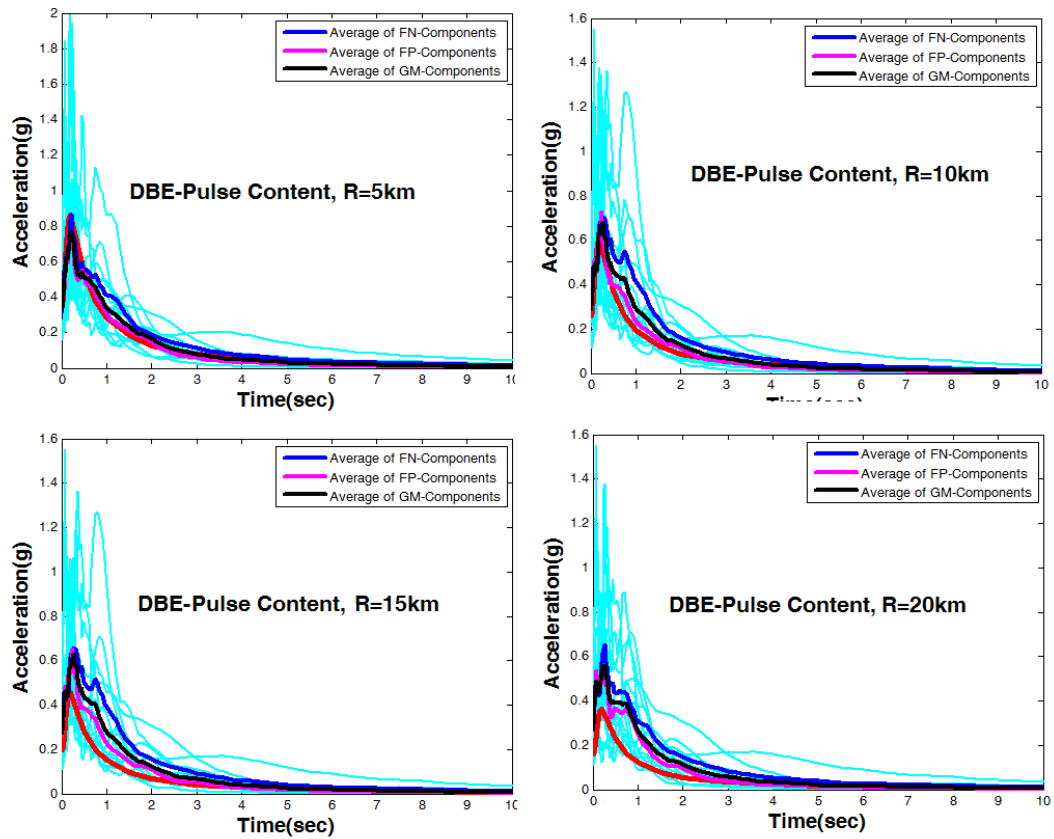


Figure 8.4. Spectral shape of selected ground motions with pulse content at DBE level earthquake.

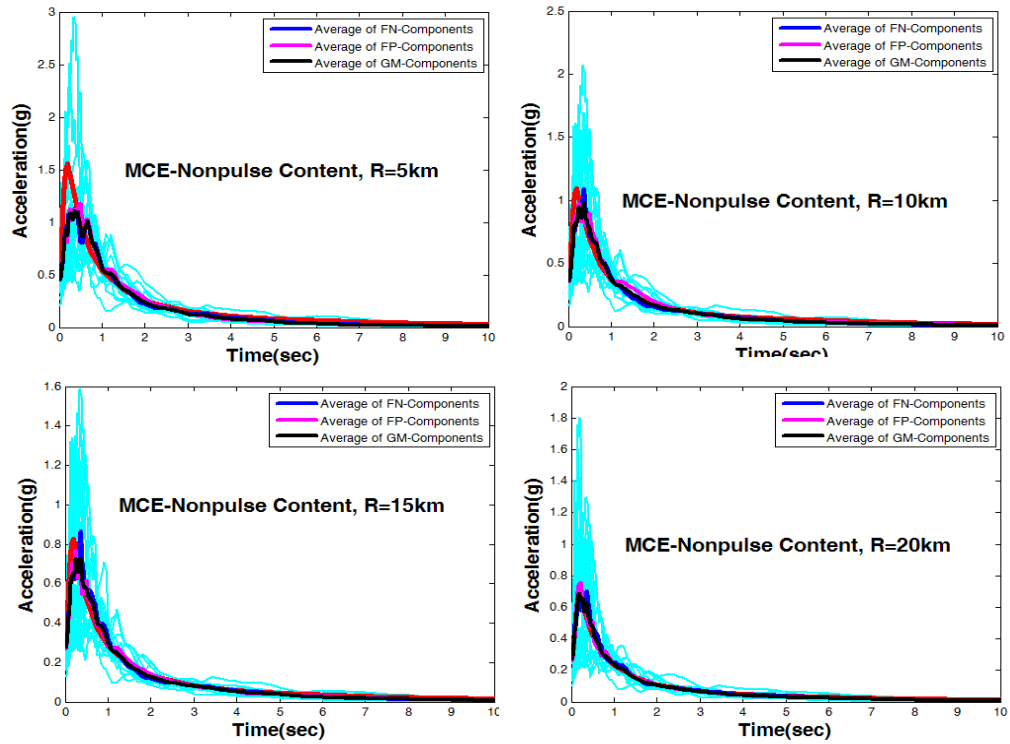


Figure 8.5. Spectral shape of selected ground motions with nonpulse content at MCE level.

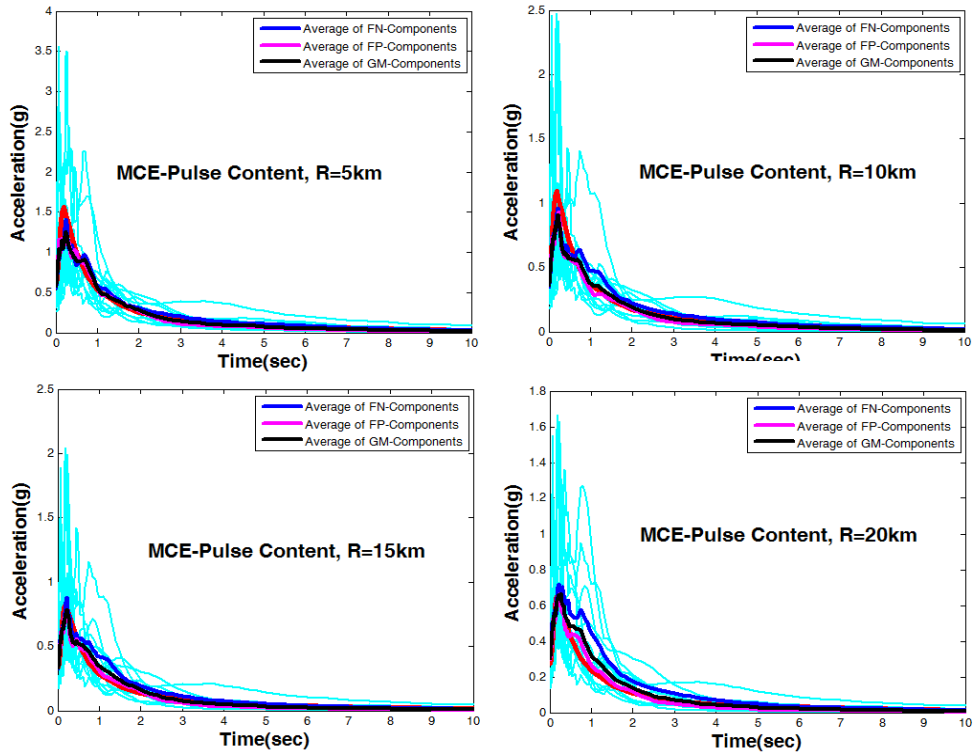


Figure 8.6. Spectral shape of selected ground motions with pulse content at MCE level earthquake.

8.2.1. Comparison of Simplified Parametric Analysis Results of Bridge-A under Different Scenario Earthquake

Four different analysis procedures are defined in the design guidelines of seismic isolators (AASHTO, 2010, Eurocode-8, 2005). Three of these methods are based on implicit representation of inherent nonlinearity through linear effective properties of seismic isolators. Among these methods nonlinear time history analysis gives the most accurate results when compared with the linear methods. Section 13.2 of AASHTO Guide Specification defines the Uniform Load Method (ULM) which approximates the earthquake loads as an equivalent static force. ULM method is an iterative method and it continues until the targeted convergence is achieved. Therefore, nonlinear time history analyses are conducted to address the accuracy of this procedure in AASHTO Guide Specification for Seismic Isolators and Eurocode 8 Part-2 (CEN, 2005) by using relevant structural characteristics of the Bridge-A. More than 2000 nonlinear response history analysis (NRHA) are conducted in OpenSees (McKenna *et al.*, 2007) for the performance assessment of the sliding type of seismic isolation systems under the selected suite of ground motions. In addition, new ground motion bins are incorporated for making evaluation on the basis of seismological features, site-to-source distance and site conditions.

AASHTO Guide Specification for Seismic Isolators (AASHTO, 2010) defines idealized shear force-lateral displacement relationship for the most common configurations of LRB and conventional curved surface friction sliders. The proposed model is valid for these two systems with appropriate characteristic strength, post-yield stiffness and yields displacements parameters. Proposed hysteretic shear-force relationship of simplified procedure relies on the rate-independent behavior. LRB can be described as a yielding type of isolator where the energy dissipation is provided by the yield of lead plug at certain amount of displacement. In contrast, curved surface friction slider dissipates the energy through friction at small yield displacements. Therefore, velocity has an important impact on the energy dissipation capacity of sliding type of isolation systems.

The friction coefficients between Teflon surfaces of sliding type of bearing are not constant and it depends on the sliding velocity, bearing pressure which has a significant influence on the behavior of isolators. The conventional curved surface friction slider and

multi-curved surface friction slider elements in computer programs contain the option for constant friction or a generalized friction model that accounts for the effect of instantaneous velocity and compression load on the friction coefficient. In order to assess the influence of velocity depended behavior, two different friction models are compared in the simplified analysis. For the simplified method of analysis, the superstructure and substructure of the seismically isolated bridge is assumed to be rigid as shown below shown below in Figure 8.7. Moreover inclusion of substructure flexibility can be achieved by applying the provisions of AASHTO Guide Specification for Seismic Isolators (2010).

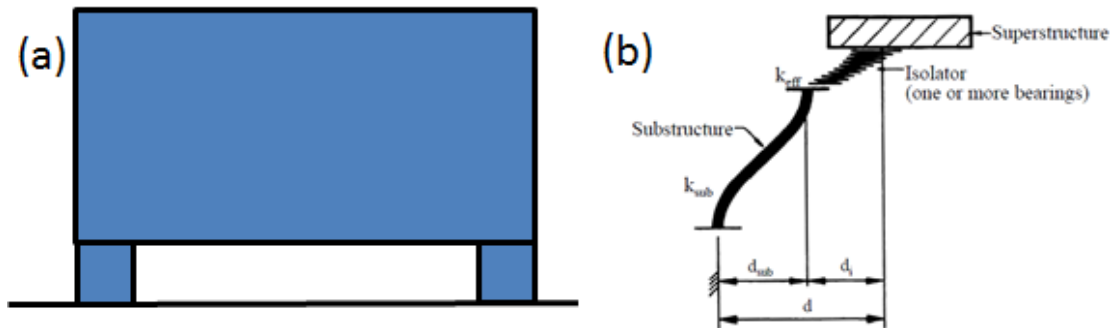


Figure 8.7. (a) Rigid substructure and superstructure assumption for isolators
(b) Inclusion of substructure flexibility regarding to AASHTO-2010.

The AASHTO Guide Specifications for Seismic Isolation Design (2010) and Eurocode-8 Part-2 for Bridges (CEN,2005) describe the analysis and design procedures of seismic isolation systems and the required test protocols of seismic isolation units. The Uniform Load Method (ULM) in the AASHTO Guide Specification is an equivalent-static procedure that accounts for the effective characteristics of seismic isolation system. In other words, ULM can be described as a simplified method of analysis that relies on the equivalent linearization of highly nonlinear systems through an idealized bilinear model in the preliminary design (Figure 8.8).

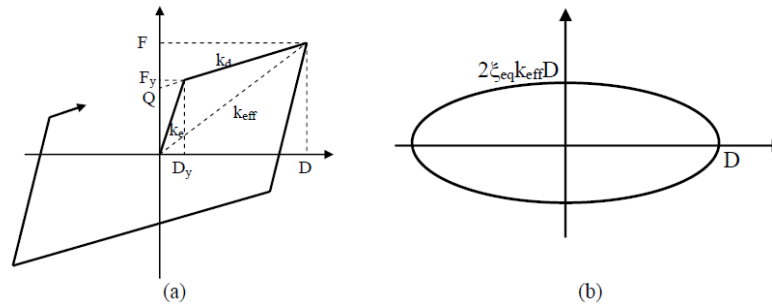


Figure 8.8 Equivalent linearization of seismic isolation system for the bilinear hysteretic behavior.

Two important parameters, maximum displacement and maximum transmitted shear force of isolators govern the design of the seismically isolated system. Since, the direct implementation is very limited under certain conditions; it has been used to establish lower bound limits for the response history analysis (RHA). With appropriate parameter assumptions, the same hysteresis curve in Figure 8.8 (a) can be used in the design and analysis of both LRB and curved surface friction slider type of isolation units.

8.2.2. Nonlinear Time History Analysis Results of the Simplified Bridge Model of the Bridge-A

Simplified analysis procedures constitute one of the key design steps for the seismic isolation systems in terms of idealized bilinear response of seismic isolation system. This analysis procedure has been presented as a part of the rate-independent analysis. The accuracy of this method needs to be comprehensively re-evaluated by using time-history analysis to avoid the main concerns of designers. Findings of researchers in the literature contradict with each other regarding to calculation of the maximum displacement demand and target spectrum (Kircher and Lashkari, 1989, Warn and Whittaker, 2004, Ryan and Chopra, 2004, and Ozdemir and Constantinou, 2010).

Earthquake ground motions may be dominated by strong velocity pulses in the vicinity of active faults. Effect of pulse content on the response of the seismic isolation units for the simplified analysis procedures is exhibited in Figure 8.10. Fault-Normal component of the recorded event is governed the behavior of the isolator for a near-fault ground motion with

pulse content. Usually ground motions within the 0-15 km distance to rupture contain strong velocity pulses. Extraction of pulse content of Fault-Normal component of earthquake ground motions are conducted by the method proposed by Shahi and Baker (2011). Example of a pulse extraction from the Landers-Lucerne Fire station record is shown in Figure 8.9. When the seismically isolated structure is exposed to this record, sufficient unrestrained movement at abutment locations are required with large displacement capacity isolators for the Bridge-A.

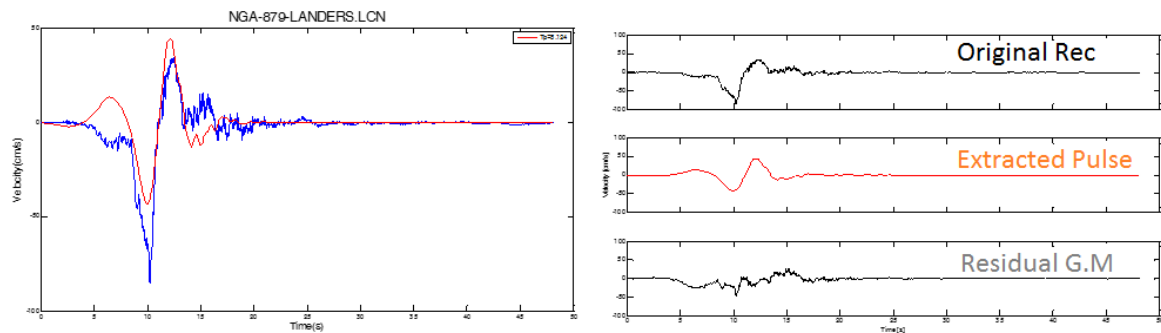


Figure 8.9 Method of analysis used to extract the pulse content of near-fault record in the PEER ground motion database (Shahi and Baker, 2011).

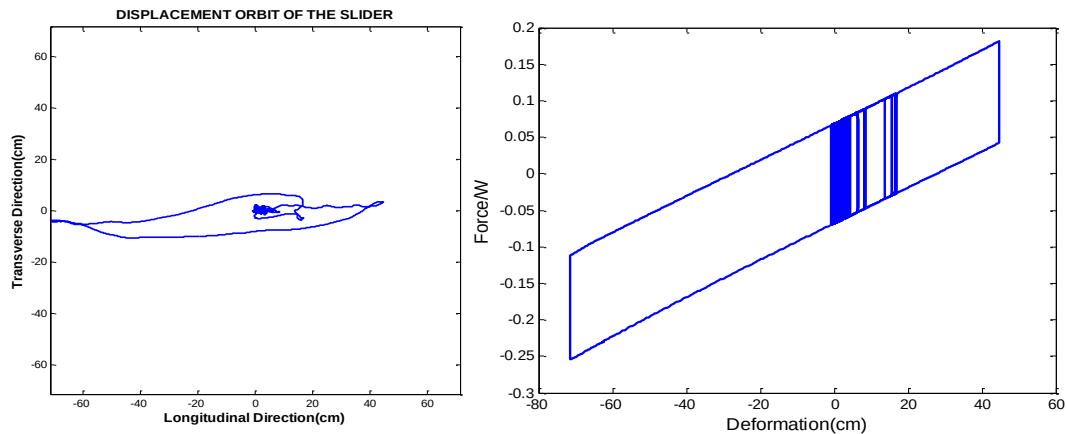


Figure 8.10. (a) Displacement orbit of the slider dominated by the pulse content of the record (b) Hysteresis loop of conventional type of curved surface friction sliders under horizontal pair of Lucerne-Fire station records.

8.2.2.1. Average Response of Simplified Bridge Model of the Curved Surface Friction Sliders (CSFS). The effect of velocity dependent behavior of seismic isolation units are shown for the single concave curved surface friction sliders. Variation of normal load acting on the isolation system directly effects the amount of the dissipated energy during the earthquake excitation. Variations of normal load on seismic isolators will be evaluated

through the multi-directional earthquake loading. Therefore, earthquake ground motions are respectively included for the pairs of horizontal ground motions, single horizontal component together with the vertical ground motion and for the three component recordings of the earthquake excitation. The effect of velocity sensitive friction behavior in the near-fault region with single pendulum characteristic will be derived. Variation of coefficient of friction, normal load, friction force and velocity of the element is given in Figure 8.11.

Simplified analysis methods are examined by exposing 2D and 3D component of selected suite of ground motions. By this way coupled plasticity effects on the average values of displacement demands and shear forces are exhibited from Figure 8.13 to Figure 8.20 for varying friction models of conventional curved surface friction slider. W is the total weight acting on isolators and the variation of transmitted shear force is represented in the form of dimensionless parameter as a ratio of the average transmitted shear force to the total weight (W). Uniaxial material in the axial direction appropriately considered for to check the uplift actions.

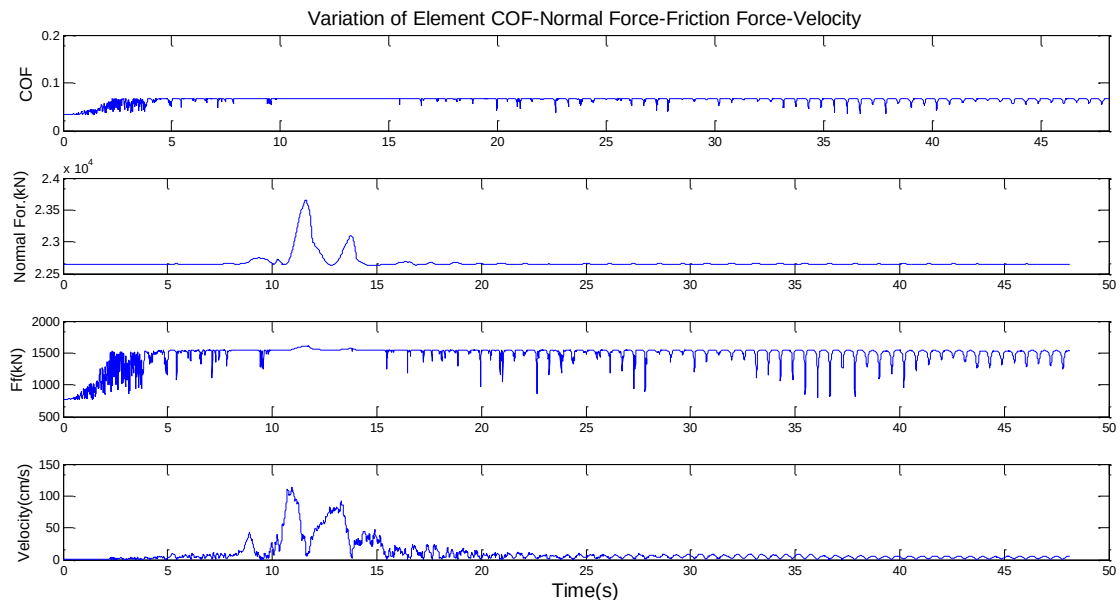


Figure 8.11. Variation of coefficient of friction (COF), normal force, friction force and computed relative velocity for the conventional type of curved surface friction slider under Landers-Lucerne Fire Station record.

Effect of coupled plasticity in conventional curved surface friction sliders for a circular yield surface is shown in Figure 8.12

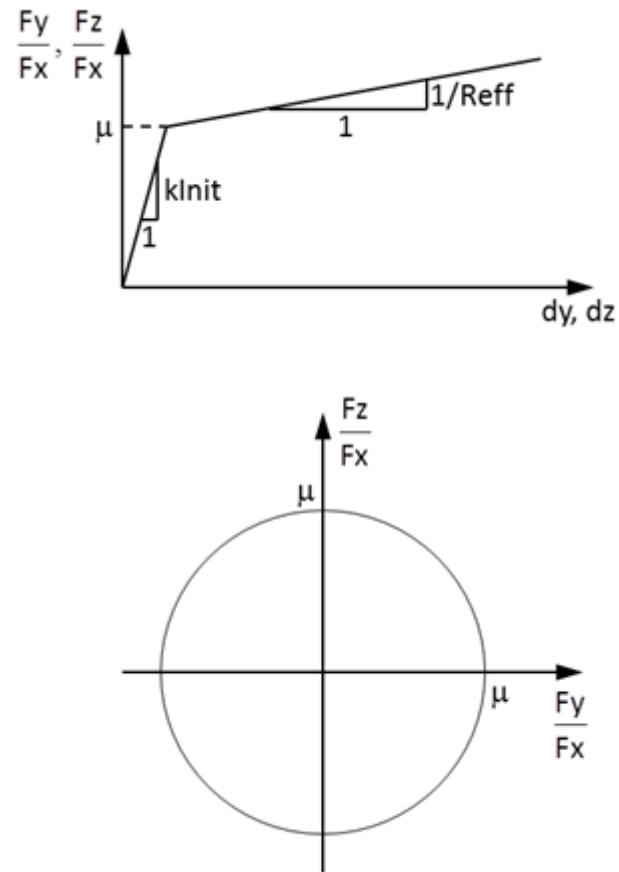


Figure 8.12. Coupled plasticity effects in conventional curved surface friction sliders.

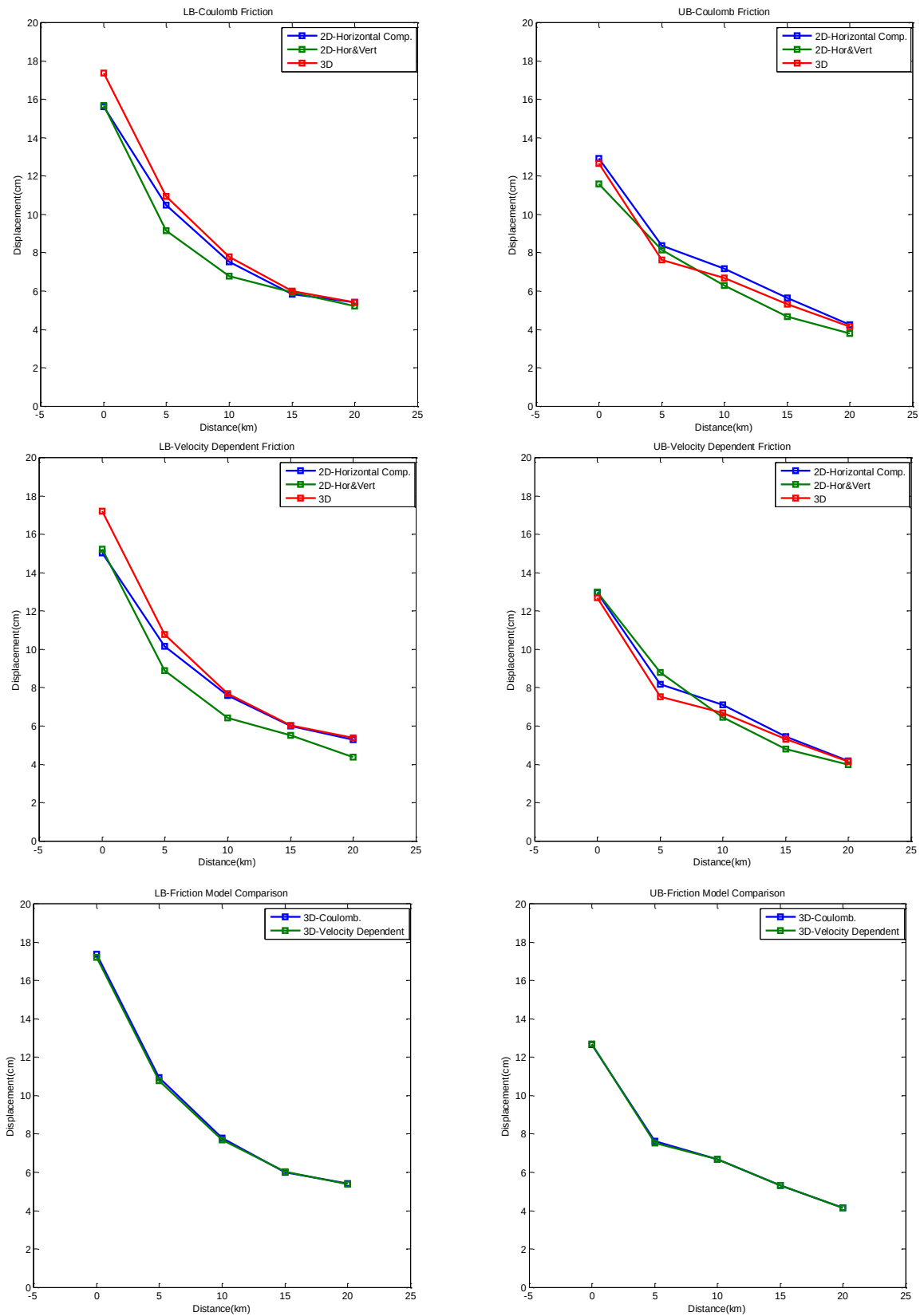


Figure 8.13. Variation of average seismic displacements of curved surface spherical friction sliders under earthquake ground motion records with nonpulse content at DBE level.

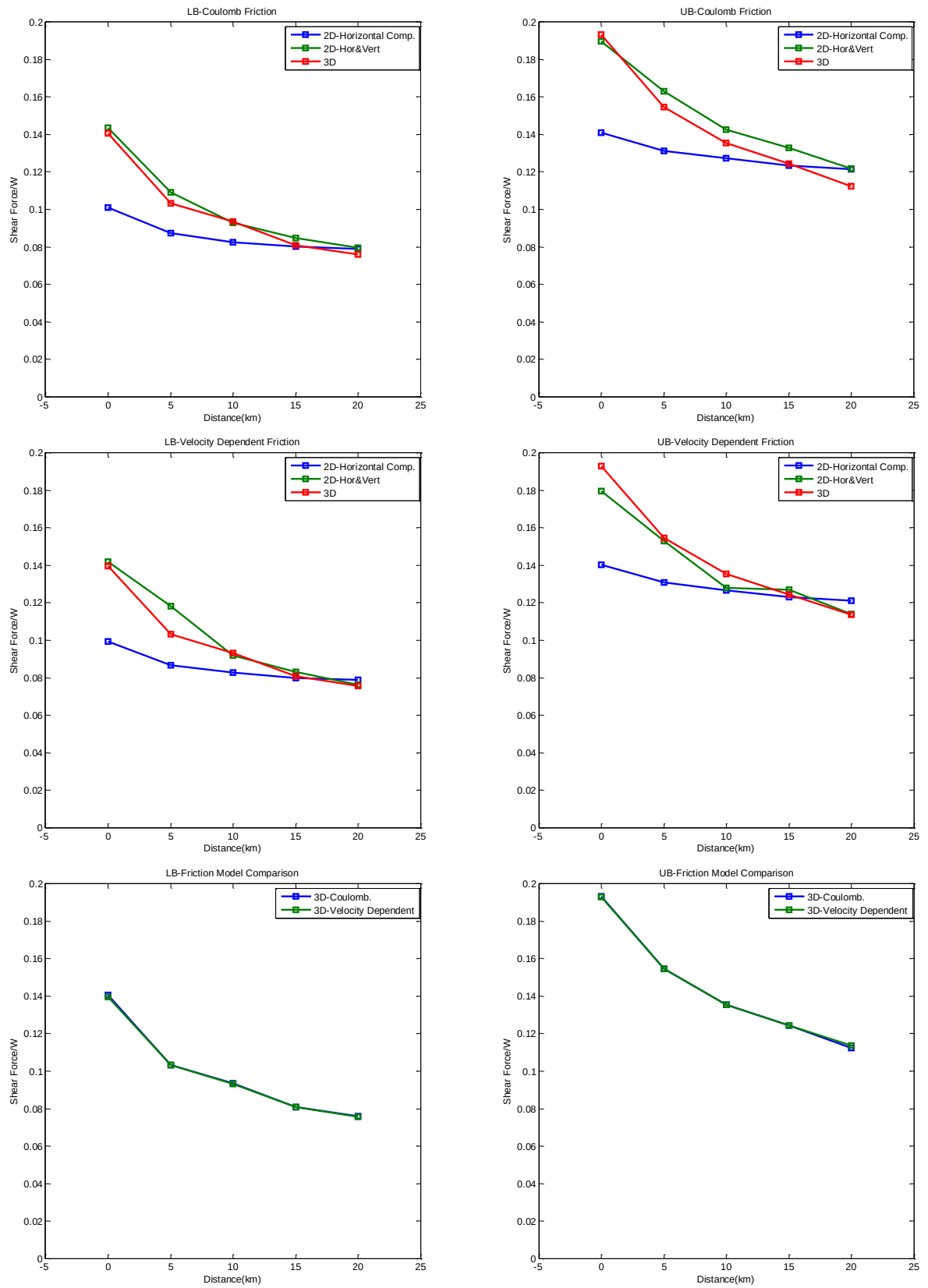


Figure 8.14. Variation of average shear force of curved surface spherical friction sliders under earthquake ground motion records with nonpulse content at DBE level.

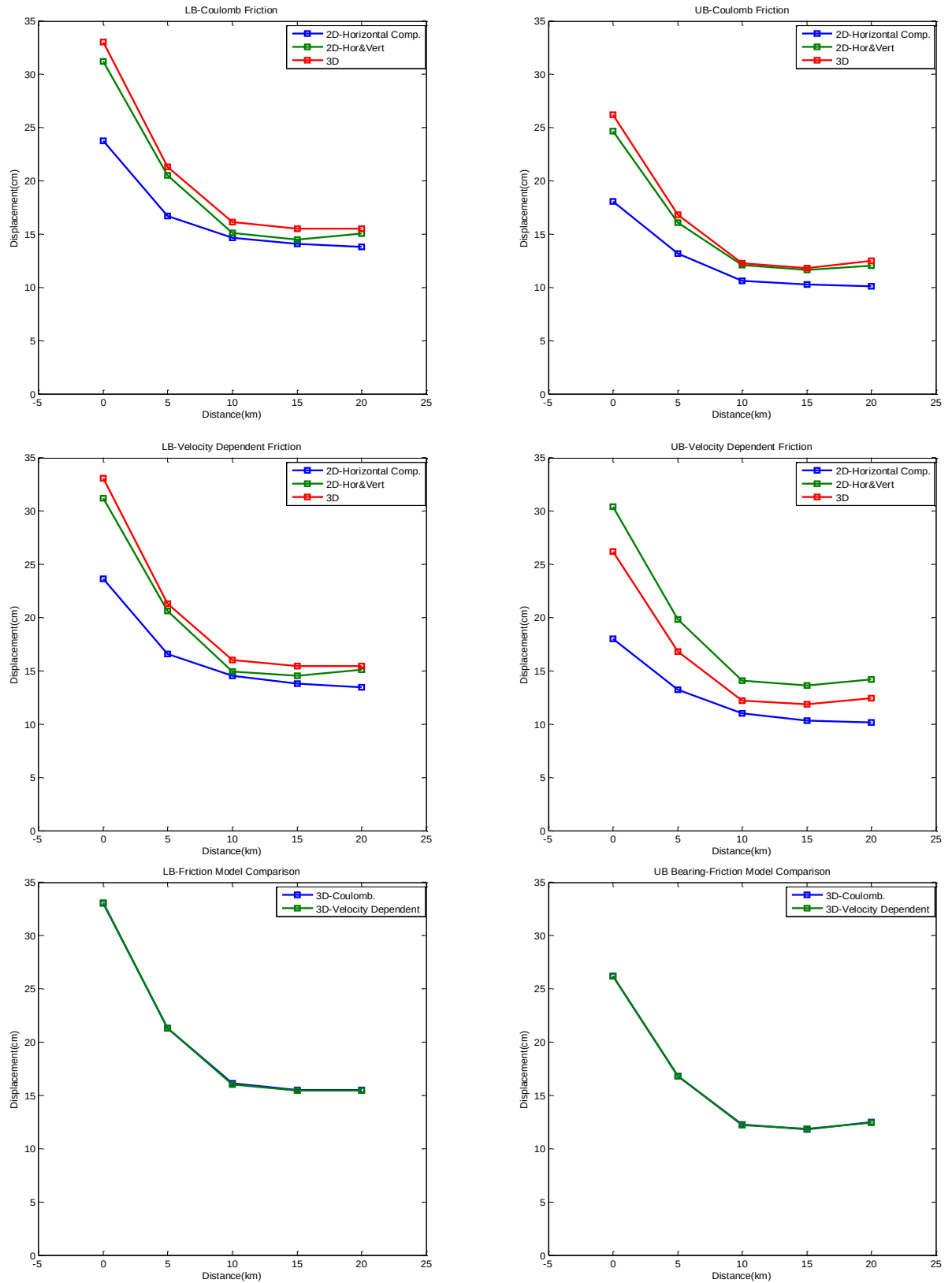


Figure 8.15. Variation of average seismic displacements of curved surface spherical friction sliders under earthquake ground motion records with pulse content at DBE level.

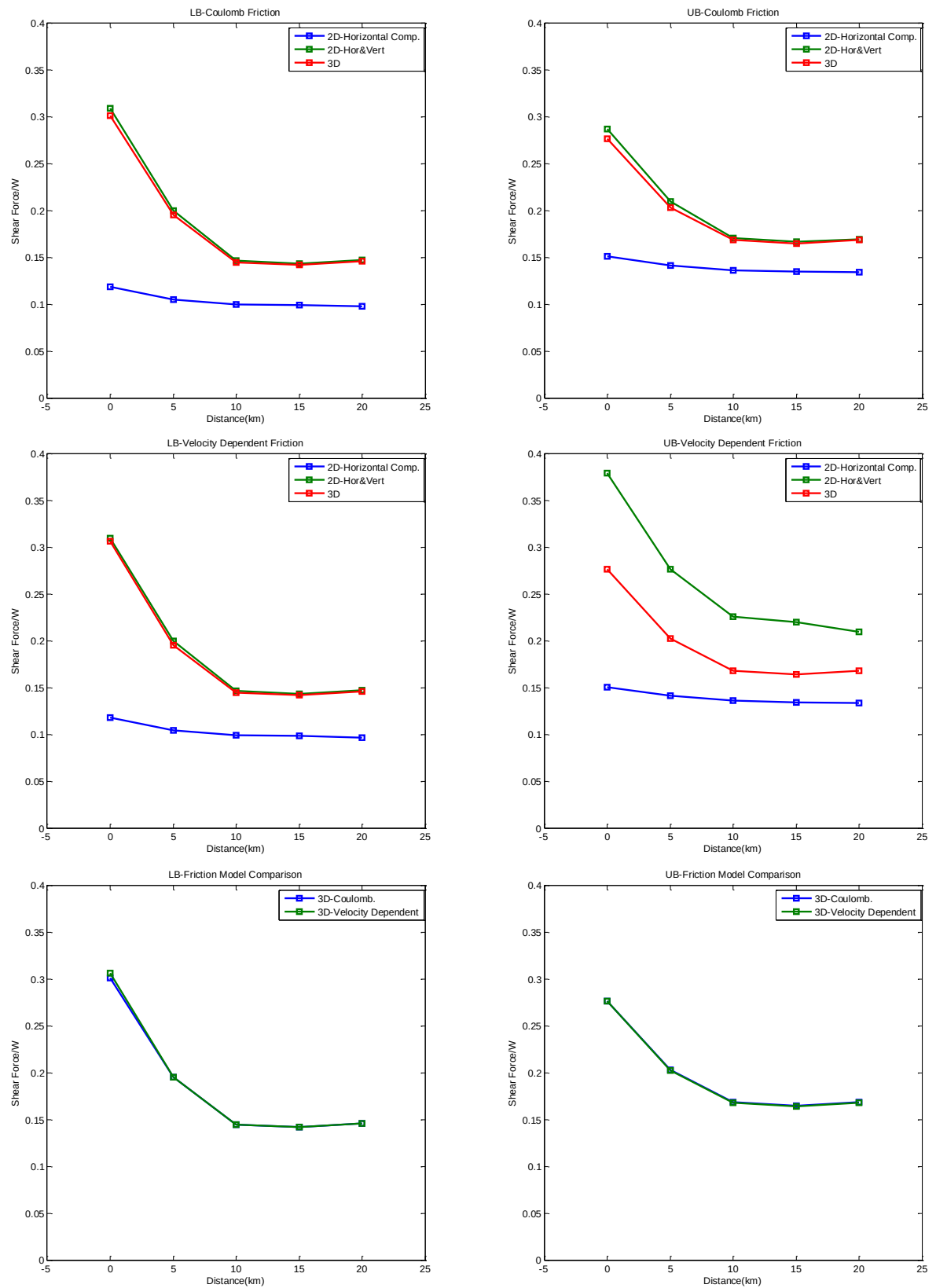


Figure 8.16. Variation of average shear force of curved surface spherical friction sliders under earthquake ground motion records with pulse content at DBE level.

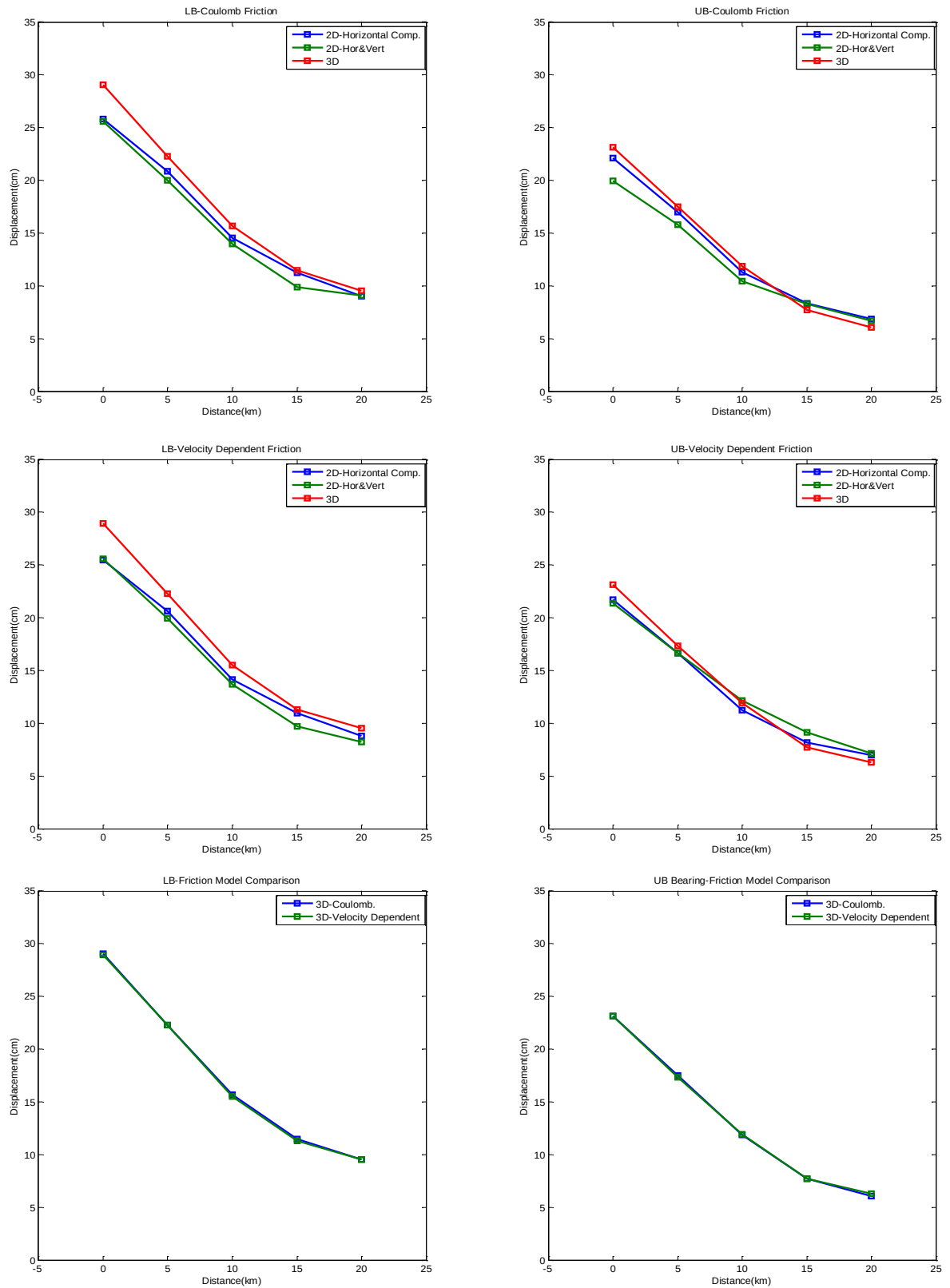


Figure 8.17. Variation of average displacement demand of curved surface spherical friction slider under earthquake ground motion records with nonpulse content at MCE level.

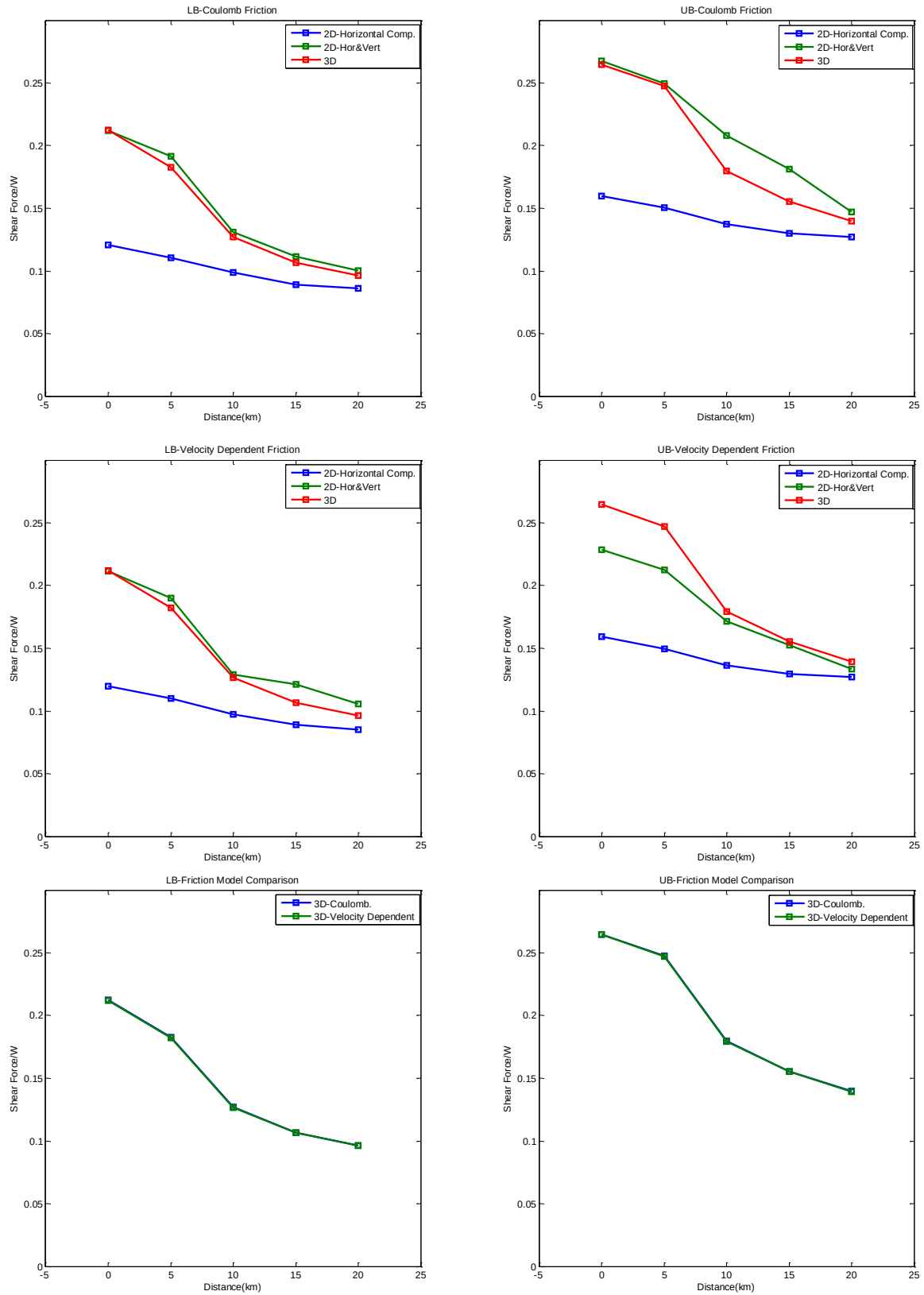


Figure 8.18. Variation of average shear force of curved surface spherical friction sliders under earthquake ground motion records with nonpulse content at MCE level.

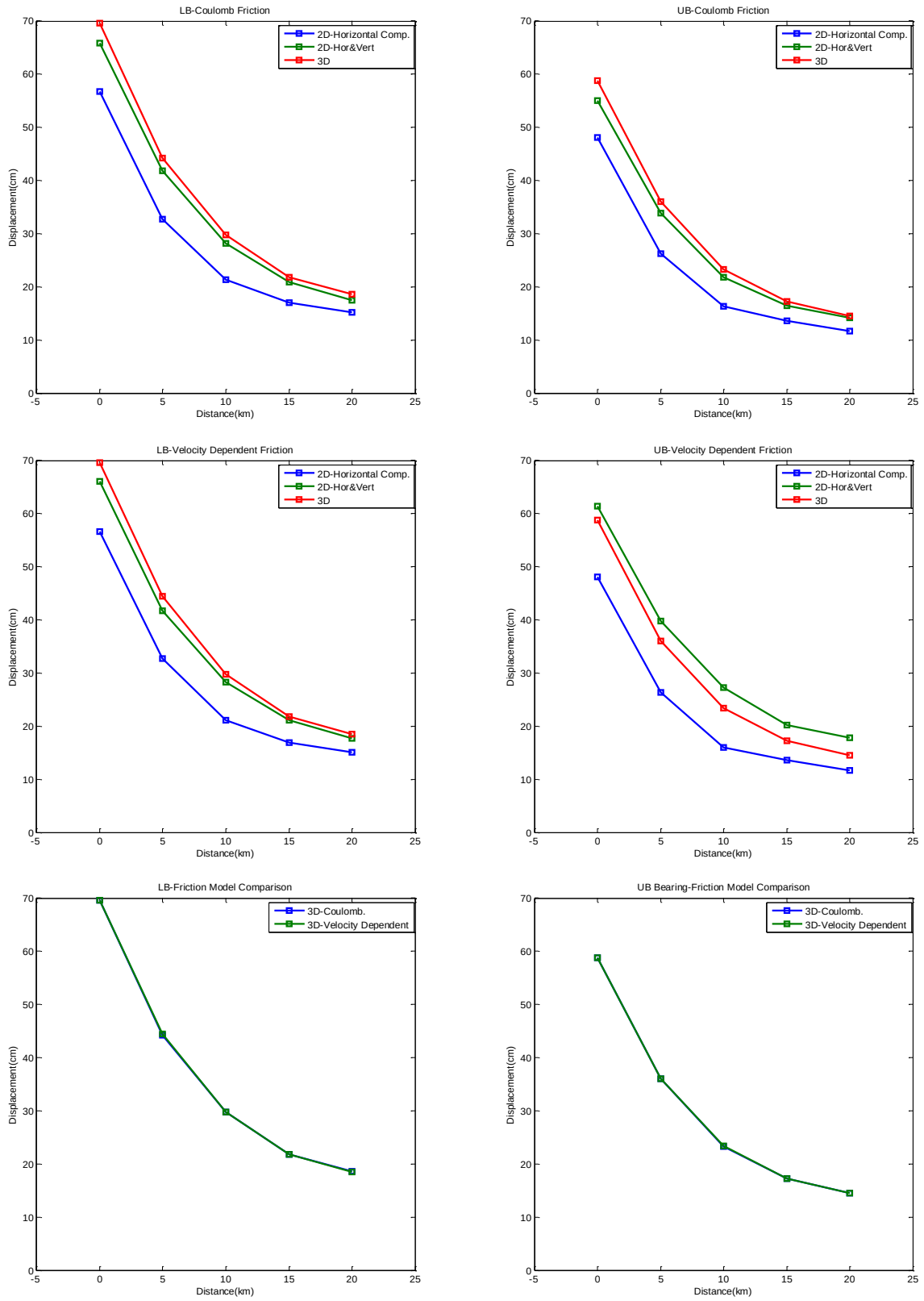


Figure 8.19. Variation of average displacement demand of curved surface spherical friction sliders under earthquake ground motion records with pulse content at MCE level.

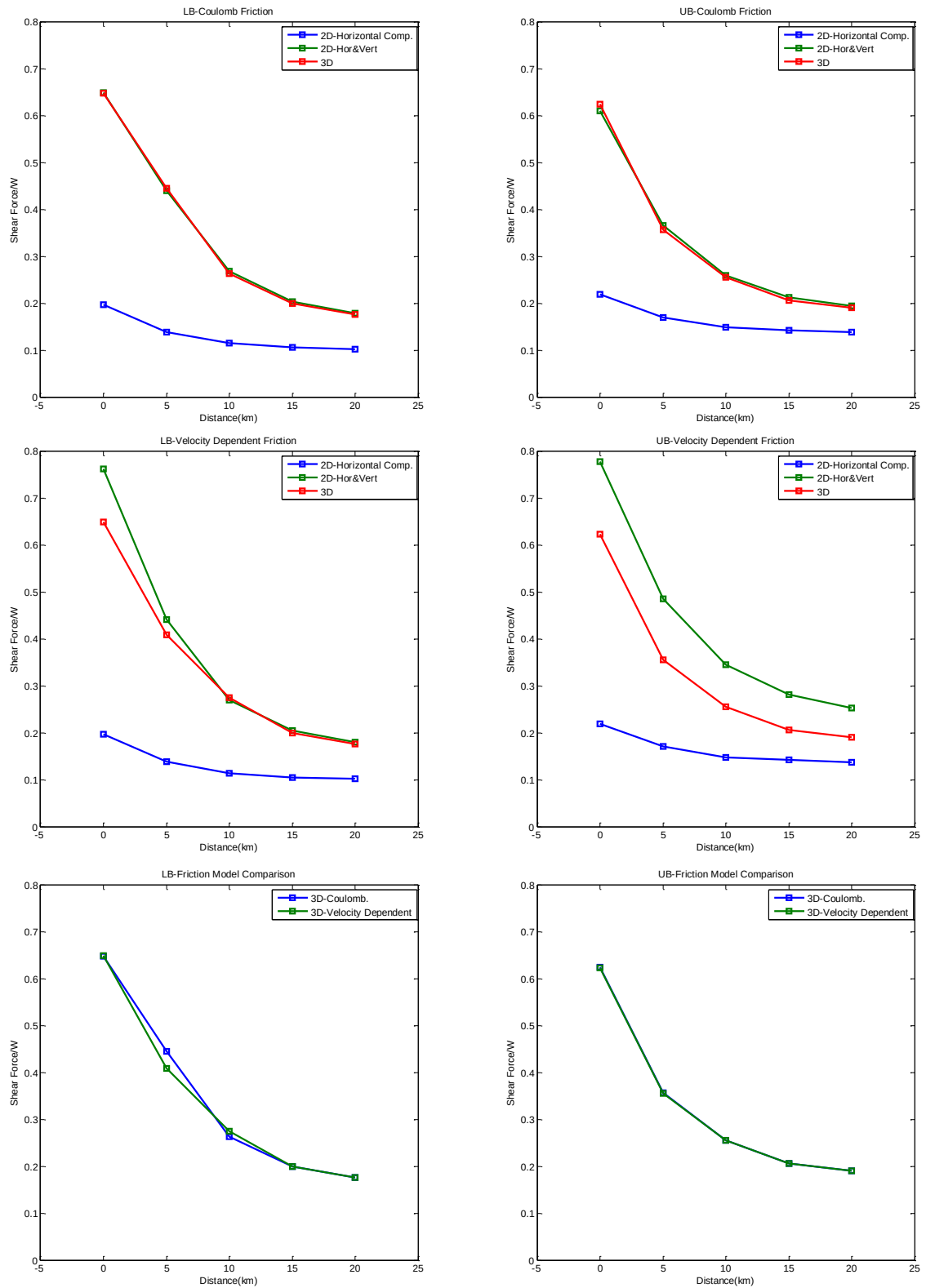


Figure 8.20. Variation of average shear force of curved surface spherical friction sliders under earthquake ground motions with pulse content at MCE level.

8.2.2.2. Average Response of the Simplified Bridge Model of Multi Curved Surface Friction Sliders (MCSFS). The conventional curved surface friction sliders has been extended to devices with multiple independent mechanisms to provide adaptable behavior. This proposal is firstly done by in 2002. Following the proposed model, various researchers have been worked on this multi-curved surface friction sliders experimentally (*e.g.* Morgan and Mahin, 2010, Fenz and Constantinou, 2008) and first analytical model through series connection is described by Fenz and Constantinou (2008). The proposed analytically series model of Fenz and Constantinou verified by experimental results and currently available single concave curved surface friction sliders and uniaxial gap elements are used to capture the overall behavior in SAP2000 (CSI, 2010). Moreover, Morgan (2007) and Becker and Mahin (2012) developed new bi-directional kinematic model of the multi curved surface friction sliders.

Multi curved surface friction sliders (MCSFS) differ from its former version on the basis of force-displacement regimes. It has a capability of performing adaptive behavior in terms of varying amplitudes of displacements. Regime five represents the reserved displacement capacity of the isolation unit as shown in Figure 8.21. This reserved displacement capacity sliding regime has been reached during the nonlinear response history analysis of simplified bridge model (Figure 8.22). Same suite of ground motions have been used to assess the main differences of sliding type of isolation bearings through the analysis results of simplified method. Nonlinear response history analysis are performed in OpenSees software under multi-directional earthquake loading under similar varying friction models. Multi-directional loading aims to capture the influence of vertical ground motions and fluctuations on the acting normal force.

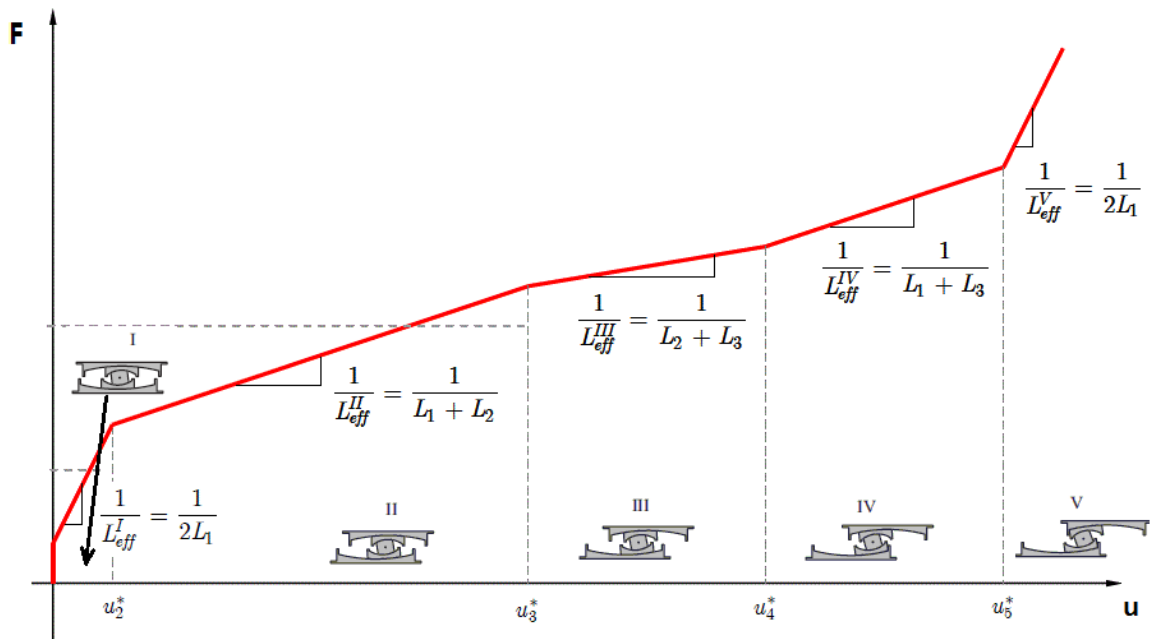


Figure 8.21. Sliding regimes and monotonic force-displacement behavior of multi-curved surface friction sliders.

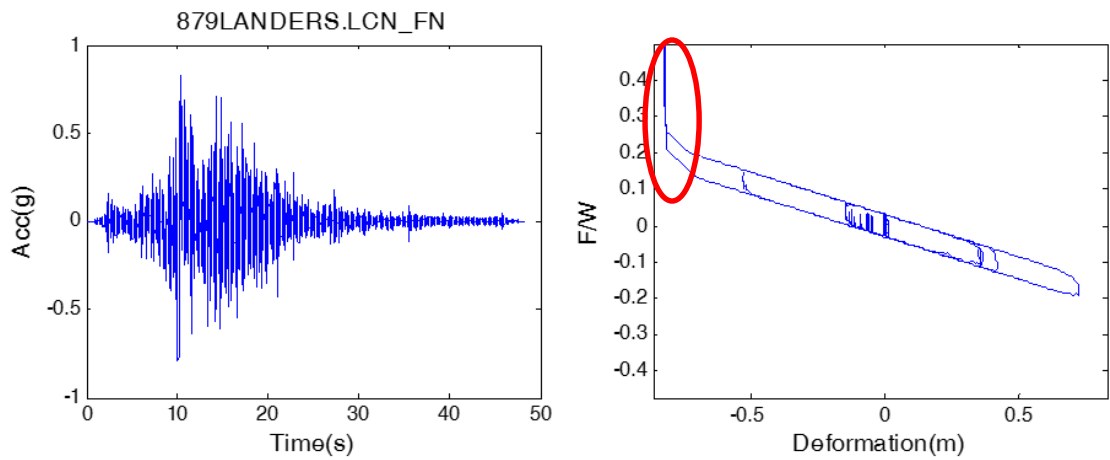


Figure 8.22. (a) Input Ground motion for the MCE-Level earthquake for pulse content sets (b) Hysteresis-loop of MCSFS that reached to the regime-5.

Variation of two important demand parameters through simplified analysis of MCSFS are depicted from Figure 8.23 to Figure 8.30.

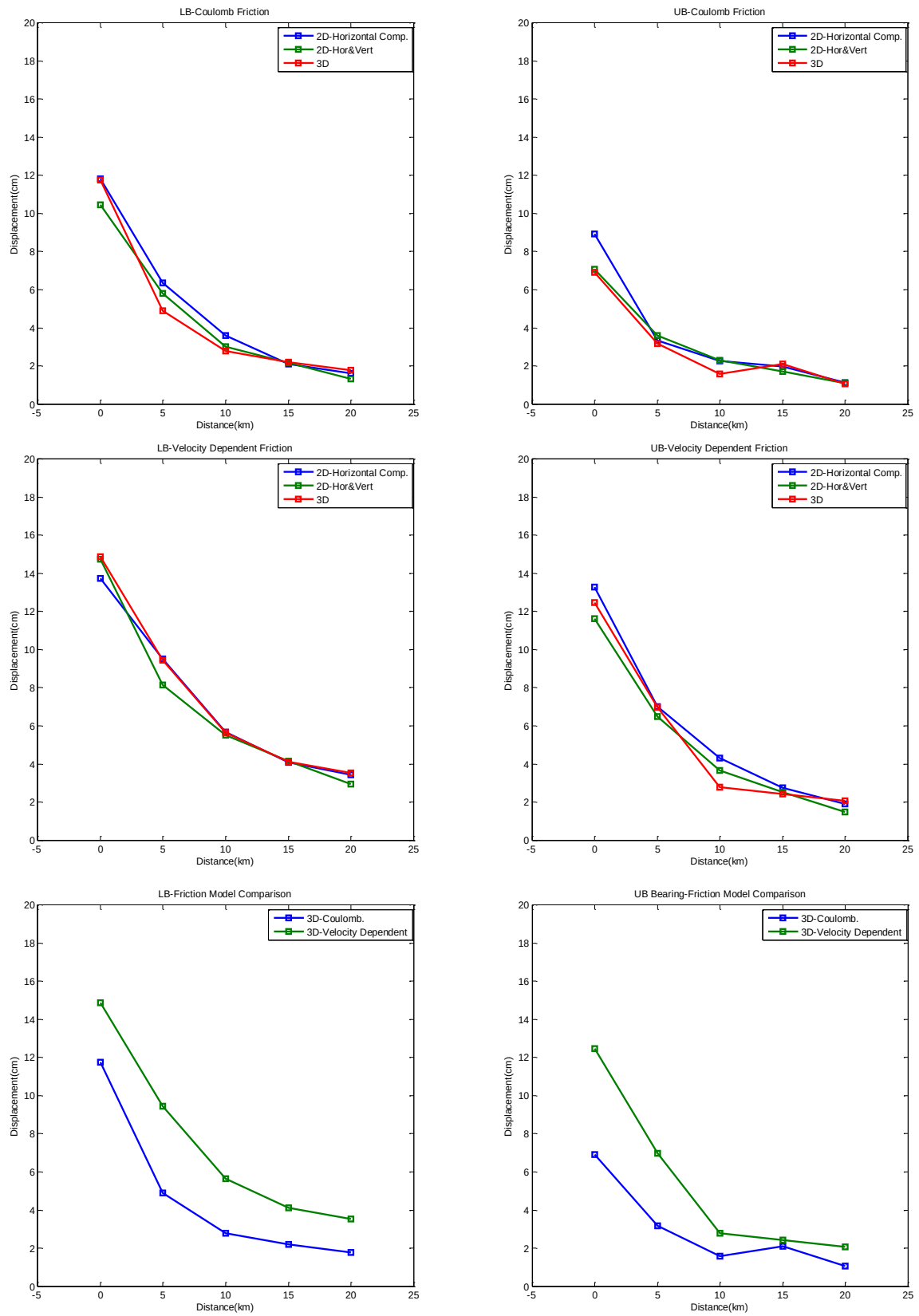


Figure 8.23. Variation of average displacement demand of MCSFS under earthquake ground motion records with nonpulse content at DBE level.

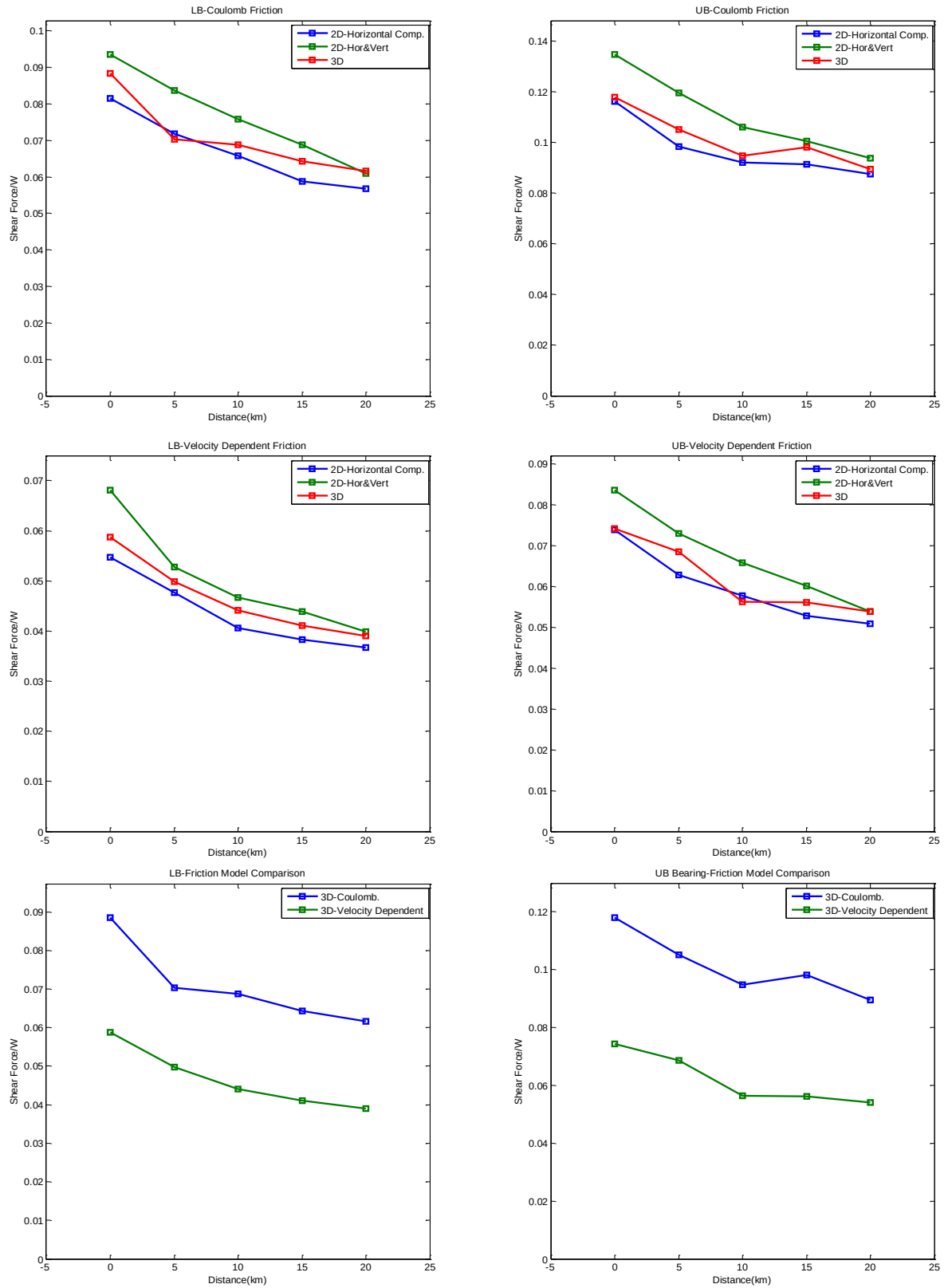


Figure 8.24. Variation of average shear force of MCSFS under earthquake ground motion records with nonpulse content at DBE level.

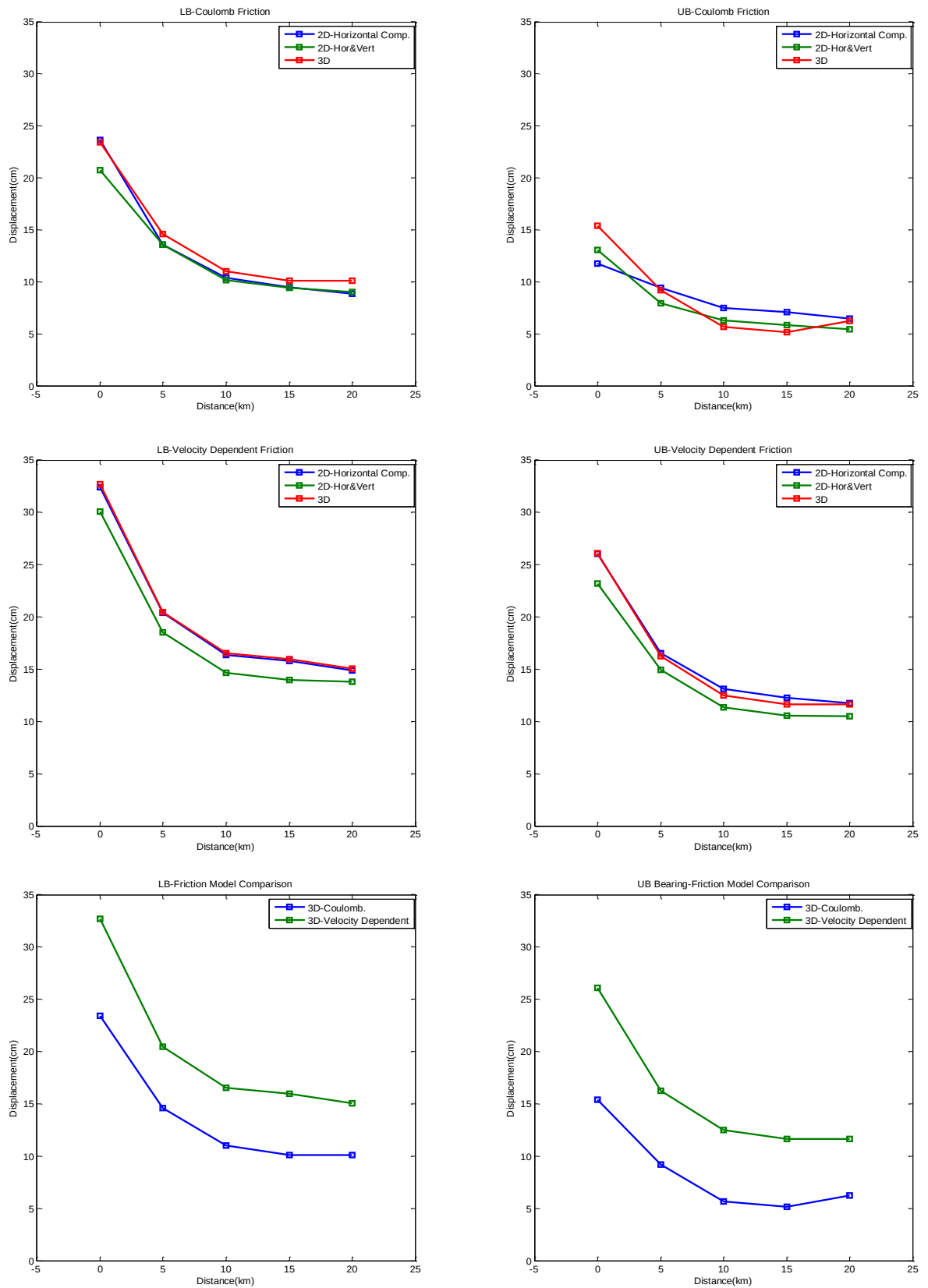


Figure 8.25. Variation of average displacement demands of MCSFS under earthquake ground motion records with pulse content at DBE level.

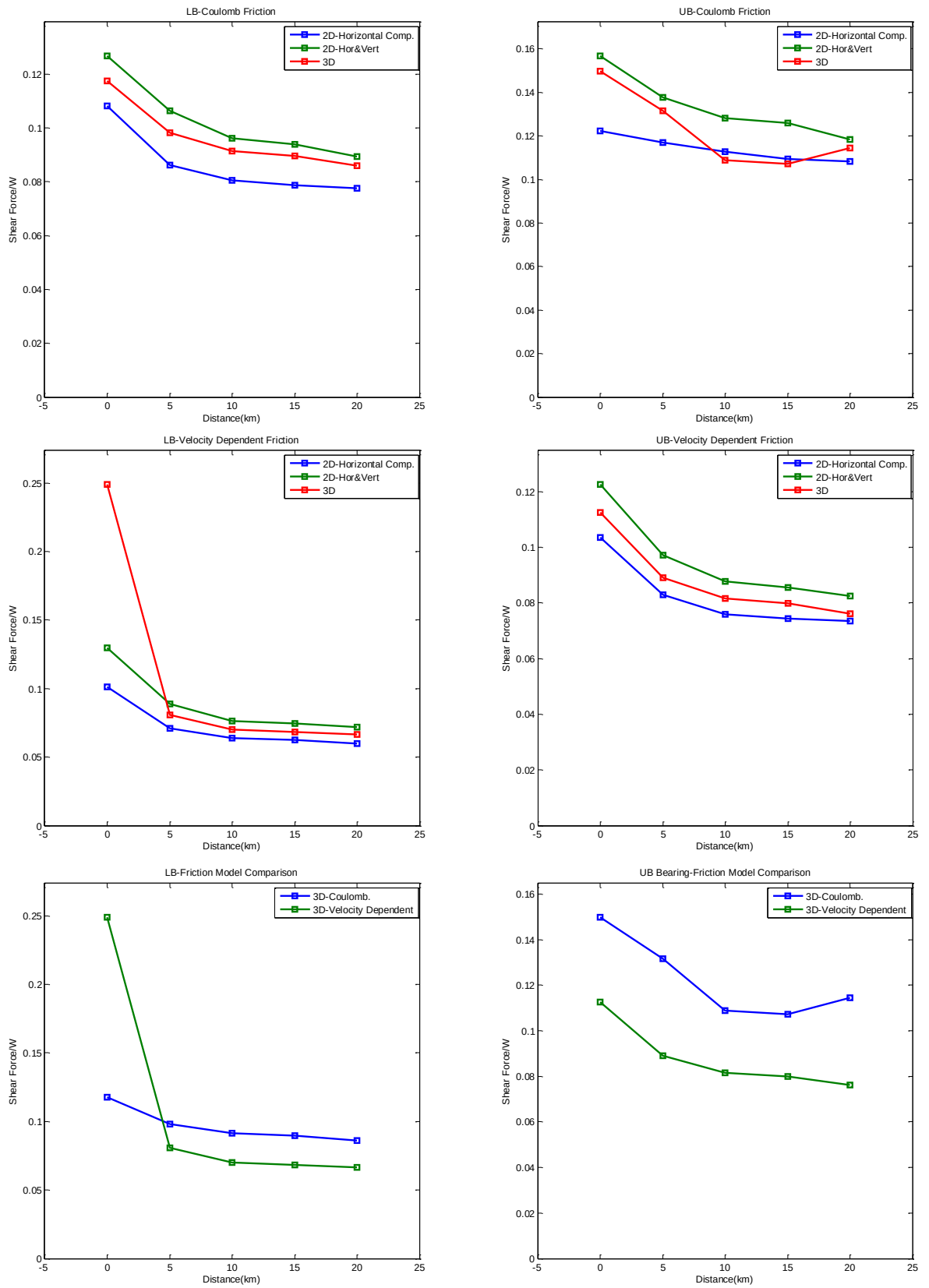


Figure 8.26. Variation of average shear force of MCSFS under earthquake ground motion records with pulse content at DBE level.

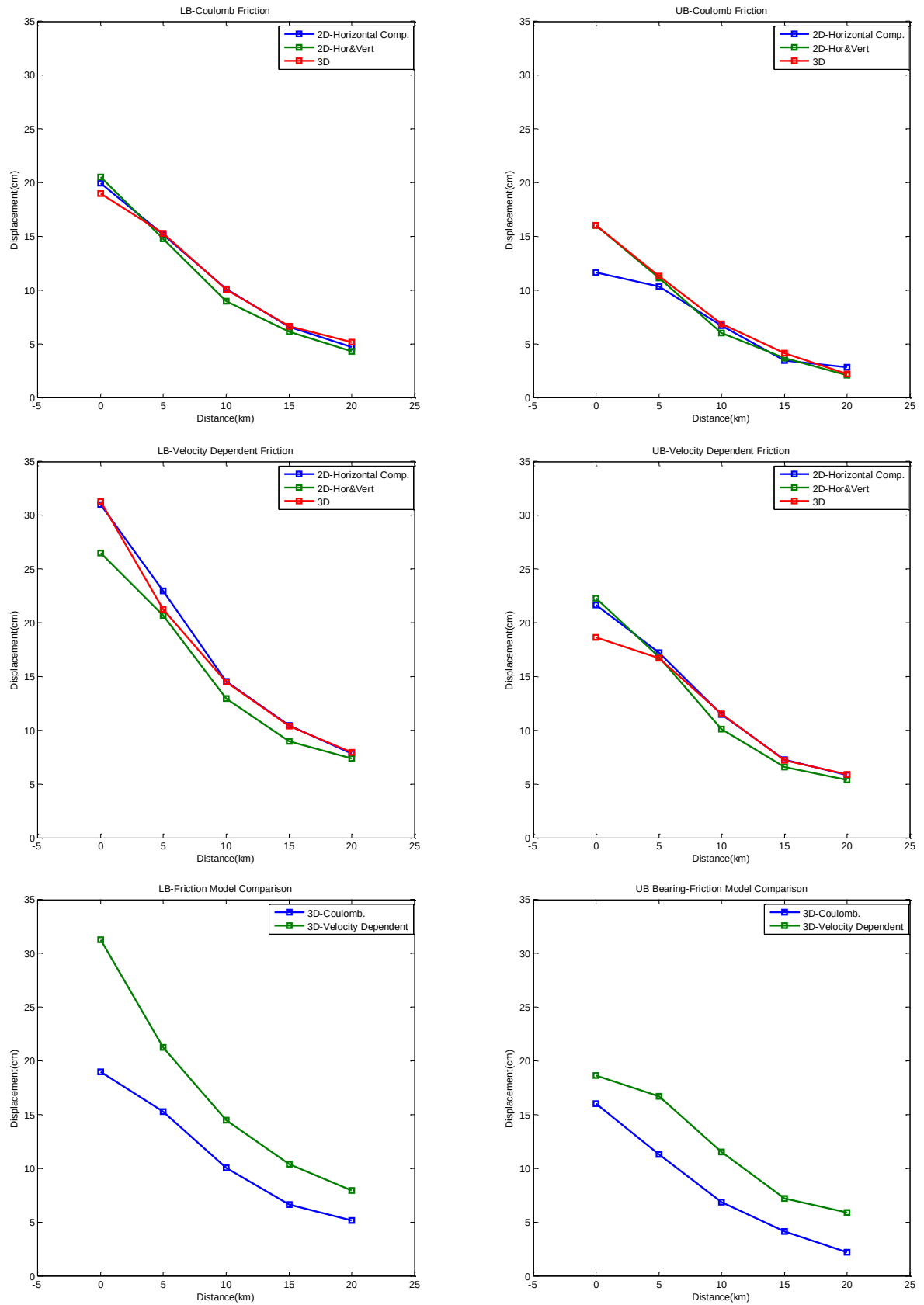


Figure 8.27. Variation of average displacement demand of MCSFS under earthquake ground motion records with nonpulse content at MCE level.

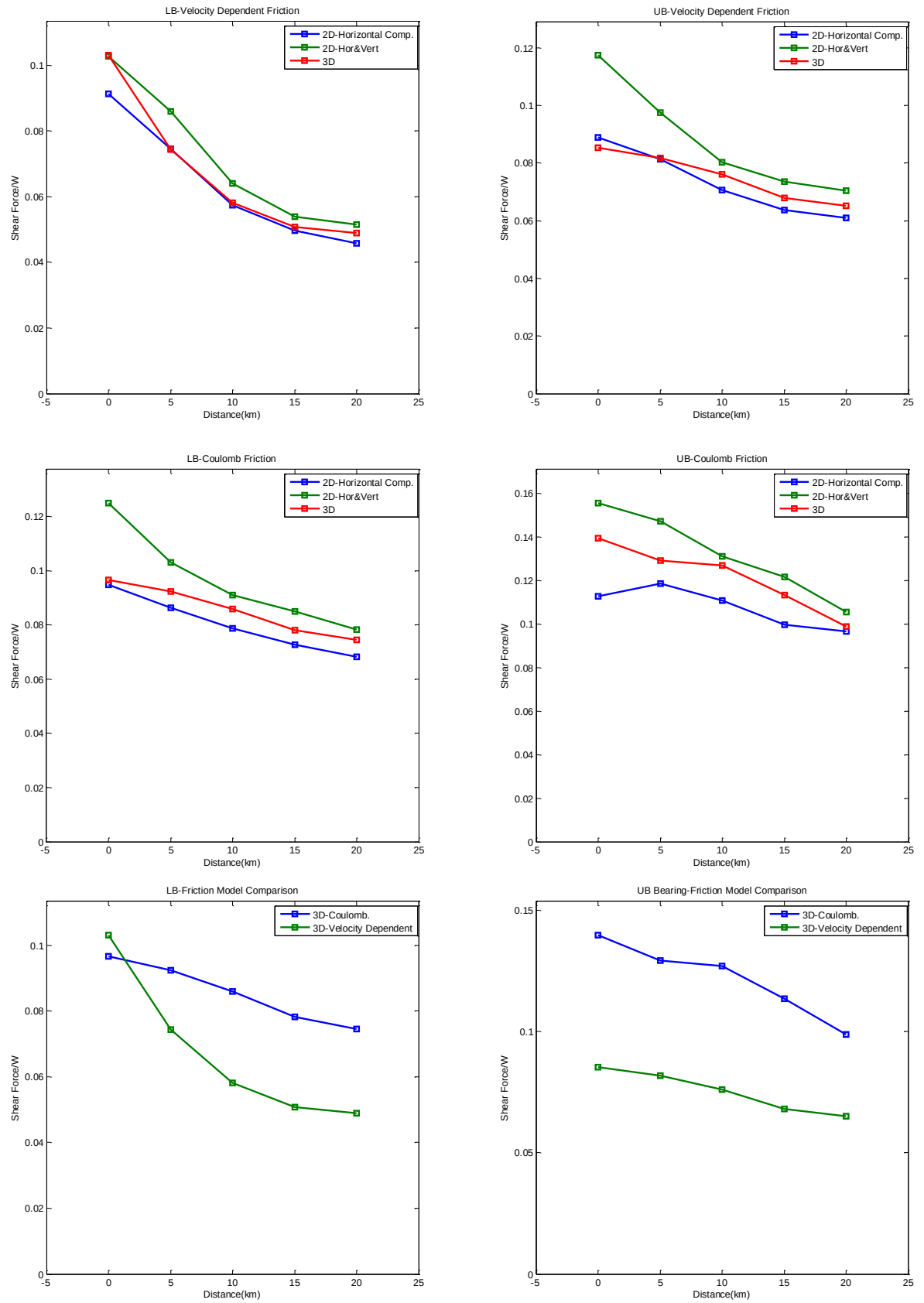


Figure 8.28. Variation of average shear force of MCSFS under earthquake ground motion records with nonpulse content at MCE level.

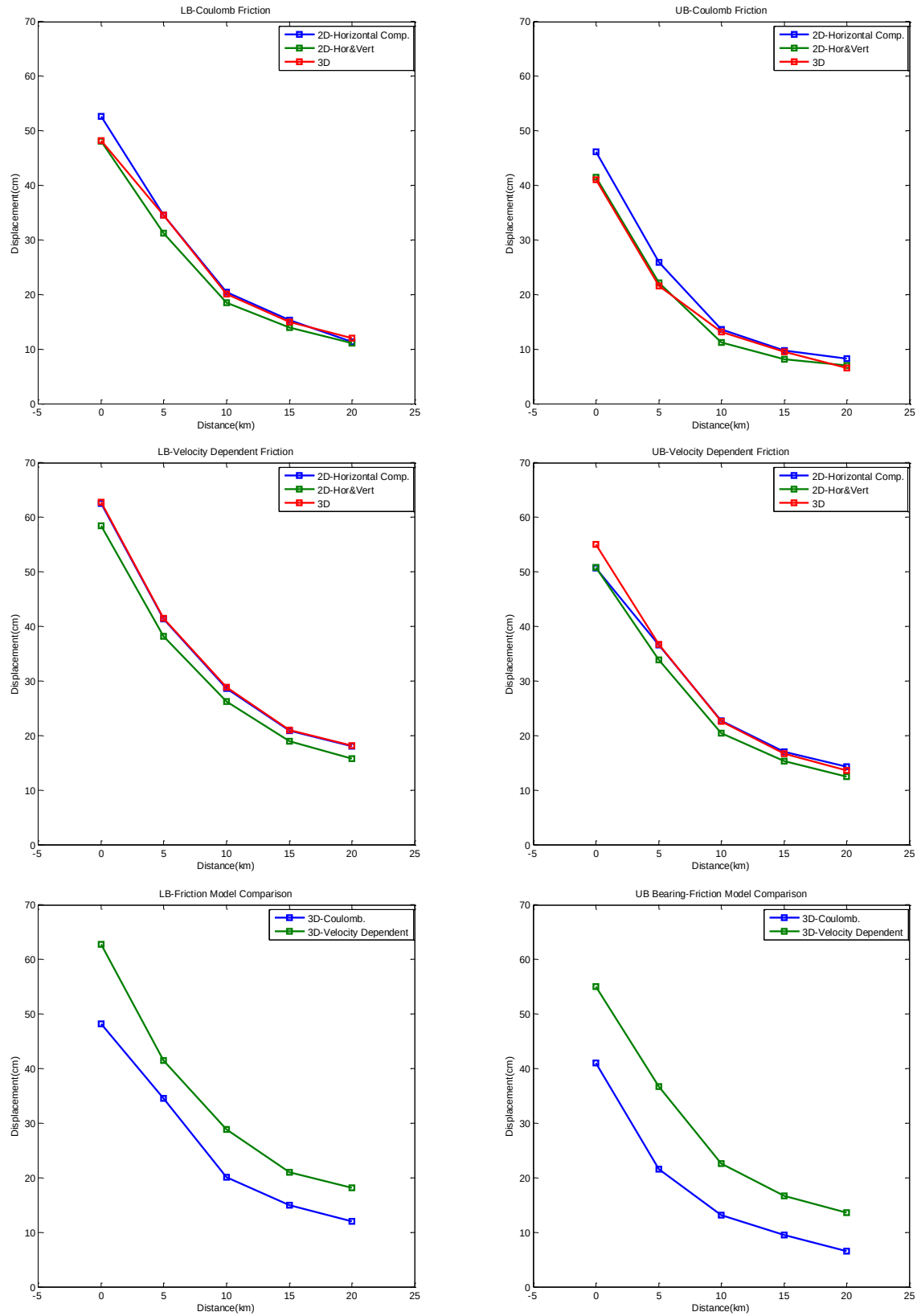


Figure 8.29. Variation of average displacement demands of MSCSFS under earthquake ground motions with pulse content at MCE level.

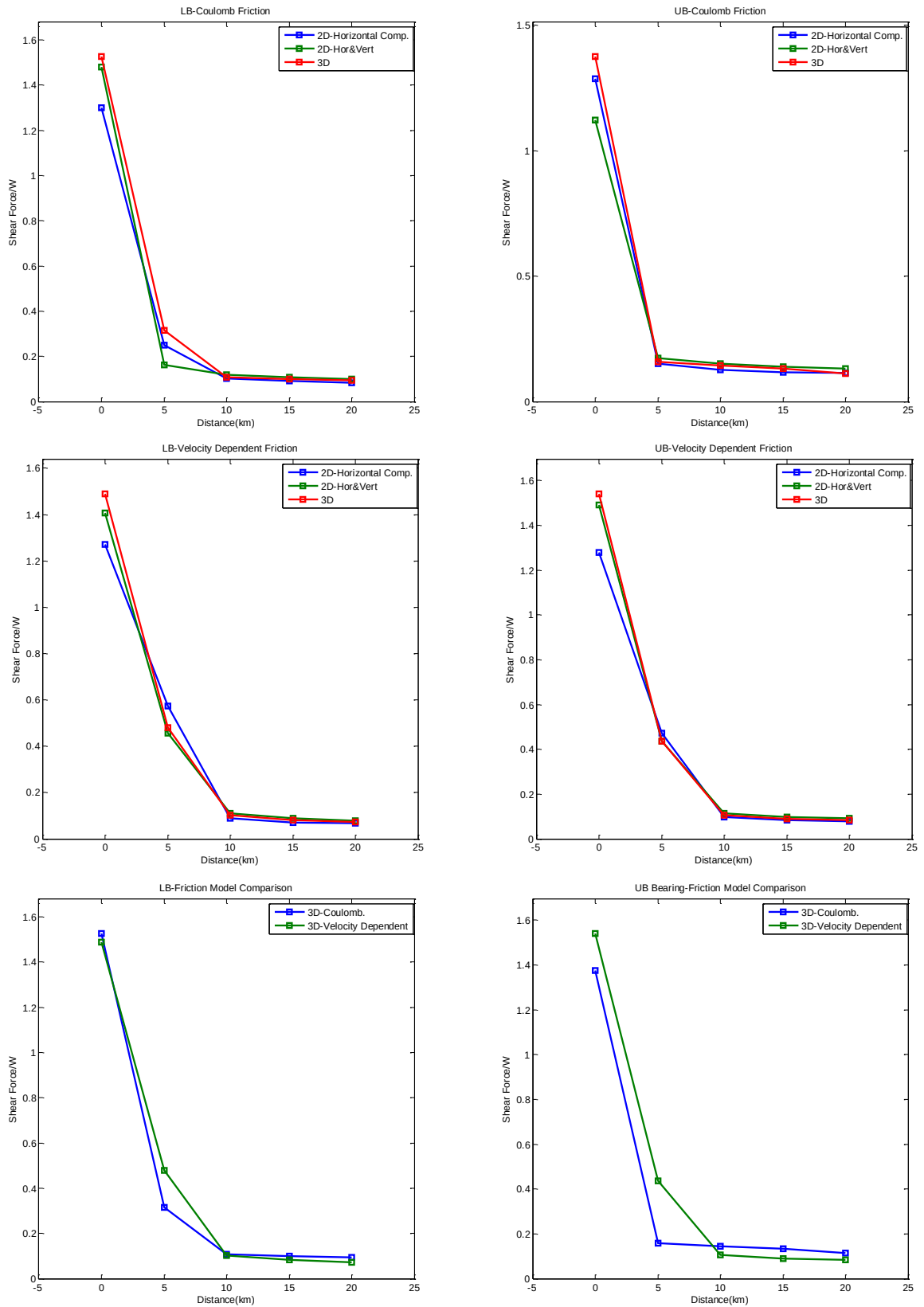


Figure 8.30. Variation of shear force of MCSFS under earthquake ground motion records with pulse content at MCE level.

pendulum systems) and its modeling assumptions. Analysis results by considering ground motions with and without pulse content at two level earthquake are illustrated in a comparative manner.

The same set of earthquake ground motions in simplified model are also applied for the 3D-Fully isolated model of the Bridge-A which accounts for the spatial variation of seismic isolators. Bridge-A has used to investigate the behavior of seismic isolators under 1D, 2D and 3D input ground motions. This will provide insight about the capability of currently available methods to capture the influence of coupled plasticity, effect of pulse content and variation of normal load on isolators. Peculiar characteristics of pulse content of ground motions on conventional curved surface friction slider are shown in Figure 8.32 and Figure 8.33.

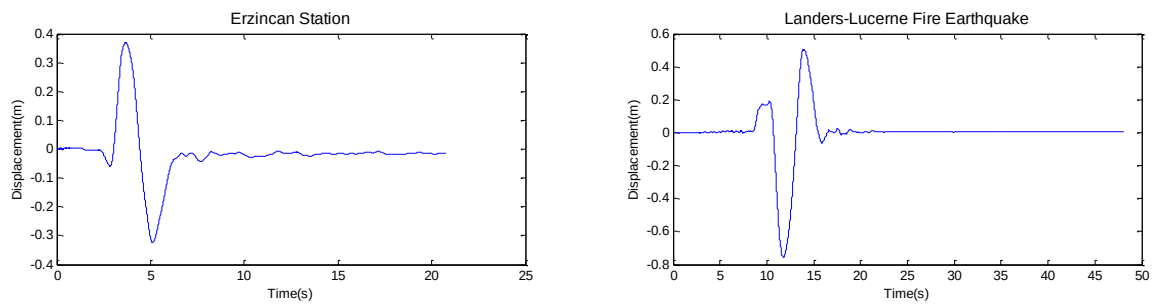


Figure 8.32 Deformation history at abutment location under pulse like ground motions of Erzincan Earthquake and Landers Lucerne Fire Station at MCE level for MCSFS type isolator.

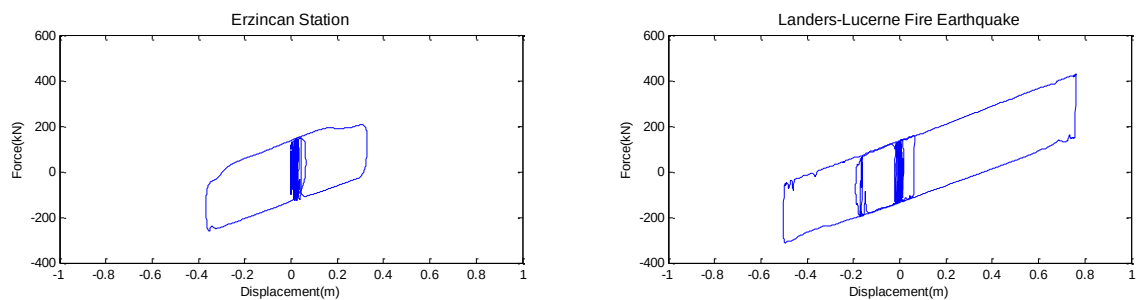


Figure 8.33. Hysteresis curve of abutment isolator under Landers Lucerne Fire Station and Erzincan.

Results of NRHA are expressed by box plots for providing a quick way of convenient visualization of maximum, minimum, median values of ground motions. Upper and lower bound of the box represents the first and third quartile, respectively. Moreover, bar graphs of

each suite of ground motion have provided the variation of seismic demands with respect to input ground motions in the selected set depicted in these graphs (Figure 8.34 to 8.49).

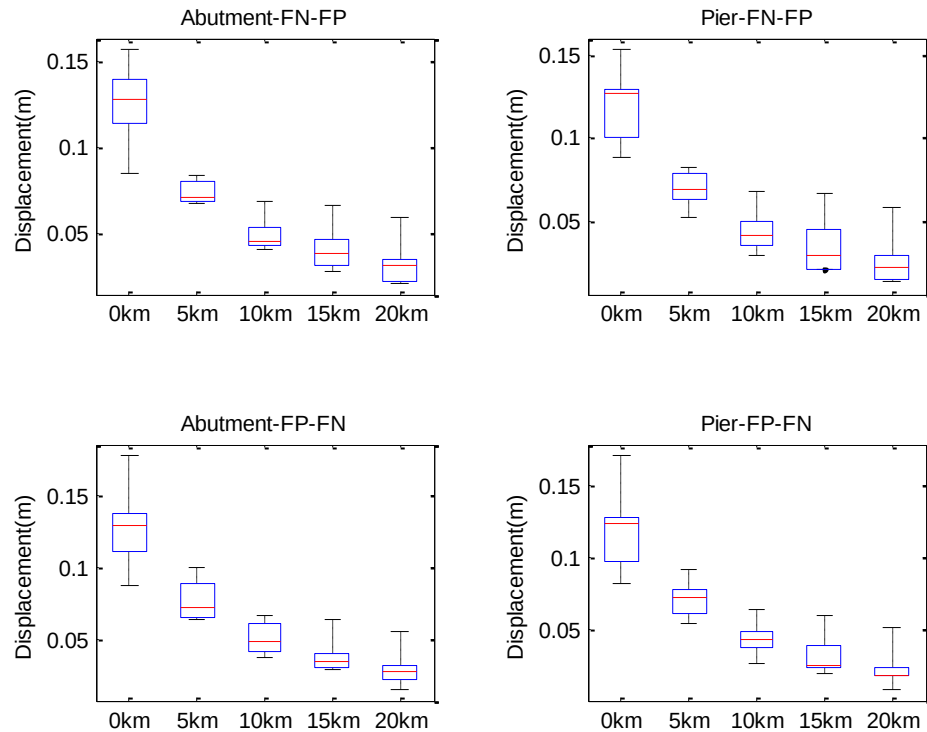


Figure 8.34. Variation of displacement demand of isolators under ground motion records with nonpulse content and for DBE level earthquake.

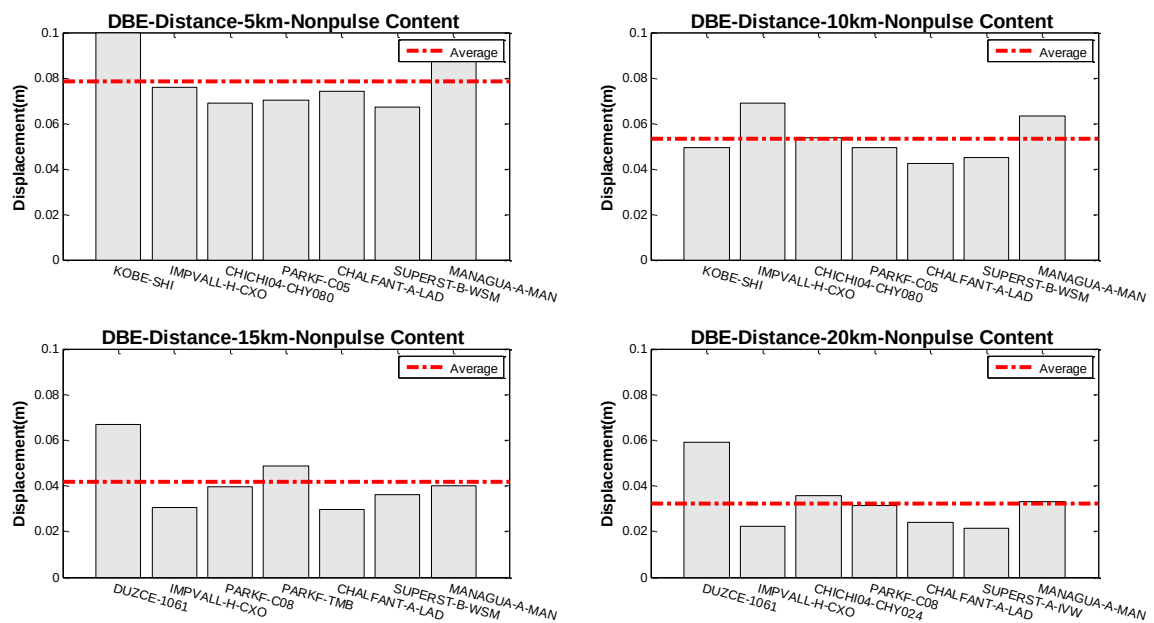


Figure 8.35. Variation displacement demand of isolators under ground motion records with nonpulse content for DBE level earthquake.

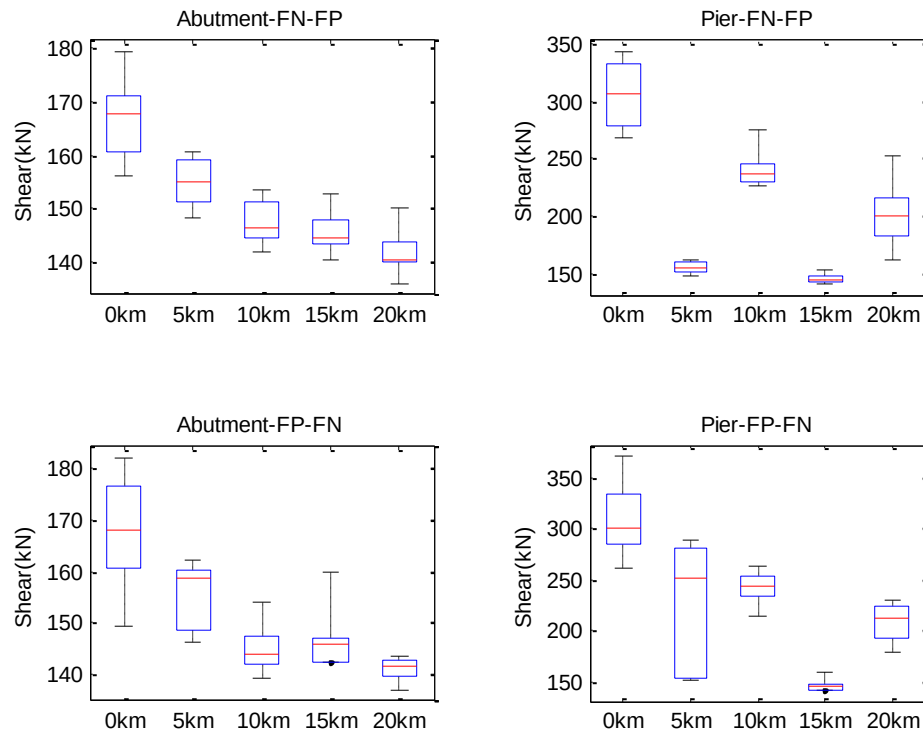


Figure 8.36. Variation of shear force of isolators under ground motions records with nonpulse content for DBE level earthquake.

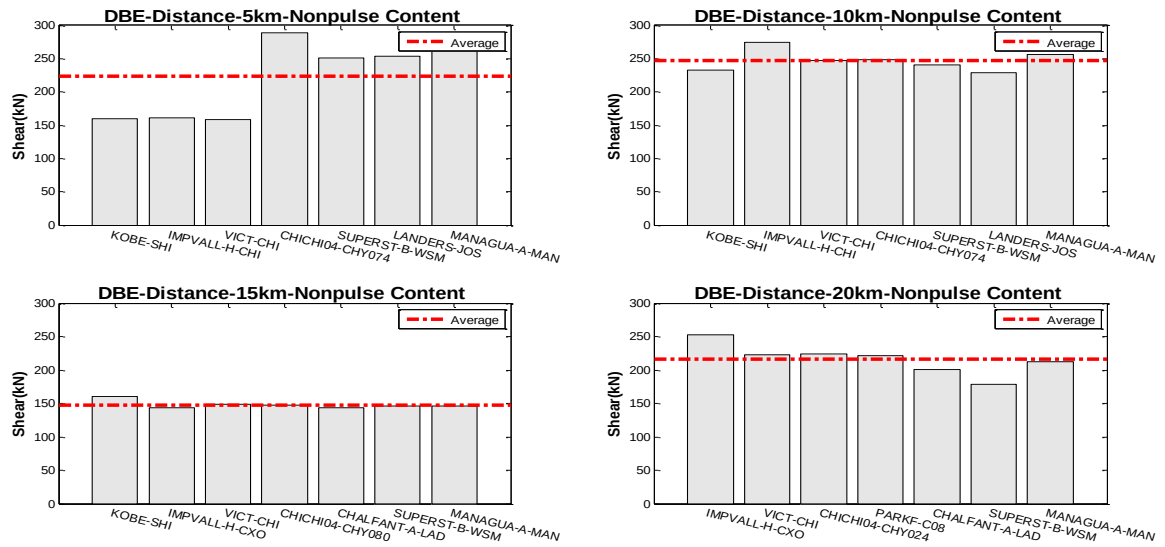


Figure 8.37. Variation of shear force of isolator under near-fault ground motion records with nonpulse content for DBE level earthquake.

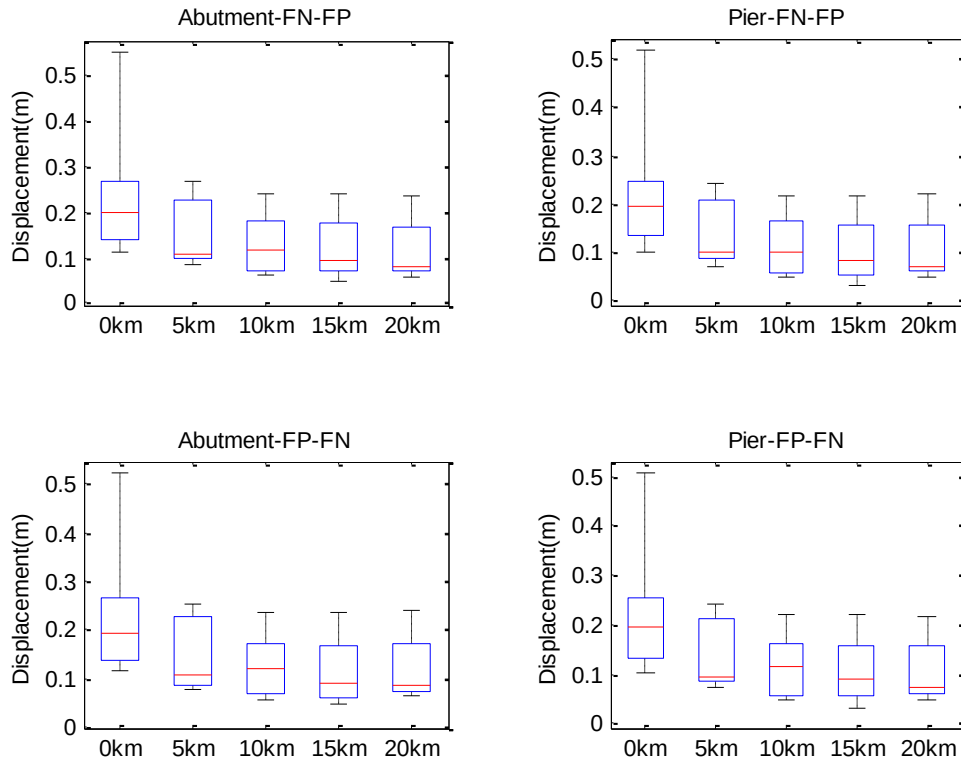


Figure 8.38. Variation of displacement demand of isolators under near-fault ground motions records with pulse content for DBE level earthquake.

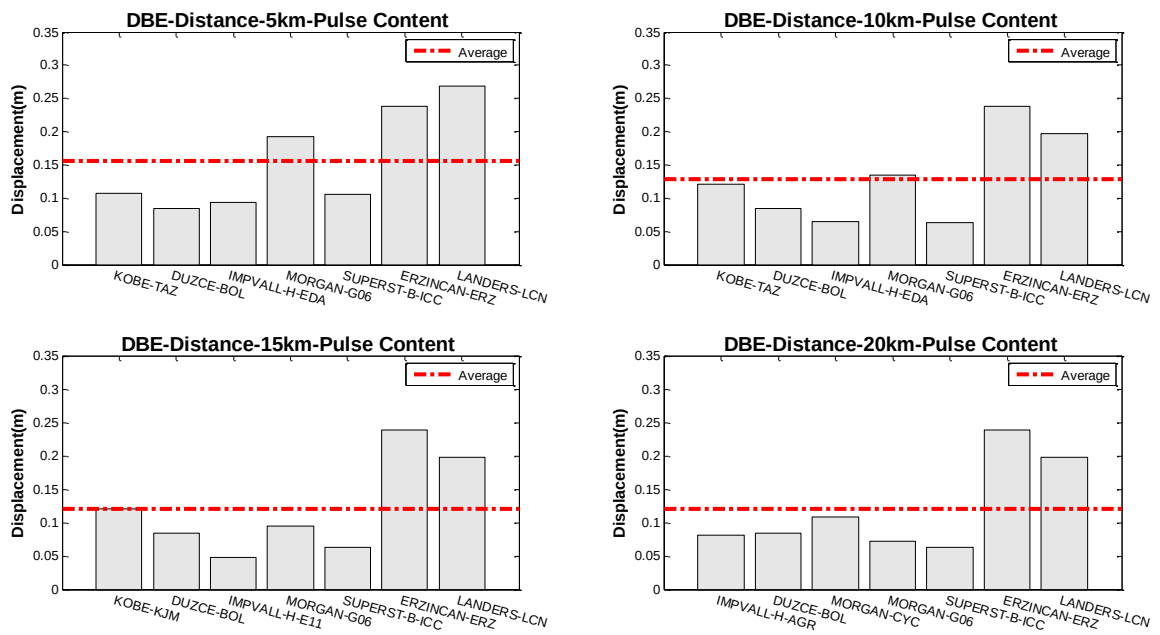


Figure 8.39. Variation of displacement demand of isolators under ground motion records with pulse content and average value for DBE level earthquake.

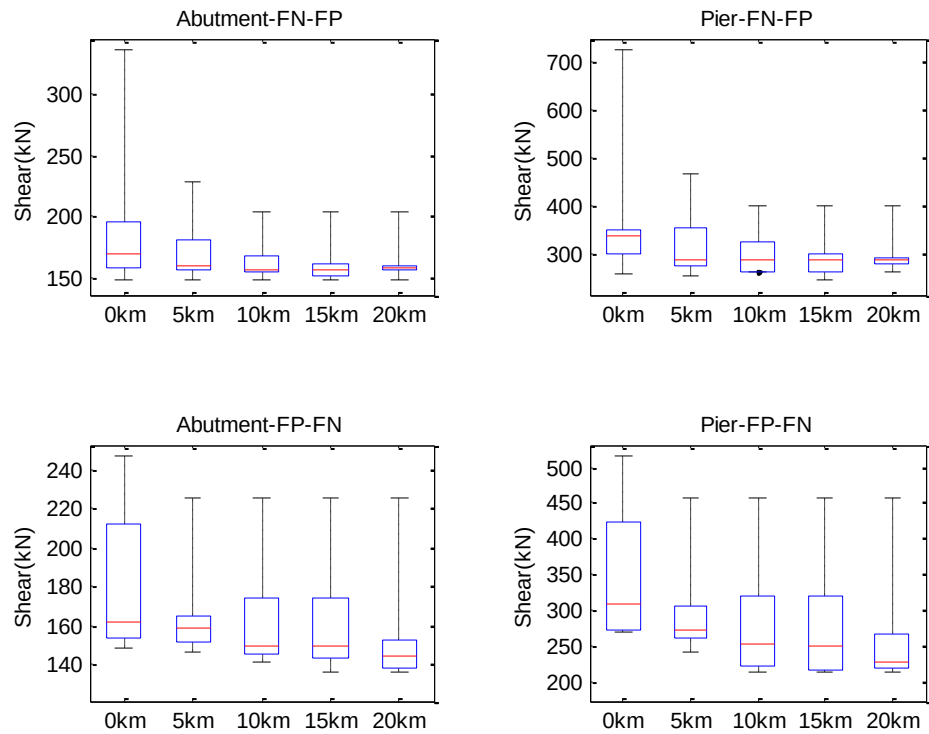


Figure 8.40. Variation of shear force of isolators at abutment and piers under ground motion records with pulse content and average value for DBE level earthquake.

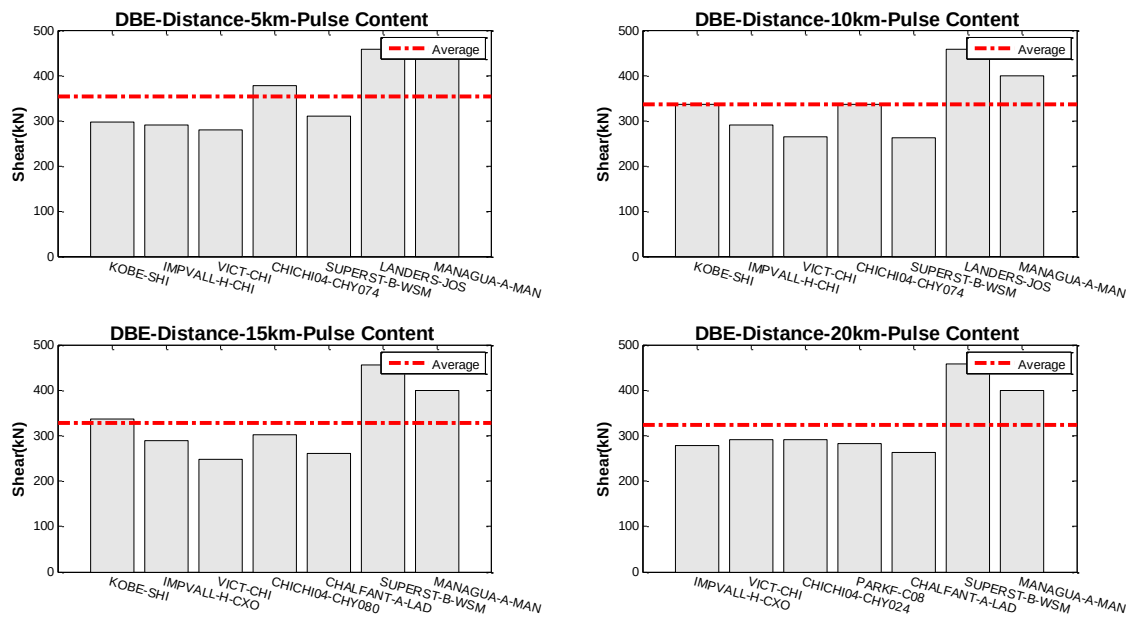


Figure 8.41. Variation of shear force of isolators under ground motion records with pulse content for DBE level earthquake.

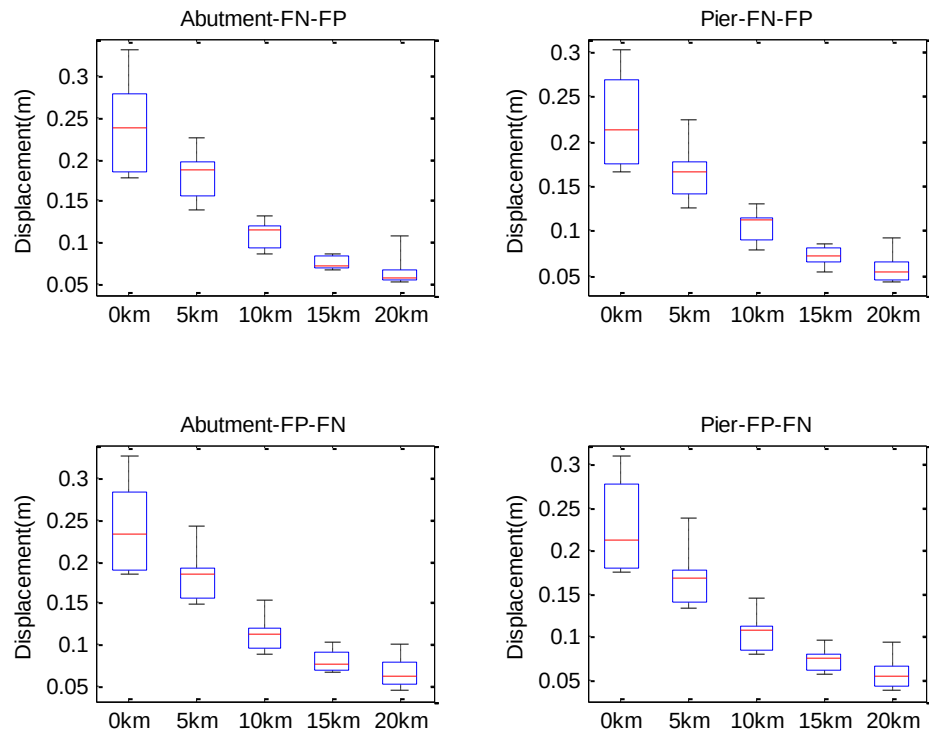


Figure 8.42. Variation of displacement demand of isolators under ground motion records with nonpulse content for MCE level earthquake.

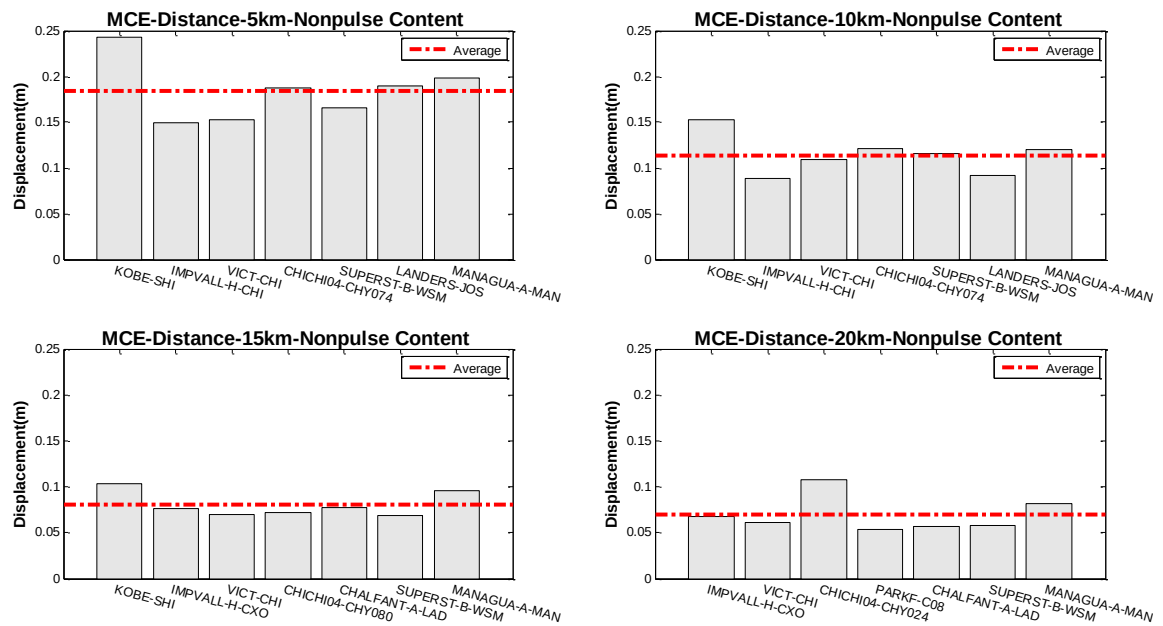


Figure 8.43. Variation of displacement demand of isolators under ground motion records with nonpulse content for MCE level earthquake.

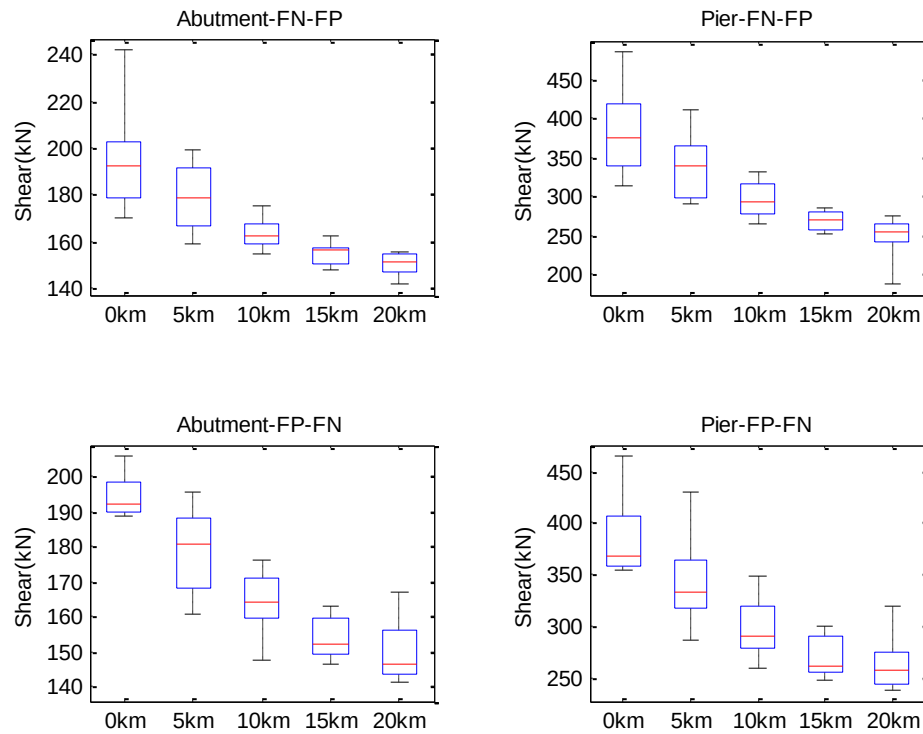


Figure 8.44. Variation of shear forces of isolator under ground motion records with nonpulse content for MCE level earthquake.

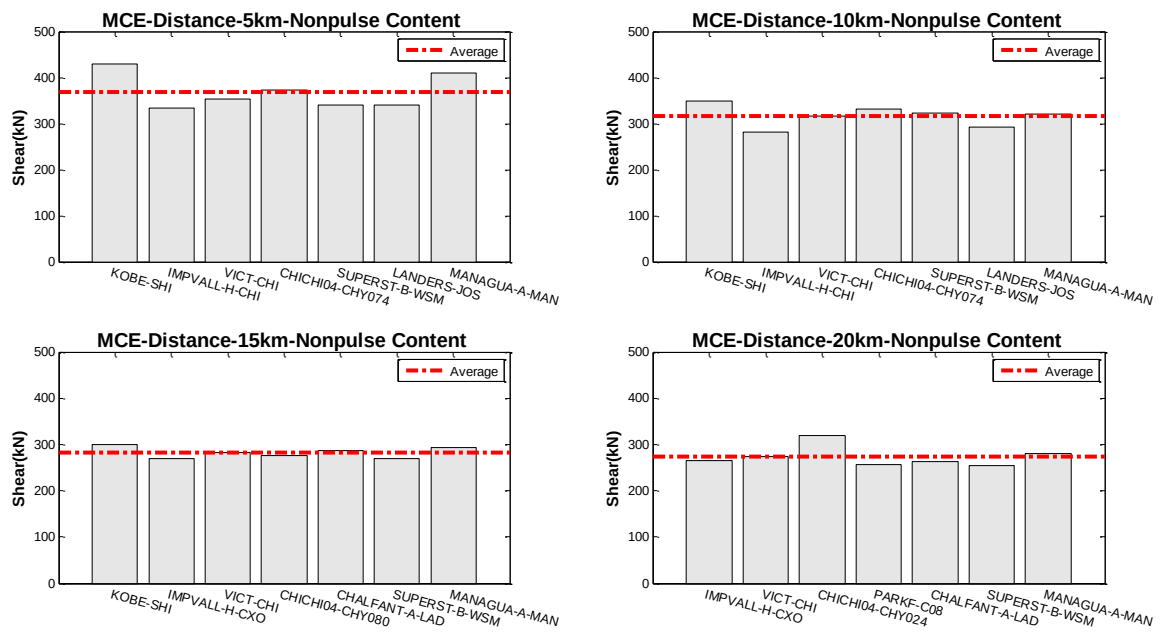


Figure 8.45. Variation of shear force of isolators under ground motion records with nonpulse content for MCE level earthquake.

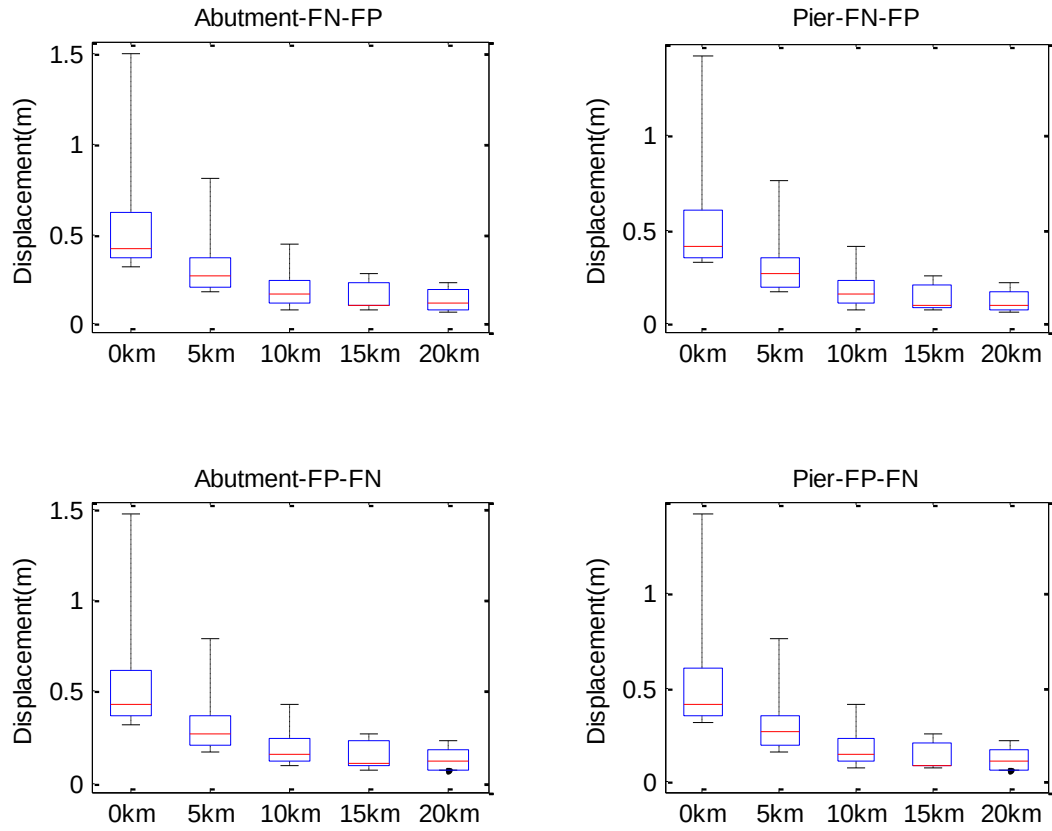


Figure 8.46. Variation of displacement demand of isolators under ground motion records with pulse content for MCE level earthquake.

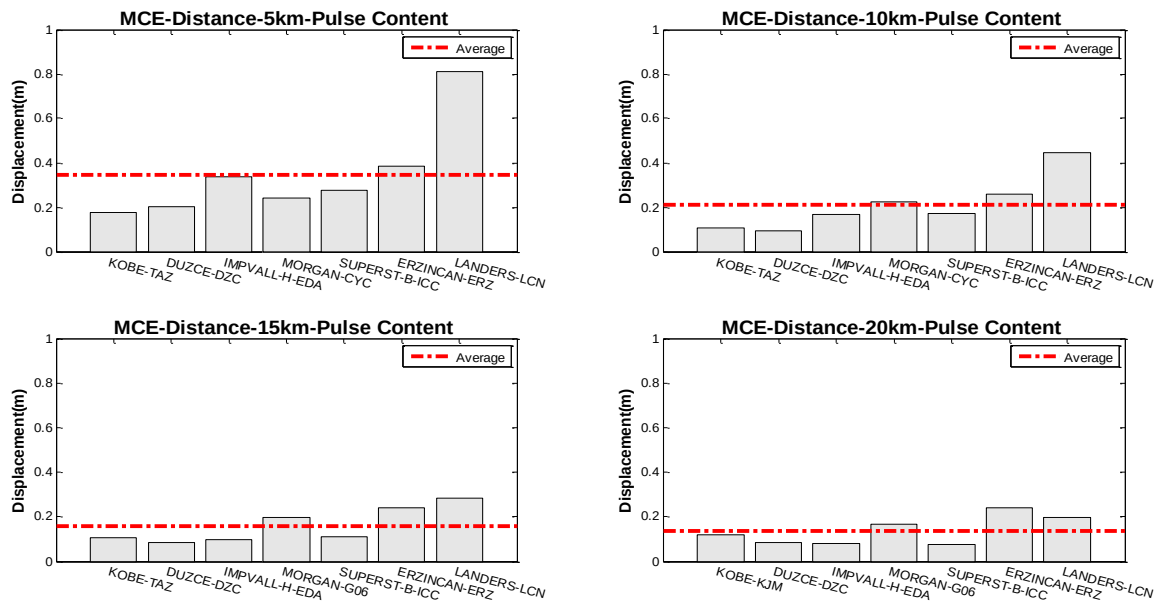


Figure 8.47. Variation of displacement demand of isolators under ground motion records with pulse content for MCE level earthquake.

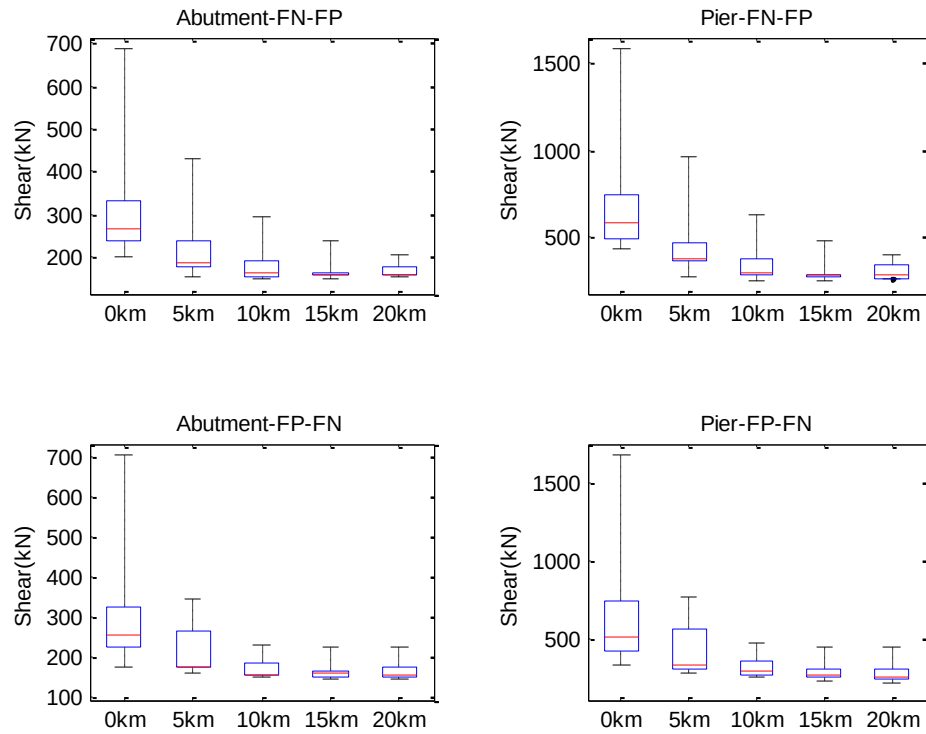


Figure 8.48. Variation of shear force of isolator at abutment and pier locations under ground motion records with pulse content for MCE level earthquake.

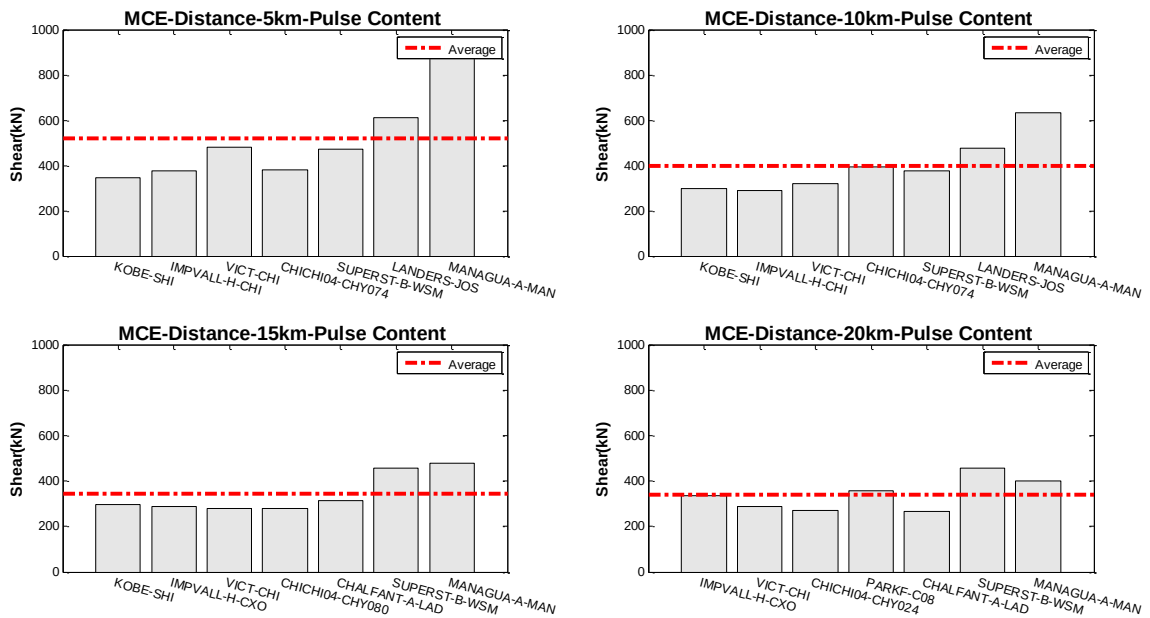


Figure 8.49. Variation of maximum shear force of isolator and at abutment and piers under near-fault ground motion records with pulse content for MCE level earthquake.

Two sets of mechanical characteristics are defined for each type of isolation system that is used in Bridge-A. Such acceptable changes or variability of design parameters provide the range of possible values that may be experienced during the entire service life of the isolation system. The upper bound values of isolation units describe the behavior of aged and contaminated bearings, low temperature effects during the first high speed cycle of seismic motion, travel accommodation which is a property of traffic loading. The largest force demand on the substructure occur as a result of this characteristic mechanical properties. In contrast, the lower bound values describe the behavior of fresh bearings at normal temperature that follows the initial cycle of the high speed motion. System property modification values are used to compute the required parameters, these parameters are given in relevant sections of guide specifications for seismically isolated bridges (*e.g.* AASHTO 2010). Variation of displacement demands and the transmitted shear forces for the lower and upper bound values of MCSFS in the Bridge-A are shown from Figure 8.50 to Figure 8.53 . Similar procedure is also applied for single concave curved surface friction sliders and lead-rubber bearings. Engineers can consult to the suppliers for these two mechanical characteristics of isolation systems.

Property modification factors will be updated with respect to currently ongoing 5 year program of full scale bridge test in the scope of NEES project coordinated by Andre Filiatrault. Currently available response modification factors were defined in the study of Constantinou *et al.* (1996) and it has been adopted to relevant code sections of design codes of structures and seismic design guidelines (*e.g.* ASCE 7-10, AASHTO Guide Specification, 2010).

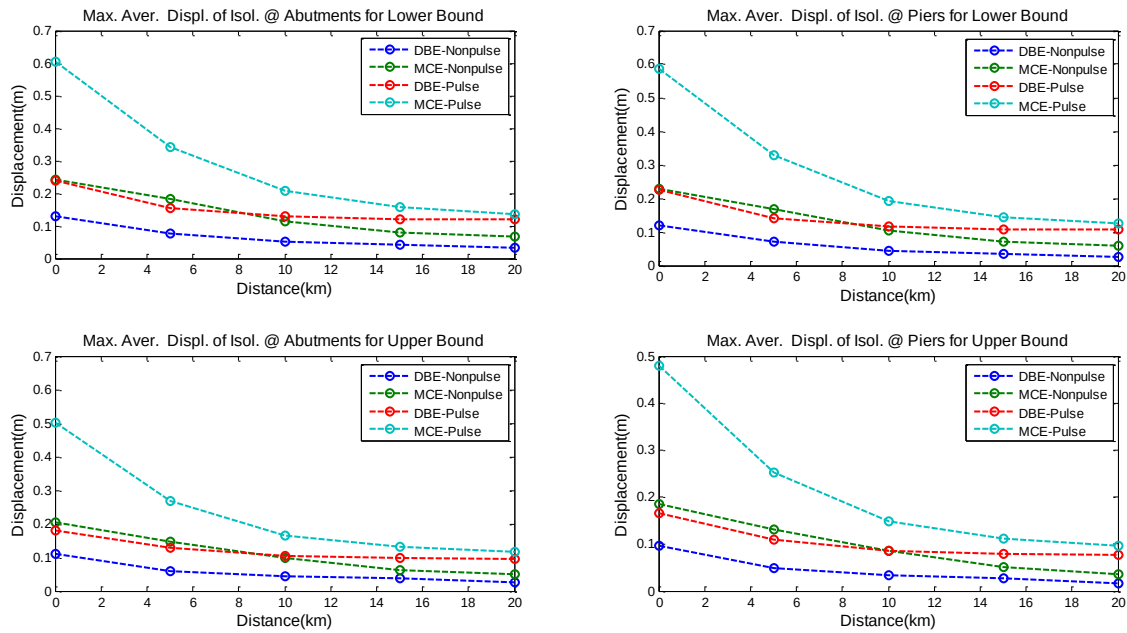


Figure 8.50. Variation of displacement demands at abutment and pier locations for the mechanical properties of MCSFS.

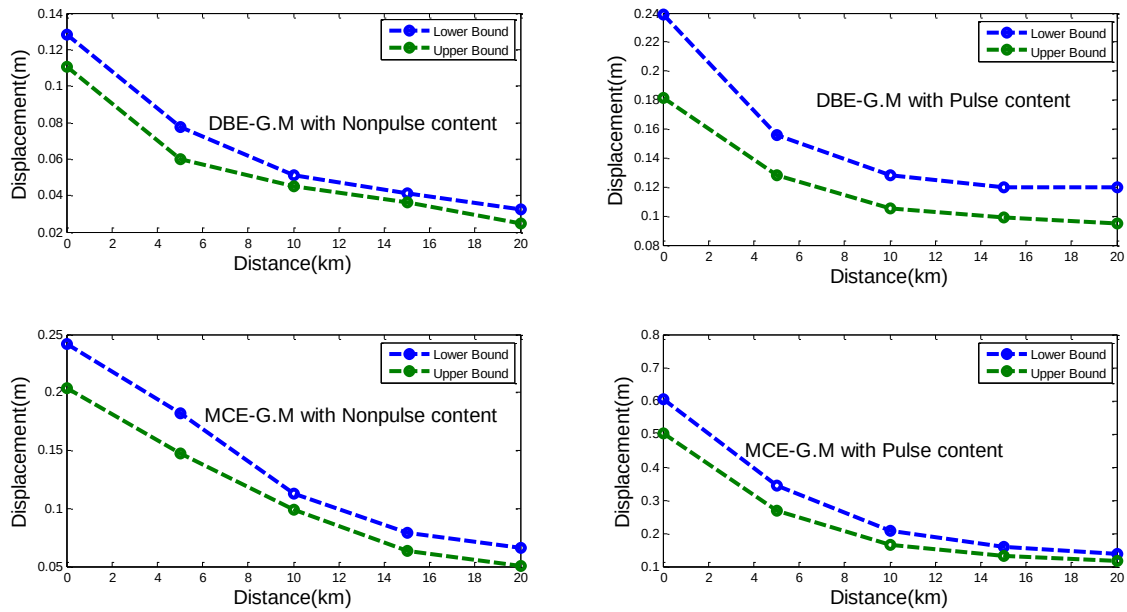


Figure 8.51. Variation of displacement demand for ground motions with and without pulse contents of MCSFS.

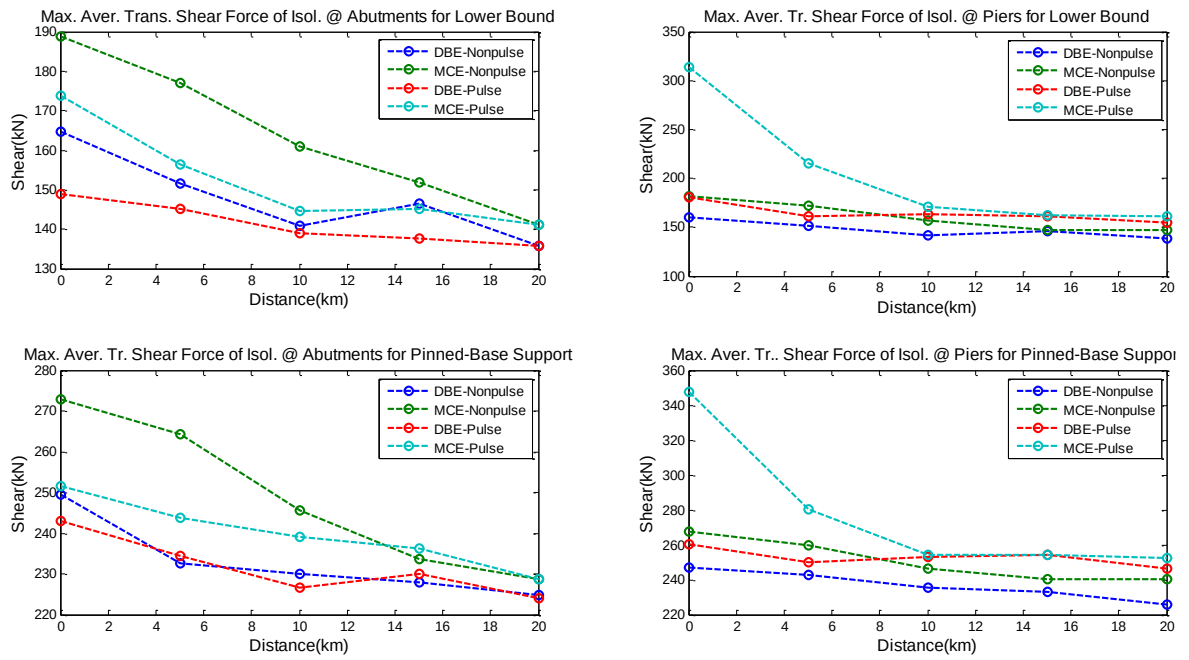


Figure 8.52. Variation of shear demands at abutment and pier locations for two mechanical properties of MCSSFS.

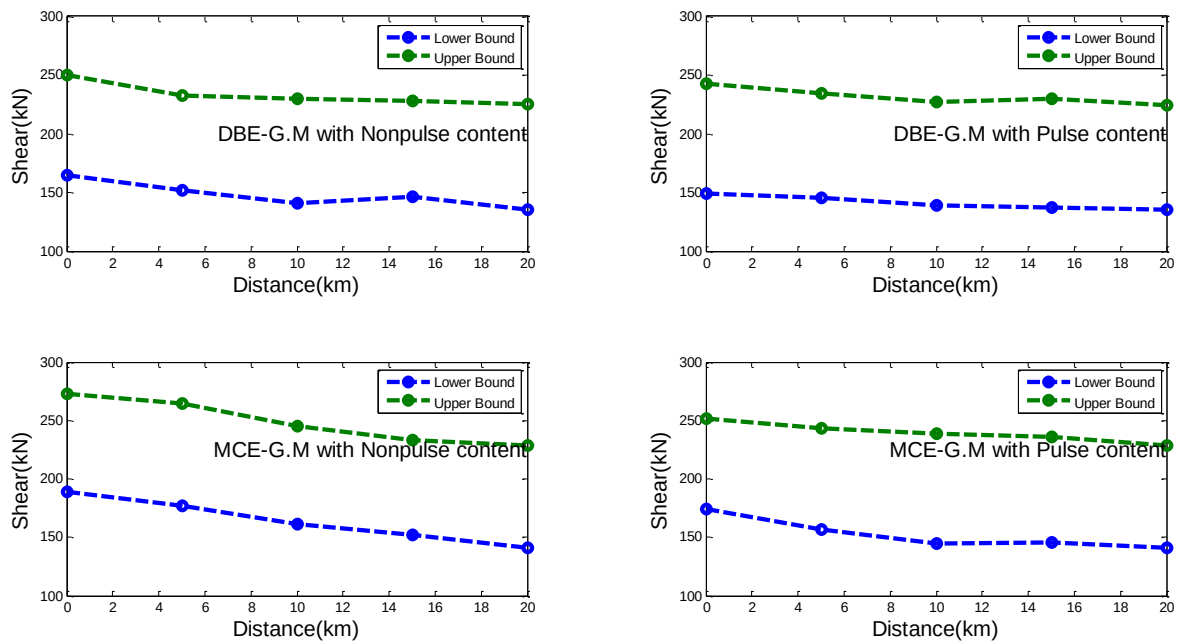


Figure 8.53. Variation of displacement demand for two mechanical characteristics of MCSSFS.

Effect of skew angle on the multi surface curved spherical friction sliders are assessed through nonlinear time history analysis in terms of displacement demand and transmitted shear forces. 3D-Fully isolated model of the Bridge-A which accounts for the spatial variation of seismic isolators is shown in Figure 8.54 for three configurations. The amount of variation in displacement and transmission of shear force are demonstrated from Figure 8.55 to Figure 8.58. Influence of directionality is assessed by rotating the FN-FP oriented scaled ground motions also in their principal direction. Additionally, comparison of skew angle effect is tested by considering the effect of directionality by rotating the FN-FP oriented ground motions to their principle directions.

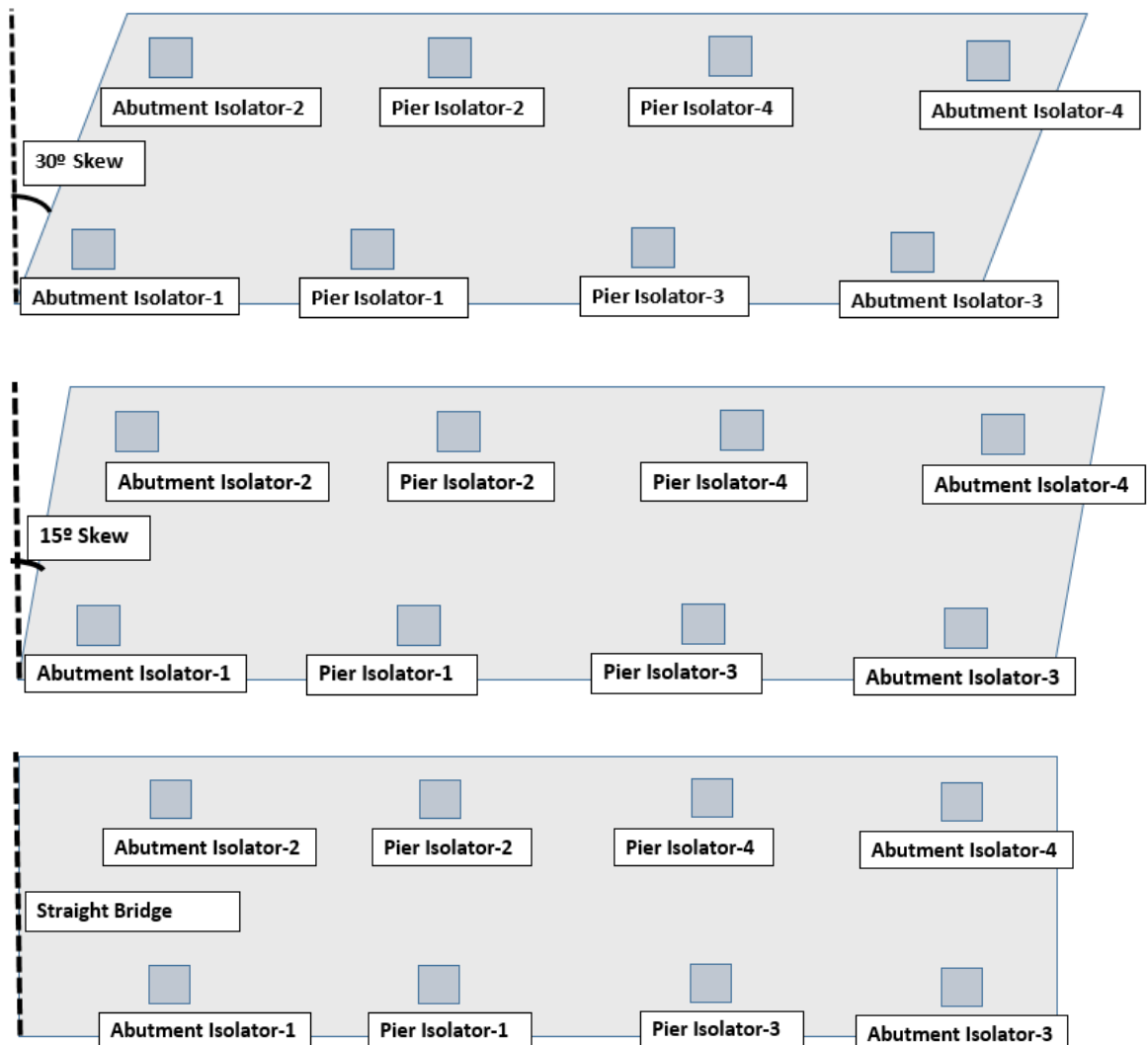


Figure 8.54. Location of seismic isolators for Bridge A with varying skew angles.

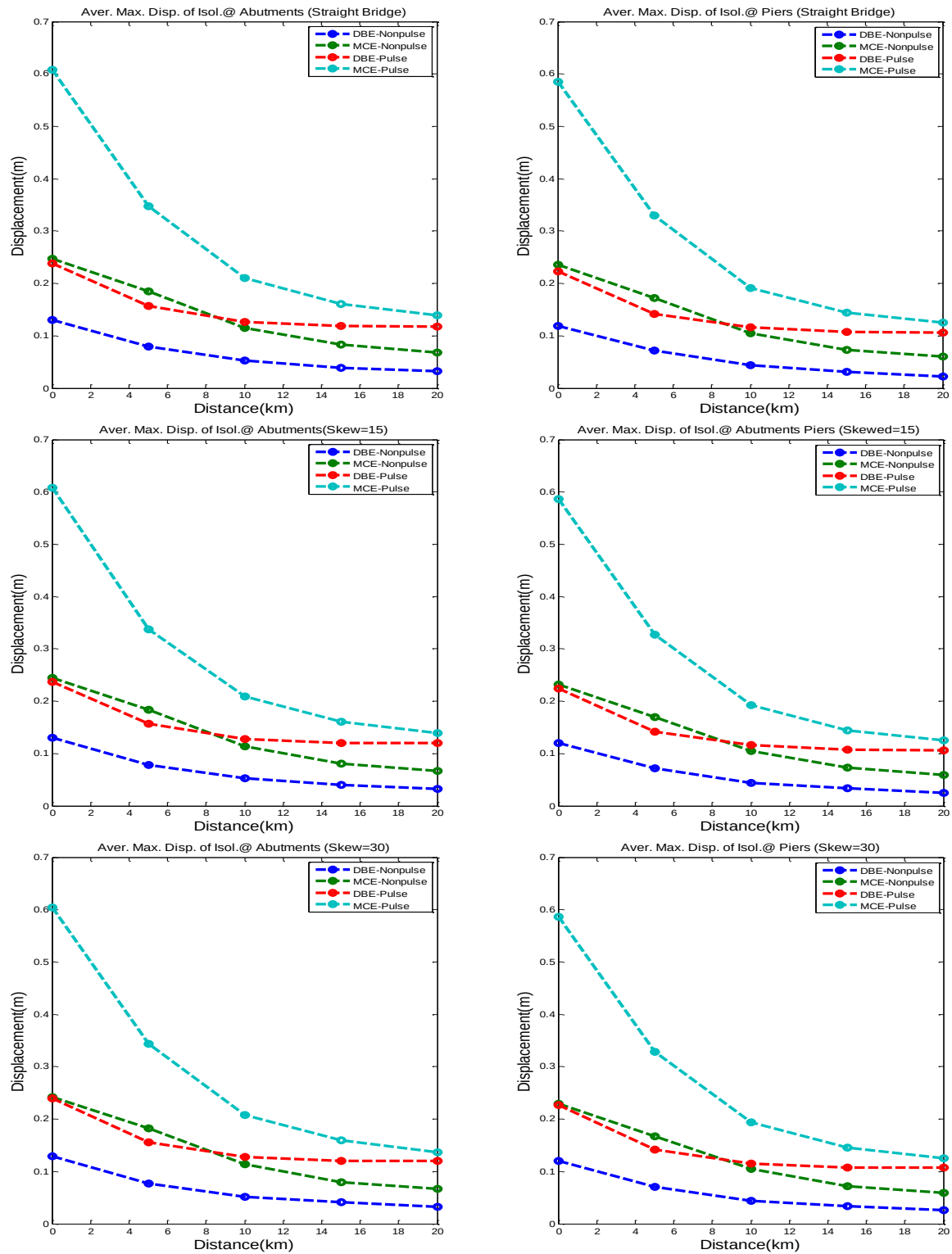


Figure 8.55. Variation of displacement demands of MSCSFS isolators at abutment and pier locations under varying skew angles.

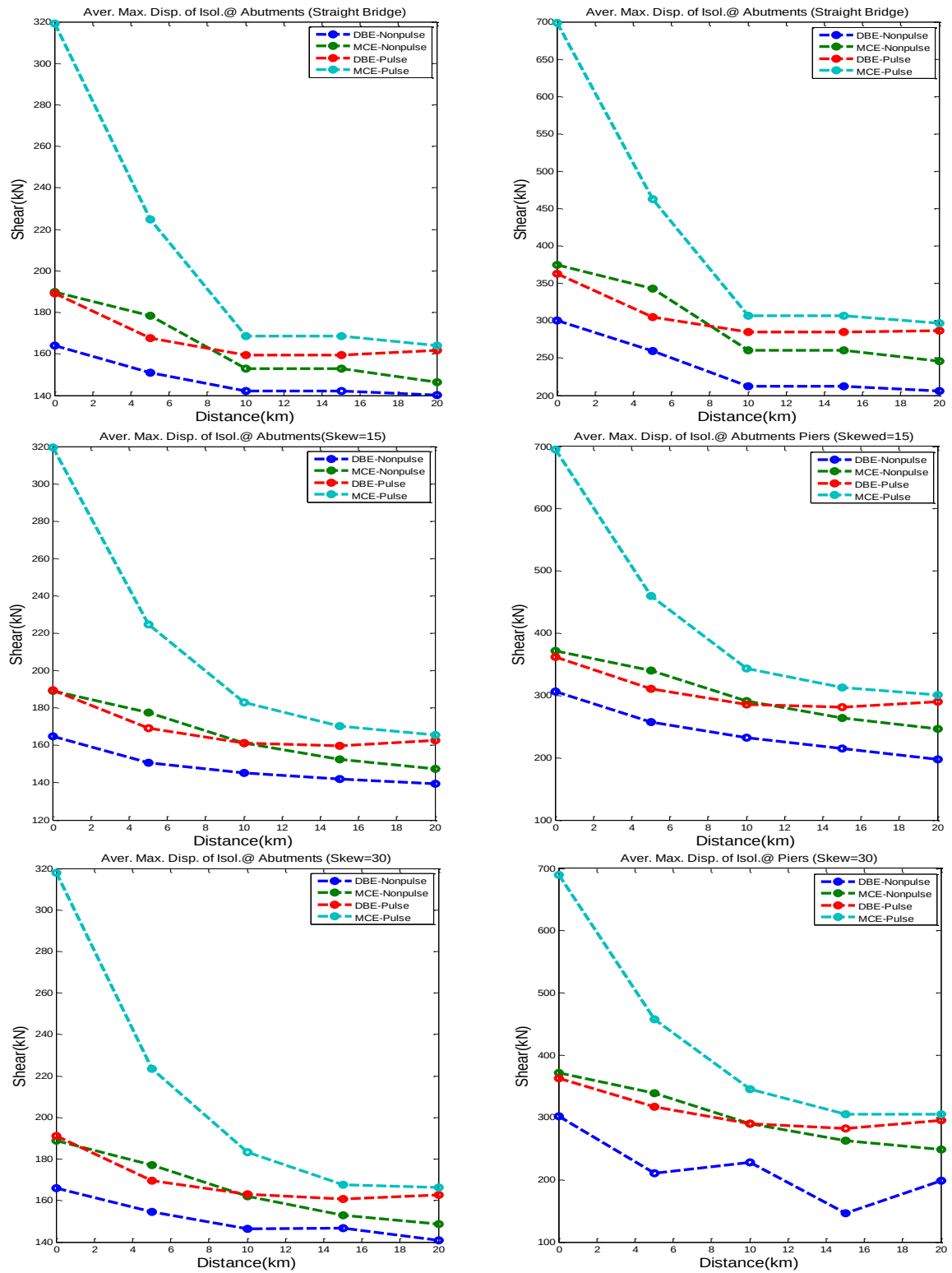


Figure 8.56. Variation of shear demands of MSCSFS isolators at abutment and pier locations under varying skew angles.

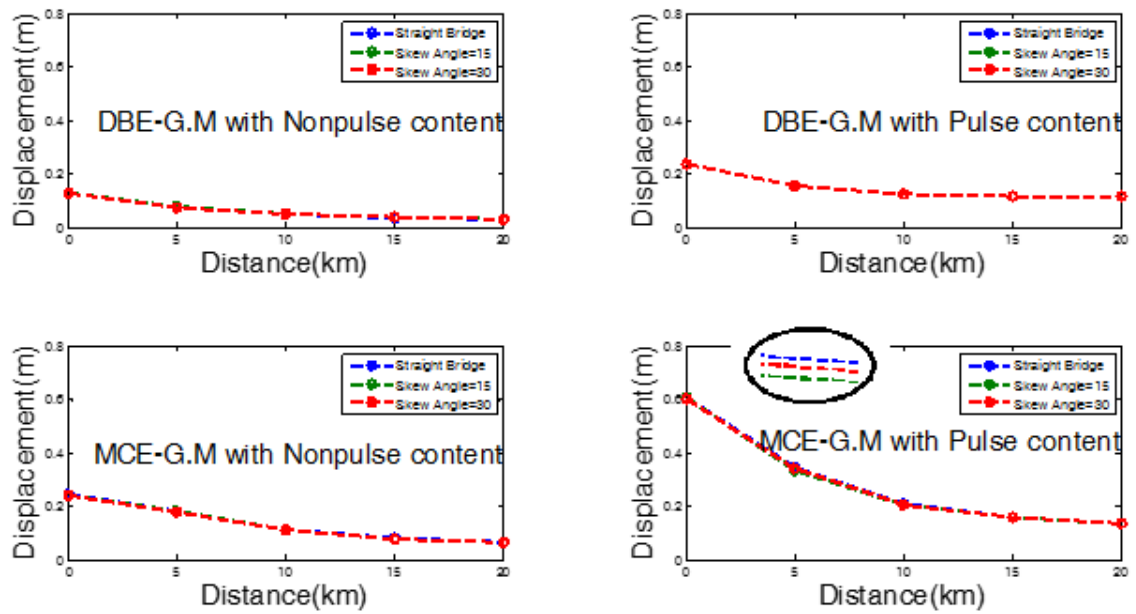


Figure 8.57. Variation of displacement demands of MSCSFS under varying skew angles.

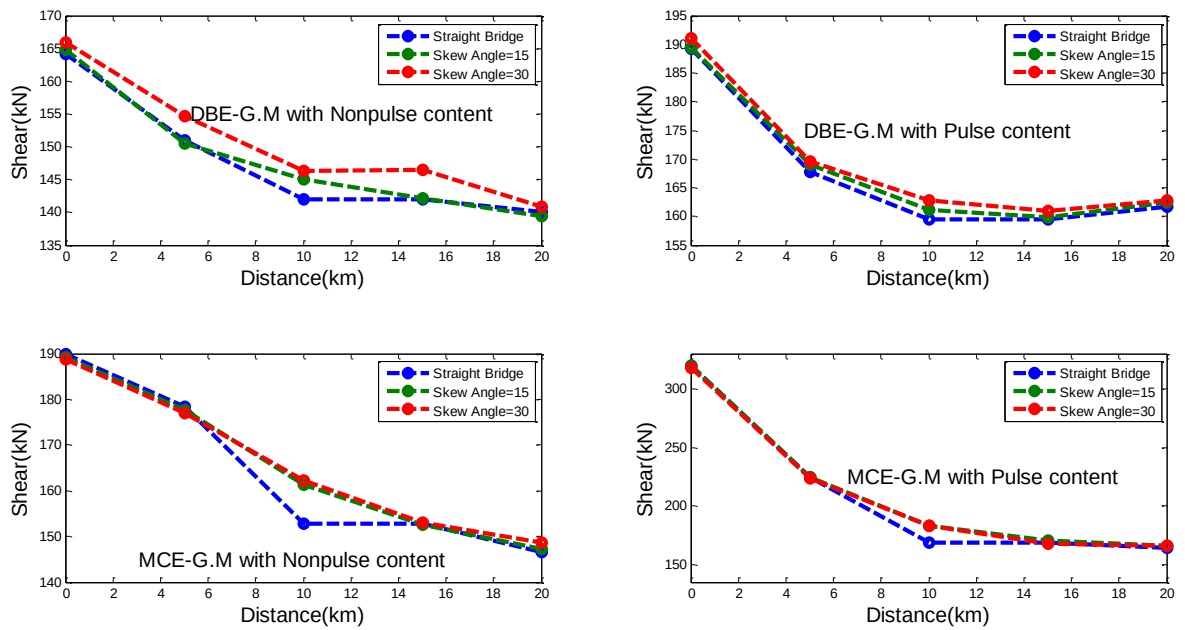


Figure 8.58. Variation of shear demands of MSCSFS under varying skew angles.

Support conditions is one of the important terms that needs to be considered for a proper design of seismically isolated system due to variation of input characteristics. Current state of practice recommends to use isolation device at sites that can be classified as engineering bedrock or stiff soils. For soft soil conditions, efficiency and effectiveness of isolation system will be reduced. In order to evaluate the suitability of isolation systems, three different type of support conditions are considered during the nonlinear time history analysis of Bridge-A as shown in Figure 8.59. Obtained analysis results are depicted from Figure 8.60 to 8.63.

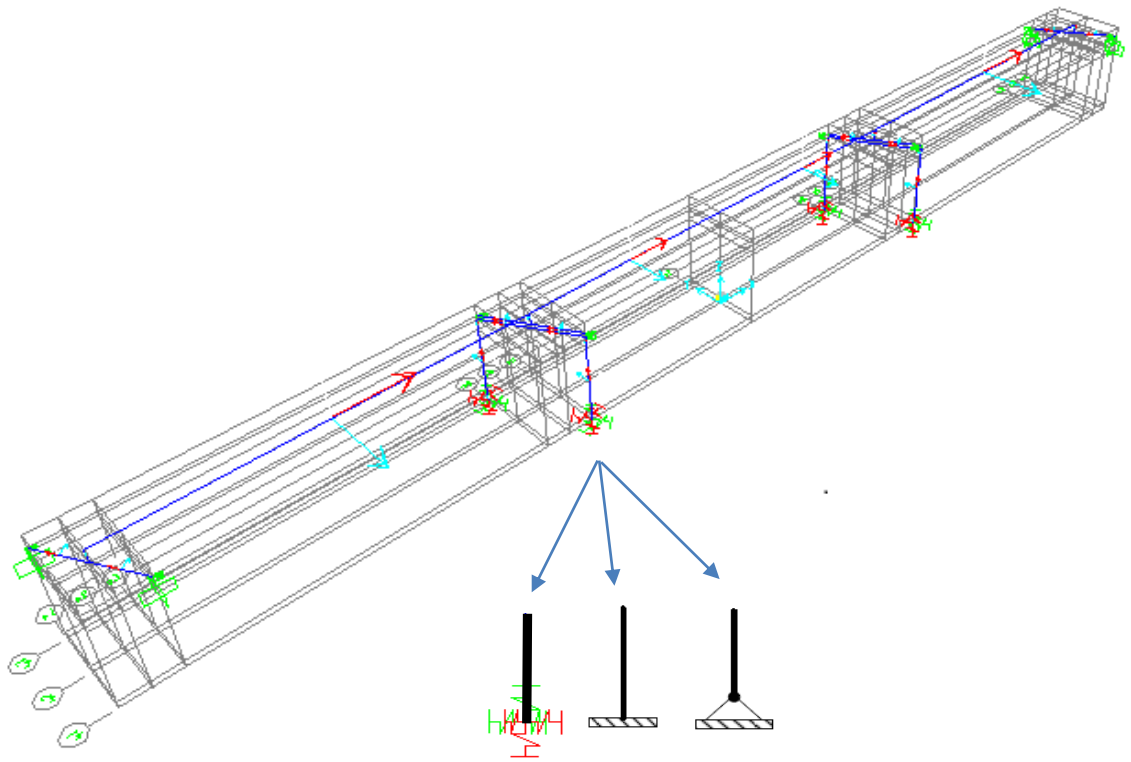


Figure 8.59. Considered support conditions in the nonlinear time-history analysis.

Furthermore, performance comparison of three isolation systems with respect to suite of ground motions prepared for two level earthquake by considering effects of pulse content. Analysis results were illustrated from Figure 8.64 to 8.67

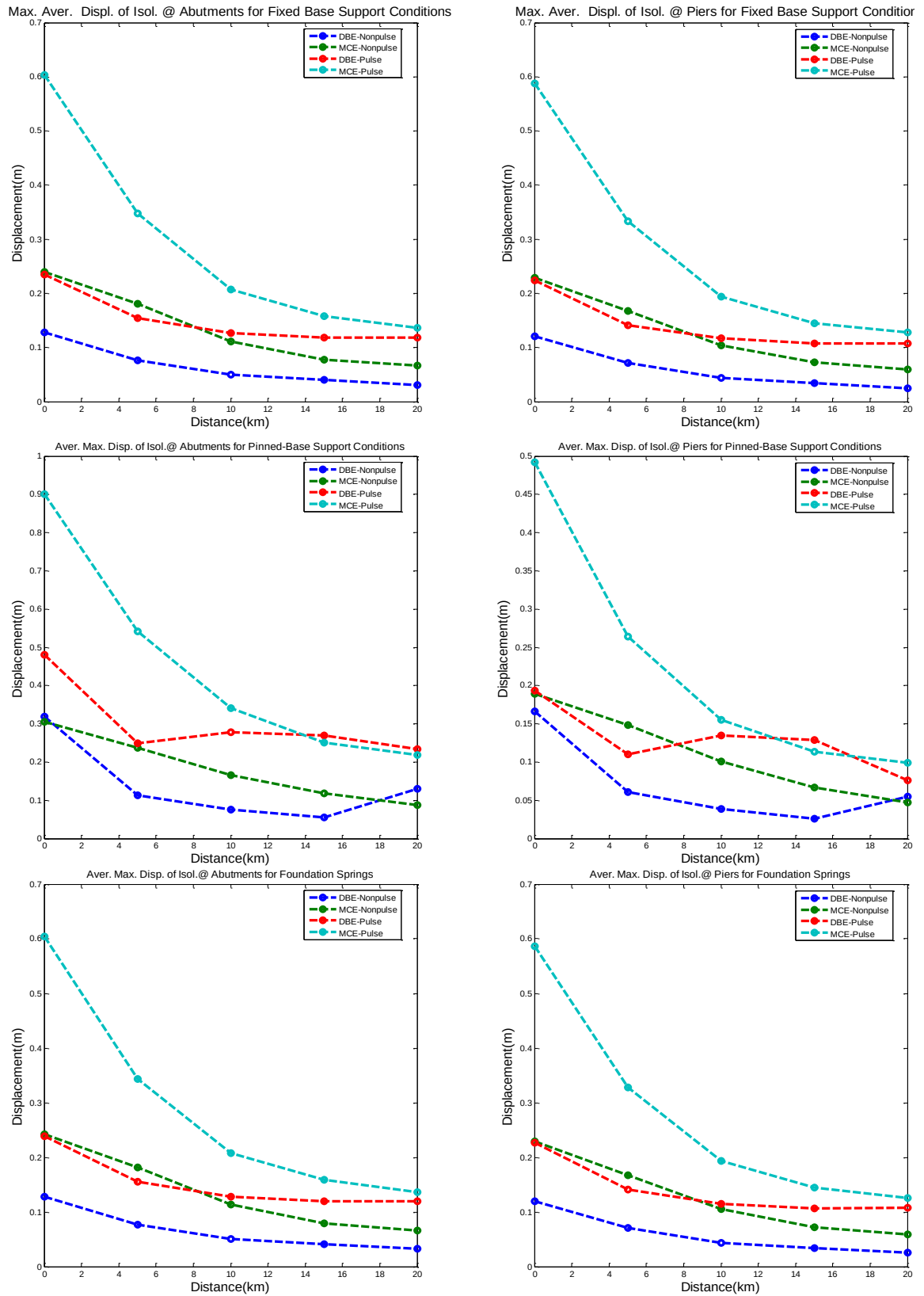


Figure 8.60. Variation of displacement demand of MSCSFS isolator at abutments and piers under varying support conditions.

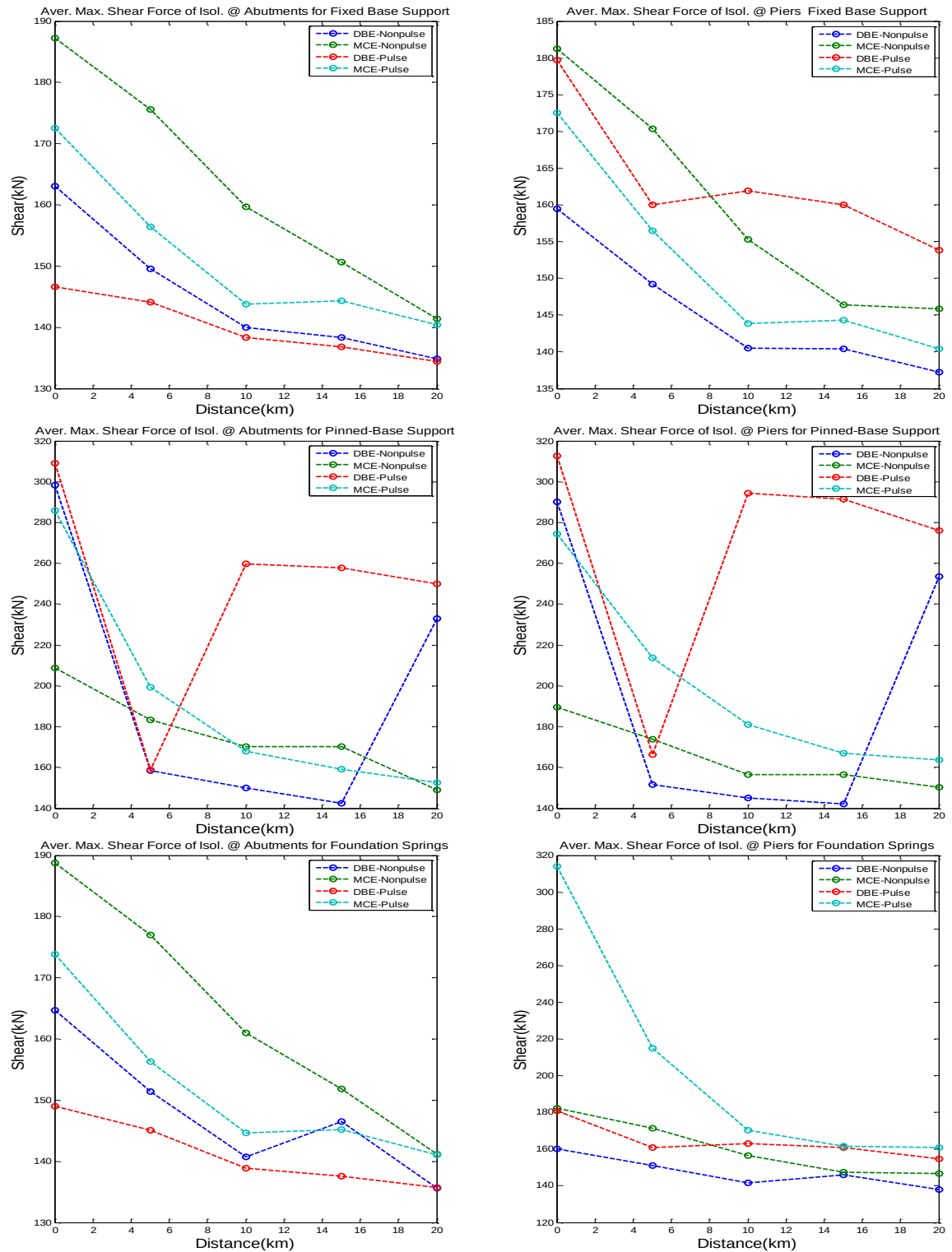


Figure 8.61. Variation of shear demands of MSCSFS isolators at abutments and piers under varying support conditions.

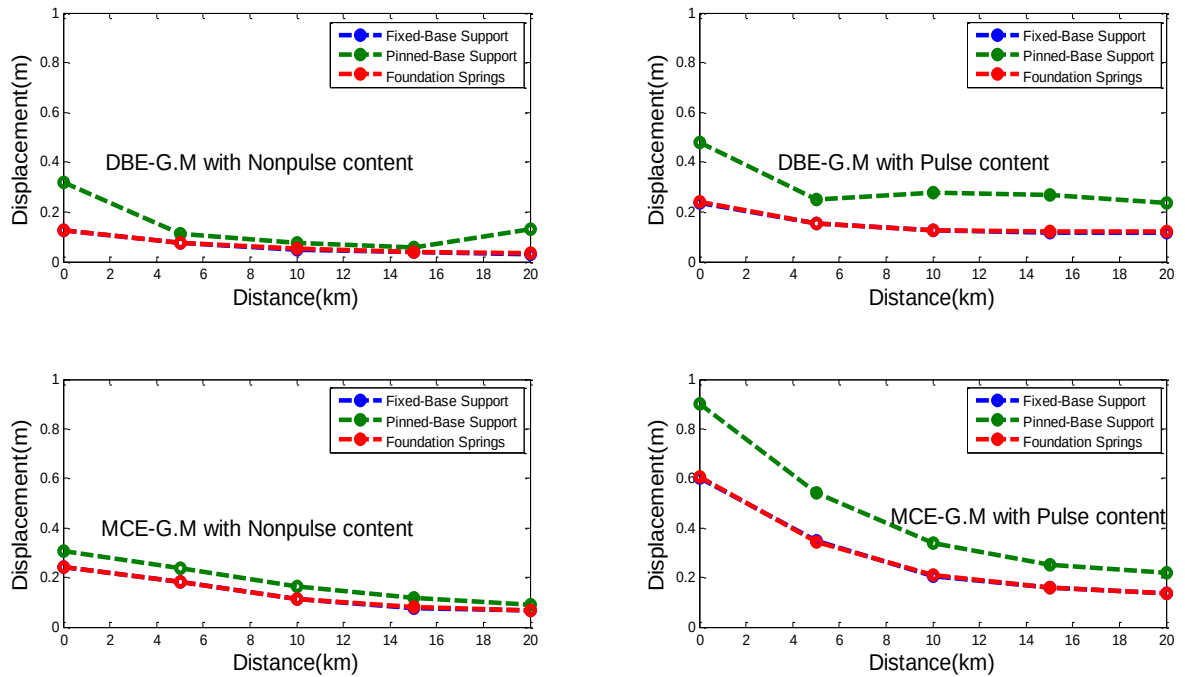


Figure 8.62. Variation of displacement demands of MSCSFS under varying support conditions.

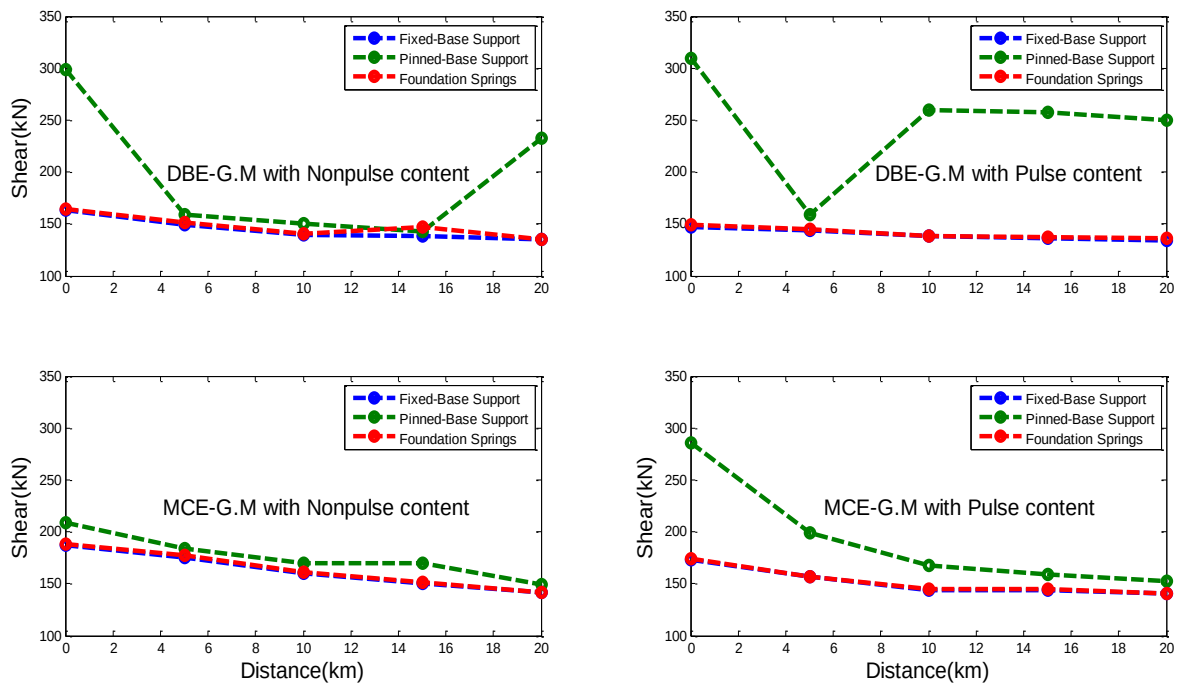


Figure 8.63. Variation of shear demand of MCSFS with respect to support conditions.

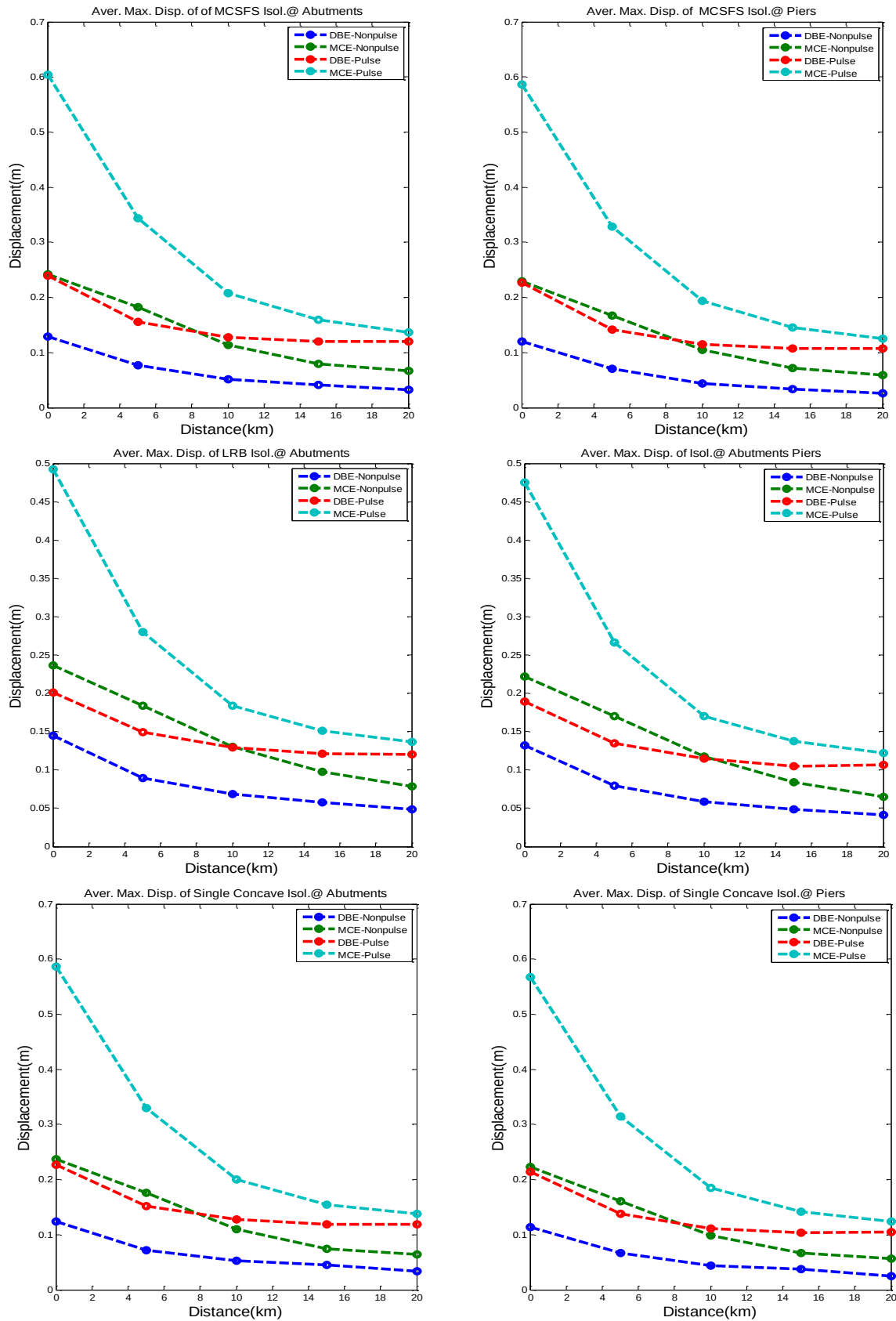


Figure 8.64. Comparison of displacement demands at abutments and piers for three different isolation systems.

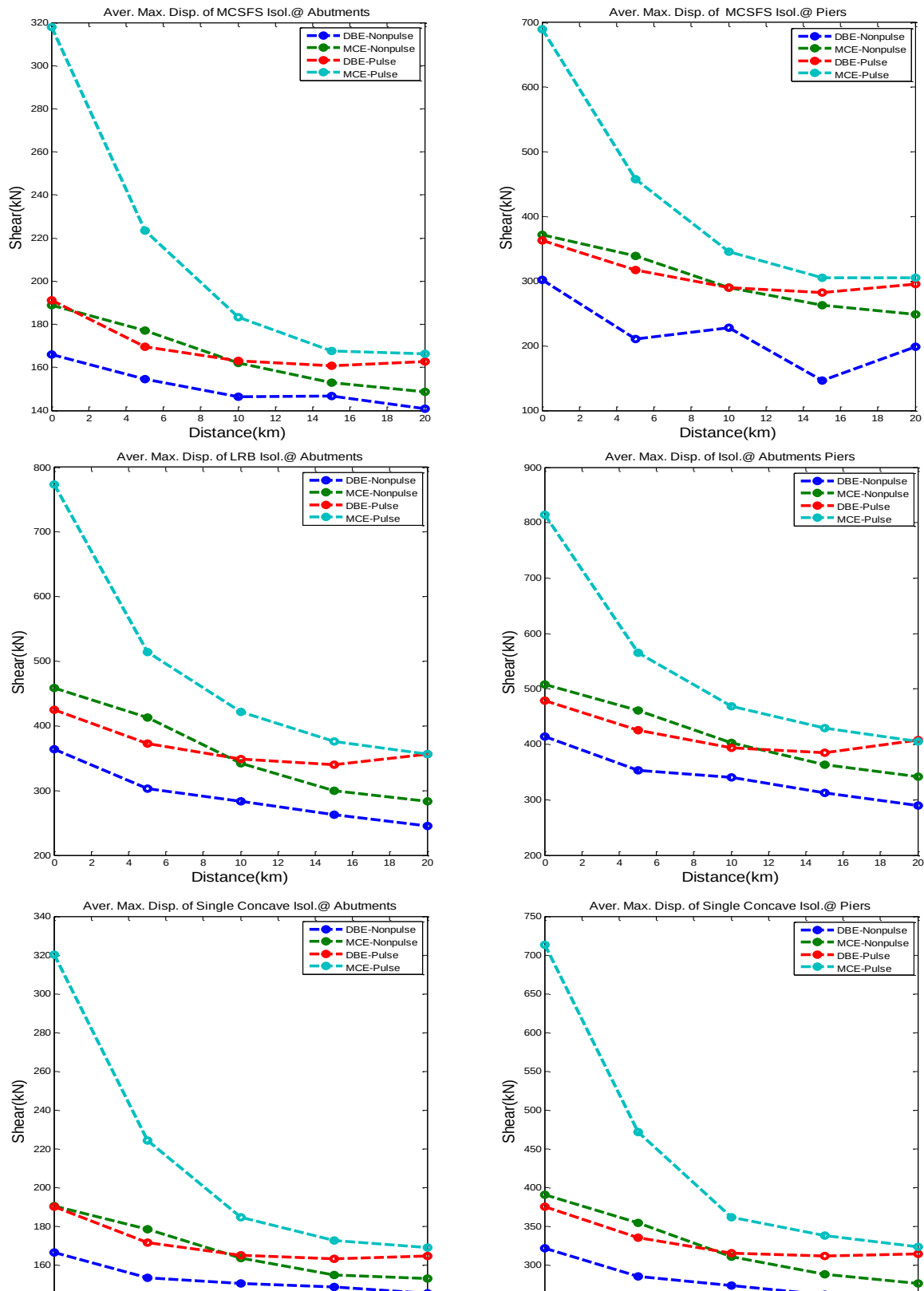


Figure 8.65. Comparison of shear forces at abutments and piers for three type of isolation systems.

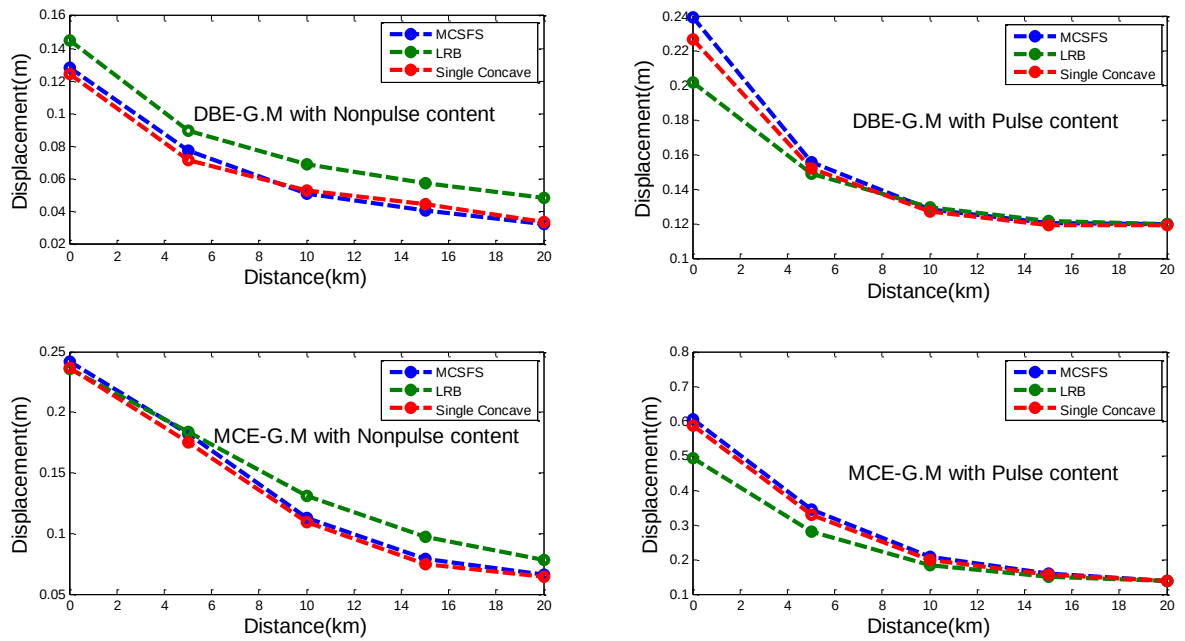


Figure 8.66. Comparison of maximum average displacement demands for the three type of different isolation systems.

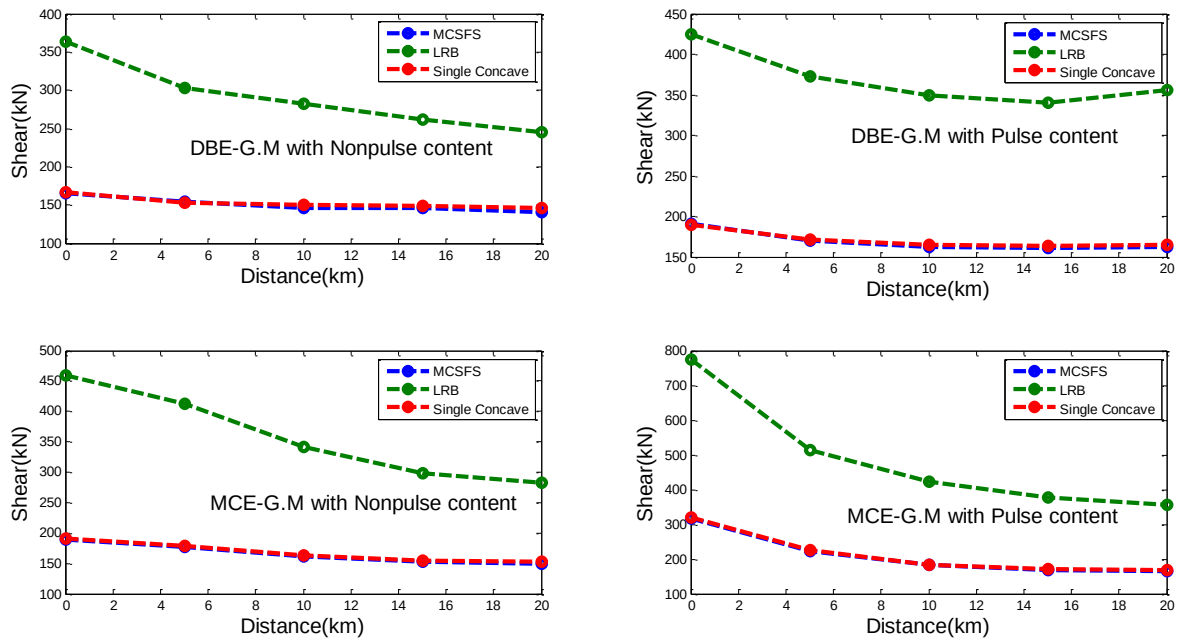


Figure 8.67. Comparison of maximum average shear force for the three different type of isolation systems.

Additional study is performed for the ground motions that are located within 0-20 km distance to the source with identified pulse content. Impact of fault-normal component and coupled plasticity assumption are evaluated for ground motions with peculiar characteristics.

8.3. Analytical and Experimental Analysis Results of the Rio-Torto Viaduct under Near-Fault Ground Conditions

The Rio-Torto Viaduct is an old prenormative RC bridge located in Emilia-Romagna region as a connection link between Florence and Bologna with a total length of 421.1m (Figure 8.68). It consists of 13 spans with two independent roadways which were supported by 12 pairs of portal frame bents where piers of the bridge are composed of circular solid and hollow sections with varying diameters. Transverse beams connect the columns of each bent. All bridge piers have plain steel bars and further investigation is considered in the task of RETRO project. Results of previous experimental tests and numerical analyses were exhibited that transverse beams are susceptible to shear failures (Giannini and Paolacci, 2009, Paolacci *et al.*, 2013). Previous quasi-static test results of Pier 12 are implemented to developed analytical models. Simplified and refined 3D bridge models are generated in OpenSees (McKenna *et al.*, 2007). In the preliminary analysis phase, SDOF bilinear model that accounts for strain hardening behavior and simplified nonlinear Opensees models are used to evaluate the damage potential of records. Results of bilinear SDOF model and simplified analytical models of the as-built configuration are evaluated on the basis of damage potential of the selected set of earthquake records.

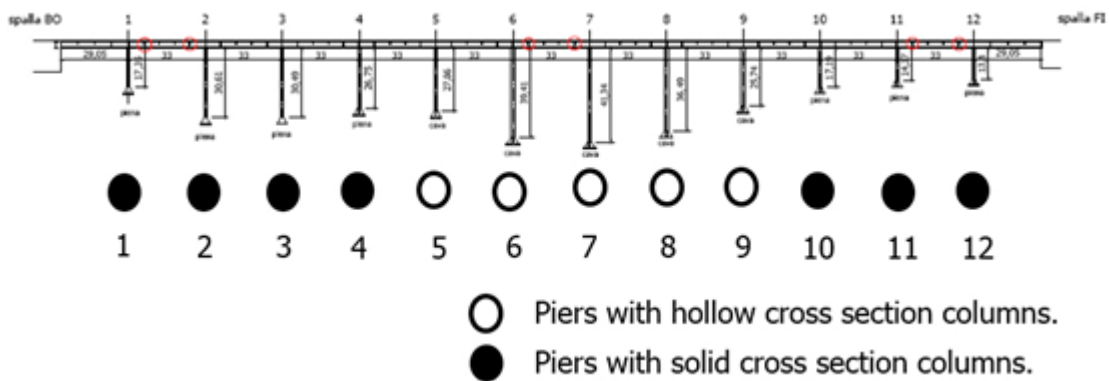


Figure 8.68. Distribution of pier cross-sections.

The Rio-Torto Viaduct has been tested for the as-built and seismically isolated cases. Development of complex analytical models were essential for the reliability and control measure of tests. Finite element models of the Rio-Torto Viaduct is generated in OpenSees for the 3D calibrated bridge model together with the simplified nonlinear model of the as-built and partially isolated configurations. The Rio-Torto Viaduct contains the most representative characteristics of aged reinforced concrete bridges. Single concave curved surface friction slider is used as an isolation unit for the seismically isolated case during the PsD tests. Seismic isolators are provided by Alga Inc. and analytic models applied for the seismic isolation system have ability to capture the behavior of LRB and energy dissipators. As an outreach of TA facility, as-built and isolated configuration of the Rio-Torto Viaduct were exposed to near-fault earthquakes to assess the sensitivity to long period earthquakes.

Due to capability of pseudo-dynamic testing procedure in ELSA laboratories, unidirectional loading was applied and analytical bridge models of the PsD tests were generated in OpenSees (McKenna *et al.*, 2007) software. Although suite of ground motions are gathered to account for earthquake loadings of probable major earthquake in the vicinity of Rio-Torto Bridge, the amount of damage that may be experienced during the 2012 Emilia Earthquake have changed the intention for the performance assessment. PsD test under the 2012 Emilia earthquake records will help to determine the most suitable retrofitting scheme in compliance with the seismic hazard study. Therefore, ground motions that are recorded from in the 2012 Emilia Earthquake are used as input motions for the PSD test sequence (Figure 8.69). Further studies were performed for contributing to the development of effective performance enhancement strategies of bridges in Eurocode-8 Part-2(CEN, 2005).

Following cases have been considered during the experimental investigation of the RETRO project

- 1) Mechanical Characterization of selected isolators
- 2) Seismically isolated bridge configuration by using physical piers with numerical isolators
- 3) Seismically isolated bridge configuration using curved surface friction sliders; physical pier with physical Isolator
- 4) “As-built configuration of the Rio-Torto Viaduct for the performance assessment

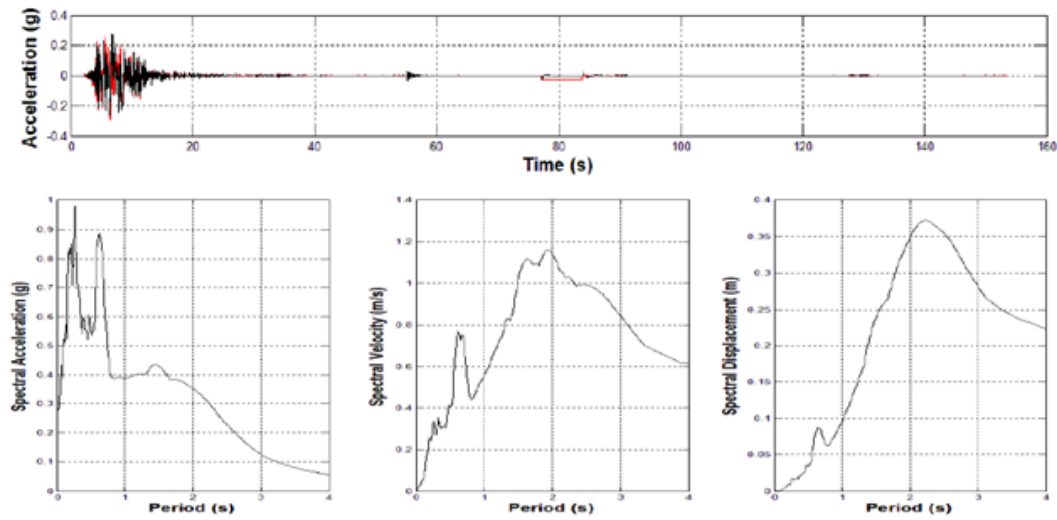


Figure 8.69. Record of the 29th May 2012 (N-S Component) and its spectral acceleration, spectral velocity and spectral displacement values.

8.4. Characterization of Mechanical Characteristics of Curved Surface Spherical Friction Sliders under Cyclic Loading

Geometric dimensions of curved surface spherical friction slider and the associated design parameters were provided by Alga (Sartori, 2011). Geometric configuration and maximum displacement capacity of the scaled version of single concave sliding bearing is shown in Figure 8.70.

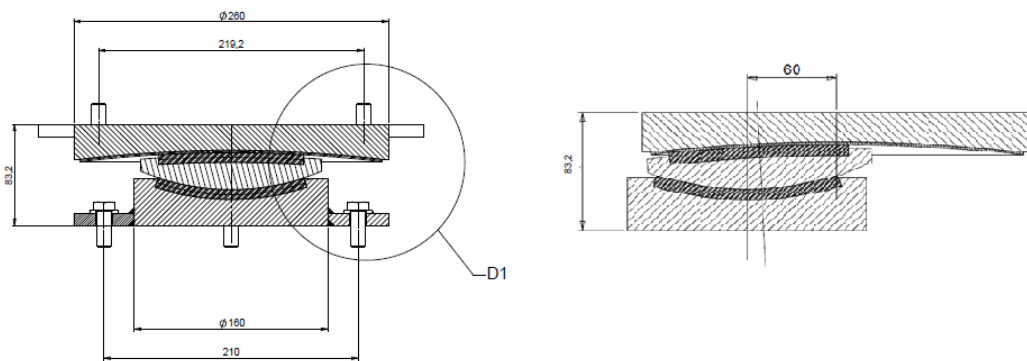


Figure 8.70. Dimensions and displacement capacity of the single concave curved surface spherical friction slider.

Series of cyclic tests are performed for the characterization of the curved surface friction sliders. Test protocol of the cyclic test is based on the procedure recommended by Lomiento *et al.* (2000). It is a well-known fact that the curved surface friction sliders are rate dependent devices in which isolators are subjected to cyclic motion with a varying amplitudes at different speeds under constant vertical load. Configuration of the test setup is shown in Figure 8.71. Consecutive tests are performed with a sufficient cooling time intervals for avoiding the effect of heating on isolators. Force-displacement hysteresis loops of applied cyclic motions are illustrated in Figure 8.72 and Figure 8.73. Similitude rules are applied for the scaled pier specimens and seismic isolators.

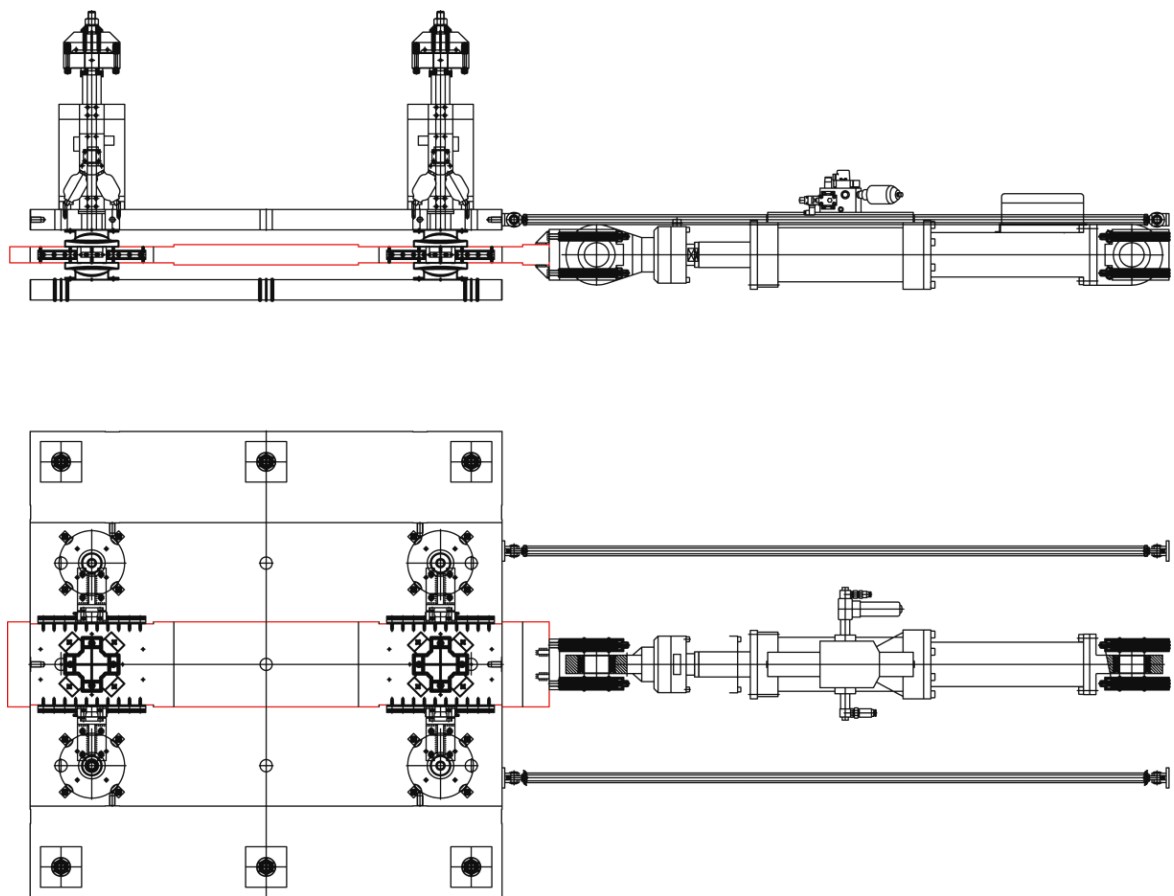


Figure 8.71. Test setup prepared in ELSA laboratories for curved surface friction sliders.

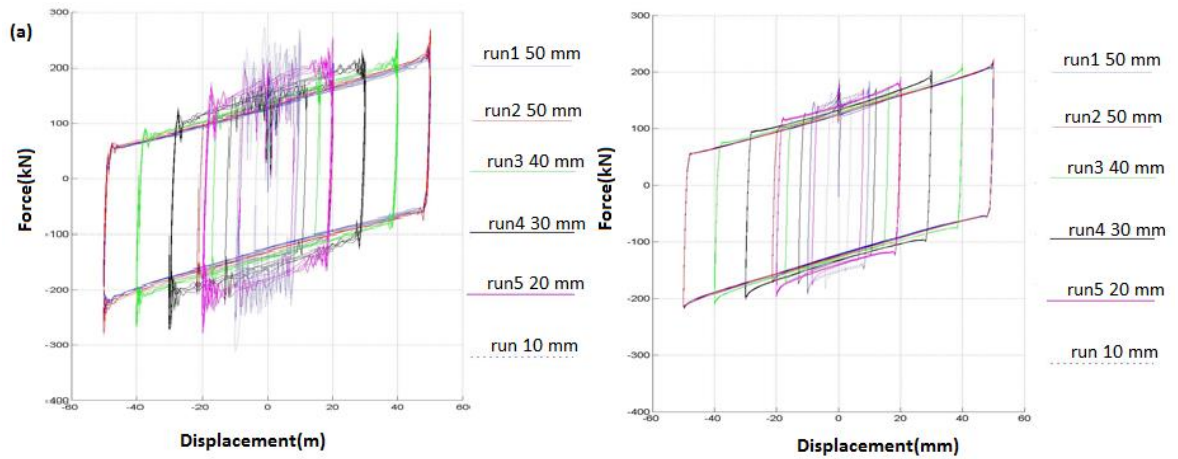


Figure 8.72. Varying displacement amplitude at short pier b) Varying displacement amplitude at tall pier.

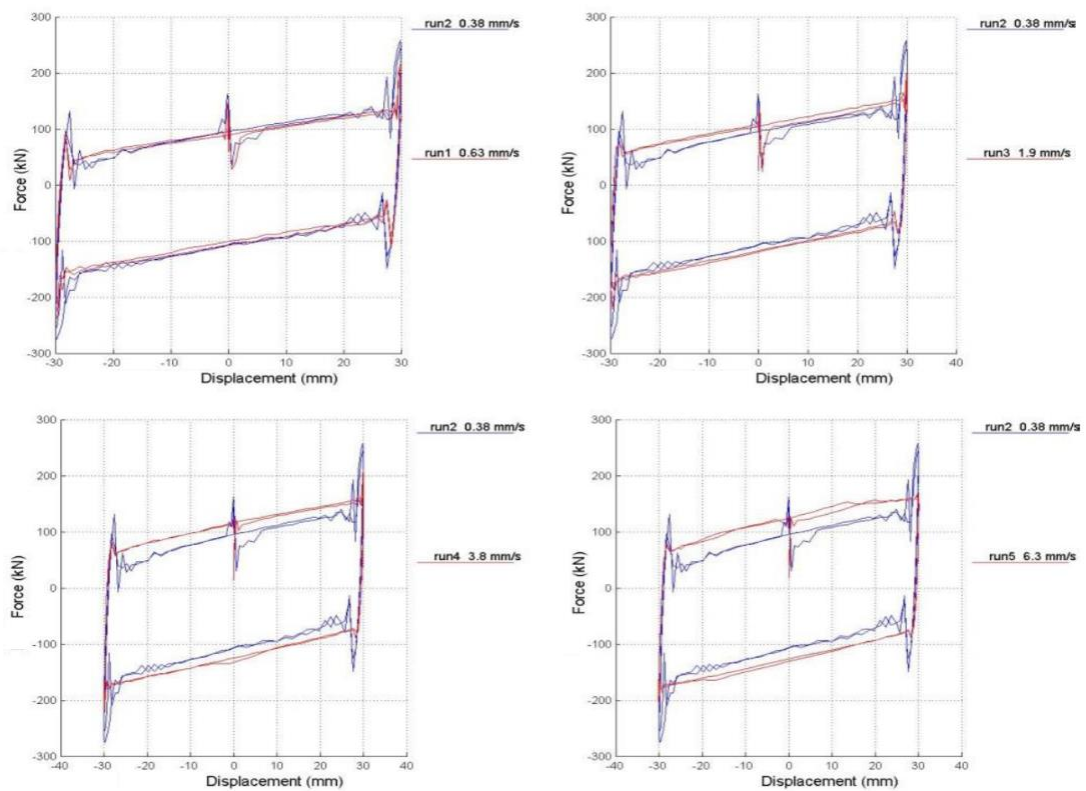


Figure 8.73. Cyclic test results of the 1/2.5 scaled curved surface friction sliders in ELSA laboratory.

SLS and ULS performance levels corresponds to the serviceability and the collapse prevention limit state in the Italian seismic design code (NTC-08).

Damage level of columns of each bent with transverse beams were found to be sensitive to the breakaway friction and stick-slip behavior of the curved surface friction sliders. The level of transverse beam deformations were computed by OpenSees and Cast3m software as shown in Figure 8.74. Analysis results of both computer programs were in the range of acceptable range when compared with the PsD test results.

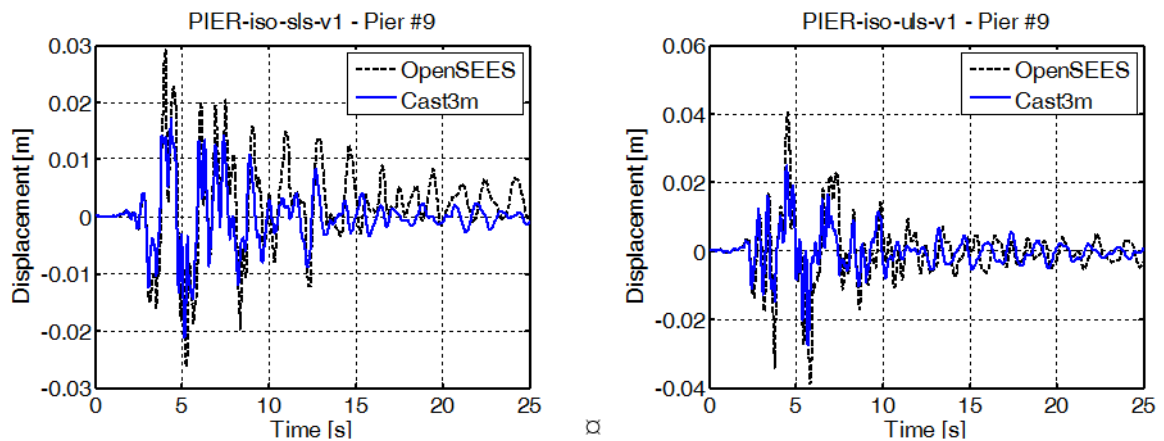


Figure 8.74. Level of transverse beam displacements under isolated case.

8.5. Parametric Investigation of As-Built and Seismically Isolated Configuration of Bridge-B

Variation in the response of Bridge-B by incorporation of seismic isolation technique is addressed with the help of the as-built and seismically isolated models. Additional study is performed to assess the effectiveness of the selected seismic isolators comprehensively for pulse like ground motions that are in compliant with the provided seismic demand representation. Envelope deformation patterns were derived for the as built and seismically isolated bridge case for ground motions that have pulse period close to the fundamental vibration period of the bridge model. The model has been created for one lane of the bridge. The numerical model in OpenSees here analyzed in full scale. (Figure 8.75). Rayleigh damping is introduced to the 3D refined model by including relevant modes (Figure 8.76). Distributed plasticity is introduced to 3D bridge model by assigning fiber elements for columns, cap beams and transverse beams that are susceptible to damage (Figure 8.77).

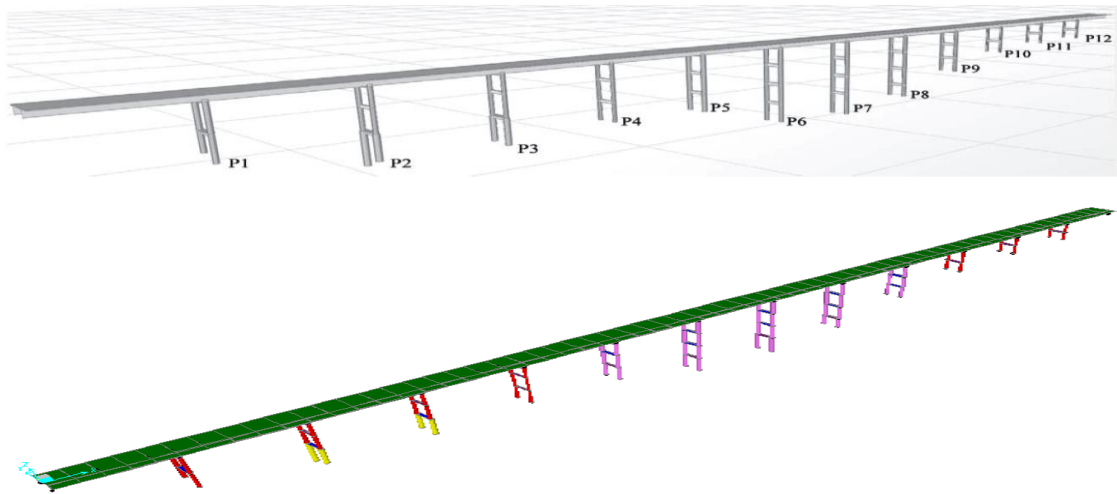


Figure 8.75. 3D models of the Rio-Torto Viaduct in OpenSees and SAP2000.

Near-fault ground motions are selected with respect to the established target spectrum as defined in Section 7 of this study where Matlab oriented code is incorporated to establish the hazard spectrum with respect to the regularly distributed grids close to the Rio-Torto Viaduct.

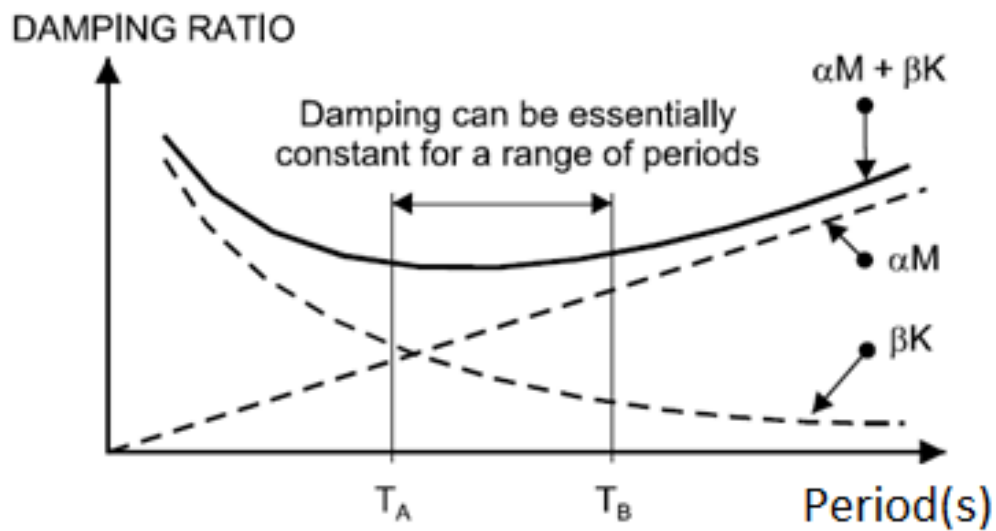


Figure 8.76. Rayleigh damping used for reinforced concrete elements in the OpenSees model of the Bridge-B.

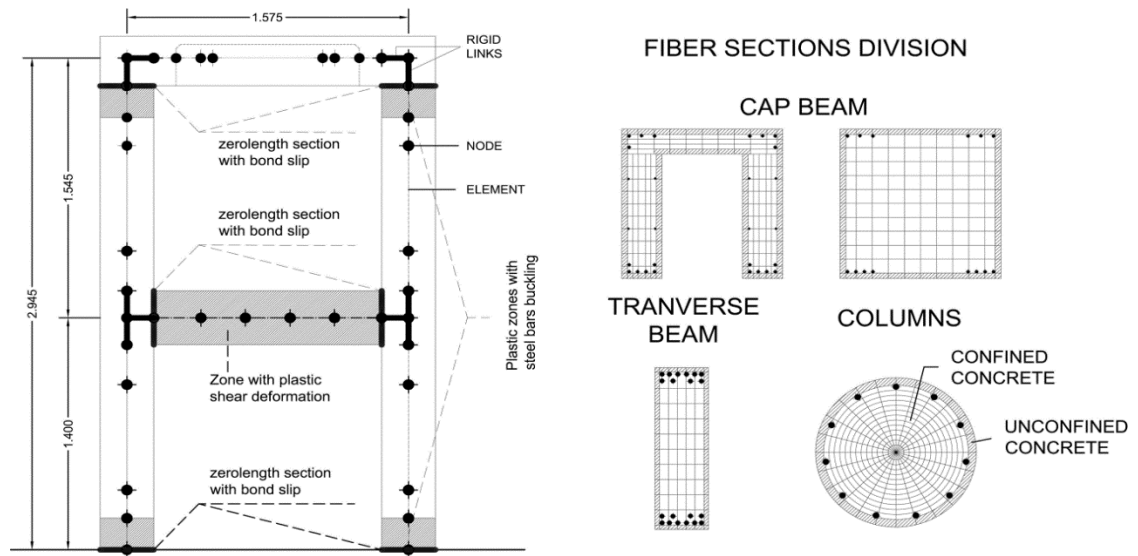


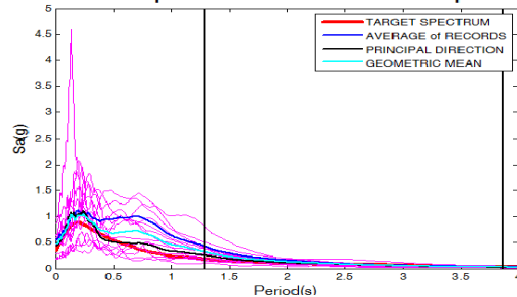
Figure 8.77. Numerical model of the pier 12 in OpenSees.

PEER ground motions database is sought to find representative input motions that fit to the established UHS spectra with a 2% probability of exceedance in Emilia-Romagna region. Selected earthquake ground motions are rotated to their principal direction to provide ground motions that can be used for the unidirectional loading in the PsD test. Suite of ground motions are listed in Table 8.1 and their corresponding spectral shapes are shown in Figure 8.78. Envelope deformation patterns under the selected suite of ground motions are shown in Figure 8.79 and 8.80, respectively for the as-built and isolated configuration of the Bridge-B. In addition to hazard compatible records, ground motions with velocity pulse periods close to the period of isolated case was assessed by utilizing PEER ground motion database (Figure 8.81).

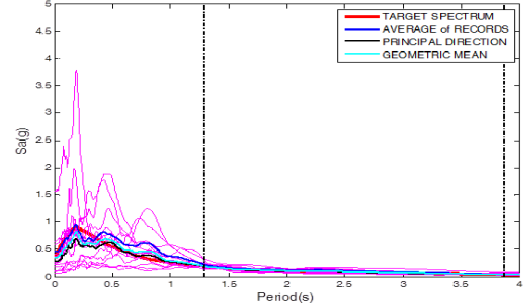
Table 8.1. Suite of ground motions used in the parametric analysis of Bridge-B.

Suite of 7 pairs of GM with pulse Content									
Event	Year	Station	Mag	Mechanism	Pulse	Tp(s)	Rjb(km)	Rrup(km)	Vs30(m/s)
Chi-Chi- Taiwan-03	1999	TCU076	6.2	Reverse	1 0	0.91 --	13	14.7	615
Chi-Chi- Taiwan-06	1999	TCU080	6.3	Reverse	0 1	-- 1	0	10.2	509
Morgan Hill	1984	Coyote LakeDam	6.19	Strike-Slip	1 1	0.95 .1	0.2	0.5	597.1
San Salvador	1986	Geotech Inv.C.	5.8	Strike-Slip	1 1	0.86 1.8	2.1	6.3	545
Coyote Lake	1979	Gilroy Array #6	5.74	Strike-Slip	1 0	1.2 --	0.4	3.1	663.3
Northridge-01	1994	LA Dam	6.69	Reverse	1 1	1.7 2.8	0	5.9	629
Morgan Hill	1984	Gilroy Array #6	6.19	Strike-Slip	1 0	1.2 --	9.8	9.9	663.3
Suite of 7 pairs of GM with Nonpulse Content									
Event	Year	Station	Mag	Mechanism	Pulse	Tp(s)	Rjb(km)	Rrup(km)	Vs30(m/s)
Chi-Chi- Taiwan-03	1999	TCU084	6.2	Reverse	0 0	-- --	3.7	9.3	680
Chi-Chi- Taiwan-03	1999	TCU129	6.2	Reverse	0 0	-- --	10.9	12.8	664.4
Northridge-01	1994	Man. And.	6.69	Reverse	0 0	-- --	33.6	39.3	405.2
Morgan Hill	1984	Dam	6.19	Strike-Slip	0 0	-- --	3.2	3.3	488.8
Northridge-01	1994	Playa Del Rey	6.69	Reverse	0 0	-- --	24.4	31.7	405.2
Imperial Valley-06	1979	Cerro Prieto	6.53	Strike-Slip	0 0	-- --	15.2	15.2	659.6
Coalinga-01	1983	Parkfld-F. Zone6	6.36	Reverse	0 0	-- --	31.6	32.9	438.3

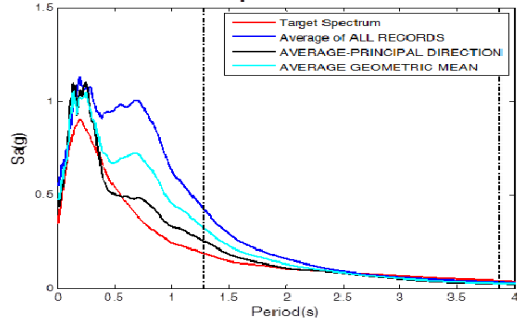
Near-Field Earthquakes with Pulse Content-Principal Direction



Near-Field Earthquakes with Nonpulse Content-Principal Direction



Near-Field Earthquakes with Pulse Content



Near-Field Earthquakes with Nonpulse Content

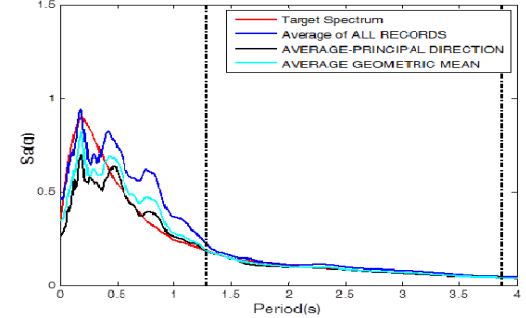


Figure 8.78. Spectral Shape and average values of earthquake records with pulse and nonpulse content.

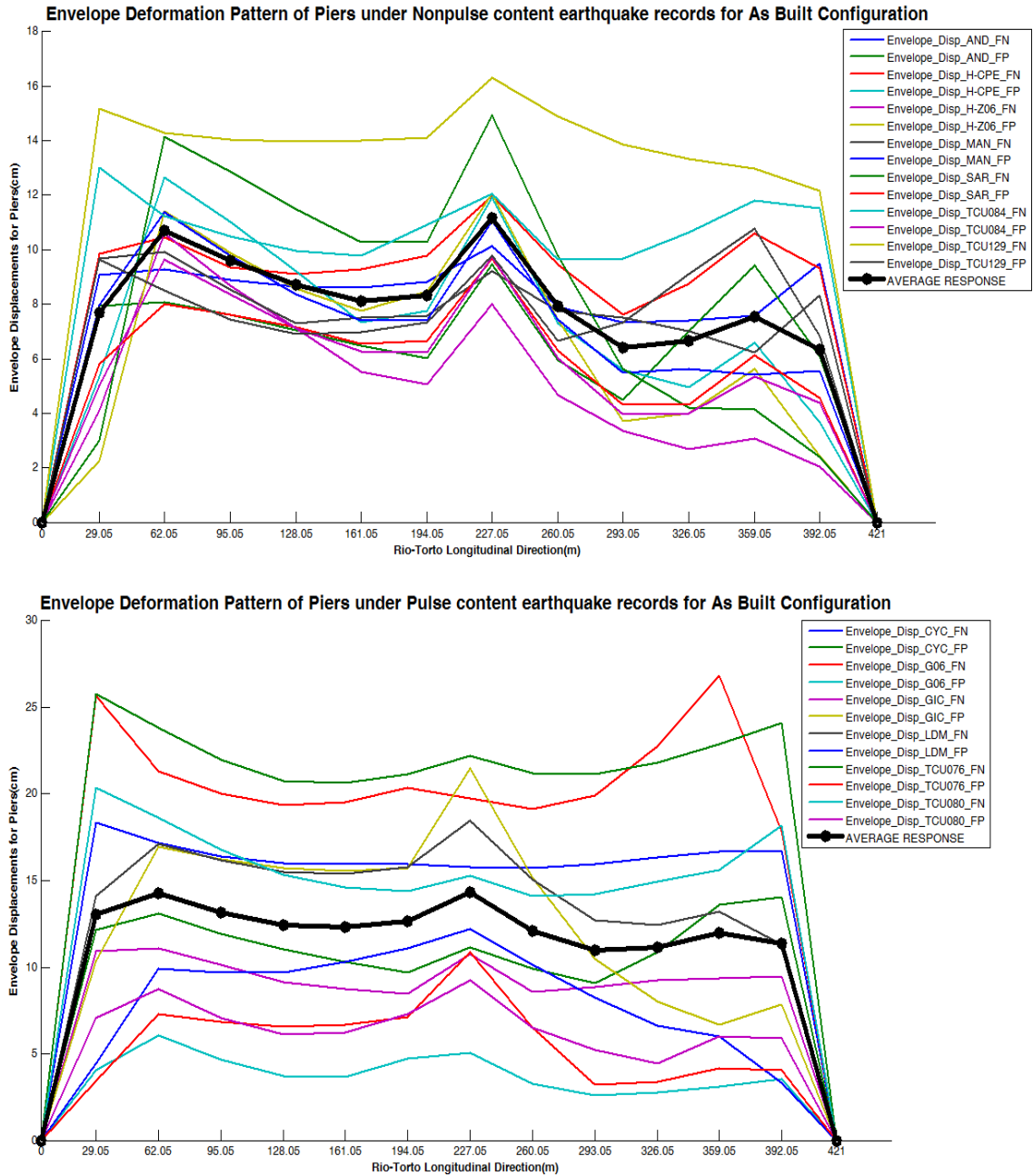


Figure 8.79. Envelope Deformation patterns of Piers for as-built model under suite of records with pulse & nonpulse content.

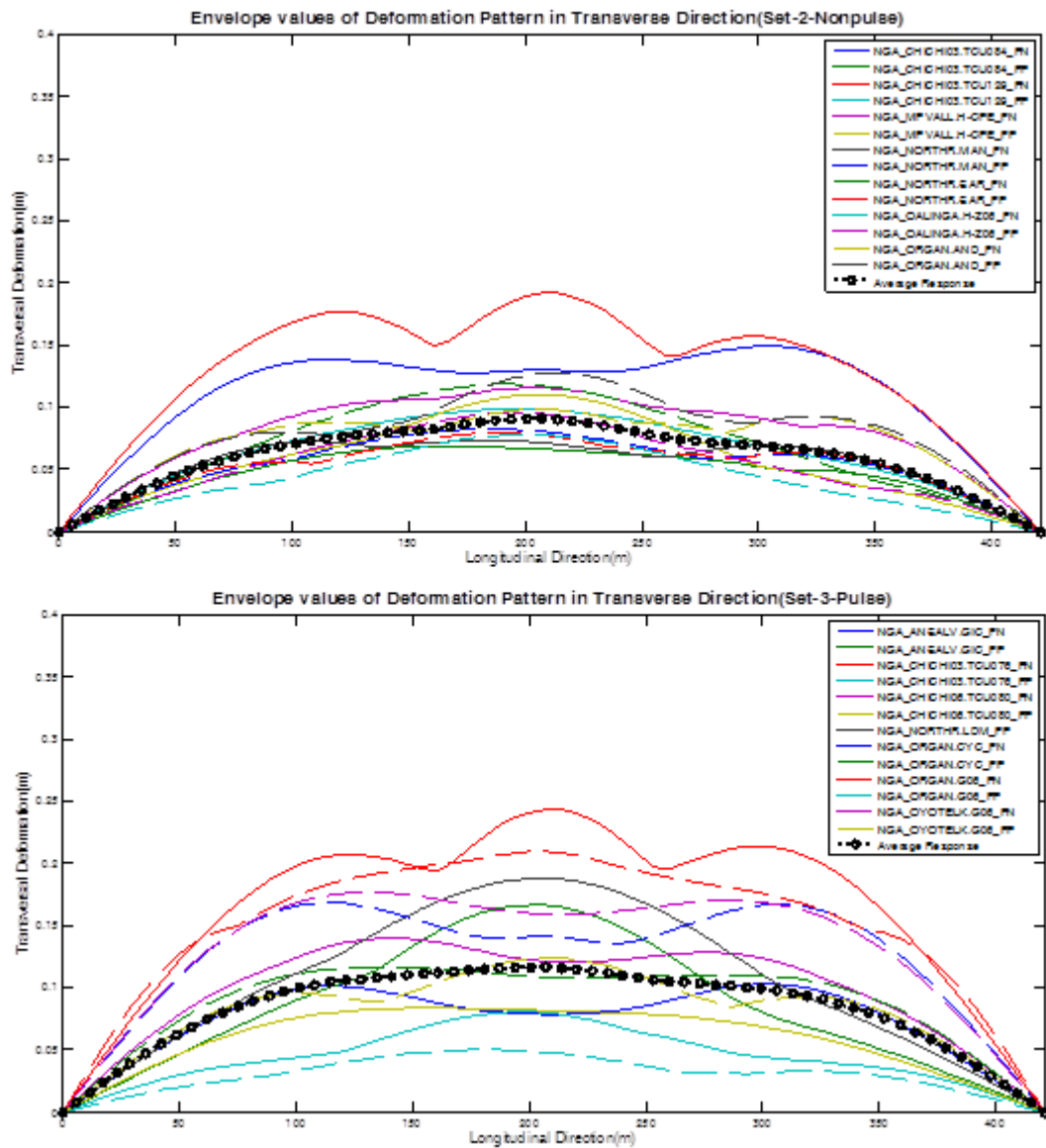


Figure 8.80 Envelope deformation patterns of piers for isolated bridge model under suite of records with pulse & nonpulse content

Likewise in Bridge-A pool of pulse like ground motions are prepared within 0-20 km source-to-distance. Pulse identification carried out by the proposed method of Shahi and Baker (2011) which relies on the wavelet analysis. One set of ground motion response history analysis will be presented in here to exhibit the effect of pulse like ground motions on seismically isolated bridges. Bins of ground motions for pulse like ground motions are prepared by considering dominant pulse period of ground motions. Irregularity of substructure elements make it difficult to correlate the damage with intensity measures.

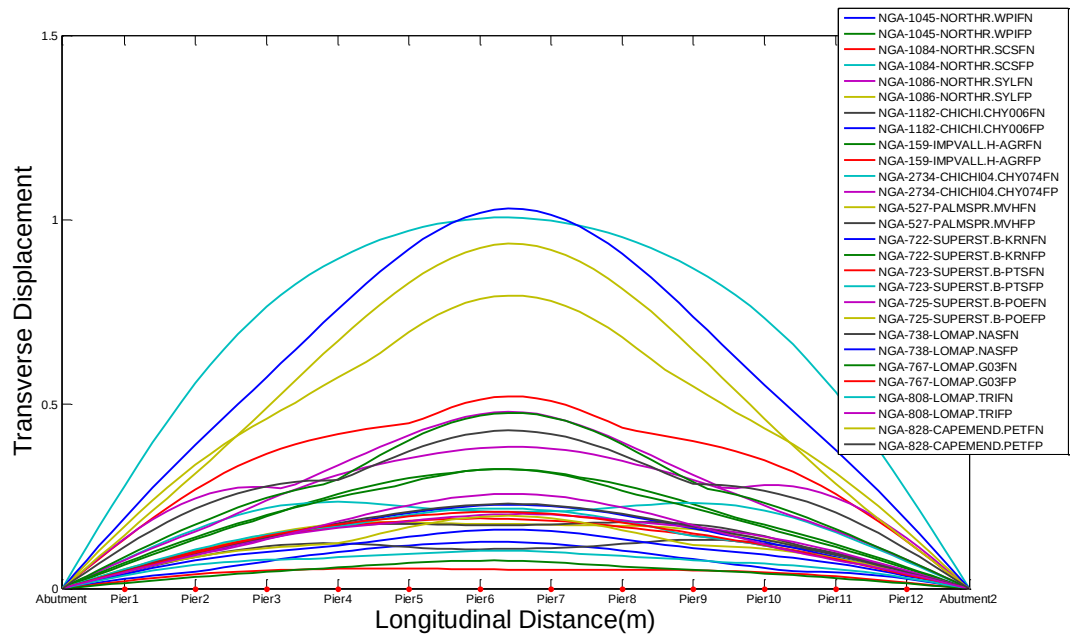


Figure 8.81. Envelope deformation under pulse-like ground motions that contain dominant pulse period between 3-4 seconds.

9. CONCLUSION

Recent advances in seismic isolation technology, in concert with the high performance expectations from structures, provide promising engineering solutions for the earthquake safety of bridges. Performance of the seismically isolated bridges in near-fault regions is one of the main concerns of the earthquake engineering community due to peculiar characteristics of near-source earthquake records. In engineering practice only the horizontal components of earthquakes were taken into consideration and their coupled vertical component is often excluded in the nonlinear time history analysis. To arrive a credible understanding of bridges in near-fault conditions, a total number of 10210 nonlinear time-history analysis were performed in SAP2000 and Opensees programs for the fully isolated (Bridge-A) and partially isolated case (Bridge-B). OpenSees and SAP2000 programs have been used for the modeling of two example bridges due to their extensive element and material library which allows the execution of large number of nonlinear response history analyses (NRHA). Ground motion records in the PGMD were compiled in 58 bins and strong ground motion parameters of each record was computed as described in Section 2.

Three different seismic isolation systems were incorporated to assess and compare the performance of the fully isolated bridge while the three translational components of near-source earthquakes were simultaneously assigned for the nonlinear response history analysis. The model parameters of the selected isolation units and bridge are varied systematically to assess the performance of the isolated bridge under multi-component earthquake excitations. Fully isolated bridge model have been designed in a way that can meet the requirements of AASHTO LRFD design procedure. 20 sets of ground motion sets are selected for the 10 different scenario earthquakes. Selection and scaling criteria of PGMD have limited the number of earthquake ground motions that were considered in the nonlinear response history analysis.

As we all know, seismically isolated bridges are classified as long period structures and the long period pulse contents of earthquake ground motions have significantly affected the response of the isolation systems in NRHA. Force-displacement hysteresis of the isolation systems are dominated by the pulse content of the input ground motion while the isolation units were exhibited pulse like shapes in their displacement time history.

Another important finding is related with the variation of displacement demands between DBE level and MCE level earthquake which were computed with the help of DSHA. Special attention is given to the design methods for isolated bridges in US and European codes with focus on the displacement demands. The AASHTO-2010 recommends 7 % probability of exceedance in 75 years and design of bridges are done with respect to the established AASHTO design spectrum. Seismic isolation systems are usually designed for a single level earthquake and then adjusted for the rare earthquake level by a given range of factor for the design purposes. Displacement amplification factor accounts for the uncertainty related with the maximum effects that the MCE level may have on the isolation system. This approach accounts for the collapse prevention of the isolation units that may be exposed to unprecedented damage under very rare earthquakes. Current state of practice recommends that this factor can range from 1 to 1.5. Similar approach was also used in the 1991 AASHTO for isolation bearings that do not have restoring force capability and scale factor of 3 was applied for the unexpected severe earthquake shakings. Nonlinear response history analysis results are confirmed that this factor is significantly exceeded the maximum recommended value of this factor proposed for bridges in California. As a consequence of this design practice, significant damages may be experienced under near-fault ground motions. In this sense, forward rupture directivity, fling-step effects and strong vertical ground motion characteristics led to significant increases in seismic demand parameters of bridges where the maximum value of the recommended amplification factor for the displacement capacity at MCE level in US code were overly exceeded.

Furthermore, nonlinear time-history analysis results were addressed that the maximum seismic displacements occurs at isolators located in abutments for the Bridge-A with varying skew angle. Considered seismic isolation systems into the design serve to diminish the detrimental effects of skew angle on the performance of bridges. This observation consistently occurred for three different skew angles (Skew angle 30, skew angle 15, skew angle 0). Particularly by the inclusion of the strong vertical components, uplift of the isolators were experienced in the nonlinear response history analysis. However, current tools in SAP2000 do not have capability to capture the occurrence of the uplift of isolation units for the multi surface curved spherical bearings (TFP) in a comprehensive way. Further studies need to be conducted to develop reliable models for the multi-curved surface friction sliders for the three dimensional analysis. Currently available elements in computer

programs only considers the bidirectional coupling in the kinematic models. In contrast, overall 3D behavior of seismically isolated building equipped with multi-curved surface friction sliders were captured in a desirable level during the full scale tests of structures (*e.g* Dao et. al.2013). Such large scale testing offers promising developments about the 3D coupled large deformation elements and give insight about the system behavior rather than only addressing the overall behavior of individual units.

Simplified bridge models are prepared on the basis of rigid superstructure and substructure assumption, where the seismic isolators are defined by their effective parameters in the preliminary design phase. The variation of demand parameters by using the simplified analysis procedures are considered to re-evaluate the accuracy of the proposed relations under the multi-directional earthquake loading. Definition of the critical site-to-source distance for near-fault earthquakes differs in ASCE/SEI 7-10, AASHTO Bridge Specification (2010) and Eurocode-8 (CEN, 2005). Nonlinear time history analysis results of the simplified bridge models were varied regarding to modeling assumptions and further analysis need to be performed with respect to the prepared bins in the Chapter 2. Critical range of distance were found to be in the range of 5 -10 km regarding to the NRHA of simplified bridge models.

Impact of the ground motion directionality on structures has been previously investigated by several researchers (*e.g.* Reyes and Kalkan 2012, Shinozuka *et al.* 2009) and it has been found that the directionality of earthquakes may increase the seismic demands of conventional bridges. Ground motions that are oriented in the fault-normal, fault parallel and principal directions are used as input motions to assess the effects of directionality. It has been found that the directionality effects of the input ground motions does not really create any deficiency regarding to the performance of isolation bearings in the considered bridge models.

The second bridge example is a part of large scale testing which requires to generate the detailed, reliable analytic models for the Pseudo-Dynamic (PsD) testing. The Pseudo-Dynamic (PsD) test of the Rio-Torto Viaduct is conducted for the as-built and isolated cases by using the Emilia Earthquake records as input motion. Large-scale response data of the as-built fixed-base bridge specimen and retrofitted bridge model provided important findings

about the prenormative bridges that are constructed in 1960s. We consider that such experimental data sets will contribute to have insights about the retrofitting procedures of existing bridges in Europe. Effectiveness of the incorporated isolation system in reducing the demands on bridges are demonstrated by analytical models and reliability of the implemented models are justified through the results of conducted PsD test. Damage level of columns and transverse beams at each bent were found to be sensitive to the breakaway friction and stick-slip behavior of the curved surface friction sliders in the conducted PsD test. The level of transverse beam deformations computed by computer programs were in the range of acceptable range when compared with the PsD test results.

As an outreach of RETRO project, generated analytical models are used to assess the effectiveness of the isolation system under near-fault earthquakes that are compatible with the seismic demand representation of the region. The results under the unidirectional loading does not reflect the coupled plasticity behavior of isolators and the current state of analysis did not consider the effect of vertical ground motion. However, it has been shown that the vertical component of ground motion can have significant detrimental effects for curved surface friction sliders in close proximity to the rupture regarding to the lose of contact under tension as experienced in Bridge-A.

In the final design of seismic isolation systems for the Rio-Torto Viaduct both coupled plasticity models of isolation devices and a model that can capture the occurrence of uplift behavior under near-field earthquakes need to be considered to understand the real performance of the bridge. Near-source earthquake records with pulse content had higher displacement demands than records with nonpulse content at pier locations. Ratio of the average response of suite of ground motions with pulse content motions to nonpulse content motions are increased approximately 43% for the isolated case and 80% for the as-built model. Moreover, the Bridge-B with partial isolation had more uniform response than the as-built model which was intentionally one of the goals of the applied performance enhancement strategy. Any opinions, findings and conclusions or recommendations expressed about the Rio-Torto Viaduct are the authors of this dissertation.

It has been concluded that the effectiveness of isolation systems in reducing the demand parameters of both bridge models are demonstrated by analytical models. Extensive parametric investigations were indicated that the suite of pulse dominated near source ground motions with coupled vertical components not only led to increase in the demand parameters of isolation systems but also exceeds the code-mandated factors for MCE level earthquakes. These results indicate that selection and scaling of near-source records could not be treated like ordinary far-field ground motion and development of new methods for their treatment in nonlinear response history analysis for practicing engineers are strongly recommended by the author.

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