Seismic Behaviour of Isolated Bridges with Soil-Structure-Interaction Effect under Near-Fault and Far-Field Earthquakes

by

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Few years ago, I started a journey carrying many worries and wishes alongside.
As an international student struggling with a new life, new culture, and new language in a new country, Canada.

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Comportement sismique des ponts isolés avec effets de l’interaction sol-structure sous des séismes proches de la faille et en champ lointain

Nastaran CHESHMEHKABOODI

RÉSUMÉ

L'isolation sismique de la base est reconnue comme étant une stratégie de conception sismique efficace pour atténuer le risque sismique et améliorer les performances structurales des ponts. Cependant, certains paramètres, tels que les propriétés des mouvements du sol et les caractéristiques du sol, influencent la réponse sismique et peuvent réduire les performances de la technologie. Cette recherche vise à étudier les effets de l'interaction sol-structure (SSI), en ce qui concerne différents mouvements du sol sismiques modérés et forts associés à différentes distances de la source au site et au contenu fréquentiel, sur les réponses sismiques des ponts conventionnels et isolés à a base. À cette fin, des groupes d'enregistrements recueillis proches de la faille (NF) modérés et forts, avec et sans impulsions de vitesse et des enregistrements recueillis en champ lointain (FF) sont appliqués aux ponts conventionnels et isolés avec et sans prise en compte du sol sous-jacent et des effets de l’interaction sol-structure (SSI). L’influence des caractéristiques du sol est étudiée en considérant trois propriétés de sol représentant du roc, un sol dense et un sol raide. De plus, l’effet de l’approche de modélisation de la SSI est étudié en comparant les résultats obtenus via deux approches de modélisation : 1-Représentation directe du sol; 2-Méthode simplifiée (sous-structure) où le sol est représenté par des ressorts linéaires équivalents. Les systèmes pont-sol sont modélisés et analysés dans le logiciel d’éléments finis général Abaqus. Des analyses temporelles non linéaires (NLTHAs) sont effectuées et les réponses structurales maximales, individuellement et en moyenne, obtenues des deux approches en termes d'accélération maximale du tablier, de cisaillement à la base et de déplacement du tablier et dans le système d'isolation sont étudiées et comparées. Les résultats démontrent que la différence entre les deux approches de modélisation du sol est significative. L'utilisation de la méthode simplifiée (sous-structure) doit être interprétée avec une attention particulière à la limite de validité de l'utilisation du modèle linéaire équivalent, car de nombreux enregistrements capturés sur des sols plus mous n'étaient pas éligibles en raison de la limitation de l'indice de déformation de cisaillement (généralement inférieur à 0,03%), et les réponses ont été très dispersées, surtout pour le pont conventionnel. Par conséquent, la méthode simplifiée d'utilisation de ressorts linéaires pour représenter la strate du sol est une approche assez simple pour capturer tous les principaux mécanismes impliqués dans le sol, la SSI et les caractéristiques de chaque enregistrement sismique. Aussi, les résultats de cette étude révèlent que la performance du pont et les effets du sol sont dominés par l'incertitude des mouvements du sol et leur contenu fréquentiel pour les enregistrements de séismes modérés et forts. Les rapports entre l'accélération maximale du sol et la vitesse maximale du sol (PGA/PGV) jouent un rôle décisif dans toutes les réponses dynamiques. Les enregistrements avec un faible PGA/PGV<12 provoquent des demandes sismiques plus élevées en termes de forces et de déplacements, quelle que soit la distance de la faille. Cependant, malgré l'importance des enregistrements FF, ces enregistrements ne sont pas aussi exigeants que les enregistrements NF pour les ponts isolés sur sols meubles, et les enregistrements NF montrent des demandes sismiques plus élevées. De plus, les enregistrements de type pulsé, probablement observés pour les enregistrements NF forts et contenant en même temps un faible rapport PGA/PGV, provoquent des demandes sismiques e jusqu'à 50% plus élevée en force et jusqu'à 75% plus élevée en déplacement, en termes moyens et par rapport aux enregistrements NF sans impulsions. Toutefois, les réponses des sols plus mous ont montré que la présence du sol diminue l'effet négatif de l'impulsion et diminue la demande sismique en force par rapport à la réponse d'un modèle avec base fixe, n'incluant pas l'effet du sol. Par conséquent, une attention particulière doit être accordée à la conception des systèmes d'isolation pour éviter de sous-estimer la demande de déplacement pour des enregistrements de type impulsion, en particulier sur des sols plus mous. Les réponses des différents systèmes d'isolation démontrent que la résistance caractéristique (Q_d), la rigidité post-élastique (K_d) et la capacité de déplacement plus élevées sont nécessaires pour que les enregistrements de type impulsion NF afin de répondre à la demande sismique de déplacement, en particulier sur les sols plus mous.

Mots-clés : Isolation sismique de la base, caractéristiques des tremblements de terre, effets de l'interaction sol-structure, enregistrements de faille proche, enregistrements de champ lointain

Seismic behaviour of isolated bridges with soil- structure- interaction effect under near-fault and far-field earthquakes

Nastaran CHESHMEHKABOODI

ABSTRACT

Seismic base-isolation is recognised as an efficient seismic design strategy for mitigating seismic risk and improving structural performance of bridges. However, some parameters, such as earthquake inputs and soil characteristics, influence the seismic response and may reduce the technology's performance. This research aims to investigate the effects of soil-structure interaction (SSI) with regard to different moderate and strong earthquake ground motions associated with different source distances to the site and frequency content on the seismic responses of conventional and isolated bridges. To this end, groups of moderate and strong Near-fault (NF) records, with and without velocity pulses, and far-field (FF) ground motions are applied to the conventional and isolated bridges with and without considering the underlying soil and soil-structure interaction (SSI) effects. The influence of soil characteristics is investigated by considering three soil properties representing rock, dense and stiff soils. Furthermore, the effect of the modeling approach of SSI is investigated by comparison of results from two modelling approaches: 1-Direct representation of soil; 2-Simplified (substructure) method where soil is represented by equivalent linear springs. The bridge-soil systems are modeled and analysed in the Abaqus general purpose finite element software. Nonlinear time history analyses (NLTHAs) are carried out, and the individual maximum, as well as the average of individual maximums of the structural responses obtained from both approaches in terms of deck acceleration, base shear, and displacement of the deck and within the isolation system, are studied and compared. Results demonstrate that the difference between the two approaches of modeling the soil is significant. Using the simplified (substructure) method should be interpreted alongside careful attention to the validity limits of using the equivalent linear method as many of the records captured on softer soils were not eligible based on the limitation of the shear strain index (generally under 0.03%), and the responses were very scattered, especially for the conventional bridge. Therefore, the simplified method of using linear springs to represent the soil stratum is a rather simple approach to capture all the major mechanisms involved in soil, SSI, and characteristics of each earthquake ground motion. Furthermore, the results of this study reveal that the bridge performance and soil effects are governed by the uncertainty in the ground motions and their frequency contents for both moderate and strong earthquake records. The ratios of the peak ground acceleration to peak ground velocity (PGA/PGV) play a decisive role in dynamic responses. Records with low PGA/PGV<12 cause higher seismic demands in terms of forces and displacements, regardless of the distance associated with the ruptured fault. However, despite FF record's importance, these records are not as demanding as NF records for isolated bridges on soft soils, and NF records show higher seismic demands. In addition, pulse-type records, likely observed for strong NF records and at the same time containing a low PGA/PGV ratio, cause up to 50% higher seismic force demand and up to 75% higher seismic displacement demand on average compared to NF records without pulses. However, responses of softer soils showed that presence of the soil diminishes the negative effect of the pulse and decreases the force demand, compared to the response of a fixed-base model, not including soil. Overall, careful attention should be paid to properly incorporate soil structure interaction in order to estimate correctly the displacement demands in the isolation systems, especially on soft soils under pulse-like records. Responses of the different isolation systems demonstrate that the higher characteristic strength (Q_d), post-elastic stiffness (K_d), and displacement capacity are needed for strong NF pulse-like records to provide the displacement demand, especially on softer soils.

Keywords: Seismic base-isolation, Earthquake characteristics, Soil-Structure Interaction effects, Near-Fault records, Far-Field records, Pulse-type records, Bridges

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LIST OF ABREVIATIONS

AI	Arias Intensity
BEM	Boundary Element Method
CSA	Canadian Highway Bridge Design Code
DFP	Double Friction Pendulum
EDC	Energy Dissipated per Cycle
EOM	Equation of Motion
FEM	Finite Difference Method
FDM	Finite Difference Method
FF	Far Field
FN	Fault Normal
FP	Fault Parallel
FPS	Friction Pendulum System
HDRB	High Damping Elastomer
LRB	Lead Rubber Bearing
LDRB	Low Damping Elastomer
NF	Near Fault
NLTHA	Nonlinear Time History Analysis
MMSA	Multi-Mode Spectral Analysis
PEER	Pacific Earthquake Engineering Research center
PGA	Peak Ground Acceleration
PGD	Peak Ground Displacement

PGV	Peak Ground Velocity
SDOF	Single Degree of Freedom
SSI	Soil Structure Interaction
SIB	Seismic Isolated Bridge
SIS	Seismic Isolation System
SMSA	Single Mode Spectral Analysis
SPSI	Soil Pile Structure Interaction
TFP	Triple Friction Pendulum

LIST OF SYMBOLS

Q_d	Characteristic strength (N)
K_d	Post-elastic stiffness (N/m)
K_u	Elastic stiffness (N/m)
K_{eff}	Effective stiffness (N/m)
K_{sub}	Substructure stiffness (N/m)
D_{max}	Maximum displacement (mm)
F	Force (N)
C	Viscous damping (N s/m)
d_{sub}	Displacement within the substructure (mm)
d	Total displacement (mm)
d_i	Isolator displacement (mm)
d_y	Yield displacement (mm)
S_d	Design displacement spectrum(mm)
S_a	Design acceleration spectrum (g)
W	Weight (N)
ΔG	Ground movement (m)
ΔH	Horizontal movement (m)
ΔR	Rocking movement (m)
ΔS	Swaying movement (m)
G	Shear modulus (MPa)
M	Mass (Kg)
u	Displacement vector (m)
T	Period (s)
μ	Friction coefficient
E	Elastic modulus (GPa)
ρ	Density (Kg/cm^3)
C	Cohesion stress (N/m^2)
ϑ	Poisson's ratio

ø	Friction angle (°)
$V_{s\,(30)}$	Average shear wave velocity in top 30 m (m/s)
Ψ	Dilatancy
ξ	Equivalent Viscous damping ratio (%)
R_{rup}	Ruptured fault distance, (km)
L	Length (m)
B	Width (m)
D	Depth (m)
$K_{x,y,z}$	Stiffness, Translation along x,y,z-axis (N/m)
$K_{xx,yy,zz}$	Stiffness, Rocking about x,y,z-axis(N-m/rad)
B	Embedment ratio

LIST OF UNITS

m	Meter
cm	Centimeter
mm	Millimeter
km	Kilometre
kg	Kilogram
m/s	Meter per second
cm/s	Centimeter per second
m/s^2	Metre per second squared
g	Gravitational acceleration, $g=9.81\ m/s^2$
N	Newton
kN	Kilonewton
N/m	Newton per meter
N/m^2	Newton per square metre
Pa	Pascal = Newton per square metre
GPa	Gigapascals = 10^9 Pascal
MPa	Megapascal = 10^6 Pascal
kg/m^3	Kilogram per cubic metre
N.s/m	Newton second per metre
N-m/rad	Newton metre per radian
°	Angle degree
Rad	Radian

INTRODUCTION

Bridges are one of the most critical infrastructures in today's modern society, serving as a crucial artery in transportation systems, especially in times of crisis, such as the period following a major earthquake. Strong earthquakes may cause huge damage to a bridge in terms of socio-economic losses, or fatalities. It should be noted that even if one component of a bridge fails, the entire structure will be affected because failure at one specific part, such as span failure due to support length at joints or failure in a column, will interrupt the operation of the entire bridge, which is a key component of the land communicating system between different places.

In modern codes, bridges are designed to achieve a target seismic performance. Levels of target seismic performance are set as a function of the importance of the bridge and the severity of the earthquake so that loss of functionality after strong seismic events is not an acceptable performance for important bridges such as lifeline or major route bridges in the high seismic zones (Constantinou et al., 2016).

In conventionally constructed bridges, earthquake-resistant performance typically relies on a significant inelastic action (energy dissipation) in selected substructure components of the bridge, such as piers and bent caps designed to resist earthquakes. It is generally targeted by bridge owners and design codes that bridge structures would (1) not collapse in very rare earthquakes, (2) provide life safety for rare earthquakes, (3) suffer only limited repairable damage in moderate shakings, and (4) be undamaged in more frequent, minor earthquakes. Such a philosophy and target performance are similar to that adopted for buildings. Higher performance is implicitly or explicitly targeted for more important bridges by increasing the seismic resistance and controlling the inelastic deformations (Constantinou et al., 2007).

Inelastic action results in damage, which is often substantial in scope and difficult to repair. Importantly, structural damage will generally result in bridge closure with attendant direct and indirect economic losses (Constantinou et al., 2007). After observing significant damages in early conventionally designed bridges due to a partial or complete collapse of piers and spans caused by major seismic events, researchers noticed that the strength alone would not be a sufficient factor for the safety of the bridges during the earthquakes (Tongaonkar & Jangid, 2003).

For the past few decades, researchers have continuously focused on studying various technologies in order to prevent or minimize the damage caused to the structures from severe seismic activities. One of the rational and fundamental solutions for mitigating the effects of earthquakes is the isolation technology, which has shown great potential for improving seismic performance over the conventional design philosophy (Buckle & Mayes, 1990; Kelly et al., 1980).

Seismic isolation, which attempts to reduce the seismic forces to the elastic capacity of the members, eliminates or drastically reduces inelastic deformations. The main concept in seismic isolation of bridges is to reduce the structure's lateral fundamental frequency of vibration to a value lower than the predominant energy-containing frequencies of earthquakes. The other purposes of an isolation system are to provide means of energy dissipation and to have adequate rigidity for service loads such as wind and vehicle braking while accommodating environmental effects such as thermal expansion, creep, shrinkage, and pre-stress (Buckle et al., 2006; Buckle & Mayes, 1990; Guizani, 2003; Kelly et al., 1980; Soneji & Jangid, 2008).

Although seismic isolation is an effective technology for improving the seismic performance of bridges, it has some limitations. As mentioned earlier, seismic isolation improves the performance of a structure under seismic loading partially by changing (increasing) the fundamental (first mode) vibration period of the structure. Thus, the vibration period of the structure is moved away from the high-energy seismic ground period, and seismic energy transferred to the structure is minimized. Therefore, using a seismic isolation system for the case of the structure supported on soft or weak foundation soil, where high period ground motion is dominant, reduces the benefits this technology offers. Due to the principle of dynamic resonance, a larger difference between the natural frequencies of the isolated soil-structure system and the predominant earthquake input frequencies results in a minimized seismic energy transferred to the superstructure. Therefore, seismic isolation is an effective method in relatively rigid structural systems and should be designed carefully for bridges on softer soils and prone to severe earthquakes with different frequency ratios (Buckle & Mayes, 1990; Kelly, 1993; Kelly et al., 1980).

Consequently, seismically isolated bridges can suffer severe damage if their seismic isolation system is not designed properly to withstand the expected displacement demand. Among different pivotal parameters on structural responses of the bridges, evidence from past earthquakes has indicated that the earthquake characteristics and the site conditions are two of the most critical parameters affecting the seismic performance of infrastructures in earthquake-prone areas and should be carefully considered in the design process, modeling, and analysis (Beresnev & Wen, 1996; Castaldo & Tubaldi, 2018; Jónsson et al., 2010; Roussis et al., 2003).

The case of the Bolu Viaduct bridge is a good example of record characteristics effect on the isolated bridges, which caused a complete failure of the installed seismic isolation system and narrowly avoided total collapse due to excessive superstructure movement during the Duzce earthquake (1999). Duzce earthquake was considered a NF earthquake for the Bolu Viaduct bridge. NF records are generally characterized by long-duration pulses that subject the structure to very high input energy at the early stage of the record (Liao et al., 2004; Roussis et al., 2003; Somerville et al., 1997).

As another important factor, SSI is the term used to define the type of interaction between the structural system and the soil at the foundation level and the abutments. This reciprocal action has an important effect on the structural responses under horizontal and vertical accelerations (Mazloom & Assi, 2022). Many studies have been conducted and showed the potentially detrimental role of softer soils in intensifying the structural responses. Soft soil can result in elongation of the system' period, resulting in severe effects such as large displacements and deformations in critical regions of the bridge. Furthermore, soft soil can result in amplifying the ground motion, causing a more severe structure response (Rayhani et al., 2008). For long period bridges such as the isolated bridges, not considering the effect of site soil in the designing process will lead to underestimating the displacements demand for the bearings (Dimitriadou & Fardis, 2007).

One of the evidences where SSI had a major contribution to bridge collapse is the Hanshin (Fukae) expressway bridge during the Kobe earthquake in Japan in 1995. An analytical study of the bridge showed that soil characteristics had a significant effect on the behaviour of the bridge. The elongation of the fundamental period of the system and modification of earthquake frequency content due to soft soil resulted in intensifying the structural response (Mylonakis & Gazetas, 2000). This shows seismic responses of soil and structure system together are highly influenced not only by the characteristics of the superstructure but also by the characteristics of the foundation and the underlying soil. In such systems, responses of the structure and soil are inter-dependent (Mylonakis & Gazetas, 2000; Tuladhar et al., 2008).

To minimize seismic risk, avoid catastrophic failures of bridges, and ensure an acceptable seismic reliability of bridges, taking into account the changes in the structural response due to soil-structure interaction needed to be progressively addressed and adjusted from edition to edition of bridge design codes. This is especially important for bridges in regions with soft soil and high earthquake activities.

For example, to reflect such lack of knowledge on site effects on seismically isolated bridges, the Canadian Bridge design code in its edition 2006, recommended conservative site effects which are about 25-35% higher than those for conventionally designed bridges (CSA, 2006) on different soils and the current revision of the code (CSA-S6:19), more details and parameters is considered to determine the site effects based on the seismicity of the area and also the period of the bridge (CSA, 2019).

Aside from various conclusions drawn from research studies, most of these studies have focused on either ground motion characteristics or soil properties with or without isolation systems, with little attention paid to the combined effects of both on conventional and isolated bridges for earthquake prone areas.

Additionally, to avoid the modeling process's complexity and shorten the analyses' time, the soil supporting the foundation is considered very stiff in many research studies and its effects have been ignored during the analyses. In many research works, the substructure method for considering the soil effects is used without considering the validity of the method on soft soils and considering the limitations on the shear strains within the soil for such a method. Not only the number of research studies using the direct method is rather limited, especially on base-isolated bridges; most of the research works using both methods have considered soils to behave within their elastic range which is questionable, or used rather oversimplified soil behaviour models.

As a result, there is a need for more investigation on the effects of soil structure interaction and different key parameters on the seismic response and performance of seismically base-isolated bridges. Of particular importance, investigation of the effects of various record characteristics, associated to NF and FF ground motions, such as presence of pulses, frequency contents based on the ratio of PGA/PGV, the seismicity level where these records originated. Furthermore, the effects of different soil types on the seismic responses of the bridge with and without the isolation systems are scrutinized through results obtained by the direct and substructure approaches. In this context, the main objective of this study is as follows:

Thesis objectives

The general objective of the research undertaken within the frame of this thesis is to contribute to the development of a better and more accurate understanding about the seismic behaviour of isolated bridges with SSI effect subjected to strong and moderate NF and FF earthquakes with different characteristics and frequency contents.

To this end, the general objective is detailed into three specific objectives as follows:

1. Quantify the effect of soil properties on the modifying of different earthquake records, including NF and FF earthquakes.

2. Quantify the effect of site soil characteristics and the distance of earthquake source, (NF and FF) with different frequency contents on the structural performance of isolated bridges for moderate and high seismicity areas.

3. Evaluate the performance and effectiveness of the seismic isolation located on different soil properties under strong earthquakes including NF records with and without pulses, and FF earthquakes with different frequency contents.

To achieve these objectives, numerical parametric studies, including modelling and seismic analysis of different bridge variants and case studies under different seismic records are carried out in Abaqus and SAP2000 software programs. The proposed methodology and procedure are detailed hereafter.

Methodology of the thesis

An overview of the methodology for the current research study is shown in Figure I. A review of the recent studies in the field of seismic responses of conventional and isolated bridges considering soil effects and SSI is required for pursuing the objectives specified in this study. To this end, a literature review is first carried out to identify the effective and crucial parameters on seismic responses of bridges when soil effect is considered to determine the research gaps in this area. Following that, the effective parameters such as soil properties and earthquake record characteristics are selected and categorized. For soil, three groups of hard rock (A), very dense soil (C) and stiff soil (D) based on the site classification in Canadian Highway Bridge Design Code are selected (CSA, 2019).

A Mohr-Coulomb based constitutive model has been adopted in this study to simulate the nonlinear behaviour of the soil medium. The model uses Mohr-Coulomb criterion to derive the primary and envelope curve of soil response and is coupled with an elastic-perfectly plastic model to describe inelastic force-deformation relation, unloading and reloading branches, with no degradation. Such a relatively simple mode is believed to catch efficiently, in terms of computation effort, the main features of soil hysteretic behaviour and has been used by many researchers (e.g., Conniff and Kiousis 2007; Rayhani and El Naggar 2008, Ghandil & Behnamfar 2015, Khazaei, Amiri, & Khalilpour, 2017 , among others) in modeling the dynamic SSI as a means to simulate soil behaviour under seismic loads in soil-structure systems (Conniff & Kiousis, 2007; Khazaei et al., 2017; Rayhani et al., 2008; Tabatabaiefar & Fatahi, 2014). Mohr–Coulomb material model requires conventional and easy accessed soil parameters including unit weight, friction angle, cohesion intercept, shear modulus (Rayhani et al., 2008).

In addition, as the main purpose of the study is focusing in the dynamic structural responses not the detailed behaviour of the soil, this model considerably reduces the sensitivity of selection of a specific soil nonlinear behaviour for the purposes of this study where the criteria are accurately in prediction of structural responses with acceptable errors being up to 10% for engineering design applications. This makes the simplicity of the nonlinear soil model a prime advantage for a response analysis of soils with acceptable accuracy (Brinkgreve & Vermeer, 1998; Ghandil & Behnamfar, 2015). Furthermore, the soil is assumed to be non-liquefiable and the ground water level, consequently the effect of pore pressure on the effective stress is ignored. The soil model is taken in this study for analysis with Abaqus (ABAQUS, 2019).

Consequently, for earthquake records, two groups of moderate (with the intensity from 4 to 6 in Richter scale), and strong earthquake (with the intensity more than 6.5 in Richter scale) for NF records within the distance of a fault less than 20 km and FF areas for distances more than 20km are chosen (Agrawal & Shrikhande, 2006; United States Geological Survey, 2012; Yatan et al., 2017). The strong NF earthquake records are also categorized to two groups of pulse-like and no-pulse records. In the next step, the case study bridge is selected and the conventional and isolated bridge are modeled in Abaqus and Sap2000 software considering the substructure (simplified) method and the direct method. Following the sensitivity studies for the mesh size and time steps, responses of NLTHA for the conventional and isolated bridges with and without the presence of the underlying soil, all subjected to abovementioned earthquake records are studied and compared to investigate the effect of changes in the soil properties, earthquake characteristics, and isolation properties on the seismic responses of selected bridge.

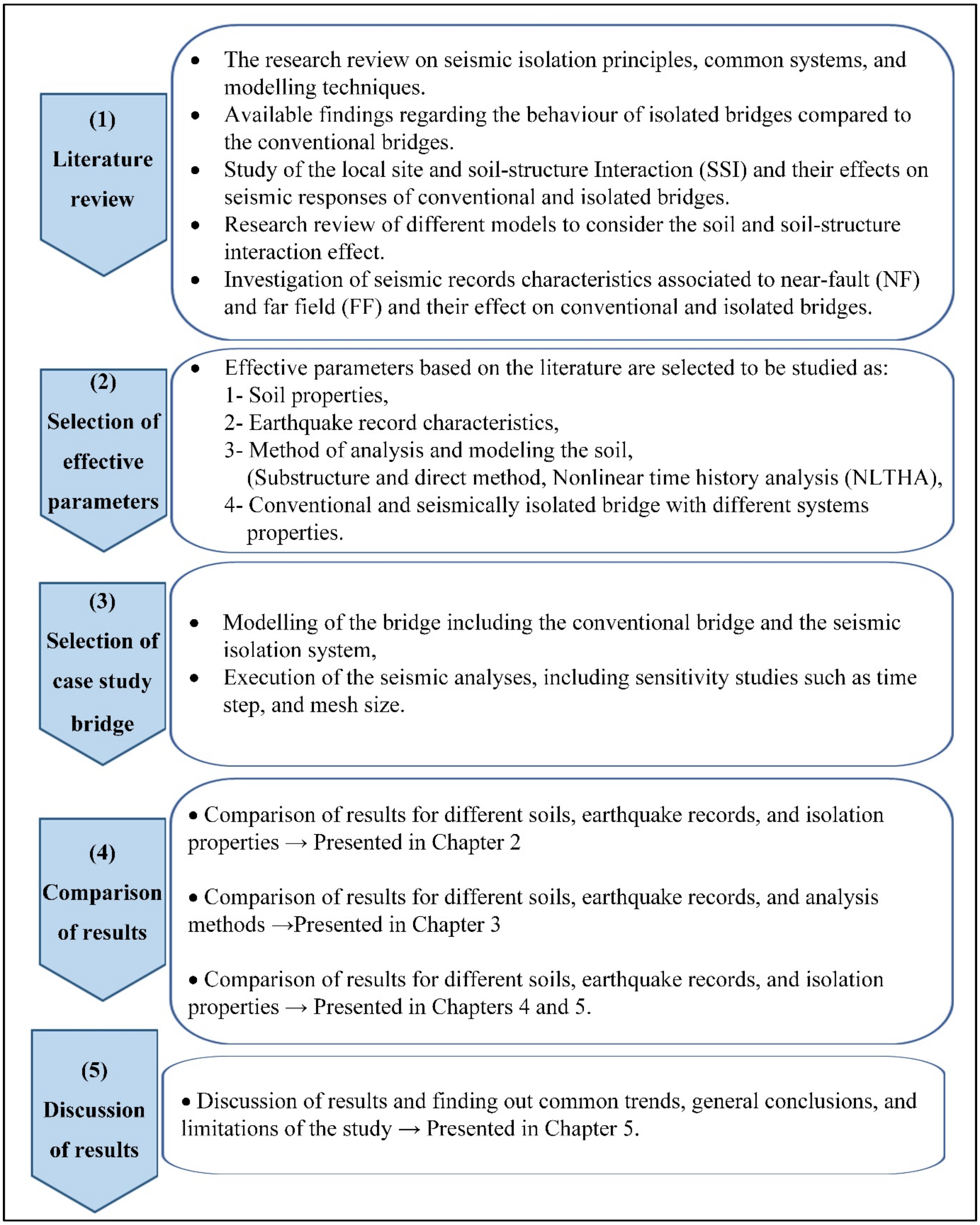

Figure I Methodology of the current research study

Original contributions of the thesis

The contribution of this thesis can be categorized into two parts:

First, it presents different comprehensive techniques of advanced modeling and seismic analysis of conventional and isolated bridges, including SSI. Also, the findings allow a better understanding of the effect of different parameters that impact the seismic response and performance of base-isolated and conventional bridges and the design of the seismic isolation systems. This understanding leads to more precise and effective isolation strategies by choosing more appropriate methods and models, when necessary, to catch the SSI and different record characteristics' effects.

Second, the findings of this research will contribute to putting forward more reliable and effective design rules through codes' specifications and more effective design strategies for seismically isolated bridges for safer and more reliable bridges during earthquakes. Additionally, it will help the researchers to enhance their knowledge and perspectives about the soil effects and record characteristics on conventional and isolated bridges.

Content and structure of the thesis

This thesis is essentially an article-based thesis. It consists of the present introduction and five main chapters and a conclusion.

Chapter 1: This chapter presents a state-of-art, focusing on the three main subjects. First, different types of isolation systems and their behaviour is surveyed. Then, the dynamic-soil-structure interaction (SSI) and different modeling approaches are discussed and the effects on seismic response is outlined. The last part elaborates on the effects of earthquake characteristics, including NF and FF records on the seismic response of bridges.

Chapter 2: Chapter 2 presents an article covering the first specific objective to understand the effect of soil properties on the propagation of different earthquake records, including NF and FF earthquakes, on different isolation systems.

Chapter 3: This chapter pursues the second specific objective to study the effect of different soil properties and moderate NF and FF earthquakes, with different frequency contents, on the structural performance of conventional and isolated bridges.

Chapter 4: This chapter presents an article fulfilling specific objectives 2 and 3. It investigates the effect of different soil properties and the distance of strong NF records with and without pulses and different frequency contents on the conventional and isolated bridges equipped with different isolation systems.

Chapter 5: In Chapter 5, specific objectives 2 and 3 are covered as a result of an article to investigate different soil properties and strong NF records with and without pulses and FF earthquakes containing different frequency contents on conventional and isolated bridges.

The last part of the thesis is devoted to the general conclusion of the present research study. In addition, the limitations of the current study and the future complementary research works are discussed in this chapter.

CHAPTER 1

LITTERATURE REVIEW

This chapter presents a literature review on the main themes related to the studied problem. A survey on seismic isolation principles, common systems and modelling techniques is presented first. Then, follows a presentation and a summary of the literature findings on soil-structure interaction (SSI) and its effects on seismic responses of conventional and isolated bridges are discussed focusing on the soil characteristics and different method of considering soil effects. The last part covers the literature on the seismic record characteristics associated to near fault (NF) and far field (FF), as well as frequency contents of the records and the effects of pulse-like records on the seismic responses of the conventional and isolated bridges.

1.1 Seismic isolation

Nowadays, seismic isolation technology is considered as one of the most suitable solutions to counter the seismic effects on structures, and is widely used for new bridges and the rehabilitation of existing bridges in moderate and high seismic areas (Abe et al., 2004a; Buckle et al., 2006; Dall'Asta & Ragni, 2006; Makris, 2019; Morgan, 2007).

Seismic isolation introduces flexible link between the bridge superstructure (the deck) and the substructure units (piers and abutments with their foundations) to decouple the movement of the deck (mass) from the ground (excitation source). Therefore, when the bridge is subjected to an earthquake, the superstructure moves more independently from the substructure and typically at a frequency much lower than the range of dominant frequencies of the earthquake. Consequently, the seismic energy transmitted to the structure is reduced significantly, inertial forces transmitted between the superstructure and the substructure are considerably reduced and damage to the bridge is prevented or largely mitigated (Buckle et al., 2006; Buckle & Mayes, 1990; Kelly et al., 1980).

Seismic isolation technology is based on decreasing the fundamental structural vibration frequency to a value less than the predominant energy-containing frequencies of the earthquake in order to reduce the seismic force demand to or near the elastic capacity of the structural members, thereby eliminating or drastically reducing inelastic deformations. Numerous experimental and analytical research studies have compared the seismic responses of conventional and isolated structures and have shown that seismic isolation technology plays a great role in reducing the seismic forces induced by an earthquake in infrastructures, by the factor of five to twenty or even more (Chandak, 2013; Ghobarah & Ali, 1988; Guizani, 2007; Soneji & Jangid, 2008; Tongaonkar & Jangid, 2003; Tsopelas et al., 1996). The long-term benefit of these technologies lies in its preservation of the serviceability of the structure during and immediately after an earthquake and thus eliminating the cost of reconstruction. Seismic isolation not only has undeniable advantages for bridges where it is more widely used, but also for buildings as it allows meeting more easily seismic performance requirements of building codes for certain essential structures, making it an excellent design option for special building projects too (Dicleli & Buddaram, 2006; Guizani, 2003; Guizani & Chaallal, 2011).

During an earthquake, the inelastic deformations occur mainly in the isolation units instead of the substructure elements, which limits the damage and enhances chances of maintaining the functioning of the bridge. This protection strategy revolves around four basic characteristics of the isolation system. First, the isolation system is typically a modified version of standard bearings and in this context, it shall have a high vertical bearing capacity to support gravity loads. Second, to lengthen the lateral fundamental periods of vibration of the structure, which is the main reason leads to the reduction of transmitted seismic lateral forces between the superstructure and the substructure, the isolation system shall have a high lateral flexibility under strong lateral forces. Third, it often needs to incorporate a dissipation energy mechanism to limit the seismic relative displacements between the superstructure and substructure and fourth, it should have an adequate rigidity for service and non seismic loads such as vehicle braking and wind, while accommodating environmental and time-dependant effects such as thermal expansion, creep, shrinkage and prestress shortening/deformation (Buckle et al., 2006;

Soneji & Jangid, 2008; Weisman & Warn, 2012). The effects of the isolation system lateral flexibility and added damping is shown in Figure 1.1 and Figure 1.2

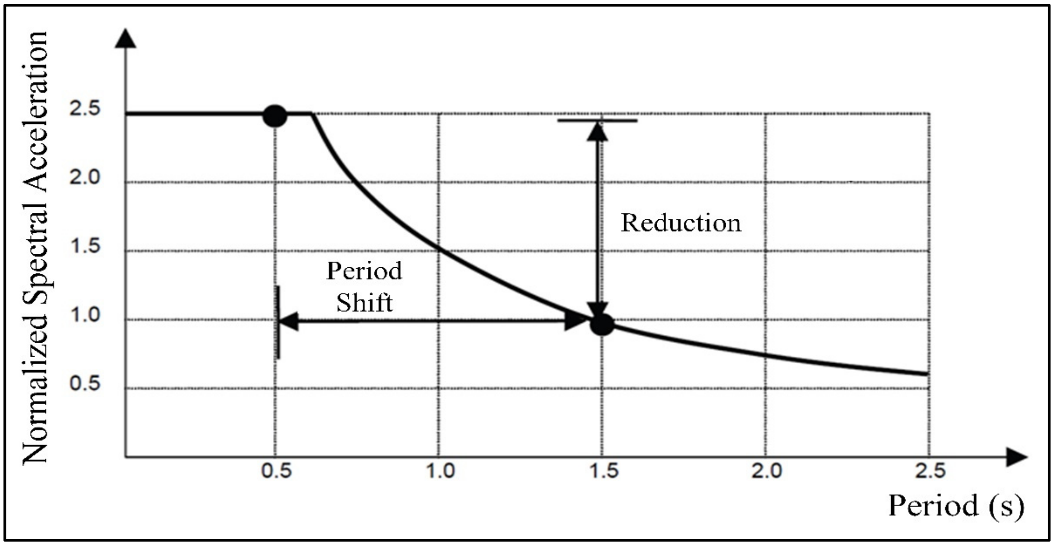

Figure 1.1 Effect of isolator’s flexibility on bridge response

Taken from Buckle et al. (2006)

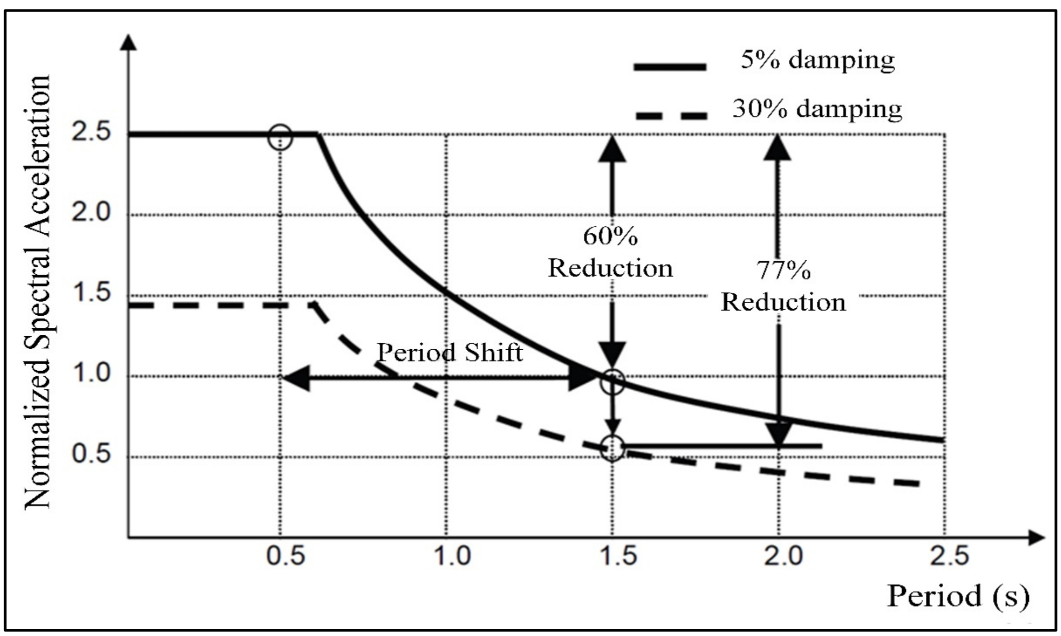

Figure 1.2 Effect of damping on bridge response

Taken from Buckle et al. (2006)

Currently, there are several types of seismic isolation systems. Commercially available seismic isolation systems fall within two main categories; Elastomeric based systems and friction /sliding based systems. Elastomeric based bearings take advantage of the flexibility of rubber under shear deformation to achieve the period of the structure lengthening, while sliding based bearings rely on the inherently low stiffness of a surface/material resting on its foundation with no connection other than friction caused by the movement at the interface (Kelly, 1993; Morgan, 2007).

The choice of the appropriate isolation system depends on characteristics of the structure such as the amount of the required bearings vertical capacity, available clearances and space, service loads to be resisted, reliability under adverse field conditions over long periods of time, environmental and service displacements to be accommodated (due to wind, thermal expansion, creep, vehicle braking, etc.), and the site seismic hazard and conditions (seismic characteristics of the area, soil conditions) which varies for each specific project (Buckle et al., 2006; Buckle & Mayes, 1990; Kelly, 1993; Kelly et al., 1980).

1.1.1 Elastomeric-based seismic isolation systems

As a broad category, there is the elastomeric-based isolation system, used on a large proportion of isolated bridges around the world (Buckle et al., 2006; Buckle & Mayes, 1990; Morgan, 2007; Tan et al., 2022). This isolation system is based on an elastomeric bearing which consists of rubber layers made of either natural or synthetic rubber (to provide horizontal movement and flexibility), sandwiched between thin steel plates (to provide vertical load capacity and stiffness). The steel plates are fully bonded to elastomers on all surfaces during the molding process. The top and bottom units of the system, are typically vulcanized to thick mounting steel plates that facilitate anchorage of the bearing to the superstructure above and the substructure unit below and allow that the whole system acts as a single unit (Buckle et al., 2006; Saidou, 2012).

The high deformation at failure and high lateral flexibility of the elastomer, working in shear, are the basis of the lateral displacement capacity of the isolator and its flexibility, respectively (Buckle et al., 2006; Buckle & Mayes, 1990; Kelly, 1993; Kelly et al., 1980).

Because natural or synthetic rubber offer low energy dissipation capacity, especially required in high seismic areas (Nguyen & Guizani, 2021b), early research has been oriented to enhancing the damping capacity of the standard elastomeric bearing (Kelly, 1993). This gave place to three main subtypes of elastomeric based isolation systems, which details of construction are shown in Figure 1.3: Low damping rubber bearing isolator (LDRB), High damping rubber bearing isolator (HDRB), which differ at the level of chemical composition and mechanical properties of the rubber, and Lead rubber bearing (LRB) which is obtained by incorporating a lead core in an LDRB.

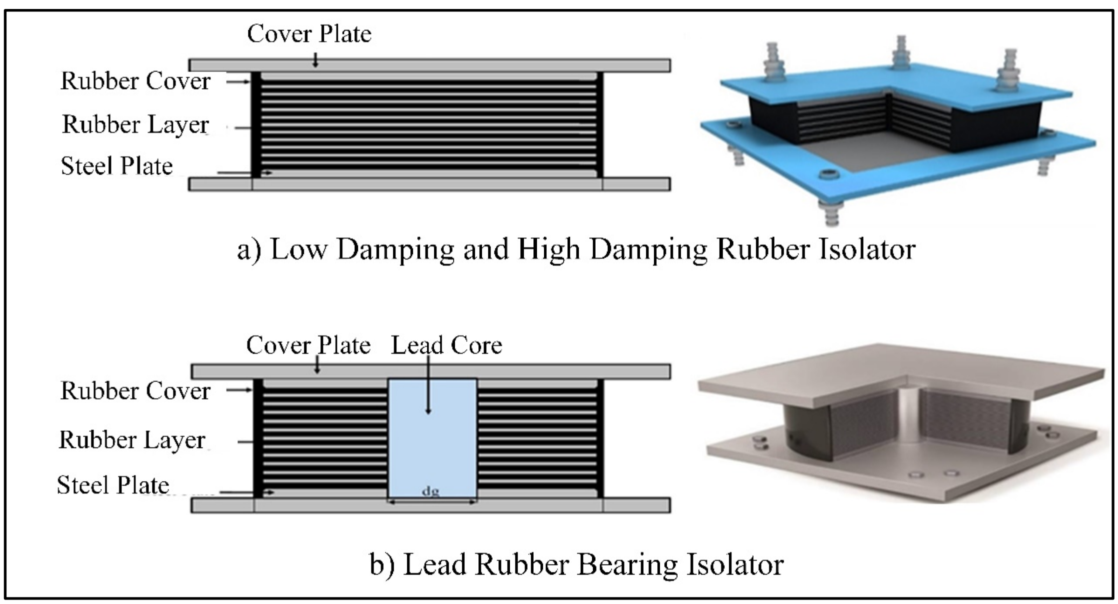

Figure 1.3 Elastomeric-based seismic isolation bearings main types
Taken from Jong Wan Hu (2015)

1.1.1.1 Low Damping Rubber Bearing (LDRB)

Low damping rubber is either the natural rubber (obtained from the Hevea plant) or the synthetic rubber (a derivative product of petroleum, more known as Neoprene) (Naeim & Kelly, 1999). Under shear deformation, low damping elastomers have the low flexibility required. However, their energy dissipation capacity, measured in terms of equivalent viscous damping, is low (around 3% to 7%) (Abe et al., 2004a; Cardone & Gesualdi, 2012; Constantinou et al., 1999; Naeim & Kelly, 1999). These elastomers (natural or synthetic) are typically not very sensitive to creep when sheared at levels below 150%. On the other hand, it presents a good long-term stability of its compressibility modulus and its shear modulus (Naeim & Kelly, 1999). The addition of steel laminae allows to increase the vertical stiffness and bearing capacity leading to the development of LDRBs in the early 1950s, and constitute a reliable and economical solution as standard bridge bearings, accommodating the thermal movement, and also the effects of prestressing, shrinkage, and creep in superstructure of bridges with stable properties (Buckle et al., 2006; Lindley et al., 1964).

The main limitation of these bearings lies in their low damping capacity which is an essential characteristic to limit seismic displacement, especially in seismic zones with low frequency records (Choun et al., 2014; Nguyen & Guizani, 2021a; Dicleli & Buddaram, 2006; Dicleli & Karalar, 2011).

To overcome this limitation, most often this system is used in association with another component with a separate energy absorbing device such as lead, steel, or viscous dampers (Abe et al., 2004a; Nguyen & Guizani, 2021a; Dicleli & Buddaram, 2006; Dicleli & Karalar, 2011; Skinner, 1993).

1.1.1.2 High Damping Rubber Bearing (HDRB)

HDRB is composed of specialized designed rubber with high damping performance which has a good horizontal shear performance to bear the horizontal force. It can absorb vibration through displacement in horizontal direction and hysteretic energy (Dall'Asta & Ragni, 2006; Li et al., 2022; Vatanshenas et al., 2021).

The difference between HDRB and LDRB lies in what the HDRB offers as a higher damping performance by means of a special formulation such as adding graphite filler (carbon black), reinforcing agent, vulcanizing agent, plasticizer and other compounding agents in natural rubber. Due to its hysteretic properties, HDRB exhibits a non-linear behaviour with an equivalent viscous damping ratio typically around 15% (Ankik, 2019; Dall'Asta & Ragni, 2006; Naeim & Kelly, 1999; Tubaldi et al., 2018; Vatanshenas et al., 2021).

High damping rubber has large horizontal deformation capacity, horizontal restoring force as well as a higher seismic energy absorption and dissipation capacity than natural rubber (Ankik, 2019; Dall'Asta & Ragni, 2006; Naeim & Kelly, 1999; Vatanshenas et al., 2021).

1.1.1.3 Lead Rubber Bearing (LRB)

Lead rubber bearing (LRB) is based on a standard rubber bearing, typically a LDRB, to which a lead core which allows to easily achieve an equivalent viscous damping up to 30%, due to high absorption capacity of the lead core (Buckle et al., 2006; Choun et al., 2014).

The lateral flexibility is provided by the rubber layers while steel reinforcing plates provide lead core confinement, axial load, and axial stiffness capacity. The lead core resists wind-generated and braking forces of a vehicle, allowing the structure to control the movements in service loading condition. In addition, when the lead core is well confined by the elastomer, under shear deformation, it acts in an almost perfectly elastoplastic behaviour (Baig et al., 2022; Monzon et al., 2016; Robinson, 1982). Because of their high damping and initial high stiffness under service loads with a relative economic cost of manufacturing, LRBs are the most common elastomeric isolators globally used in bridges (Buckle et al. 2006, Gimenez & Himeno, 2018).

1.1.2 Limitations and shortcomings of elastomer-based isolation bearings

As shown above, the different types of elastomer-based isolation bearings have common and distinct features as can be put in evidence by their typical hysteresis curves shown in Figure 1.4. Although their numerous advantages, such as low manufacturing cost, high stability of properties, long experience in the field and modulable properties, elastomer-based isolators suffer some disadvantages and limitations. Among these limitations there is their sensitivity to low temperatures and their crystallisation with the duration of exposition, which greatly affect their mechanical properties such as stiffening and isolation efficiency. Other minor limitations include the effect of aging, which can lead to their stiffening or degradation over time, the dependence of their properties on loading history, especially for the HDRB, and the need for large dimensions in the presence of high vertical loads which can be limiting, notably on rehabilitation projects, where available room is limited (Buckle et al., 2006; Nassar et al., 2022; Stanton & Roeder, 1982; Zhao et al., 2017).

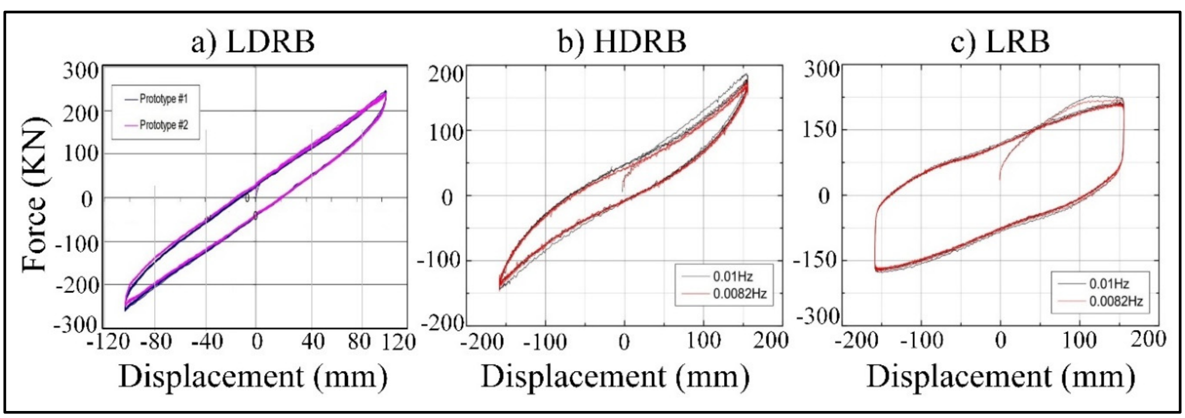

Figure 1.4 Comparison of hysteresis curves for different elastomeric bearings obtained by laboratory testing
Taken from Velev et al. (2011) and Zhenyuan et al. (2021)

1.1.3 Sliding-based seismic isolation systems

The second family of seismic isolation systems is based on accommodation of the lateral displacement through a sliding interface. The friction pendulum bearing isolator is the most extensively used kinematic system in base isolation technology. The pendulum system has a globe that is placed in two steel concave curved surfaces using sliding interfaces to allow lateral movement and decouple the movements of the superstructure from the foundation units as illustrated in Figure 1.5. The contact surface is spherical and concave, allowing the support to be re-centered and the system works like a pendulum. The radius of curvature of the surface controls the period of the pendulum while the coefficient of friction of the sliding interface controls the energy dissipation and the damping of the isolation system (Buckle et al., 2006; Morgan, 2007).

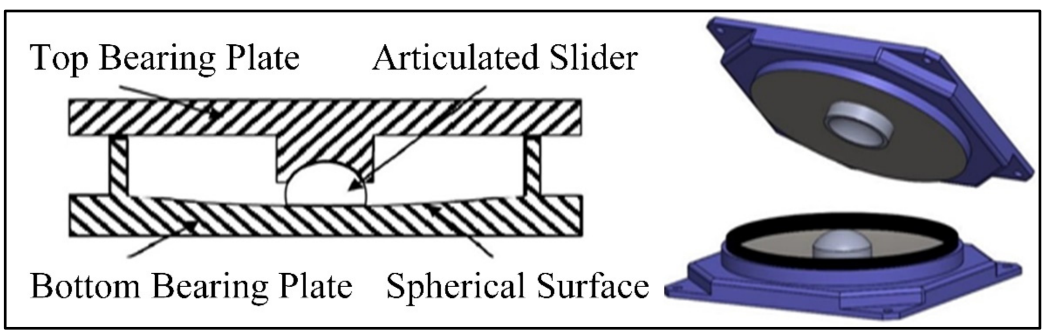

Figure 1.5 Simple Friction Pendulum Bearing (FPS)
Taken from Fatemi (2018)

The simple friction pendulum system (FPS) has all the advantages of a rubber bearing with higher vertical load capacity. It has a bilinear hysteresis loop that can meet the required performance for moderate events but is not very effective in severe earthquakes especially for NF ground motions (Becker, 2011). Therefore, other types of sliding systems were studied and introduced to cover different levels of performance depending on the project's demands using multiple pendulum mechanisms such as Double-Pendulum Bearings (DFP), Triple-Pendulum Bearings (TFP), etc.

Double Friction Pendulum (DFP) systems were proposed to achieve a device with a displacement capacity two times larger than FPSs and also to reduce the heating effects (Calvi & Calvi, 2018). Therefore, DFPs might have two different types of hysteresis loops, a bilinear in the case of equal radii and friction coefficients, and a trilinear in the case of unequal radii and friction coefficients. As a consequence, in some specific conditions, limited adaptive behaviour can be obtained by DFPs (Fenz & Constantinou, 2008; Morgan, 2007).

Triple Friction Pendulum (TFPs) bearings were developed to create a more adaptable behaviour with a smoother transition between stiffness regimes and increase the displacement capacity with a smaller plan size of bearing (Becker et al., 2017; Fenz & Constantinou, 2008). The bearing consists of four spherical sliding surfaces and exhibits three distinct pendulum mechanisms (Becker, 2011). Theoretical and experimental studies have been conducted on modified SFPs, DFPs, and TFPs systems with a variety of configurations, including various displacement capacities and friction coefficients. The results showed that adaptive behaviour could be obtained in each case when properly configured. However, the hysteresis behaviour of a TFPs is more complex and can contain up to five different stiffness regimes depending on the combination of the sliding surfaces (Becker et al., 2017; Calvi & Calvi, 2018; Fenz & Constantinou, 2008).

Adaptive behaviour of TFPs allows the design of the optimized isolation system for different amplitudes of displacements as well as different performance objectives. It can be designed to have high stiffness and low damping at lower displacements to accommodate service conditions and loads. At moderate displacement, the softening behaviour may reduce base shear at the level of a design earthquake and at a larger events, the stiffening behaviour limits the isolation displacement preventing unseating of the bearing or the bridge span (Calvi & Calvi, 2018; Fenz & Constantinou, 2008).

Therefore, the ability of TFPs to limit the displacement in rare events while maintaining drifts and acceleration in the appropriate range for low to moderate levels of earthquakes makes it superior in specific areas or projects over the SFPs and DFPs. Different Friction pendulum systems and the typical force- displacement curves are shown in Figure 1.6.

Other sliding-based systems are also commercially available, although less widely used. A Canadian system based on a pot-bearing equipped with helicoidal springs, known as the Izolatech (ZTS), was first used in Quebec in 2002-2003 and later used in some other projects in the province (Guizani, 2003) A similar system, based on a disc bearing and polymer springs, known as the Eradiquake system (EQS), is being used on many projects in the US and in Canada (Fatemi & McGinn, 2018).

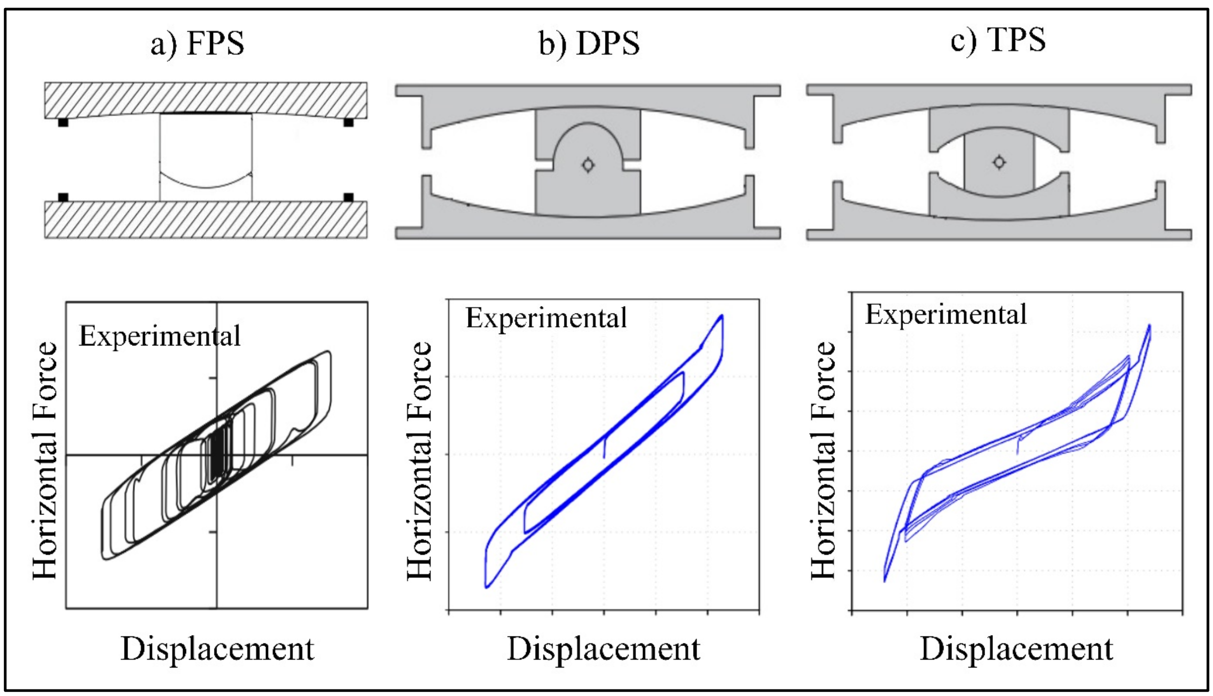

Figure 1.6 Friction pendulum systems and force-displacement typical behaviour
Taken from Cardone et al. (2015) and Fenz et al. (2007)

1.1.4 Modeling of isolation systems and their behaviour

The hysteresis behaviour of seismic isolation devices strongly depends on several parameters and different conditions such as materials, friction interface surfaces, aging, loading history, temperature conditions, interaction of mechanical properties, velocity, level of deformation, coupling between different directions of deformation, level of normal stress, and the effect of internal temperature resulting from heating caused by energy dissipation due to inelastic deformation or friction (Abe et al., 2004a; Buckle et al., 2006; Constantinou et al., 2007; Naeim & Kelly, 1999).

Researchers have introduced different hysteresis models with various degrees of complexity and accuracy, to represent the force-displacement relationship of isolation systems such as one dimensional linear model with equivalent stiffness and damping ratio, two-dimensional model which is theoretically derived based on the three-dimensional elastoplastic constitutive law, bilinear (elastoplastic) hysteresis model, etc. (Abe et al., 2004b; Buckle et al., 2006; Fragiacomo et al., 2003; Inaudi & Kelly, 1993; Naeim & Kelly, 1999). The proposed models and approaches are based on the required complexity of the analysis. Among different available models, the nonlinear models are the most complicated but the best and accurate models compared to the real behaviour of the isolation systems (Abe et al., 2004b; Kikuchi & Aiken, 1997).

However, such models are rarely used in practice due to the complexity of the calculations. Nevertheless, the bilinear model is the simplest nonlinear model and also the most widely used for the analysis of the isolated bridges, making it possible to capture the essential behaviour of the most common isolation systems. Additionally, the viscoelastic model, which is an equivalent linear model, is more commonly used in practice and design within a spectral multimodal analysis and is typically based on the bilinear model (Buckle et al., 2006; Gai et al., 2020; Guizani & Chaallal, 2011; Koval et al., 2016; Naeim & Kelly, 1999).

These two models, the bilinear model and the equivalent linear model, are also adopted as a basis for the design in CSA-S6, 2019, and implicitly recognized as being sufficiently reliable and accurate for seismic isolation systems (CSA, 2019). The bilinear force-displacement behaviour, as well as its main parameters and characteristics, representing typical isolation systems behaviours, are illustrated in Figure 1.7, along with the features of the equivalent linear model (K_{eff} and β_{eff}) (Buckle et al., 2006; CSA, 2019; Naeim & Kelly, 1999).

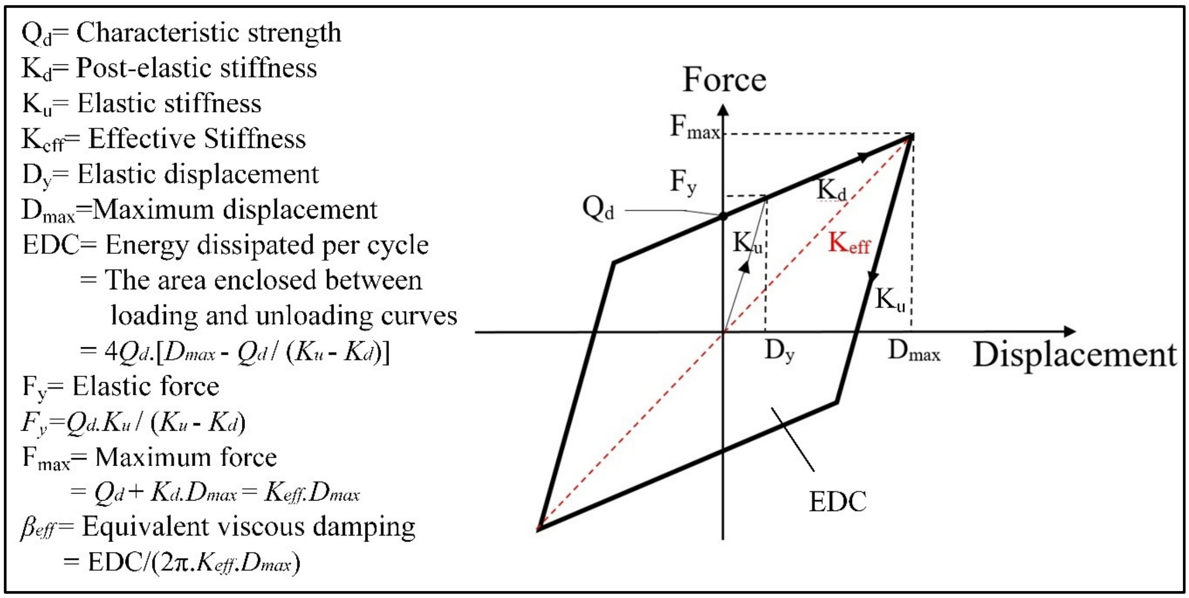

Figure 1.7 Bilinear hysteresis and its equivalent viscoelastic model parameters and features

The initial elastic stiffness (K_u), is typically a very high value, while the displacement at the yield point (D_y), is typically 0 to 1-2 millimeters, depending on the isolation system. The characteristic strength (Q_d), and the post-elastic stiffness (K_d) are the most important parameters directly affecting the performance of the isolation system and the performance of the structure under large earthquakes (Buckle et al., 2006; CSA, 2019; Dicleli & Buddaram, 2006; Naeim & Kelly, 1999). The equivalent linear viscoelastic model can be used for the purpose of elastic seismic analyses.

This model is a combination of an elastic spring and a viscous damper, working in parallel as shown in Figure 1.8. The model will be defined by the effective stiffness (K_{eff}) of the linear spring, and the equivalent viscous damping rate (β_{eff}) of the damper, calculated at the design displacement. The equivalent damping ratio is obtained by equating the energy dissipated per cycle (EDC) of the bilinear model to that of a viscous damper for the same deformation amplitude (Chopra, 1995; Nguyen & Guizani, 2022).

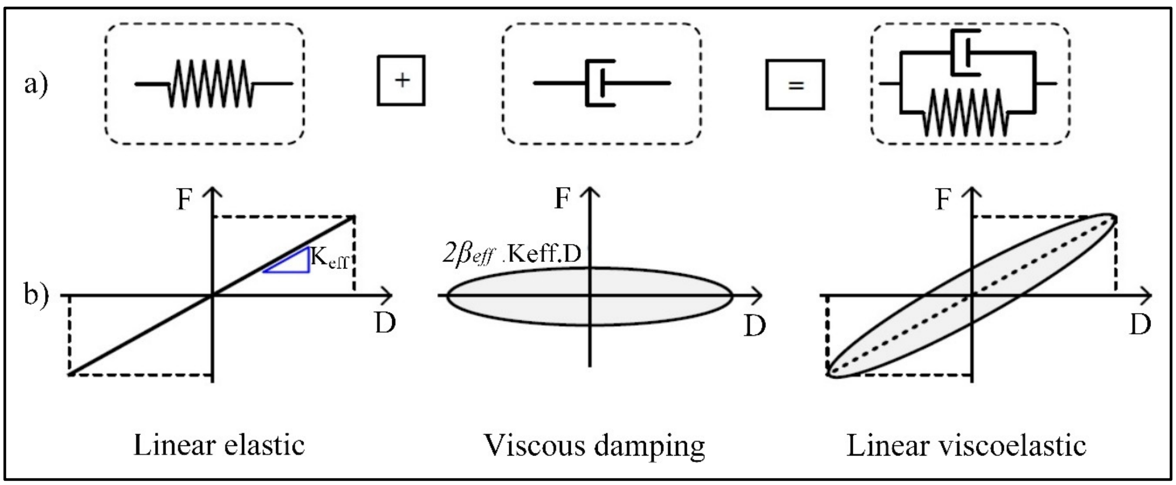

Figure 1.8 Linear viscoelastic model: (a) schematics; (b) component behaviour
Taken from Xuan Dai and Guizani (2022)

1.1.5 Seismic analysis and design of isolated bridges

The required degree of refinement of the model and analysis method, specified by the codes for seismic design purposes, depend on the importance of the bridge and target performance, its geometry and complexity, and the level of the seismic excitation (return period). Three methods of seismic analysis are commonly used for designing isolated bridges. These are Single Mode Spectral analysis (SMSA), Multi-Mode Spectral Analysis (MMSA), and Nonlinear Time History Analysis (NLTHA), etc. (AASHTO LRFD Bridge Design Specifications, 2017; Buckle et al., 2006; CSA, 2019).

Both SMSA and MMSA are spectral analysis methods, in which the isolation system is replaced by an equivalent linear model, and only the expected maximum response and seismic demand are estimated for design purpose. Under some conditions or for preliminary design purposes, these methods are acceptable in Canadian bridge code and the bilinear behaviour is generally used as a basis for calculating the effective parameters (CSA, 2019). The effective parameters such as K_{eff} and β_{eff} are evaluated at the design displacement which is an unknown parameter by itself. In addition, the calculated force for the design displacement shall be the same force obtained through the bilinear model as well as by the design spectrum for the effective period (T_{eff}) and the effective stiffness (K_{eff}) that are associated to the effective parameters of the isolation system. Therefore, an iterative procedure is used to estimate the seismic demand, including the design displacement and the effective parameters. Figure 1.9 shows the steps of the iteration process for SMSA method (Buckle et al., 2006; CSA, 2019). Usually, the design displacement is used as an unknown. In the case of the multimodal spectral method, the approach is similar but the model of the bridge is represented by multiple degrees of freedom, and the evaluation of the seismic response for each iteration is usually carried out within a specialized software.

A preliminary estimation of the displacement can be obtained through SMSA. At each iteration, the seismic displacement is estimated, the effective properties of the isolation units are calculated, and then the structure's model is modified to incorporate these properties. Subsequently, the calculation spectrum is adjusted (for the range of isolated periods with the equivalent viscous damping). The seismic response for the obtained model is evaluated through spectral analysis, and the convergence is checked. When the estimated displacements differ from the assumed displacements, the procedure is repeated with a new estimate of the seismic displacements until convergence. The nonlinear time history analysis method (NLTHA) allows to provide a complete history of the response of structural elements under seismic records. For the design, many records calibrated on the target spectrum are needed. The behaviour of isolation systems can be represented by the bilinear behaviour model or a more sophisticated model using advanced software programs such as Sap, Abaqus, OpenSees, etc. (ABAQUS, 2019; Mazzoni et al., 2006).

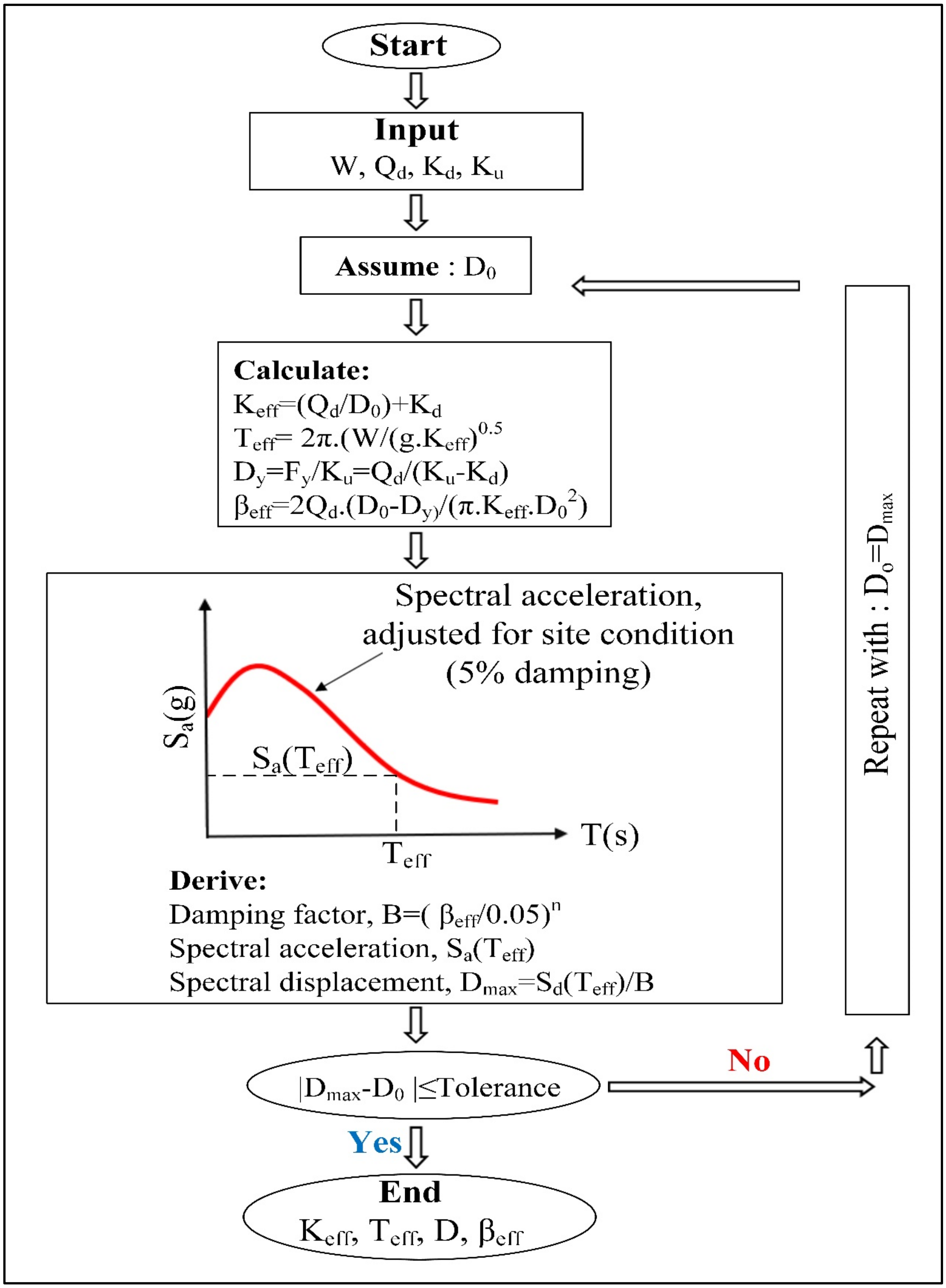

Figure 1.9 Seismic analysis scheme of the iterative process of SMSA for seismic isolated bridges with stiff substructures

This method provides the precise response of structures from the simplest to the most complex behaviour level of the isolation systems and it is considered as the most accurate method. It allows to validate the results of preliminary analyzes by SMSA or MMSA and to finalize the design. This method is fastidious, complex and time consuming in terms of computational effort and processing and is used only for research purposes or when required by the code, typically for important and/or complex/irregular bridges subjected to high level of seismic excitation (Buckle et al., 2006; CSA, 2019).

As mentioned earlier, regardless of the method and the choice of the systems, the whole purpose of the seismic isolation strategy is the reduction of the transmitted seismic loads to the structure by the period shift through providing more flexibility and the control of the seismic displacement by adding damping in the system. Numerous experimental and numerical research works on seismic responses of conventional and isolated structures have indicated that seismic isolation technology significantly improves the seismic performance of infrastructures (Cardone et al., 2022; De Luca & Guidi, 2019; Haque & Bhuiyan, 2013; Tsopelas et al., 1996).

The hysteretic properties of the isolation devices govern the seismic response and performance of isolated bridges. As different isolation devices are commercially available, their properties are affected by many factors, such as temperature, aging, velocity, and fabrication tolerances (Buckle et al., 2006; Nassar et al., 2022). Nowadays, advancing in laboratory equipment and technology leads to more accurate experimental tests and numerical simulations. Therefore, innovative methods and models using different and new materials and performances, such as unbonded fiber-reinforced elastomeric isolators, high-damping rubber bearings strengthened with glass fiber fabrics, quasi-zero stiffness isolation system, LDRB combined with steel hysteresis dampers, etc., have been introduced in recent years to provide new isolation systems or improve the existing isolation systems to offer better performance, more convenient and economical solution for a wide range of structures and ground motion excitations (Abe et al., 2004a; Nguyen & Guizani, 2021a; Domenico et al., 2023; Mordini & Strauss, 2008; Ye et al., 2020).

Other researchers carried out parametric studies to identify the optimal range of hysteretic properties of seismic isolation devices leading to better compromise between the reduction of seismic forces and increase of seismic displacement, depending on the characteristics of the area's seismic records such as high seismicity or medium seismicity areas (Castaldo & Ripani, 2016; Inaudi & Kelly, 1993; Jangid, 2005; Nguyen & Guizani, 2021b). All the research studies aim to provide more information and choices to make a safe and accurate design of the isolation systems and to take advantage of this technology in reducing the damage induced to the bridges in prone areas.

1.2 Dynamic Soil-Structure Interaction (SSI)

Traditional assessments of structural seismic reaction presume that a structure is rigidly supported (Carbonari et al., 2011; Dicleli et al., 2005; Tochaei et al., 2020). However, a structure embedded in the soil has a rather different dynamic response than a rigidly supported structure (Alam & Bhuiyan, 2013b; Carbonari et al., 2011; Dezi et al., 2012; Fraino, 2013; Gazetas & Mylonakis, 2001; Mylonakis & Gazetas, 2000; Tongaonkar & Jangid, 2003). Therefore, two major components are evaluated for examining the effect of soil conditions on the dynamic responses of bridges.

1. The response of the soil when seismic waves travel through the soil deposit,
2. The coupled foundation–superstructure response.

Foundation–superstructure responses are typically supposed to be a superposition of the foundation's response to the excitation in the absence of the superstructure, known as the kinematic response, and the effect of the additional flexibility caused by the foundation to the response of the superstructure, known as the inertial response. Finally, the soil response, also known as the free field response, is one of the most important aspects of earthquake engineering since it indicates the ground motion experienced at the top of the soil without a structure and which in turn is the input motion at the base of the structure.

The analysis of soil effects includes estimating the seismological properties of the region and accordingly modeling the soil profile and its dynamic characteristics. It also considers the various reflections and refractions at the interfaces between soil layers as seismic waves propagate through the soil deposits. When foundation soil is subjected to seismic ground motion, the intensity of the structural response is highly dependent on the nature of the ground motion and soil parameters. In fact, according to wave propagation theory, soil strata alter the characteristics of input seismic waves as they pass through them, thereby affecting free field data (Gazetas & Mylonakis, 2001; Wolf & Obernhuber, 1985; Wong & Trifunac, 1975).

According to research studies, SSI induces additional flexibility and damping into the bridge system. Figure 1.10 shows a simplified representation of the effect of SSI on the response of a base-isolated structure, according to a smooth design spectrum. As a general trend, it is expected that the period of the structure is further lengthened and displacement demand increased due to SSI. For long-period structures, the presence of a flexible foundation soil influences the nonlinear dynamic behaviour of the structure. Under strong shaking, the soil in the vicinity of the structure will have nonlinear behaviour with permanent deformations and will cause changes in the natural period compared to the fixed-base condition in the structural analysis. Such changes may cause problems, i.e. due to underestimation of displacement demand, if they are not taken into account properly during the design process (Ucak & Tsopelas, 2008).

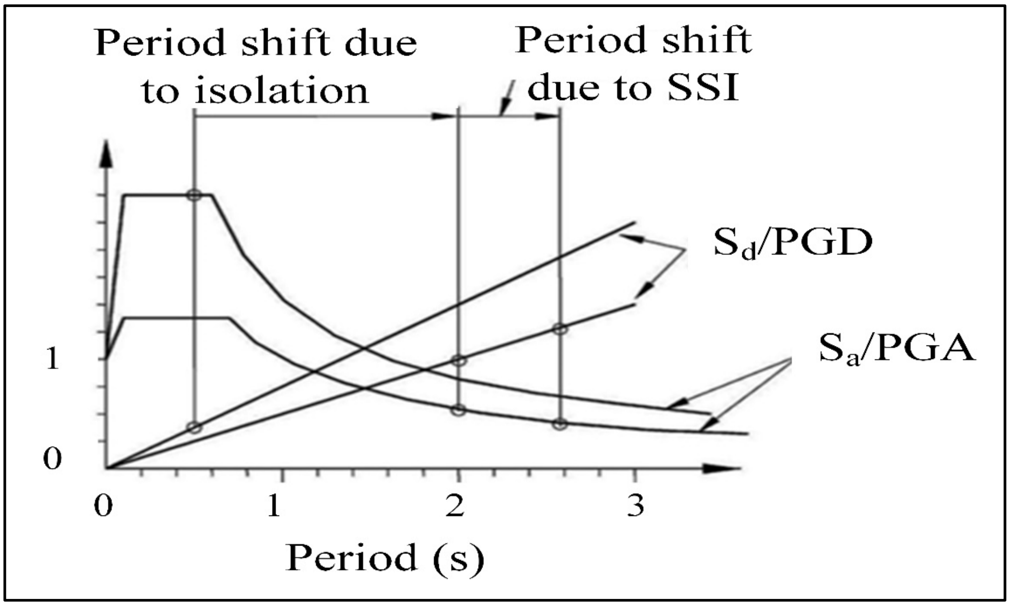

Figure 1.10 Effect of SSI on the response of seismically isolated structures
Taken from Ucak & Tsopelas (2008)

In the case of very stiff soil, the dynamic responses of the structure are much less affected by the soil's presence, and the structure can be considered fixed at its base. However, if the structure is resting on a flexible medium soil, the dynamic responses of the structure will be different from the case of the fixed base condition due to the interaction between the soil and the structure, as well as the frequency changes in earthquake records as they propagate through soil layers. Therefore, a complete dynamic analysis should be considered to study the effect of SSI in the model and to accurately evaluate the seismic response and/or the performance level of the structure (Gazetas & Mylonakis, 2001; Ucak & Tsopelas, 2008; Wolf & Obernhuber, 1985; Wong & Trifunac, 1975). Although special computer programs exist for modeling SSI, the validity of the results depends greatly on how accurately dynamic soil properties and the governing aspects are included, which is still a challenging task despite improvements in in-situ testing.

1.2.1 Inertial interaction

The inertial interaction part of the SSI phenomenon occurs when the inertial forces induced in the superstructure by the seismic excitation produce base shear and moments, at the ground level, which cause differences in the displacements of the foundation system compared to the surrounding soil (Alam & Bhuiyan, 2013b; Carbonari et al., 2011) (Fraino, 2013). Figure 1.11 shows a Single Degree of Freedom (SDOF) system subjected to earthquake excitation. The initial position is labeled as “0”, and the surrounding foundation soil is represented inside the rigid box. The figure at the right, represents the situation in which the earthquake excitation is acting in the first system (left side). The final position of the mass is labeled as “1”, and its total displacement deformation is the sum of the ground movement (Δ_G) and the displacement induced by structural deformations (Δ_S). In this case, there are no inertial SSI effects acting on the system (Avilés & Rocha, 1998; Betti, Abdel-Ghaffar, & Niazy, 1993; Fraino, 2013; Gazetas & Mylonakis, 2001).

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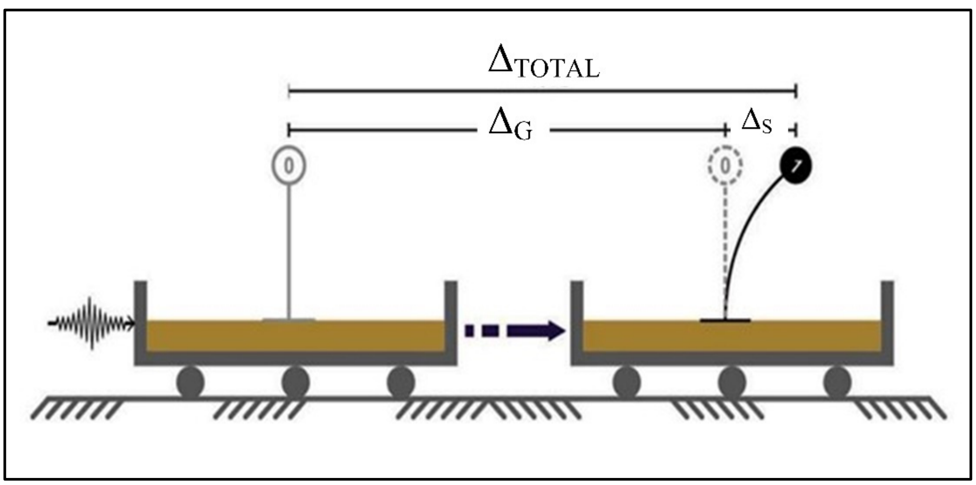

Figure 1.11 Displacements induced on a SDOF system under seismic excitation without inertial SSI effects
Taken from Fraino (2013)

The case of an SDOF system under seismic excitation which is affected by inertial SSI is shown in Figure 1.12. Deformed conditions due to the seismic excitation are shown on the right side labeled from 1 to 3.

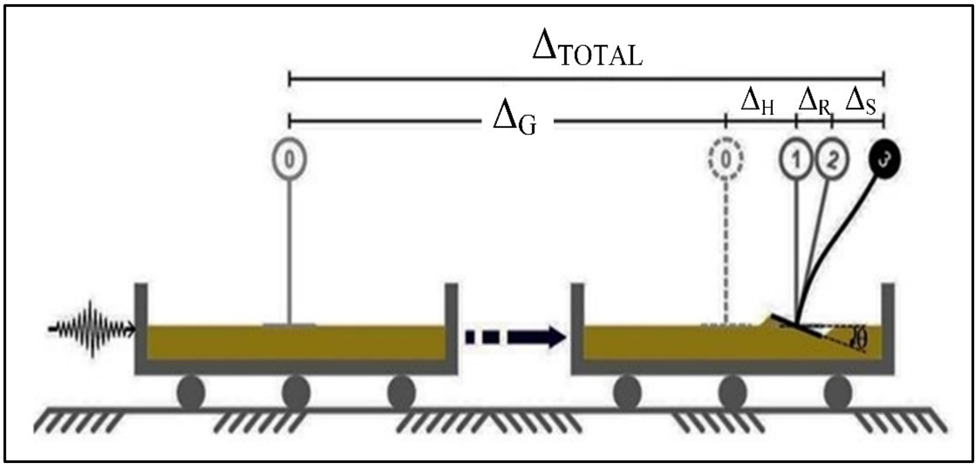

Figure 1.12 Displacements induced on a SDOF system under seismic excitation with inertial SSI effects
Taken from Fraino (2013)

The total displacement for this case is defined as the sum of the following components:

I. Ground movement (Δ_G),

II. Horizontal movement of the column base (Δ_H), leads to position 1,

III. Contribution from rocking at the base (Δ_R), leads to position 2. This component is caused by the rotation of the column base and transmitted to the structure,

IV. Contribution from swaying of the structure (Δ_S), leads to position 3. This component is induced by the structural deformations in the system.

This previously described mechanism of Figure 1.12, implies deformations in the foundation soil. The consequence of this from the structural point of view is to affect the level of fixity at the foundation, deviating from the condition of fixed-base systems (Avilés & Rocha, 1998; Betti et al., 1993; Fraino, 2013; Gazetas & Mylonakis, 2001).

1.2.2 Kinematic interaction

The difference in stiffness between the foundation and the soil causes the kinematic interaction. This phenomenon arises because the foundation system and surrounding soil are separate pieces and are not rigidly coupled and will not move together. This naturally produces differences between free-field and structural base motions (Alam & Bhuiyan, 2013b; Avilés & Rocha, 1998; Betti et al., 1993; Carbonari et al., 2011; Fraino, 2013; Gazetas & Mylonakis, 2001).

The possible mechanisms behind this effect are as follows:

I. Base-slab averaging: the waves from the free-field motion that enters the system are "averaged" within the footprint area of the base slab due to the kinematic constraint applied by the slab moving as a rigid body,

II. Embedment effect: reduction of seismic ground motions with depth for embedded foundations,

III. Scattering of seismic waves off of corners and asperities of the foundation.

The extent of the importance of inertial and kinematic interactions depends strongly on various parameters. For example, kinematic interaction is very predominant for foundation embedment depth. As the embedment depth increases, there is a greater decrease in the short period spectral response. This happens because the ground motion decrease with depth, which is a common feature of site response (Anand & Kumar, 2018; FEMA, 2020). Another example of the kinematic interaction importance is in soil-pile structure interaction. In this case, strong mobilized kinematic interaction effect causes significant bending moments when the ground is excited at or close to the natural frequency of the coupled soil-pile-structure system (Hussien et al., 2016). In contrast, in the case of the heavy structures such as dams and nuclear power plants, the inertial interaction becomes more pronounced (Anand & Kumar, 2018).

1.2.3 Dynamic behaviour of soils

Earthquakes make irregular cyclic soil loadings which result in various levels of stresses and strains. Soils exhibit nonlinear hysteretic behaviour when cyclically loaded in shear and the nonlinear behaviour depends on different factors such as amplitude of loading, number of loading cycles, soil characteristics and in situ confining pressure (Beresnev & Wen, 1996; Hashash & Park, 2001). For monotonically increasing loads, they exhibit nonlinear behaviour with gradual softening, which manifests as a decrease in shear modulus with increasing strain (Beresnev & Wen, 1996; Hardin & Drnevich, 1972; Hashash & Park, 2001).

Soil materials exhibit a diverse range of complex constitutive behaviours, making the development of numerical models quite challenging. Several models of varying complexity have been developed in earthquake engineering, which are categorized by as:

I. Equivalent-linear models,
II. Cyclic nonlinear models,
III. Advanced constitutive models.

Equivalent and cyclic nonlinear models are based only on the average shear modulus which is used for entire analyses. This approach is computationally effective and yields satisfactory outcomes particularly in the case of small strains (less than 1-2%) and moderate accelerations (less than 0.3-0.4 g). There are some drawbacks to employing the equivalent linear model. Due to its linearity, the model is unsuitable for calculating permanent displacements as the shear strain resets to zero after the loading concludes. Additionally, the inherent linearity of the soil might cause misleading resonances that wouldn't happen in the field.

Nonlinear models are assessed by directly numerically integrating the equation of motion in small time increments, such as employing the explicit finite difference technique. Nonlinear models typically have a primary or envelope curve and unloading, reloading and degradation rules. They have the capacity to consider the nonlinear characteristics of soil by incorporating diverse soil models that encompass features such as updated stress-strain relationships, pore-pressure generation, and cyclic modulus degradation. These features, which are not present in the equivalent linear model, enable a more accurate representation of soil behavior. Advanced constitutive model incorporates the two or three-dimensional nonlinear behaviour of the soil and are used for more comprehensive site response and SSI analyses (Beresnev & Wen, 1996; Gutierrez & Chopra, 1978; Hashash & Park, 2001; Iswanto & Yee, 2016; Kramer & Paulsen, 2004; Rajasekaran, 2009). A typical cyclic shear stress-strain relationship of the soil is shown in Figure 1.13.

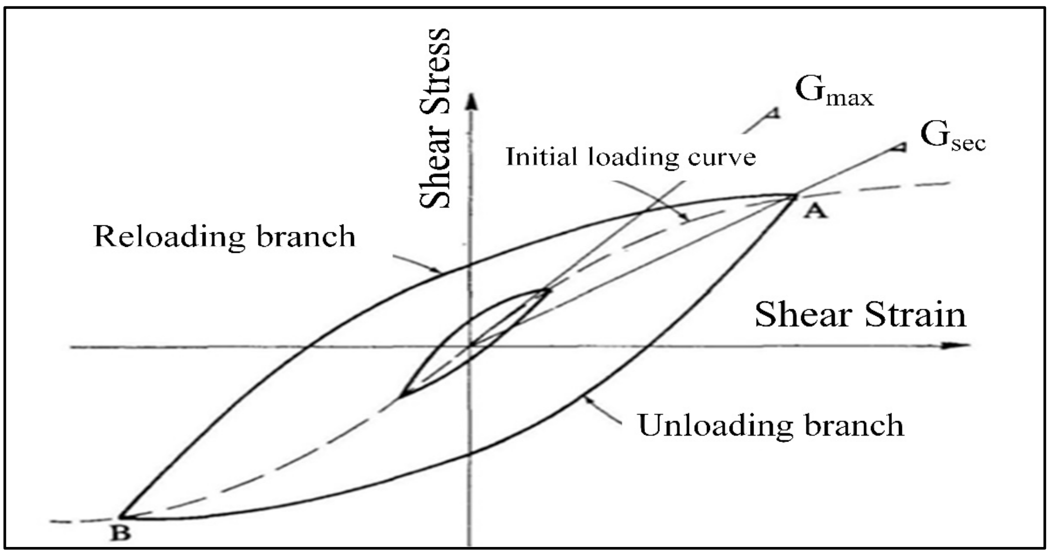

Figure 1.13 Typical stress-strain relationship of soil in cyclic shear deformation
Taken from Beresnev & Wen (1996)

As the earthquake waves propagate upwards in the soil profile, selected frequencies in the input motion could be amplified, de-amplified, changed in characteristics or modify differently depending on the records characteristics, structure's properties, and sub-soil behaviour.

The effects of local soil site conditions such as rock outcrop, stiff site conditions, soft to medium clay and sand, and deep cohesionless soils on the response spectra shapes for 5 percent damping are shown in Figure 1.14. Normalized spectral shapes were computed by dividing the spectral acceleration by the peak ground acceleration (PGA) at the surface. These spectral shapes were computed from motion records made on rock and soil sites at close distances to earthquakes with magnitude $6 \leq Mw \leq 7$ (Richter scale) (Dhakal et al., 2013; Kamatchi et al., 2013; Seed et al., 1976).

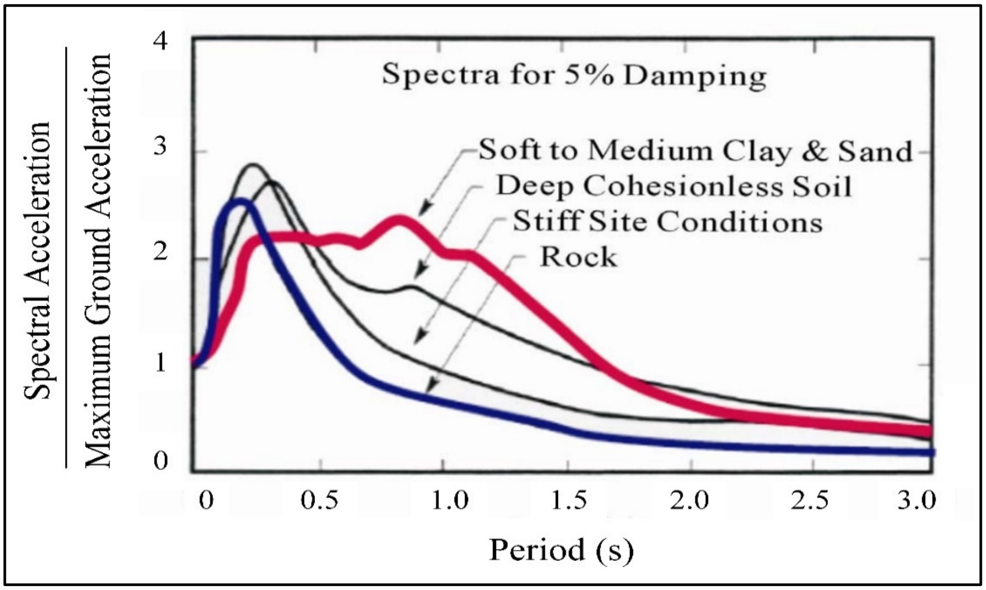

Figure 1.14 Soil site effects on average normalized response spectra
Taken from Seed et al. (1976)

Researchers have shown that the pattern of damage during past earthquakes was partly attributed to the resonance effect of time period of soil deposit and the time period of the structures. Initially, they noticed that soil amplification can be observed mostly for small amplitude shaking and that for strong shaking there may not be considerable amplification due to the damping effects of the soil (Kamatchi et al., 2013). Therefore, properly assessing the seismic hazard at a site requires a reasonable estimation of these amplifications and the related frequencies.

When earthquake waves move through the soil, certain frequencies in the input motion are amplified, causing new spike-like to appear in the surface acceleration response spectrum. For weak ground motions that cause small strains in soil (nearly linear site response), these frequencies are aligned with the resonant frequencies of the soil deposit. More intense ground motions will cause larger soil strains and probable nonlinear site effects. Nonlinear site effects can decrease the resonant frequencies of amplifications because of the softening of the soil, and in addition, they can decrease the magnitude of amplification or even cause de-amplification because of the hysteretic damping (Bolisetti, 2015).

1.2.4 Approaches to dynamic SSI

Analysis of SSI can be conducted by two general approaches explained as follows:

1.2.4.1 Direct SSI analysis approach (coupled system)

The direct approach stands out as the preferred method for conducting nonlinear analysis, especially when employing a detailed 3D model of both soil and structure, along with appropriate nonlinear constitutive relations for the soil and/or the structure. In the direct method, the entire soil-structure system is modeled as a unified unit, offering the most comprehensive approach to capturing SSI effects compared to alternative methods like the substructure method (Asli et al., 2019; Pitilakis et al.,2006).

The model encompasses a restricted section of the soil domain, incorporating the foundation system, superstructure, transmitting boundaries of the soil domain, and interface elements between the foundation system and the soil. Therefore, the direct solution involves considering the entire soil-structure system and solving the complete model in a single step. This approach automatically incorporates both kinematic and inertial interaction effects into the numerical model. For this method, it is necessary to evaluate the input ground motion at the base of the numerical model. The input ground motion is typically obtained using deconvolution or using rock outcrop motions available.

The approach offers advantages in terms of improved accuracy and realism in soil modeling, particularly in capturing nonlinear behaviour for more precise responses in SSI. However, this comes at the cost of significant computational demands and time-intensive processes. The drawbacks include the necessity for substantial computational resources and the time-consuming nature of the method, leading to its limited practical implementation. The schematic figure of direct approach is shown in Figure 1.15. Coupled soil and structure models using robust constitutive soil formulations have the potential to overcome the deficiencies of the substructure method, presented below (Pitilakis et al., 2006; Stewart et al., 2012; Tileylioglu, 2008).

Accurately defining the soil properties, material nonlinearities, and careful treatment of interface and boundary conditions in order to compute the coupled responses are the most important factors. This complex and time-consuming method can be performed in commercial finite element and finite difference programs such as Abaqus, ANSYS, LS-DYNA, FLAC, or the open-source finite element program, OpenSees, among others. The Equations of Motion (EOM) for an SSI finite element model subject to a uniform base acceleration, $\ddot{u_g}$, can be written as:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [k]\{u\} = -[M]\{\ddot{u_g}\} \qquad (1-1)$$

Where [M], [C] and [k] are respectively mass, damping and stiffness matrices, u is the relative displacement to the base of the model vector, $\{\dot{u}\}$. and $\{\ddot{u}\}$ are the first and second derivatives of displacement $\{u\}$ relative to time (relative velocity and acceleration) corresponding to the degrees of freedom of the model, including soil and the structure (Dehghanpoor et al., 2019; Stewart et al., 2012; Stewart et al., 1999; Tileylioglu, 2008).

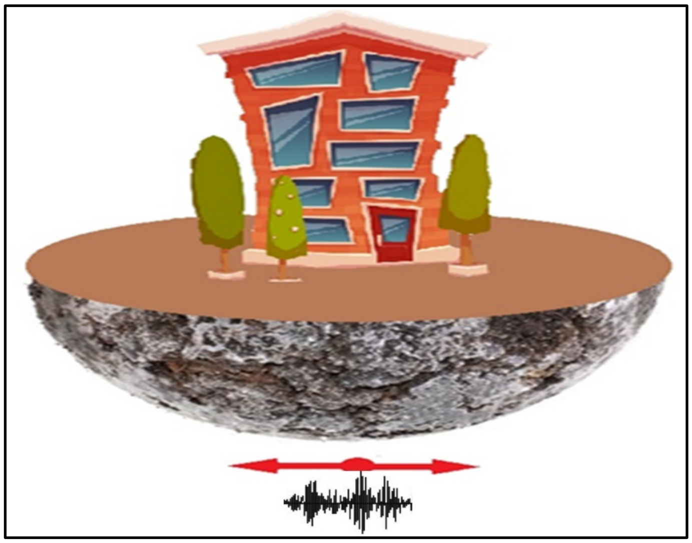

Figure 1.15 Direct approach of SSI analysis

As all finite element models must be restricted by a series of finite boundaries, in the direct approach, simulating infinite domains is another challenge in performing nonlinear SSI analysis. For example, the infinite soil domain must be defined by modeling a finite domain in the analysis so that waves do not reflect and scatter into the soil domain from its lateral boundaries. To achieve that, either a very large soil domain should be modeled with sufficient plan dimensions to dissipate the radiating waves before they reach the lateral boundaries, or artificial and absorbing boundaries must be defined to represent a non-reflecting boundary condition, ensuring no spurious reflections are caused by the finite domain.

Absorbing boundary models have been developed and implemented in commercial finite element programs such as viscous and infinite boundaries (Beer & Watson, 1989; Liu & Jerry, 2003; Wang et al., 2013).

1.2.4.2 Indirect analysis approach (substructure system)

In this approach, kinematic and inertial responses are calculated independently. The substructure is connected by applying equal and opposite interaction forces to each substructure model. The responses of all the substructures will be super-positioned for the total response of the system (Ostadan & Ghiocel, 2006; Stewart et al., 2012; Stewart et al., 1999; Tileylioglu, 2008). The schematic figure of indirect approach is shown in Figure 1.16.

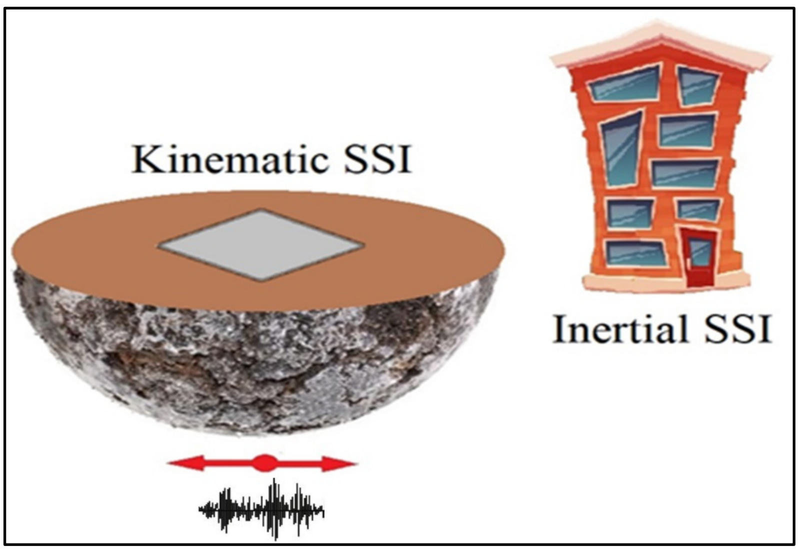

Figure 1.16 Indirect (substructure) approach of SSI analysis

The substructure approach incorporates either springs or springs and dashpots working together to represent the flexibility and damping of the soil.

The requirements to take into account the substructure approach are as follows:

i) Evaluate of free field motions with soil material properties,
ii) Convert free field motions to foundation input motion,
iii) Use of springs and dashpots to represent stiffness and damping of soil,
iv) Assess the response with coupled springs and dashpots and structure system.

The springs and dashpots in the substructure approach represent the frequency-dependent stiffness and damping characteristics of the soil-foundation interaction. To compute the elastic solution for the spring's stiffness and damping, different factors including the geometry of the foundation, structure natural period, and the characteristics of the soil are considered (Stewart et al., 2012; Stewart et al., 1999; Tileylioglu, 2008).

1.2.5 Site effects and recent changes in the Canadian codes

In Canadian Highway Bridge Design Code in the 2006 revision (CSA-S6:06), the effect of soil was considered only by introducing a multiplying factor of the design spectrum without the effect of the specified period of the structure or the seismicity of the region (CSA, 2006). This factor was considered as neutral on rocks and getting amplified on softer soils, as it is shown in Table 1.1. To reflect the lack of knowledge on site effects on seismically isolated bridges, this revision recommended conservative site effects which are about 25-35% higher than those for conventionally designed bridges (CSA, 2006). In the previous revision of Canadian Highway Bridge Design Codes (CSA-S6:14), variable site coefficient factors F(T) were introduced, considering different seismicity areas and also based on different periods of the bridge. The site coefficients are the same for isolated and non isolated bridges.

Table 1.1 Site classifications for seismic site response
Taken from CSA (2006)

CSA-S6-06				
Soil type	**Site coefficient for non isolated design**	**Site coefficient for seismic isolation design**	**Soil profile**	**Shear wave velocity (m/s)**
I.	1	1	Rock, stiff soil conditions where the soil depth is less than 60 m	>750
II.	1.2	1.5	Stiff clay or deep cohesionless soils where the soil depth exceeds 60 m	
III.	1.5	2	Soft to medium-stiff clays and sands, characterized by 9 m or more	
IV.	2	2.7	Soft clays or silts greater than 12 m in depth	< 150

In this revision, soil C is considered as the reference soil with the F(T)=1 for all the scenarios and classification of soil groups is refined, relatively to the precedent edition, and is shown in Table 1.2. More classes are defined and shear wave velocity is taken as the average value in the top 30 m, known as V_{s30}.

Table 1.2 Site classifications for seismic site response
Taken from CSA (2014)

<table>
<tr><th rowspan="2">Site class</th><th rowspan="2">Ground profile name</th><th colspan="3">Average properties in top 30 m</th></tr>
<tr><th>Shear wave average velocity, $\bar{V}_s$ (m/s)</th><th>Standard penetration resistance, $\bar{N}_{60}$</th><th>Soil undrained shear strength, s_u</th></tr>
<tr><td>A</td><td>Hard rock</td><td>$\bar{V}_s > 1500$</td><td>Not applicable</td><td>Not applicable</td></tr>
<tr><td>B</td><td>Rock</td><td>$760 < \bar{V}_s \leq 1500$</td><td>Not applicable</td><td>Not applicable</td></tr>
<tr><td>C</td><td>Very dense soil and soft rock</td><td>$360 < \bar{V}_s \leq 760$</td><td>$\bar{N}_{60} > 50$</td><td>$s_u > 100\ kpa$</td></tr>
<tr><td>D</td><td>Stiff soil</td><td>$180 < \bar{V}_s \leq 360$</td><td>$15 \leq \bar{N}_{60} \leq 50$</td><td>$50 < s_u \leq 100\ kpa$</td></tr>
<tr><td rowspan="2">E</td><td rowspan="2">Soft soil</td><td>$\bar{V}_s < 180$</td><td>$\bar{N}_{60} \leq 15$</td><td>$s_u < 50\ kpa$</td></tr>
<tr><td colspan="3">Any profile with more than 3 m of soil with the following characteristics:
- Plastic index PI>20;
- Moisture content W≥40%; and
- Undrained shear strength s_u<25 kpa</td></tr>
<tr><td>F</td><td>Other soil</td><td colspan="3">Site-specific evaluation required</td></tr>
</table>

Site coefficient for all the soils stiffer than soil C has a value less than unity, meaning the de-amplifying of the responses while softer soils are associated to values equal or more than unity. F(T) is changing based on the reference PGA (PGA_{ref}) of the site, which reflects the intensity of the excitation, and also the period of the bridge. Considering a period of T=0.5 s and T=2 s, to represent the conventional and flexible or isolated bridges, Table 1.3 shows the F(T) for three PGA_{ref}. For soils stiffer than soil type C, F(T=2) values are lower than those of F(T=0.5). In contrast, for soils softer than soil type C, F(T=2) has higher values than F(T=0.5), showing the stronger negative effect of softer soils in flexible or isolated bridges.

For soils stiffer than soil type C, F(T=0.5) is slightly increasing with the increase in PGA_{ref} while F(T=2) remains constant for all PGAs. In contrast, for soils softer than soil type C, F(T) is decreasing from lower PGAs to higher PGAs for both F(T=0.5) and F(T=2) (CSA, 2014). This reflects the observation that the stronger is the earthquake record, the less is the amplification (of accelerations) due to soft soil presence for short and long period structures, owing to the nonlinear behaviour of soft soil (softening under high deformation amplitude).

Table 1.3 Comparison of F(T) for different soils and PGA_{ref} in CSA-S6:14

Soil type	**F(T=0.5)**		
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.3$	$PGA_{ref} \geq 0.5$
A	0.46	0.48	0.49
B	0.58	0.6	0.61
C	1	1	1
D	1.47	1.2	1.1
E	2.47	1.48	1.17
Soil type	**F(T=2)**		
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.3$	$PGA_{ref} \geq 0.5$
A	0.4	0.4	0.4
B	0.52	0.52	0.52
C	1	1	1
D	1.57	1.36	1.27
E	2.9	1.92	1.58

In the current revision of the Canadian Highway Bridge Design Code (CSA-S6:19), there have been a few adjustments in the F(T) values specifically for soils stiffer than soil type C (CSA, 2019). As it is shown in Table 1.4, in comparison to CSA-S6-14, the F(T=0.5) values for soil A have increased by 24%, 19%, and 16% respectively for $PGA_{ref} \leq 0.1$, $PGA_{ref} = 0.3$, and $PGA_{ref} \geq 0.5$. For soil B, the increase in percentage is comparatively lower, with 12%, 8%, and 6% for the same respective PGA_{ref} values (CSA, 2014). The increase in F(T=2) is nearly twice as much and remains constant for various PGA_{ref} values.

In comparison to CSA-S6-14, the F(T=2) value for soil A has experienced a 45% increase, while for soil B, it has risen by 21%, and it remains consistent across different PGA_{ref} levels. In fact the de-amplifying effect of soil A and B is observed to be less pronounced in the current revision (CSA, 2019).

Table 1.4 Comparision of F(T) for different soils and PGA_{ref} in CSA-S6:19

Soil type	**F (0.5)**		
	PGAref ≤ 0.1	PGAref = 0.3	PGAref ≥ 0.5
A	0.57	0.57	0.57
B	0.65	0.65	0.65
C	1	1	1
D	1.47	1.2	1.1
E	2.47	1.48	1.17
Soil type	**F (2)**		
	PGAref ≤ 0.1	PGAref = 0.3	PGAref ≥ 0.5
A	0.58	0.58	0.58
B	0.63	0.63	0.63
C	1	1	1
D	1.57	1.36	1.27
E	2.9	1.92	1.58

In the 6th Generation of seismic hazard Canada, more precise data is available for any location in Canada identified by its latitude, longitude, and site designation. Additionally, it offers seismic hazard values at various probabilities and time intervals. Moreover, there has been a change in the approach used to determine seismic hazard values for different site designations. Previously, under the NBC 2015, the seismic hazard values were computed for a reference site class C, and then adjusted for other site designations using a site coefficient. However, with the NBC 2020, the seismic hazard values for each site designation are directly calculated based on the average velocity of shear waves at the site, allowing for direct derivation of values for different time intervals and probabilities (NRCAN, 2022).

1.3 Effects of earthquake characteristics

After the 1994 Northridge and the 1995 Kobe earthquakes, structural engineers showed increasing attention to investigating correlations between structural damage and impulsive characteristics of earthquakes. Records from earthquakes show that NF records differ from FF records in that they are generally characterized by long-duration pulses that subject the structure to very high input energy at the early stage of the record (Hall et al., 1995; Liao et al., 2004; Makris & Roussos, 1998; Somerville et al., 1997). Because many NF ground motions have been recorded mostly in recent years, the importance of earthquakes with high-velocity pulses on structures is still an ongoing research subject (Choi et al., 2010; Galal & Naimi, 2008; Jia et al., 2023; Jiang et al., 2020; Jiao et al., 2021; Mangalathu et al., 2019).

Long-period impulsive earthquake ground motions impose large strength and ductility demands, especially on intermediate and long-period structures, such as high-rise buildings and isolated structures. Several important factors have been pointed out in previous studies, and all of them have shown that NF records cause severe damage to structures. They can cause high values of story drift in structural members, forcing the structure to behave in the inelastic range and leading to severe damage (Attalla et al., 1998; Chouw & Hao, 2005; Ismail et al., 2014; Malhotra, 1999a; Neethu & Das, 2019; Shen et al., 2004).

In NF zones, the propagation of rupture towards a site at a velocity close to the shear wave velocity causes most of the seismic energy from the rupture process to arrive in a single large pulse of motion. These pulses occur at the beginning of the record and represent the cumulative effect of the seismic radiation from the fault. Sites in the opposite direction of fault rupture, or backward sites, have lower amplitude pulses of longer duration than the fault-normal component of the ground motion in the rupture propagation direction. These features are shown in Figure 1.17, which compares the velocity time series and displacement spectra (ξ=5%) for two horizontal components of ground motions at the Imperial Valley earthquake.

The two components are perpendicular and parallel to the strike, defined here as fault-normal (FN) and fault-parallel (FP), respectively. Spectral ordinates of the NF motion are larger at periods longer than 2s, and larger displacement demands can be expected on long period structures compared to FF records. The effects of rupture directivity are obvious, and the FN component of a "forward rupture directivity" record usually has a longer period and larger amplitude than its FP counterpart (Archila, 2014; Somerville et al., 1997).

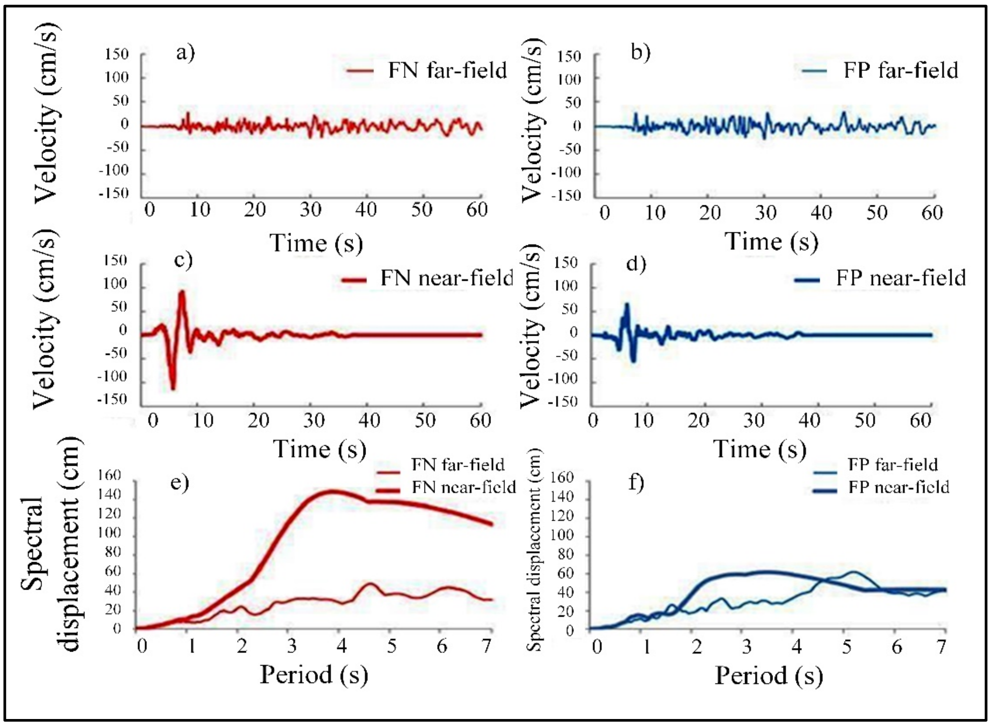

Figure 1.17 NF and FF records for fault normal and fault parallel directions from Imperial Valley earthquake (1979)
Taken from Archila (2014)

The complex nature of NF ground motions is one of the reasons why the effects of the earthquake ground motions on structures located at close distances of the causative fault, especially on complex structures, still need to be fully understood.

Dicleli and Buddaram (2006) studied the maximum seismic responses of the isolated bridges based on the characteristics of the earthquakes with various PGA/PGV ratios (5.5 1/s - 21.5

1/s), and various PGV^2/PGA ratios. The PGV^2/PGA ratio is related to the intensity of the acceleration pulses in a ground motion and therefore represents the damage potential of the earthquake (Dicleli & Buddaram, 2006). The seismic displacement demand of isolated bridges showed a decreasing trend with increasing of PGA/PGV or decreasing PGV^2/PGA ratios. Moreover, with high ratios of PGA/PGV>12 (1/s), which represents high frequencies in the earthquake records, the characteristic strength (Q_d) of the isolation system has a minor effect on the responses of maximum displacements.

On the other hand, for earthquakes with low PGA/PGV ratios (less than 10.6 1/s), characteristic strength (Q_d) has a large effect on controlling the maximum displacement demand and higher values of characteristic strengths are recommended to dissipate seismic energy and maintain the displacement within reasonable limits. In addition, for earthquakes with low ratios of PGA/PGV, the post-elastic stiffness (K_d) has a considerable effect on the responses of maximum displacement and maximum force (from 1.5 to 4 times greater than those with ratios high PGA/PGV) (Dicleli & Buddaram, 2006).

The same conclusion was resulted from Choun, Park et Choi (2014) research work showing that for earthquakes with low PGA/PGV ratios (PGA/PGV< 8 1/s), the increase in seismic responses is greatly amplified and the base shear responses can be 6 times more than the earthquakes with high PGA/PGV ratios (PGA/PGV> 12 1/s). For earthquakes with high ratios of PGA/PGV, the properties of the isolator have a minor effect on the response of the isolated structures (Castaldo & Tubaldi, 2018).

More recently, Guizani and Nguyen (2021) conducted a study on optimal seismic isolation characteristics for bridges in areas with moderate and high seismic activity, and they obtained similar findings. High values of the characteristic strength ($0.04 \leq Q_d/W \leq 0.12$) are beneficial to control displacement for seismic records with low PGA/PGV values, associated to high seismic areas such as West coast of North America and South East of Europe while intermediate and low values of the characteristic strength ($0.015 \leq Q_d/W \leq 0.05$) are better suited for moderate seismic areas, associated to high PGA/PGV ratios, such as East coast of

North America and North West of Europe. High values of Q_d (damping) in moderate seismic areas (high PGA/PGV) are rather harmful as they do not noticeably reduce the displacement but increase the force demand (Nguyen & Guizani, 2021b).

The results from many studies indicate that NF seismic ground motions are frequently characterized by intense velocity and displacement pulses of relatively long period, higher input energy and lower PGA/PGV ratio than FF ground motions, where most of the records are bounded between PGA/PGV ratios of 7–12 1/s, regardless of the earthquake magnitudes and focal distance as it is shown in Figure 1.18. This ratio, referred to in some papers as PGV/PGA, is larger than 0.1 seconds and influences their response characteristics, so that by increasing the PGV/PGA ratio of the ground motions, the NF effect is more obvious (Galal & Ghobarah, 2006; Hatzigeorgiou, 2010; Kitada, Umeki et al., 2004; Liao et al., 2004; Makris & Roussos, 1998; Shen et al., 2004; Somerville et al., 1997).

According to a study by W.I. Lia et al. (2006), when a bridge structure is subjected to NF earthquake ground motions, the amount of ductility demand is higher, and the induced base shear is about 1.5 to 2.0 times larger than that from FF input ground motions. Then, the effect of NF earthquakes should be carefully considered for prone areas (Liao et al., 2000).

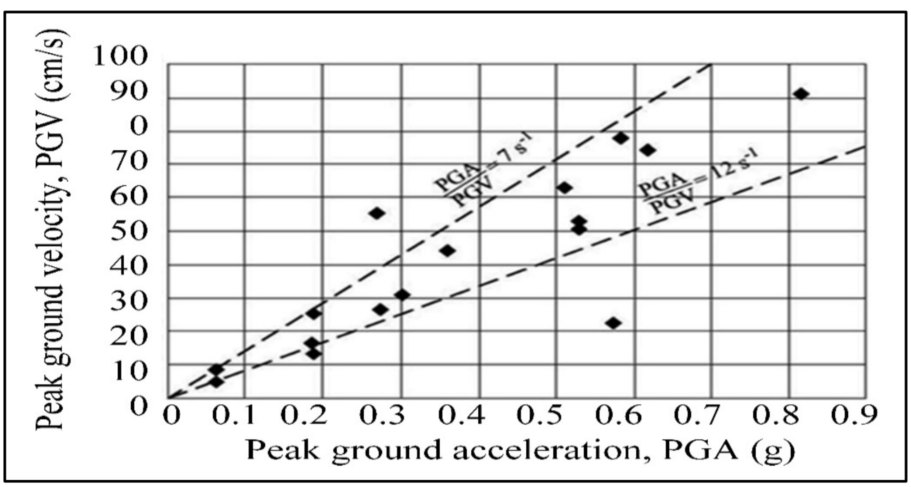

Figure 1.18 Relationship of PGV vs. PGA for NF records on stiff soil
Taken from Kitada, Umeki, & Takashema (2004)

Records of two real NF pulse like records, one synthetic NF pulse like record and one NF record without pules are shown in Figure 1.19. The comparison of the responses shows that peak values of ground acceleration, velocity and displacement are the controlling parameters in the response characteristics of NF pulse-like ground motions. Additionally, pulse-like ground motions with low ratios of PGA/PGV have wide acceleration-sensitive region in their elastic response spectrum and it causes higher base shear and inter-storey drifts and ductility demand as well as reduces the effectiveness of supplemental damping in high-rise buildings and isolated structures (Liao et al., 2004; Malhotra, 1999b; Shen et al., 2004).

Studies have shown that the ratio of the pulse period (T_p) to the fundamental period of the structure (T_1) have a significant effect on structural seismic performance (Shen et al., 2004). The curvature ductility demand of concrete columns has shown different trends of ductility demands with the increase of T_1/T_p for various heights of the columns. As Figure 1.20 shows, the seismic demand increases with the increase of T_1/T_p when its value is relatively low. On the contrary, a downward trend is observed when the value of T_1/T_p is relatively high depending on the heights of the columns (Shen et al., 2004; Zhong et al., 2022).

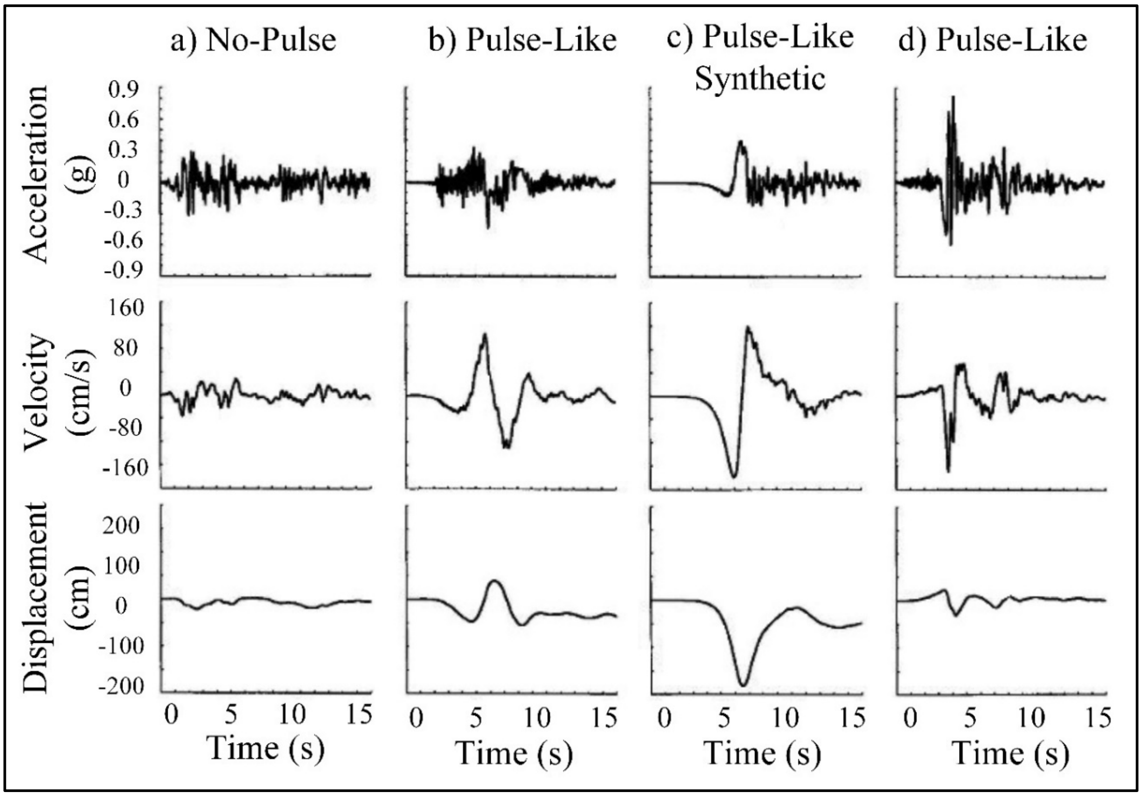

Figure 1.19 Acceleration, velocity and displacement records of different earthquakes

Taken from Malhotra (1999)

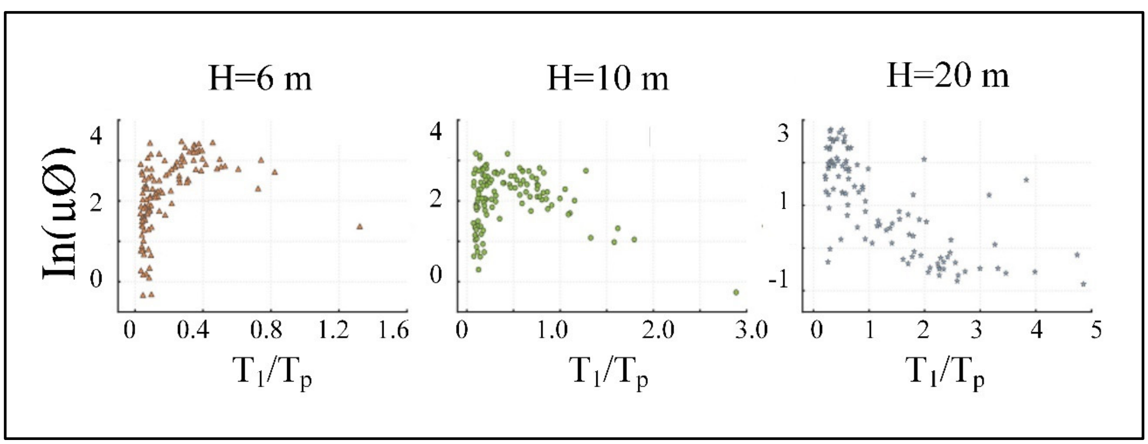

Figure 1.20 Structural responses of different columns under pulse-like ground motions

Taken from Zhong, Yang, & Wang (2022)

The effect of NF ground motions with pulses in their velocity records on the Bai-Ho Bridge as a typical seismically isolated bridge showed that the NF effect amplifies the seismic responses of the isolated bridge when the pulse period is close to the effective period of the isolation system, and the ratio of dissipated energy by the LRBs to the total input energy is influenced by the NF effect (Shen et al., 2004).

Isolated bridges are impacted by two aspects of NF ground motion records. First, as explained, the ground motion normal to the fault trace is richer in long-period spectral components than that parallel to the fault. The second aspect of NF ground motion that strongly impacts seismic isolation systems is the presence of long duration pulses. The ground motions may have one or more displacement pulses, with peak velocities of the order of 0.5 m/sec and durations in the range of 1-3 sec. In forward directivity records with low ratios of PGA/PGV <10 1/s, the ground motion is considered as directivity affected. Therefore, pulses are expected to exist in the time history of earthquakes, and they tend to result in higher displacement and shear force responses compared to ground motions with higher PGA/PGV ratios. The presence of pulses may be especially problematic for isolated structures in terms of the displacement demand (Bray & Marek, 2004; Jangid & Kelly, 2001; Jia et al., 2023; Malhotra, 1999a) and the isolation system might not be effective in the amount of energy dissipation in the first part of the pulse (Dicleli, 2007; Jónsson et al., 2010)..

Studies on both rubber and sliding based isolation systems showed that the responses of seismically isolated bridges (SIBs) are highly related to the PGA/PGV ratios of the ground motions. Thus, the choice of seismic ground motion with regard to the characteristics of the bridge site is crucial for the correct design of the SIBs. It is also found that the characteristic strength of the isolators may be chosen based on the intensity and frequency characteristics of the ground motions. Furthermore, the isolator post-elastic stiffness is found to have a notable effect on the response of SIBs (Dicleli & Buddaram, 2006).

The responses of isolated bridges equipped with FPSs showed significant displacement in the FPS for low friction coefficient values under NF motions and the increase in the friction coefficient can reduce the bearing displacement significantly without altering the superstructure accelerations. Also, a particular value of the friction coefficient leads to an optimum design for which the pier base shear and deck acceleration attain a minimum value for a predetermined seismic hazard and level (Jangid, 2005). However, a more resilient design is using adaptive behaviour of DFPs or TFPs to design the optimized isolation system for different amplitudes of displacement as well as different performance objectives which can be designed to have high stiffness and low damping at lower displacements (Becker et al., 2017; Calvi & Calvi, 2018; Fenz & Constantinou, 2008; Morgan & Mahin, 2010).

In addition, although seismic isolation can effectively reduce the base shear and acceleration responses of structures subjected to FF records without a notable increasing in the displacement responses even in long periods, some studies show that isolation systems with intermediate periods are more effective for NF earthquakes. In fact, when the initial period of the isolated structure is relatively long (e.g., greater than 2.5 s), NF excitations can impose significantly large displacement demands on the isolation systems and superstructure. For NF excitations, a range of initial period (1.5–2.5 s) and lateral yield-strength (10–15%) of the seismically effective weight) of the isolation system parameters has been recommended in the literature as a good compromise between cutting the force demand and controlling the displacement demand. Such a range could noticeably reduce enough the deck accelerations and force demands while the displacement demands remain within an acceptable range used in the practice (Anajafi et al., 2020; Haque & Bhuiyan, 2013; Haque et al., 2010).

Large isolator displacements either can be accommodated by using large isolators incorporating higher damping features or using other supplementary energy dissipation and/or recentring devices like passive dampers (fluid viscous dampers, supplemental clutching inerter dampers, Tuned inerter dampers, shape memory alloy (SMA)-spring dampers, etc.) to control the displacement demand and reduce the base shear as explained (Cao & Yi, 2021; Jangid, 2022; Talyan et al., 2021; Wen & Hui, 2022). These dampers mostly are installed in the longitudinal direction of the bridge between the piers (bent caps) and the superstructure to control the large superstructure displacement. A detailed SMA-spring damper installation is illustrated in Figure 1.21.

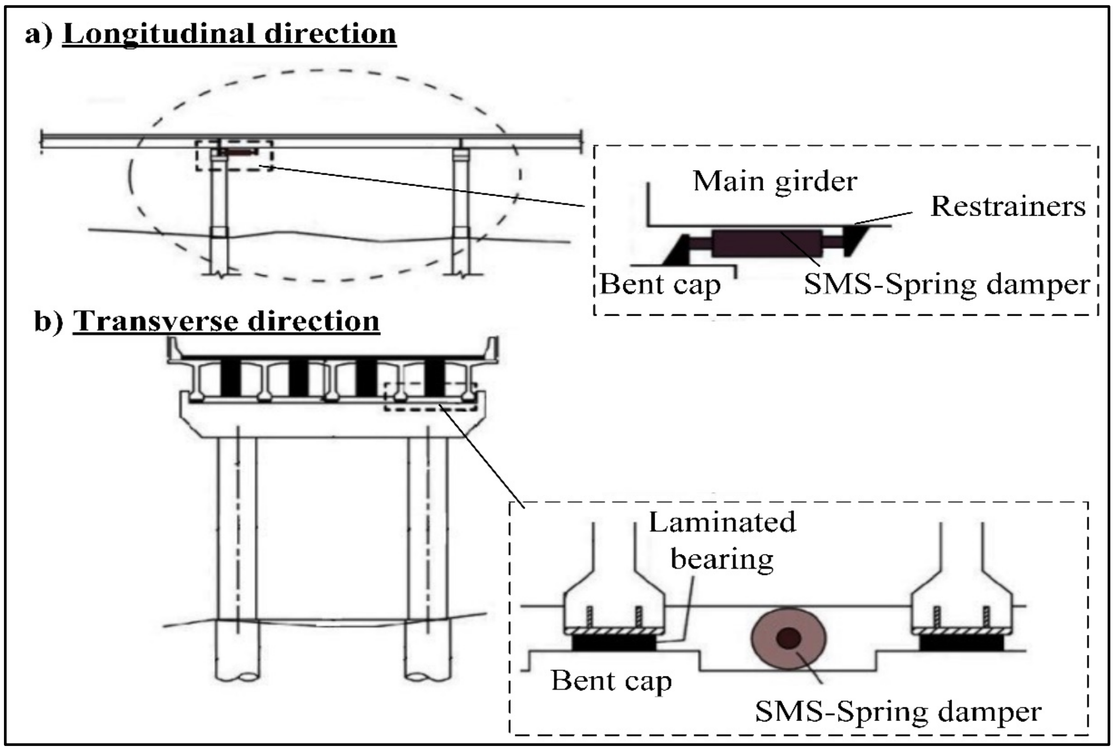

Figure 1.21 Location of the isolation system and dampers in an isolated bridge,
Taken from Sasa Cao and Jiang Yi (2021)

However, these damping mechanisms are non-linear and become less effective at large displacements. For example, if a level of 20 percent damping is required at a specified displacement, the level of damping at smaller displacements becomes extremely high. Therefore, the isolation system will not be effective if the structure experiences a small or moderate earthquake. As a result, the benefits expected from isolation, namely, the reduction of accelerations and inter-story drift to protect sensitive internal equipment and non-structural elements in structures, may not occur (Jangid & Kelly, 2001).

1.3.1 Effects of earthquake characteristics on seismic isolation performance and recent changes in seismic hazard in Canada

In Canada, seismic isolation strategy was introduced in the 2000 edition of the Canadian Highway Bridge Design Code (CSA-S6, 2000). For the period between 2000 and 2015, the design of the isolation systems was generally calculated according to the specifications of the 2000 and 2006 editions of this code (CSA-S6, 2000; 2006), where the design spectra were developed from accelerograms taken from the west coast of North America, similar to those of the AAHSTO applied in the United States. These spectra did not adequately reflect the seismic hazard and level of the seismicity in Eastern Canada (CSA, 2006; Guizani, 2003; Koval et al., 2016; Mitchell et al., 2003). The current version of the Canadian Highway Bridge Design Code (CSA-S6, 2019) is aligned with the latest seismic hazard data in Canada for eastern and western parts with different seismicity categories (CSA, 2019; NRCAN, 2022).

In general, the seismic performances of isolated bridges are considerably affected by the different characteristics of the earthquake ground motion records such as their frequency contents and their spectral properties. Several studies have been devoted to this effect and as it was explained in detail in previous sections, the PGA/PGV ratio is strongly associated with the frequency content of the earthquake and strongly impacts the seismic response and performance of isolated bridges (Castaldo & Tubaldi, 2018; Choun et al., 2014; Dicleli & Buddaram, 2006; Saritas & Hasgur, 2014; Yang et al., 2019; Yang et al., 2017)

In addition, the shape of the acceleration spectrum is representative of the variation of the seismic energy concentration in the frequency domain. In fact, the higher values of the spectral acceleration contain the higher energy for a given range of frequencies. The variation of the spectral values with the extension of the period can be considered as an important indicative parameter for estimating the seismic performance of flexible or isolated bridges. From this point of view, two locations in Canada, Vancouver as a representative of high seismicity areas located in the West, and Montreal as a representative of medium seismicity areas, located in eastern Canada, illustrated in Figure 1.22, shows the effect of the spectral acceleration based on the seismicity zone and the frequency contents of the earthquakes, which leads to a significant impact on the isolated bridges. In fact, while the CSA-S6-06 code adopts the same spectral acceleration for the two cities of Montreal and Vancouver, the CSA-S6-19 defines very distinct spectral acceleration based on their different seismicity areas. This difference, is particularly important for the design of isolated bridges in terms of the spectral values for the long periods of vibration versus those specified for the short periods.

Figure 1.22 shows that very strong similarity between the CSA-S6-06 and CSAS6-19 for Vancouver is maintained, in accordance with the previous discussions. For Montreal, the spectral accelerations decrease rapidly with the lengthening of the period, which indicates that the seismic isolation would be even more effective. On the other hand, for Vancouver the values of the spectral acceleration decrease more slowly with the lengthening of the period and where these values remain quite high at relatively long periods (2s to 3s), the isolation approach seismic is complicated and more efficient systems would require higher isolation periods and be required to dissipate the induced seismic energy into the bridge in the range of long periods.

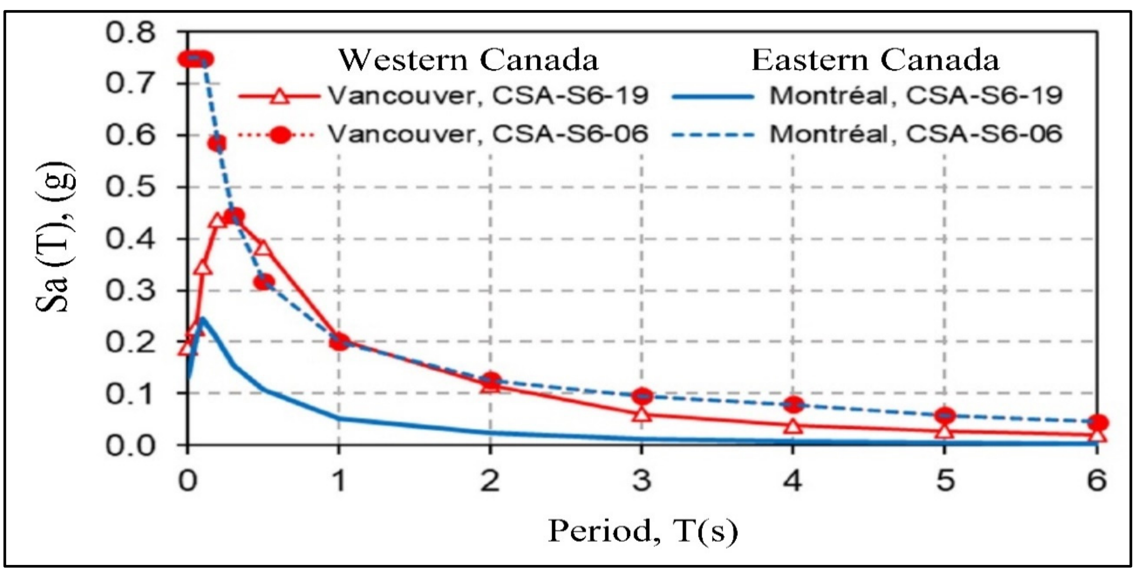

Figure 1.22 Comparison of spectral acceleration in CSA-S6-06 and CSA-S6-19, with a probability of exceedance of 10% in 50 years
Taken from Xuan Dai and Guizani (2022)

1.4 Simultaneous effects of SSI and earthquake characteristics on bridges

The challenge among researchers is a better understanding of structural manners in a critical situation like an earthquake time under different circumstances. Experimental and analytical methods and simulation software programs have been suggested and applied with the aim of helping to create more accurate and reliable design and construction methods. Recent detailed studies show that a safe and economical seismic design of a bridge structure depends strongly on the level of seismic excitation and the influence of supporting soil on the structural dynamic response. Based on the literature, long-span bridges are more susceptible to a relatively more severe SSI effect during earthquakes than regular structures (Alam & Bhuiyan, 2013a).

The seismic responses of four real isolated bridges supported on groups of piles considering soil and SSI effects were investigated by Chaudhary, Abe, and Fujino (2001). To simplify the complication of the model, they considered equivalent linear behaviour for the isolation system, piers, and soil-pile system. Simplified models of one pier and isolation system were used to study the seismic responses of the bridges subjected to different real earthquake ground motions captured at the area with various PGAs. The SSI effect was applied using the substructure method and considering the fact that soil strains should be less than 0.03% in all cases of earthquakes to avoid significant non-linearity in the soil behaviour. Their results showed that SSI is more highly related to the stiffness of the isolated bridges than soil shear modulus, and ignoring the effect of SSI will cause errors in the analysis results of the bridge (Chaudhary et al., 2001).

A theoretical study to investigate the effect of SSI on the dynamic performance of a seismically short-span isolated bridge was carried out by A.G Vlassis and C.C. Spyrakos (2001). The bridge piers were supported on a spread footing and a shallow soil layer subjected to horizontal moderate to strong ground motions. The shallow layer of soil was modeled as a system of linear springs and dashpots acting in the horizontal and rotational directions, which created a four-degree-of-freedom system at the base of the bridge. All the bridge components were modeled as linear elastic behaviour, and an equivalent linear system characterized the isolation system. Their results showed that SSI caused a lengthening of the fundamental period of the bridge system and a reduction of the base shear, especially in stiff soil conditions. Including SSI in seismically isolated bridge design increases the safety level of the bridge and reduces the design cost. Additionally, they concluded that because of the presence of the isolation system, which reduces considerably the stiffness of the system, in terms of dampening, SSI does not appear to play a significant effect on the isolated structures (Vlassis & Spyrakos, 2001).

The effect of soil structure interaction on peak responses of a three-span continuous deck bridge seismically isolated by the elastomeric bearings consisting of alternate layers of rubber and steel plates was assessed by NP. Tongaonkar and RS. Jangid (2003). SSI was included using equivalent linear springs and dashpots model with frequency-independent coefficients. Linear elastic behaviour for the piers and the isolation system was considered, and a time–history analysis using the complex modal analysis was carried out. The response quantities of interest were the deck acceleration, base shear in the piers, and the relative displacement of the isolation devices at the abutment and pier locations. Three types of soil (Hard, Medium, and Soft) under three real earthquakes (El-Centro, Northridge, and Kobe) were considered. Both longitudinal and transverse directions of the bridge were analyzed simultaneously. They concluded that SSI would result in the enhancement of safety and a reduction in design costs. In addition, when the soil stiffness is more than 10 times the isolation system stiffness, the SSI may be ignored in the design of the isolated bridges. In other cases, SSI influences the bearing displacement at the abutment, and ignoring these effects will underestimate the design displacement. A sizeable reduction in the pier base shear with SSI effects has been observed. They reported that the flexibility of the surrounding soil tends to reduce the earthquake forces induced in the isolated bridges. In the end, they showed that the effect of SSI are found to be more pronounced for stiff bridges and stiff isolation in comparison to flexible bridges and isolation system, and the variation in the damping in the isolation system does not have noticeable effects on the response of seismically isolated bridge with SSI effects (Tongaonkar & Jangid, 2003).

From the past seismic performance of bridges, piers have been the most vulnerable component during seismic excitations. Muhammad Tariq Amin Chaudhary (2004) investigated the stiffness degradation in reinforced concrete piers in isolated bridges and the SSI effect. Eighteen real earthquakes and four existing isolated bridges in Japan were studied. Bridges were equipped with various isolation systems such as laminated rubber bearings, lead rubber bearings, high-damping rubber bearings, and Teflon sliding bearings. All bridges were isolated in the longitudinal bridge direction, and side stoppers were installed to prevent movements in the transverse direction. In order to obtain soil shear wave velocities, dynamic soil shear modulus, and soil damping ratios, one-dimensional site response analysis was used by SHAKE software. In contrast, the frequency dependence of dynamic foundation stiffness was ignored. The influence of peak ground acceleration, dynamic soil shear strain, soil shear modulus, reinforced concrete piers stiffness, and wave parameters on SSI were investigated. Results revealed that pier stiffness does not degrade before the column's drift reaches 0.015%. In addition, a substantial reduction is observed for moderate drift levels (0.12%) during the relatively stronger seismic excitations. SSI effect identified in these bridges is independent of free field acceleration and weakly dependent on dynamic soil properties. This apparent contradiction with the popular belief of strong SSI in weaker soil is prompted to consider that similar pier stiffness degradation also occurs with increasing seismic intensity. The ratio of pier and foundation stiffness should be examined to determine the influence of SSI. A relatively strong relationship between these variables supports the hypothesis that SSI is more strongly related to the stiffness ratio of pier and foundation than dynamic soil properties (Chaudhary, 2004).

The effects of SSI and non-uniform ground motions on pounding between two adjacent bridge frames were investigated by Nawawi Chouw and Hong Hao (2005). The pounding behaviour of girders is analyzed using a combined boundary element and finite element method. They modeled the bridge as a two-dimensional structure and studied responses in the longitudinal direction. To study the effect of the characteristic of NF earthquakes on the pounding behaviour, they selected simulated ground motions on soft soils and medium soils matched to the Japanese design spectra. The soil behaviour was considered elastic with the shear wave velocity of 100 and 200 m/s for soft and medium soil. They concluded that neglecting the SSI effects results in inaccurate predictions of pounding responses of bridges. In soft soils, SSI causes a larger required gap to avoid pounding. In addition, neglecting the ground motion spatial variations will underestimate the bridge damage potential due to poundings or separation between the adjacent bridge girders (Chouw & Hao, 2005).

The effect of ground motion spatial variation, SSI, and site condition on the seismic pounding of a typical two-span bridge structure was investigated by Kaiming Bi, Hong Hao, and Nawawi Chouw (2011). The soil surrounding the pile foundation was modelled as linear frequency-dependent horizontal and rotational spring and dashpot systems, and the soil–abutment interaction was not considered in the study. Four site conditions, base rock, firm soil, medium soil, and soft soil, were selected, and the structure was assumed to be rigid in the vertical direction in the analysis. All the foundations were assumed rigidly fixed to the ground surface, while the pier was founded on a rigid cap supported by pile groups. Results showed a significant influence of SSI on the separation distances and a need for larger separation distances considering SSI is proved especially on softer soils. Additionally, when the structure resonates with the local site, the SSI effect is most evident (Bi et al., 2011).

The effect of SSI on multi-span seismically isolated bridges was studied by Alper Ucak and Panos Tsopelas (2008). For this reason, two isolated bridge systems with periods of 2 s and 4.5 s, representing short stiff highway overpasses and tall flexible multi-span highway bridges, were considered. A bilinear hysteretic model for the isolation system and linear frequency-dependent springs and dashpots for the foundation system was used. At the same time, the behaviour of the pier was assumed to be linear. In addition, nonlinear time history analyses were employed with two sets of seismic motions, one containing 20 FF records and one with 20 NF records. Their results showed that for most of the record ground motions of both NF and FF, isolation system drift increases due to SSI. Therefore, SSI effects are significant and must be considered during the design and analysis of seismic isolated bridge systems. They also reported that SSI could be either beneficial or detrimental depending on the details of the individual motions and their relation to the dynamic properties of the pier and foundation.

Another observation was related to the tall flexible bridge when it showed more isolation drift under the FF records, while in contrast, the effect for NF motions was rather minor. The modeling approach of the foundation system with the SSI effect considerably influences the base shear force at the pier of the tall flexible bridge for both FF and NF sets of motions. In contrast, the short, stiff bridge responded differently to both categories of ground motions. Considering the frequency dependence of foundation impedances but neglecting rocking damping resulted in conservative designs for the base shear force. In contrast, considering the frequency dependence of the foundation spring and overestimating rocking damping leads to non-conservative estimates of the base shear forces at the pier (Ucak & Tsopelas, 2008).

The effect of local geologic and soil conditions on the intensity of earthquake ground motions was investigated by MHT. Rayhani, MH. El Naggar, and SH. Tabatabaei (2008). In order to evaluate the effect of soil nonlinearity on the ground response, a series of nonlinear site responses were analyzed and compared with the equivalent linear approach. The nonlinear behaviour of the soil was applied by the elastoplastic constitutive model approach using a one-dimensional profile in the FLAC program (Itasca, 2019). The strongest NF earthquake record captured from Bam Earthquake on the Iranian strong motion network was used in their study. It was reported that the local soil, known as a thick alluvium deposit, amplified the ground motion and significantly damaged residential structures. The comparison of results indicated similar response spectra of the motions for both equivalent and nonlinear analyses, showing peaks in the period range of 0.3–1.5 s.

Also, the ground motion record’s shear waves were amplified during wave propagation in soil layers from the base to the surface. The amplification levels of nonlinear analysis were less than the equivalent linear method, especially over long periods and the observed response spectra were above the National Earthquake Hazards Reduction Program requirements, especially at high frequencies (Rayhani et al., 2008).

Using a new and simplified procedure for the rapid assessment of the effectiveness of seismic isolation devices was conducted by Edward H. Stehmeyer and Dimitris C. Rizos (2008) to study the SSI effects. They investigated the importance of SSI phenomena on the response of seismically isolated bridges and compared the results with fixed-base bridges. They used coupled boundary element method (BEM) and the Finite element method (FEM) to assess the seismic behaviour of one pier of the bridge supported on a spread footing. Eight different soil properties (from soft to stiff) with linear behaviour were used. A simple numerical model of the bridge and the surrounding soil was formulated. An equivalent linear stiffness and damping model for the isolation system represents a high-damping rubber bearing. The seismic responses were compared with the fixed base bridge subjected to a real earthquake ground motion. They concluded that SSI would elongate the damped period of vibration, and the relative pier displacement would be increased. Also, they observed that the composite damping ratio for non-isolated bridge increases with SSI while this parameter decreases for isolated bridges. The important fact is that all these identified effects are amplified for softer soils. In the end, the effects of considering SSI in the analysis of seismically isolated structures are shown to reduce the effectiveness of the isolation system (Stehmeyer & Rizos, 2008).

The effect of SSI on the seismically isolated cable-stayed bridge response was studied by B.B.Soneji and R.S. Jangid (2008). The bridge was isolated by using high-damping rubber bearings. Pier consisted of two H-shaped concrete towers supported on rigidity-capped vertical pile groups passing through moderately deep, layered soil overlying rigid bedrock. The soil–pile interaction was idealized as a beam on a nonlinear Winkler foundation using continuously distributed linear and nonlinear springs and viscous dashpots placed in parallel.

The response of the superstructure was investigated under three different types of soil surrounding the pile foundation, namely soft, medium, and firm, and the bridge was subjected to a series of NF records. They concluded that bearing displacement may be underestimated for soft soils if SSI is ignored, especially in the longitudinal direction. The base shear response in the transverse direction will be affected by SSI, especially in soft to medium soils, and for accurate prediction of the dynamic behaviour of the soil-pile system, more accurate nonlinear soil modeling needs to be used (Soneji & Jangid, 2008).

The simultaneous effect of NF earthquake ground motions and SSI on the dynamic responses of cable-stayed bridges was investigated by Emad Norouzzadeh Tochaei, Todd Taylor, and Farhad Ansari (2020). The Twin River bridge in Chongqing, as a typical cable-stayed bridge with a laboratory scale of 1/60, was designed and fabricated for the investigation. The results were verified with the 3D numerical model of the bridge. Three different types of real records from the Chi-Chi earthquake, NF with a pulse, NF with non-pulse, and FF, were employed in the experiments and the numerical analysis of the bridge. A box-spring system with interchangeable springs was defined for the laboratory simulation of the soil-structure interaction for the bridge towers. The stiffness of the soil-foundation system and the surrounding soils were modeled by springs, while the damping effect was neglected. Due to the limitations of the experimental setup and to simplify the soil-foundation system, only the horizontal stiffness of the soil-foundation system was considered in the present study. The properties of springs were determined based on a series of systematic lumped-parameter models that were developed without involving the mass of the surrounding soil of the foundation.

The bridge towers, cables, and deck remained elastic for the range of seismic motions considered in this study. Therefore, the computations only involved elastic analysis. Results from the numerical and experimental investigations indicated that the type of input ground motions influenced the effects of foundation soil stiffness on the response of the bridge.

Neglecting soil-structure interaction can lead to an underestimation of the displacement response at the deck and towers, particularly when dealing with softer soil and pulse-type NF ground motions. Moreover, pulse-type motions cause the highest force demand on the bridge cables. Additionally, when considering the effects of soil-structure interaction (SSI), softer soils increase the tension force in the cables under the same seismic motions. Pulse-type NF motions generate the highest bending moment at the base of the tower across all soil-foundation stiffnesses. Furthermore, it has been reported that the tower base shear decreases with decreasing soil stiffness (Tochaei et al., 2020).

Research on a real three-span isolated bridge supported on piles and equipped with high-damping rubber bearings to study the effect of SSI and site amplification was conducted by Francesca Dezi, SandroCarbonari, AlessandroTombari, and Graziano Leoni (2012). Using the substructure method, the kinematic interaction analyses were performed using the 3D finite element model in the frequency domain. For the superstructure, the inertial interaction analysis was conducted in the time domain by adopting Lumped Parameter models to account for the compliance of each soil-foundation system. The results of each case study were compared with the conventional fixed base models. For this purpose, five bridge locations were considered, characterized by five different soil stratigraphy obtained from real laboratory tests and categorized within the soil type C. Ten real earthquake ground motions were selected from the European Strong Motion Database. Suitable shear modulus degradation and damping ratio curves were used to obtain the convenient equivalent linear system properties (damping and stiffness) of the soil. The substructure method was carried out under the assumption of linear behaviour for both soil and structure to evaluate the dynamic responses of the bridge subjected to real ground motions with different PGAs. Seismic responses showed isolated bridges are less sensitive to SSI effects than conventional bridges. In the case of the isolated bridge, SSI increases the base shear, while in the conventional bridge, various results of increase or decrease were observed from one record to the other. In addition, SSI slightly increases the maximum relative displacements of the isolation system (Dezi et al., 2012).

The effects of SSI and liquefaction on the fragility of a three-span continuous steel bridge were studied by Zhenghua Wang, Leonardo Dueñas Osorio, and Jamie E Padgett (2014). The bridge was equipped with LRBs between eight girders, piers, and abutments. Bilinear isolation bearings adapted from the AASHTO 2010 were used, and the bridge was assumed to be on stiff, soft, and liquefiable soils. Bridge and soil layers were modeled by Opensees software. Sixty of the 240 synthetic ground motions introduced by Fernandez and Rix (2006) for seven cities within the Upper Mississippi Embayment with hazard levels of 10, 5, and 2% probabilities of exceedance in 50 years were selected. Nonlinear time history analyses were carried out to derive key component fragility curves of bridges on different soil sites. The fragility analyses revealed that the failure probability of the isolated system is less than that of the non-isolated bridge for both stiff and soft soils. In contrast, SSI tends to decrease the effectiveness of the isolation system. In addition, even though liquefaction provides effective natural isolation by reducing the curvature demands on the piers, it increases the isolation bearing displacement and pile curvature (Wang et al., 2014).

A full finite element 3D model of a historical masonry bridge to study the SSI effect using NLTHA, and the direct method approach was employed by Hamza Güllü and Handren Salih Jaf (2016). The adjustment of the support motion due to the stiffness of the foundation is accounted for by the inertial interaction, and the kinematics interaction considers the modification of the free-field ground motions. The SSI effect was applied with the combination of kinematics and inertial interactions regarding the nonlinear behaviour of soil and structure. Solid elements for the bridge and soil were used, and the results of the analyses were compared with the fixed-base bridge. NLTHA responses showed that the influence of SSI becomes relatively prominent on the acceleration, displacement, rotation, modal shape, base shear, and overturning moment responses, and considering the soil effects is a crucial matter. The changes in the responses between the fixed-based SSI solution could be interpreted basically due to the soil characteristics (nonlinearity and softness) (Güllü & Jaf, 2016).

An original benchmark two-span bridge equipped with two different isolation devices including the SSI was studied by Forcellini (2017). The direct method was conducted by modeling the soil and bridge with OpenSees software. The combined effects of soil and isolator non-linearities have been investigated to assess the best isolated configuration able to efficiently accommodate the different non-linear conditions of the soil. The evaluation of the isolation technique involves examining two representative models: elastomeric bearings and frictional/sliding bearings. Using the OpenSees software, a study on soil deformability was conducted to determine the conditions that necessitate the consideration of SSI. Results proved that linear modeling of isolators and the soil leads to incorrect evaluation of the structural behaviour and the study assessed the benefit of nonlinear isolators in protecting structural elements (Forcellini, 2017).

One recent study about the SSI effect on typical multi-span continuous isolated bridges was conducted by B. Neethu and Diptesh Das (2018). Laminated and lead rubber bearings with non-linear behaviour and real NF and FF earthquake ground motions with a magnitude of more than 6 Mw and various PGAs were used. A simplified model of the deck-pier system with the elastic behaviour, attached together with the isolator, were employed. The numerical simulation, computations, and modeling of the bridge, isolation system, and soil–structure interaction was studied using MATLAB and Simulink programs. Linear springs and dashpots with frequency-independent coefficients for modeling the soil were used, and the effect of the nonlinearity of the soil and the effect of kinematic interaction was not considered.

Results showed that considering the effect of SSI reduces the efficiency of the isolator in controlling displacement and pier shear responses compared to the fixed base condition. The isolator's efficiency in terms of percentage reduction of seismic responses decreases, and the control value is lower when SSI is taken into account. This is due to the detrimental effect of combining isolation and SSI, which increases the structural flexibility. Considering SSI leads to an increase in both isolator force and displacements. Additionally, residual displacement occurs in soft soil conditions due to soil deformation, which intensifies with softer soil. Based on the findings, the authors noted that SSI has a tendency to decrease the structural demand by reducing its overall stiffness. Conversely, the foundation's ability to rotate and translate contributes to an overall increase in displacement, potentially leading to higher pier shear values during most earthquakes (Neethu & Das, 2018).

The effect of combined vertical and horizontal components of ground motions on three different bridge configurations constructed on a 4 and 6 m layers of soft and medium clay, which are overlaid on 14m Fontainebleau sand, was studied by Ahmad Dehghanpoor, David Thambiratnam, Ertugrul Taciroglu, and Tommy Chan (2019). Using a soil plasticity model to include SSI and conducting NLTHA showed that seismic earth pressure, side friction acting on the pile-cap, the superstructure's fundamental period, the ground motion's intensity, and the ground surface's natural period have a considerable effect on pile-cap displacement. Additionally, the beneficial role of elastomeric bearings in reducing the axial loads, bending moments under vertical components, and soil-pile-structure-interaction (SPSI) effects were observed (Dehghanpoor et al., 2019).

Yazhou Xie and Reginald DesRoches (2019) examined how changes in SSI modeling parameters affect the seismic demands and fragility estimates of a typical highway bridge in California. The study focused on assessing the sensitivity of these parameters and their impact on the bridge's response to seismic forces. They developed a comprehensive modeling approach based on rigorous p-y methodology. By utilizing 18 random variables that encompassed various soil zones, they established a range of possible variations in SSI modeling parameters, ensuring comprehensive coverage of different scenarios and providing a realistic representation of SSI effects. They employed bilinear behaviour for the bearings and utilized a nonlinear curve for the p-y springs and dashpots. The consistent findings from both regression analyses suggest that modeling parameters related to near-ground soils have a significant impact on the demand models of bridges. The research findings indicate that the performance and fragility curves of bridge columns and decks are primarily influenced by the uncertainty associated with ground motions. However, the propagation of potentially variable SSI parameters significantly affects the estimation of fragility for bridge foundations and abutment components, including span unseating, bearings, and shear keys (Xie & DesRoches, 2019).

1.5 Research gaps in the literature

Despite all the crucial findings and recommendations in research studies, several issues related to SSI effects, seismically isolated bridges, and earthquake characteristics have yet to be thoroughly investigated. The reason could be the extensive scope of the study containing many details, variables, and uncertainties in infrastructures, soil properties and seismic records. Therefore, each researcher has tried to cover a related part to their interests and specialties; still, many details have remained uncovered.

Understanding more accurate responses of bridges, considering site effects in NF and FF earthquake zones, is a crucial task in order to make informed decisions regarding the seismic design of bridges as the fundamental keys for the transportation system. The following gaps in the current state of knowledge were found:

Many research works have been conducted mostly by modeling soil through linear springs and dashpots, and few studies have tried to cover the soil's nonlinear behaviour, especially in the direct method. As a result, contradictory results have been reported due to the SSI effect, some showing the beneficial aspect of SSI, while others indicating its detrimental and unfavourable aspects with the results in lower or higher seismic responses based on different factors such as structural elements, soil stratum properties, and ground motion characteristics.

Aside from various conclusions drawn from the literature, most of these studies have focused on either ground motion characteristics or soil properties with or without isolation systems, with little attention paid to the combined effects of both factors on isolated bridges for prone areas. As a result, as presented in the precedent chapter, this research aims to look into the effects of multiple record characteristics, such as NF and FF, their frequency content and presence or not of pluses, as well as the effect of different soil types on a bridge with and without seismic base-isolation system.

CHAPTER 2

ON THE INFLUENCE OF EARTHQUAKES AND SOILS CHARACTERISTICS ON SEISMIC RESPONSE AND PERFORMANCE OF ISOLATED BRIDGES

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2.1 Abstract

Seismic isolation technology is an effective means of reducing seismic risk and enhancing the structural seismic performance. However, some parameters, such as earthquake inputs and soil characteristics, affect and mitigate the efficiency of this technology. The main purpose of this study is to investigate the simultaneous effects of different records and flexible soils on isolated bridges. To this end, an isolated bridge is assumed to be at different distances from the ruptured fault (Rrup) in order to represent near-fault (NF) and far-field (FF) situations. These records are extracted on different soils, which are categorized based on their shear velocity, to represent different soil behaviours and characteristics. Nonlinear Time History Analysis (NLTHA) is carried out on a typical isolated bridge model using the SAP2000 software. Responses in terms of deck acceleration, base shear, displacement of the isolation system, and the performance of the isolation units are studied. Results demonstrate that for NF zones, the soil effects must absolutely be taken into account. In soft soils, all seismic responses are amplified, leading to higher forces and displacement demands. In such zones, failing to consider this fact during the design process results in largely underestimated seismic displacement and force demands for the isolated bridge system, seriously harming its seismic performance. Likewise, the amplification of responses in soft soils is observed for FF records and it should be considered

Keywords: Seismic isolation, Earthquake characteristics, Near-Fault, Far-Field, Soil effects, Bridges.

2.2 Introduction

Seismic isolation technology is based on decreasing the fundamental structural vibration frequency to a value less than the predominant energy-containing frequencies of the earthquake in order to reduce the seismic force demand to or near the elastic capacity of the structural members, thereby eliminating or drastically reducing inelastic deformations. Numerous experimental and analytical research studies have compared the seismic responses of conventional and isolated structures and have shown that seismic isolation technology plays a great role in reducing the seismic responses of infrastructures (Ghobarah & Ali, 1988; Soneji & Jangid, 2008; Tongaonkar & Jangid, 2003; Tsopelas et al., 1996). The long-term benefit of these technologies lies in its preservation of the serviceability of the structure after an earthquake and thus eliminating the cost of reconstruction (Guizani, 2003; Guizani & Chaallal, 2011).

Bridges rank among the most important infrastructures in modern societies because they are the vital artery in transportation systems, especially in crisis conditions, such as the period following a strong earthquake. It should be mentioned that even when just a part of a bridge collapses, the stability of the whole structure is affected. Because failure at a particular point, such as a span failure due to an inadequate support length at the joints, or a failure in a pier, will interrupt the operation of the whole bridge, which is a key component of the land communication system (Andrawes & DesRoches, 2005; Forcellini, 2017).

To ensure that reliable installations are available to respond to damaging earthquakes, all effective parameters of the performance of isolated bridges must be taken into account at the design stage. According to recent studies, and based on evidence from past earthquakes, record characteristics and soil effects are two of the most important parameters affecting the seismic performance of isolated structures (Beresnev & Wen, 1996; Castaldo & Tubaldi, 2018).

Ground motion records obtained in major earthquakes have shown that the characteristics of NF are particularly different from FF records. NF and FF ground motions differ in the distance to the ruptured fault so that if the structure under consideration is inside the specified distance of a fault (10-20 km), it can be classified as NF. Ground motions having a source-to-site more than the abovementioned distance are classified as FF motions (Billah et al., 2013; Bray & Marek, 2004). NF records often contain strong and long-period velocity pulses that could cause severe structural damage (Chouw & Hao, 2005; Neethu & Das, 2019). Studies on the effect of NF records and isolated bridges showed that in the NF zone, records contain a higher PGV/ PGA ratio, velocity pulse, and input energy than FF records and that these records lead to a greater ductility demand and a larger base shear on structures (W. I. Liao et al., 2000; Shen et al., 2004).

Furthermore, NF records amplify the seismic responses of the isolated bridge when the pulse period is close to the effective period of the isolation system and the ratio of the energy dissipated by the isolation system to the total input energy is slightly influenced by the NF effect (Shen et al., 2004). Several important factors are highlighted in the literature, showing that NF records cause severe damage to conventional and isolated bridges, and produce high story drift values, which force the structure to behave in the inelastic range and lead to severe damage (Attalla et al., 1998; Malhotra, 1999b; Ordaz et al., 1995).

It has been shown in the literature that the frequency contents of the seismic waves transform while the waves are passing through the different soil layers (Saritas & Hasgur, 2014; Worku, 2014). These changes are drastic where the underlying soil is soft, resulting in a higher displacement and shear forces (Kulkarni & Jangid, 2003; Saritas & Hasgur, 2014; Worku, 2014). For this reason, isolated bridges located on soft soils have a greater potential for severe damage, while the isolation system provides a better efficiency on rocks or stiff soils during earthquakes (Dicleli & Buddaram, 2006; Kulkarni & Jangid, 2003).

Often, a rigid base is assumed in seismic structural design, while soil effects or soil-structure interaction (SSI) effects are either ignored or considered separately, whereas they are indeed coupled, and ignoring the soil and its effects lead to erroneous assessments (Stehmeyer & Rizos, 2008). Studies have shown that while neglecting soil effects is not consequential for all bridges, in the case of heavier bridges in particular, as well as for soft soil conditions, such neglect leads to an underestimation of the bridge damage potential (Castaldo & Ripani, 2016; Stehmeyer & Rizos, 2008).

Various methods and software programs used by researchers to study soil and SSI effects provide reliable soil models and structural behaviours. Developing numerical models is quite challenging since soil materials exhibit a diverse range of complex constitutive behaviours (Beresnev & Wen, 1996). Methods used for seismic analysis of soils and structures are based on analytical, experimental, and numerical procedures, combined with observations of physical behaviour and lessons learned from past events. The finite element method (FEM), the boundary element method (BEM), and the coupled BEM_FEM are among the most popular numerical techniques used for the rigorous modeling of soil effects on bridges (Stehmeyer & Rizos, 2008).

The effect of SSI on the response of an isolated cable-stayed bridge was investigated by Soneji and Jangid (Soneji & Jangid, 2008). Springs and dashpots were used to simulate the SSI effect and the bridge was subjected to a series of NF records. Results showed that the soil has significant effects on the responses of isolated bridges and that bearing displacements may be underestimated if SSI effects are ignored. Further, it was found that nonlinear soil modeling is essential in properly reflecting the dynamic behaviour of the soil-pile system.

Similar studies have been conducted by using springs and dashpots to model the SSI effects, and show that SSI causes higher isolation system drifts and higher pier shears as compared with fixed-pier bridges (Ates & Constantinou, 2011; Bi et al., 2011; Ucak & Tsopelas, 2008; Zhang & Makris, 2002). As well, numerous studies have shown that seismically isolated bridges are very sensitive to NF records and soft soil effects (Castaldo & Tubaldi, 2018; Chouw & Hao, 2005).

Despite the difficulties inherent in modeling soils and structures in the direct method, many related studies have been conducted by researchers using different methods such as FEM, BEM, and the finite difference method (FDM) to show the effect of different soils and records on infrastructures. The SSI effect for a historical masonry arch bridge using full three-dimensional (3D) nonlinear time history analysis and the FEM approach was studied by Güllü and Jaf (Gillich et al., 2018). A comparison of the results with the 3D model showed that the influence of SSI becomes relatively prominent on the displacement, acceleration, rotation, frequency, modal shape, base shear, and overturning moment responses.

Another approach to investigate the influence of soil characteristics in terms of frequency content on the seismic performance of isolated bridges was used by Castaldo and Tubaldi (2018) (Castaldo & Tubaldi, 2018). A series of artificial ground motions corresponding to the stiff, medium, and soft soil conditions was used to assess the seismic responses of an isolated bridge and define the optimal isolator properties. Results revealed the importance of different soil characteristics in the design of the isolator's parameters and the need to consider the frequency and characteristics of the records at the design stage.

While most of the studies focused on the NF records or compared the NF and FF records, a seismically isolated highway bridge pier was evaluated by considering the SSI effect subjected to FF records. Spring and dashpot components for SSI and laminated rubber bearings for the isolation system were modeled. The results showed that SSI effects play an important role in increasing the responses of seismically isolated bridges constructed on soft soils, which should be carefully considered in such conditions (Alam & Bhuiyan, 2013b).

Many studies have examined different record characteristics, such as NF and FF records, soil effects, and SSI effects on conventional and isolated bridges. Most such studies have focused either on the record characteristics or the soil characteristics with or without isolation systems, but limited attention has been paid to the concurrent effects of both characteristics on isolated bridges for prone areas. For this reason, this study aims to investigate the simultaneous effects of record characteristics, including NF and FF, and the effect of different soil properties on isolated bridges. Real earthquake records on different soil layers are extracted and seismic responses of a bridge with and without an isolation system are studied.

The purpose of this study is to understand the effect of soil layers on the propagation of different record characteristics and the performance of isolation systems. Reaching a more advanced comprehension of the responses of isolated bridges leads to better and more optimal bridge designs in future projects. Additionally, this understanding allows posing more accurate diagnostic and finding more efficient isolation solutions for retrofit projects, considering the effects of site conditions. More details about the records, soil categories, and isolation systems will be presented in the next sections.

2.3 Selected records and record properties

Twenty-one ground motion records of the Northridge 1994, Kobe 1995, and Parkfield 2004 earthquakes were extracted from the strong motion database of the Pacific Earthquake Engineering Research Center (PEER 2013). These earthquakes were chosen for two major reasons. First, they are among the most destructive earthquakes ever recorded, and second, there is a rich database of records for these earthquakes at different distances from the epicenter (R_{rup}) representing NF and FF situations, and different soil types, to represent the soil effects.

It is important to note that for all three earthquakes, records were chosen to cover nearly the same distances and soil classes. Real captured records were used in order to study the trend of real changes in NF and FF record characteristics by passing through the different soil layers instead of using software predictions and simulations. Records and site classes were classified according to Canadian Highway Bridge Design Code (CSA, 2014). Nine of the 21 records were within 10 km of the epicenter (R_{rup} less than 10 km) and were considered as NF records (CSA, 2014). As earlier stated, NF records have higher characteristic values, especially in terms of Peak Ground Velocity (PGV), specific energy density, and damage index (W. I. Liao et al., 2000; Shen et al., 2004).

These features cause higher seismic responses, and such signals impose very high input energy on the structure at the early stages of the records (W.-I. Liao et al., 2004). Different site classes were categorized based on the shear wave average velocity (V_s), which is shown in Table 2.1, while the record properties are shown in Table 2.2. It should be noted that since this study aims mainly to investigate the simultaneous effects of NF and FF records on different soils, the effect of different source mechanisms such as directivity effects, focal mechanisms (strike-slip, normal or reversing faulting), rupture duration, and slip duration are not considered.

Table 2.2 shows that NF records have higher characteristics, especially in PGV, Arias intensity (AI), and specific energy density. Arias intensity (AI), which is a cumulative ground motion intensity measure, provides a quantitative and instrumental measure of the severity of seismic shaking. According to Table 4.2, this parameter has a higher amount for soft soil records in NF areas (R. C. Wilson, 1993).

A term used for the amount of energy stored in a system is "specific energy density", which is the square of velocity at any given time integrated over the entire time range (Sandeep & Prasad, 2012). Obviously, for NF records, this parameter should carry a high value, and as is shown in Table 2.2, for soft soils in NF zones, the parameter has a higher amount of specific energy density. As a constant rule, all parameters are amplified in soft soils, as compared to stiff soils. The damping effect of soil in FF areas is clearly observed in records characteristics by passing through the soil so that by increasing the distance from the fault, a higher reduction is observed in all the intensity parameters.

Spectral accelerations of the Kobe, Northridge, and Parkfield for NF records on different soil types are shown in Figure 2.1 to illustrate the effects of different soil frequencies on the characteristics of records (PEER, 2013). Drastic changes in the maximum response acceleration and period elongation in soft soils are observed, which illustrates the importance of considering soil classes for seismic bridge analysis.

2.4 Case study bridge model and isolation system properties

The selected case study bridge model is a regular conventional bridge with a uniform solid slab deck, 7 m wide and 0.7 m thick. In this study, the bridge is symmetric with two equal spans supported on a circular 30 MPa concrete pier. In total, four different conventional bridge models are analyzed in which each pier's dimensions provide a bridge with a specified fundamental period of 0.63, 0.43, 0.24 and 0.16 s, which are referred to in this study as BR-1, BR-2, BR-3 and BR-4, respectively. Table 2.3 illustrates the dimensions and the period of each conventional bridge.

3D structural modeling of the bridge and NLTHA is performed in SAP2000 program. The concrete bridge deck and pier are modelled by frame elements with an assumed elastic behaviour during the earthquake excitation because in general the bridge piers in seismic isolated bridges are designed to perform in an elastic or nearly elastic manner (Ucak & Tsopelas, 2008). Table 2.4 indicates the value of the density, concrete's compressive strength, modulus of elasticity, and Poisson's ratio of the sections used for modelling.

As it is shown in Figure 2.2, boundary conditions are assigned as roller for free movements of the superstructure in the longitudinal direction for both spans and pin-type connections in the transverse direction. In addition, the base of the pier is restrained in all directions of translation and rotation.

Table 2.1 Site classifications for seismic site response
Taken from CSA (2019)

Site class	Ground profile name	Average properties in top 30 m		
		Shear wave average velocity, $\overline{V}_s$ (m/s)	**Standard penetration resistance, $\overline{N}_{60}$**	**Soil undrained shear strength, s_u**
A	Hard rock	$\overline{V}_s > 1500$	Not applicable	Not applicable
B	Rock	$760 < \overline{V}_s \leq 150$	Not applicable	Not applicable
C	Very dense soil and soft rock	$360 < \overline{V}_s \leq 760$	$\overline{N}_{60} > 50$	$s_u > 100\ kpa$
D	Stiff soil	$180 < \overline{V}_s \leq 360$	$15 \leq \overline{N}_{60} \leq 50$	$50 < s_u \leq 100\ kpa$
E	Soft soil	$\overline{V}_s < 180$ Any profile with more than 3 m of soil with the following characteristics: - Plastic index PI>20; - Moisture content W≥40%; and - Undrained shear strength s_u<25 kpa	$\overline{N}_{60} \leq 15$	$s_u < 50\ kpa$
F	Other soil	Site-specific evaluation required		

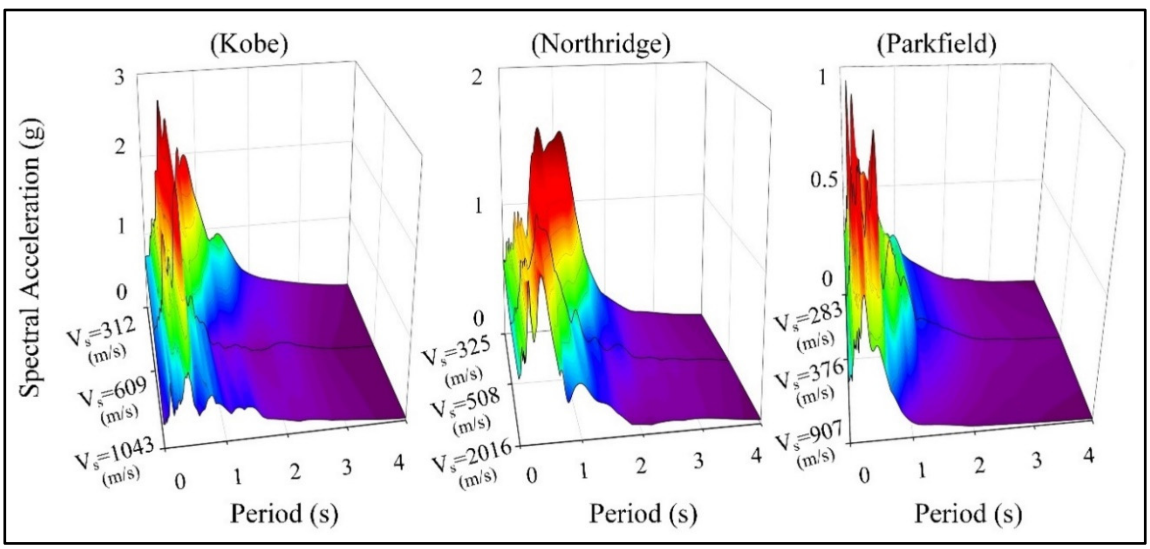

Figure 2.1 Spectral acceleration of NF records on different soils, R_{rup}<10 (km)

Table 2.2 NF and FF earthquake records on different soils based on Vs

Records	Soil Type	R_{rup}	PGA	PGV	PGD	Arias Intensity	Specific Energy Density
		km	g	cm/s	cm	m/sec	cm^2/sec
Northridge-1	A	7	0.43	30	5.5	0.73	489
Northridge-2	C	7.2	0.43	51	10.5	1.8	1812
Northridge-3	C	31.7	0.06	5.3	2.4	0.04	27
Northridge-4	C	81.7	0.05	2.9	0.6	0.03	12
Northridge-5	D	7.5	0.55	76	14.3	3.1	5281
Northridge-6	D	29.7	0.1	13.6	5.7	0.22	417
Northridge-7	D	85.4	0.05	4	0.8	0.03	15
Kobe-1	B	0.9	0.31	31	8.6	0.82	1031
Kobe-2	C	7	0.48	46.8	8.4	3.35	1978
Kobe-3	C	50	0.09	5.3	2.8	0.11	41
Kobe-4	C	119	0.02	1.1	1.9	0.01	6
Kobe-5	D	0.9	0.83	91.1	21.2	8.4	7597
Kobe-6	D	31.7	0.3	24.5	8.1	1.4	1778
Kobe-7	D	95.7	0.14	15.1	3.1	0.37	420
Parkfield-1	B	5.3	0.24	14.6	1.4	0.17	54
Parkfield-2	C	4.9	0.37	14.1	1.9	0.74	105
Parkfield-3	C	22	0.02	3.6	2	0.01	33
Parkfield-4	C	69	0.01	0.7	0.2	0.002	1
Parkfield-5	D	5.2	0.31	20.2	3.7	0.95	292
Parkfield-6	D	29.4	0.02	4.3	1.7	0.03	87
Parkfield-7	D	68.5	0.02	0.9	0.2	0.01	4

Considering the bridge is located in Montreal, Canada, the isolation system is designed for three periods of T= 1, 2, and 3 s, referred to as ISO-1, ISO-2, and ISO-3, with a displacement capacity of 3, 5.5, and 10 cm, respectively. For implementing seismic isolation, the isolation system is lumped between the deck and the pier, and only the longitudinal direction is studied. Bilinear behaviour model of the isolation system is based on the rubber isolator property and it is assigned as a link element in SAP2000 (E. L. Wilson, 2017). The hysteresis parameters, the behaviour of the seismic isolation system (SIS), and the bridge 3D model are shown in Figure 2.2, while the SIS parameters are presented in Table 2.5.

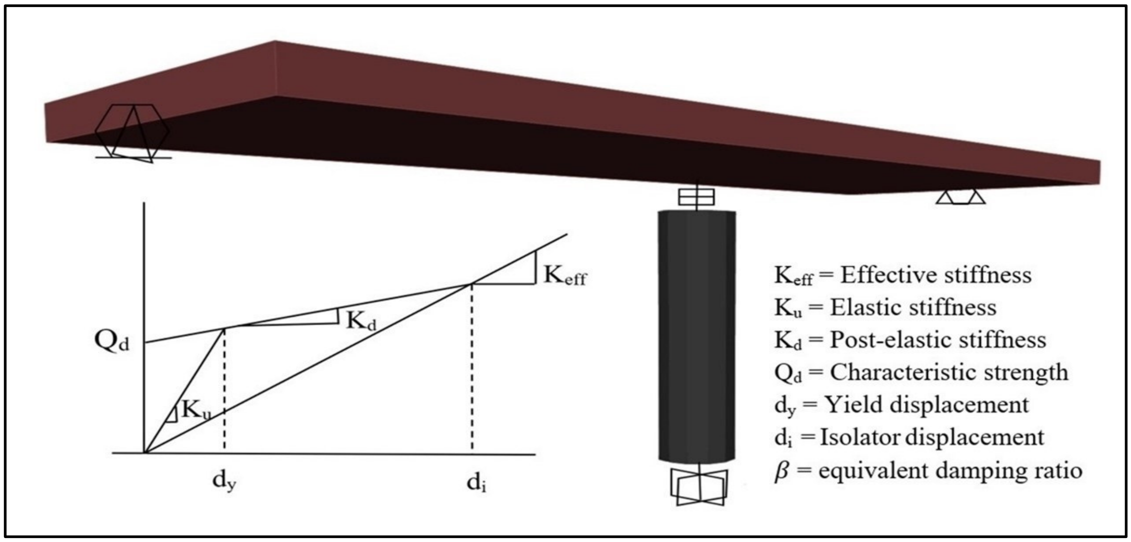

Figure 2.2 Isolated bridge model and primary hysteresis curve parameters for SIS

Table 2.3 Period and the dimension of conventional bridges

Bridge	**T**	**Pier Diameter**	**Pier Height**	**Span Length**	**Deck Width**	**Deck Thickness**
	s	**m**	**m**	**m**	**m**	**m**
BR-1	0.63	0.8	6	10	7	0.7
BR-2	0.43	1	6	10	7	0.7
BR-3	0.24	1.5	6	10	7	0.7
BR-4	0.16	2	6	10	7	0.7

Table 2.4 Material properties

Material	**Density**	**Compressive Strength**	**Modulus of Elasticity**	**Poisson Ratio**
	KN/m^3	**Mpa**	**Mpa**	
Concrete	24	30	24000	0.2

Table 2.5 Isolation properties

Isolation Name	**T**	**K_{eff}**	**K_u**	**K_d**	**Q_d**	**β**
	s	**KN/m**	**KN/m**	**KN/m**	**KN**	
ISO-1	1	9750	30000	3000	225	0.2
ISO-2	2	2400	12000	1200	70	0.2
ISO-3	3	1100	8000	800	30	0.2

Four different conventional bridges in order to represent different structural flexibility parameters, and three isolation systems to show the effect of different isolation characteristics, in total, 16 bridges are analyzed. These bridges are subjected to 21 records that contain NF and FF captured on different soil types at different ruptured faults distances. NLTHA for all cases is conducted in SAP2000 and structural responses, including the maximum acceleration on top of the deck, the maximum displacement of the isolator and the maximum base shear are studied, and the results are discussed in the following sections (E. L. Wilson, 2017).

2.5 Results

2.5.1 Maximum acceleration response

As can be seen in the maximum acceleration responses shown in Figure 2.3, increasing the isolation period leads to fewer responses in all bridges, with different natural periods and all subjected records, including NF and FF on stiff to soft soils. The highest reduction factor among the isolation systems is related to ISO-2, by a factor of 2 compared to ISO-1. This factor is almost 1.5 for ISO-3 compared with ISO-2. By increasing the isolation system period, the acceleration responses tend to approach the same value, especially in the stiffer bridges, BR-3 and BR-4.

Acceleration responses of isolated bridges are higher in NF records than in FF records, and among NF records, it is higher on soft soils. This trend shows the destructive effect of NF records, particularly on softer soils. The average increasing factor in the maximum acceleration responses for NF records on soft soils is 2, 1.8, and 1.5, respectively, as compared with NF records on stiff soil for the Kobe, Northridge, and Parkfield records, respectively. Comparisons of FF records show that there is a drastic decrease in acceleration responses for all bridges in the case of stiffer soils (type C). This reduction is more significant for records at higher distances from the ruptured fault. In softer soils (type D), the amplification of responses is observed compared with the records in stiffer soils, while in general, the lowest acceleration responses are related to FF records.

The maximum acceleration responses of all records along with the increasing factor of the maximum acceleration are shown in Figure 2.3 and Figure 2.4. It should be mentioned that the higher seismic responses in NF records and also the amplification of responses in soft soils have been reported in previous studies (Shen et al., 2004; Stehmeyer & Rizos, 2008).

2.5.2 Maximum seismic isolation system displacement

The maximum isolation system displacements are 29.4, 23.7, and 4.34 cm for Kobe, Northridge, and Parkfield records, respectively. All the maximum responses are related to NF records, (R_{rup}<10 km), on soft soils (soil type D). For the Kobe and Northridge records, the isolator displacements are nearly 6, 5, and 3 times and 8, 3, and 2 times greater than the isolator displacement capacity for ISO-1, ISO-2, and ISO-3, respectively.

In the case of the Parkfield records, the maximum isolator displacement in ISO-1 is higher than the designed displacement and the responses of other isolation systems are in the designed range. This shows that the isolation displacement demand is higher in NF zones, especially on soft soils, and so it must therefore be taken into account in the design process by increasing the isolation period or the designed displacement.

In FF records, the displacement responses are also negligible as compared with NF records and the highest responses are related to records on soft soils. The maximum displacements of the isolation systems for FF records are 6.8, 4.0, and 2.1 cm for Kobe, Northridge and, Parkfield records, respectively, all of them related to records on soft soils (soil type D). For Kobe FF records on soft soils (soil type D), the maximum isolator displacement in ISO-1 is higher than the designed displacement by only 2 mm while the responses of other FF records for all three isolation systems are less than the designed displacement.

These results are in agreement with previous studies and show that the isolation systems are more effective for stiffer soils while less effective for moderate to soft soils if the effect of the soil is neglected (Shen et al., 2004; Stehmeyer & Rizos, 2008). The maximum displacement of isolation systems is shown in Figure 2.5 and Figure 2.6.

It should be stated that the pier displacement in flexible bridges is greater than that in stiffer bridges. This means that the flexible pier is partly responsible for the displacement in isolated bridges, and this prevents the isolation system from acting perfectly. As is shown in Figure 2.6, in the stiffer bridge, BR-4, the pier displacement is less than 10 percent, making its contribution lower, and in that case, the isolation system is more efficient.

The isolation hysteresis loops for BR-1 and BR-4 subjected to Kobe records and obtained from NLTHA, are shown in Figure 2.7 to Figure 2.9. They illustrate that the isolator displacement and energy dissipation are higher in soft soils and NF records.

In FF areas, the farther the distance away from a ruptured fault, the higher the responses and displacement demands on soft soils. A comparison of the same isolation system on 2 different bridges, BR-1 and BR-4, shows that in stiffer bridges, the maximum isolation displacement is higher than the displacement in softer bridges. This is because, in softer bridges, a part of the displacement occurs in the pier, and based on the contribution percentage in Figure 2.6, the isolation performance increases in stiffer bridges.

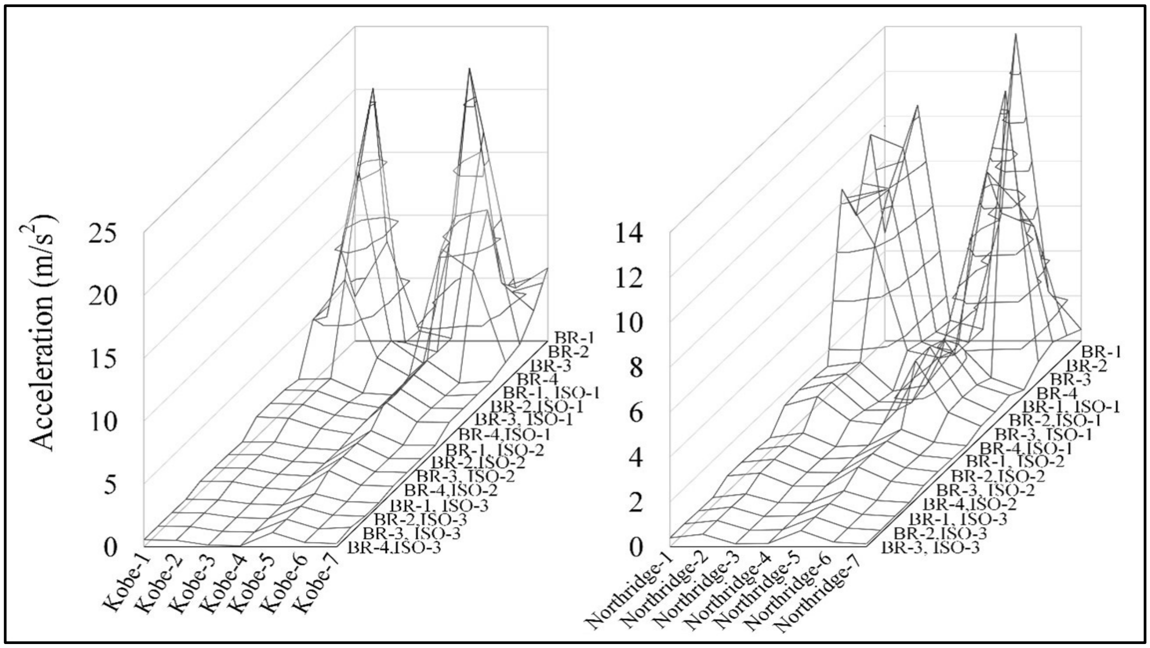

Figure 2.3 Maximum acceleration responses: Left) Kobe records, Right) Northridge records

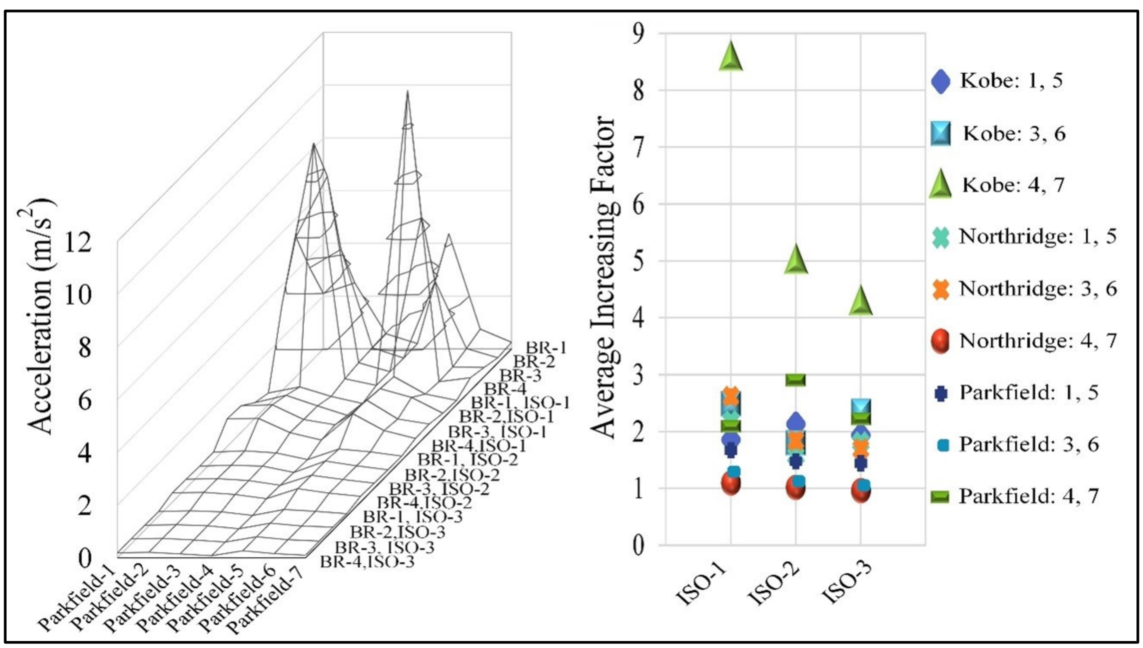

Figure 2.4 Left) Maximum acceleration responses of Parkfield records, Right) Average increasing factors in NF and FF records

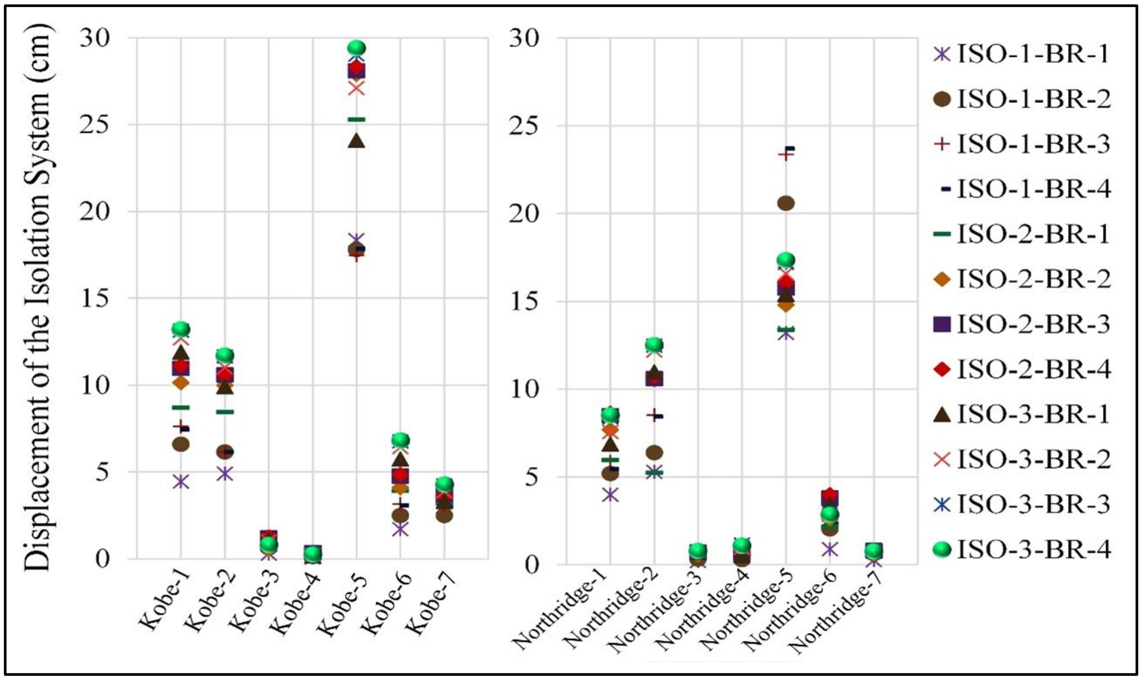

Figure 2.5 Maximum isolation displacements, Left) Kobe records, Right) Northridge records

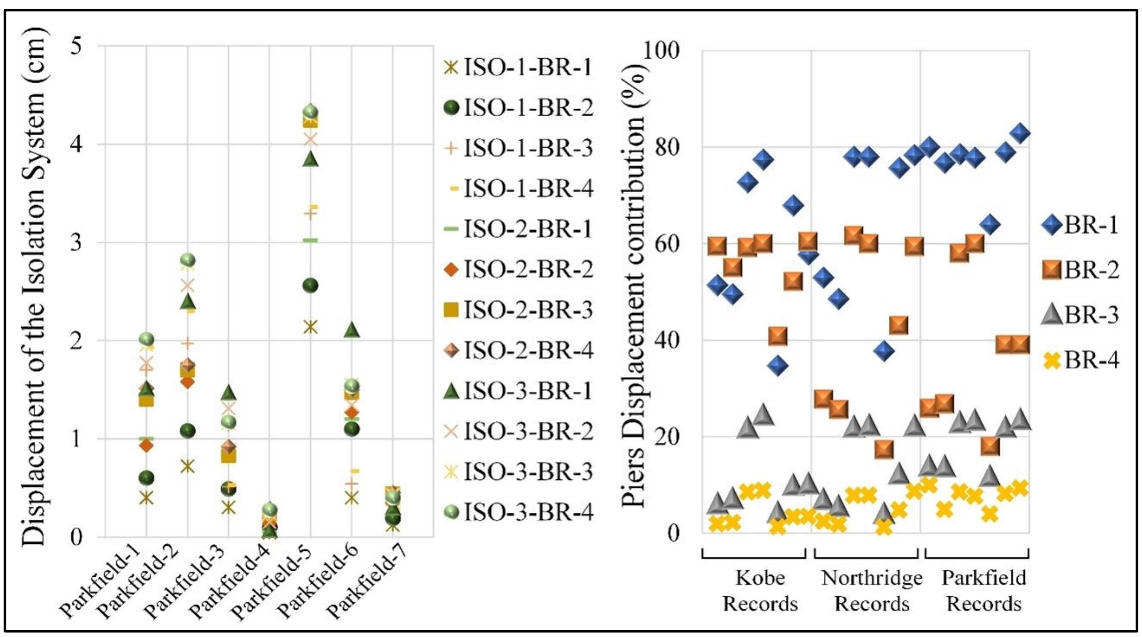

Figure 2.6 Left) Maximum isolation displacements for Parkfield records, Right) Displacement contribution of piers

2.5.3 Base shear responses

The maximum base shear responses are shown in Figure 2.10 and Figure 2.11. The isolation systems noticeably reduce the base shear of conventional bridges by an average factor of 4, 8, and 8.5 for ISO-1, ISO-2, and ISO-3, respectively. For all isolated bridges, the maximum responses are seen in the NF records on soft soils, which is almost twice as much as the responses of stiffer soils.

In the case of isolated bridges, while all three isolation systems reduce the base shear, the reduction with ISO-2 is twice as much as that with ISO-1, and ISO-3 reduces it by 1.5 times more than ISO-2. Thus, the overall reduction factor in ISO-2 is the greatest. For all isolation systems, base shear responses show the increasing trend from BR-1 to BR-4, meaning that stiffer structures endure stronger base shear forces.

In FF records, the base shear responses are much lower than NF records. Comparisons of soil types C and D show that the highest responses are related to records on softer soils (soil type D), showing amplification in responses by passing through these soils. These results are in agreement with the fact that ignoring the soil effects does not lead to an accurate prediction of the base shear in the design stage for soft soils (Soneji & Jangid, 2008). Besides, it shows that the isolation systems are sensitive to NF records and soft soil effects. Thus, these parameters will cause a higher pier shear in structural responses (Ates & Constantinou, 2011; Bi et al., 2011; Ucak & Tsopelas, 2008).

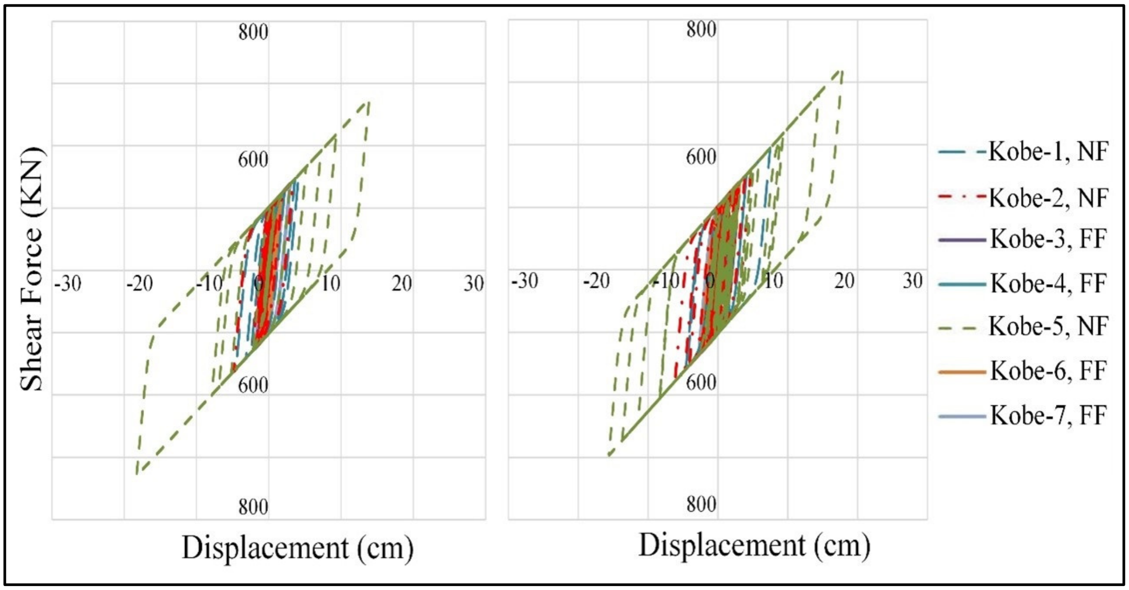

Figure 2.7 Isolation hysteresis loops: Left) BR-1, ISO-1, Right) BR-4, ISO-1

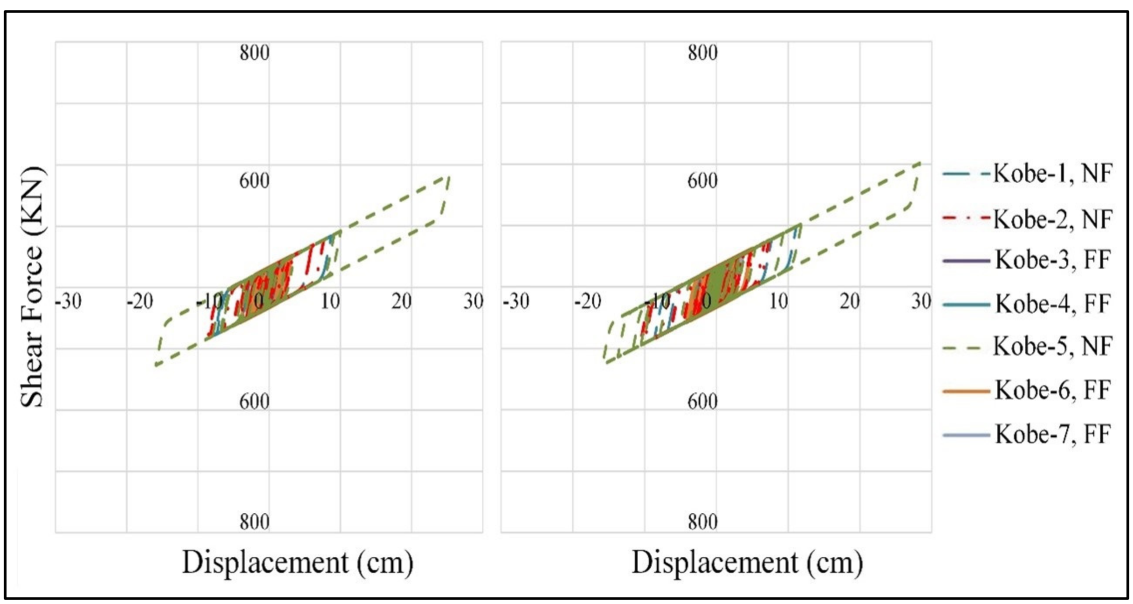

Figure 2.8 Isolation hysteresis loops: Left) BR-1, ISO-2, Right) BR-4, ISO-2

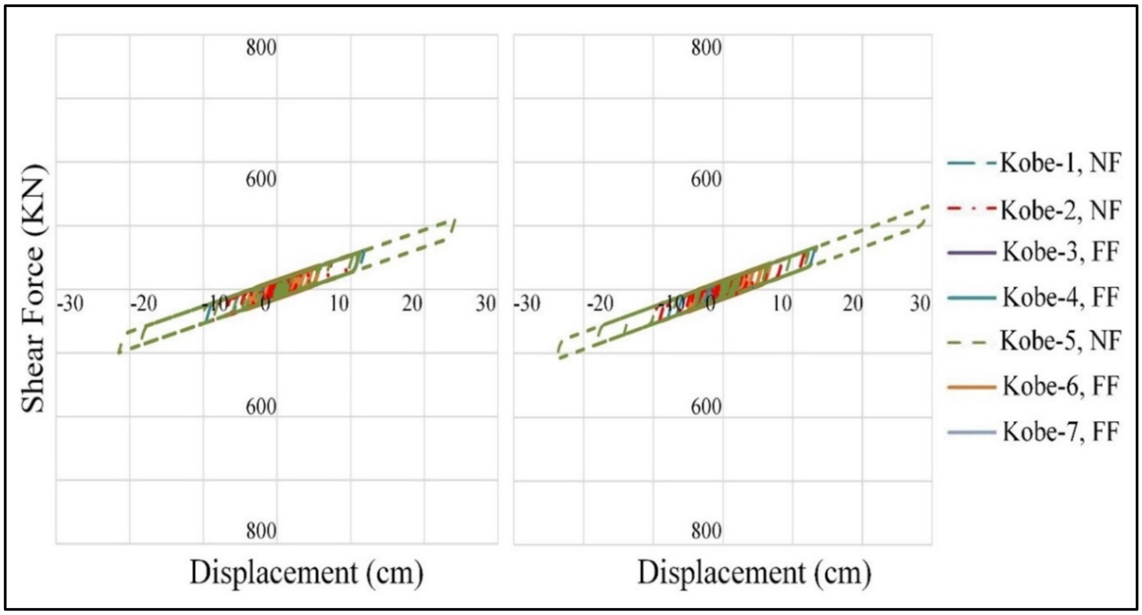

Figure 2.9 Isolation hysteresis loops: Left) BR-1, ISO-3, Right) BR-4, ISO-3

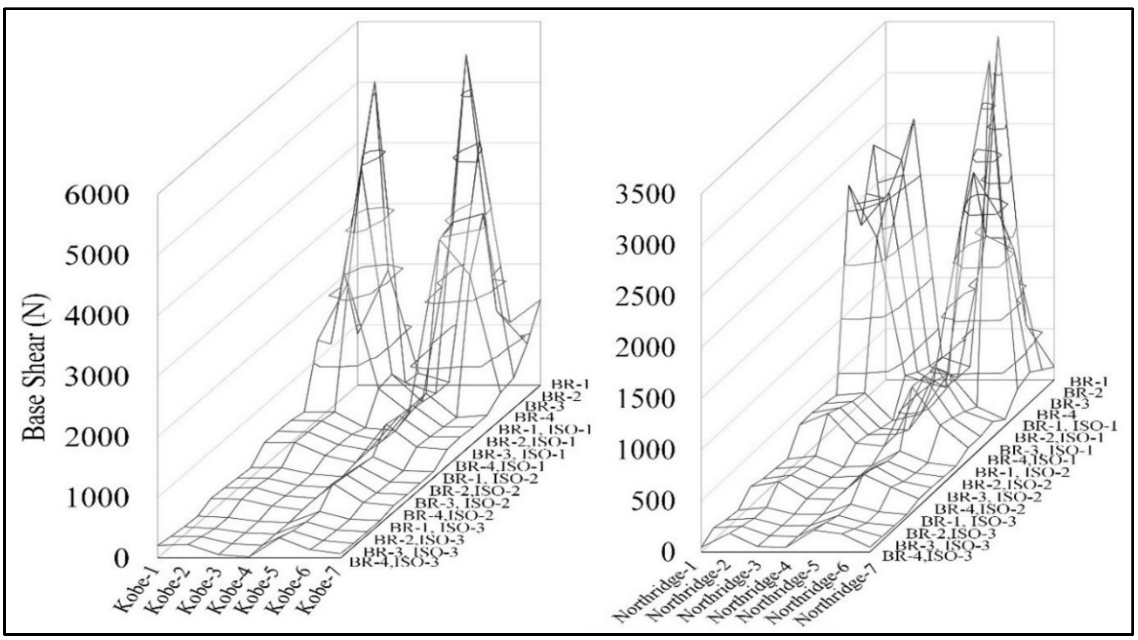

Figure 2.10 Maximum base Shear responses: Left) Kobe Records,
Right) Northridge Records

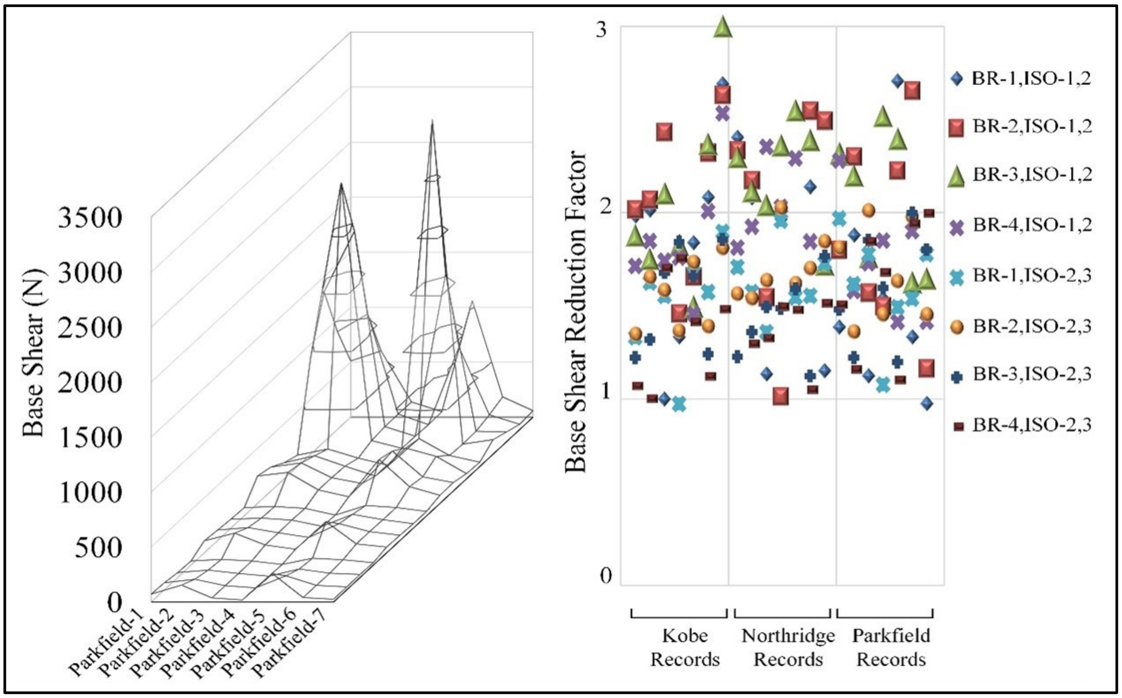

Figure 2.11 Left) Maximum base Shear responses of Parkfield Records, Right) reduction factor for different isolation systems

2.6 Conclusion

The seismic responses of four conventional bridges isolated by 3 different isolation systems subjected to real NF and FF records captured on different soil layers are examined in this paper by three-dimensional NLTHA. For this task, SAP2000 software is used to study the effect of soil properties on changes in the record characteristics and structural responses, and the following results are obtained:

1- In NF records, all mentioned seismic responses are amplified for the isolated and conventional bridges. These increasing trends are more severe on soft soils, and this effect should be considered during the design stage for the bridges in NF areas and located on soft soils. In FF zones, although the amplification of responses on soft soils is negligible compared with NF areas, soft soil effects should be taken into consideration for the design stage.

2- In the case of the isolated bridges, the concurrent effects of NF records and soft soils lead to higher demands in terms of the isolator displacements. Isolator displacements are 6, 5, and 3 times greater than the designed displacement in the case of Kobe, and 8, 3, and 2 times greater than Northridge records for ISO-1, ISO-2, and ISO-3, respectively. Ignoring this issue at the design stage could lead to a large underestimation and possible failure of the isolation systems and the bridges during a strong NF earthquake on soft soil sites. Isolator displacements for FF records were less than the displacement capacity for all records except for Kobe records on soft soils, where they are exceeding the designed displacement by only 2 mm.

3- The isolation performance is better for stiffer bridges for both NF and FF records, where pile flexibility contributes very little in the total displacement, which is mainly taking place within the isolation system.

4- In soft soils, the reduction in acceleration responses of the isolated bridges is limited. The average reduction factor of all isolation systems in NF located on stiff soils is 28, 13, and 26 for Kobe, Northridge, and Parkfield, while it drops by a factor of 3, 5, and 12 for FF records on soft soils, respectively. This clearly shows that seismic isolation is more efficient for rocks or stiff soils rather than soft soils for both NF and FF records. Careful attention and scrutiny are required in designing these technologies, depending on the distance of the relevant structure from active faults and the type of soil where it is located.

CHAPTER 3

SOIL-STRUCTURE INTERACTION EFFECTS ON SEISMIC RESPONSES OF A CONVENTIONAL AND ISOLATED BRIDGE SUBJECTED TO MODERATE NEAR-FAULT AND FAR-FIELD RECORDS

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3.1 Abstract

Seismic isolation is a powerful tool for mitigating seismic risk and improving structural performance. However, some parameters, such as earthquake inputs and soil characteristics, influence the technology's performance. This research aims to investigate the effects of soil-structure interaction (SSI) with regard to different moderate earthquakes associated with different distances of the source to the site, frequency content, and different soil characteristics on the seismic response of the isolated bridges. Near-fault (NF) and far-field (FF) records are applied to the conventional and isolated bridge with and without considering the underlying soil. For this reason, using the direct and simplified methods, three soil properties representing rock, dense and stiff soils are modeled in Abaqus software. Nonlinear time history analysis (NLTHA) is carried out, and structural responses of both approaches in terms of maximum deck acceleration, base shear, and displacement of the deck and the isolation system are studied. Results demonstrate that the difference between the two approaches is significant. Using the simplified method is a rather simple approach that roughly captures the important features of the record characteristics and SSI. Furthermore, careful attention should be paid to

the base shear responses and the isolator displacement demands, as they are significantly amplified on softer soils. In addition, the peak ground acceleration to peak ground velocity ratio (PGA/PGV) plays a decisive role in all dynamic responses. Records with a lower PGA/PGV ratio cause higher dynamic responses in terms of displacement and acceleration /force, regardless of the distance of the ruptured fault, while NF records show higher dynamic responses compared to FF records.

Keywords: Seismic isolation, Earthquake characteristics, Soil-Structure Interaction effects, Near-Fault, Far-Field, Bridges

3.2 Introduction

Strong earthquakes may create devastating effects on infrastructures in seismic prone areas. Experiences from past damaging earthquakes have proven that strength alone would not be a sufficient requirement for the safety of the structures and continuity of service. Therefore, researchers have continuously focused on studying various technologies to prevent or minimize the damage caused to structures by severe seismic activities.

Bridges are one of the most critical infrastructures in today's modern society, being crucial components in transportation systems, especially in times of crisis, such as the period following a major earthquake. Therefore, the seismic design of bridges is carried out to fulfill a set of target seismic performances, typically tuned depending on the bridge's importance and the earthquake probability of exceedance. As a result, the seismic hazard is defined in a set of design spectra associated with different probabilities of exceedance, usually varying between 2% to 40% in 50 years, and associated with different seismic performance levels. The design to reach such performance can be carried out according to a process called force-based design, where the target performances are implicit, and the designer has to apply prescriptive rules to reach them, or a performance-based design, where the target performances are explicitly expressed, and the designer has to prove that the proposed design meets such performances. Two main strategies can be adopted to design bridges for such seismic performances: 1)

Conventional design approach where the superstructure is fixed, directly or through fixed bearings, to the foundation unit, herein-called, conventional bridge; 2) Base-isolated design where special bearings with controlled lateral stiffness and eventually additional damping devices are inserted between the superstructure and the foundation units, herein-called, isolated bridge. For the conventional design, to accommodate strong seismic action, i.e., the 2% in 50 years design earthquake, typically, the design of the bridge relies on the capacity of critical components to accommodate inelastic deformations by a ductile behaviour.

One of the rational and fundamental solutions for mitigating the effects of earthquakes is seismic isolation (Tongaonkar & Jangid, 2003). Seismic isolation is based on reducing the fundamental structural vibration frequency to a value less than the predominant energy-containing frequencies of the earthquakes in order to decrease the seismic force demand to or near the elastic capacity of the structure; thereby, inelastic deformations within the structure will be obliterated or drastically diminished while large inelastic deformations take place within the isolation devices (De Domenico et al., 2020; Di Cesare et al., 2021). Numerous experimental and numerical research works on seismic responses of conventional and isolated structures have indicated that seismic isolation technology plays a significant positive role in reducing the seismic responses of infrastructures (Cardone et al., 2022; De Luca & Guidi, 2019; Tsopelas et al., 1996). The long-term advantage of these innovations is that they preserve the structure's serviceability following an earthquake, reducing the socioeconomic losses and the cost of reconstruction (Guizani & Chaallal, 2011).

Consequently, hysteretic properties of the isolation devices govern the seismic response and performance of isolated bridges. Different isolation devices are commercially available, such as the friction pendulum and the lead-rubber bearings, two of the most widely used isolation systems. Their properties are affected by many factors, such as temperature, aging, velocity, and fabrication tolerances (Buckle et al., 2006; Nassar et al., 2022). The design of base-isolated bridges uses, therefore a bounding analysis approach to evaluate the performance of the bridge at the upper and lower bound values of the hysteretic properties of the isolation devices. In addition, prototype and control quality testing on the seismic isolation units are typically prescribed to verify that the hysteretic properties and behaviour fall within the used values in design (CSA, 2019). Advancing in laboratory equipment and technology leads to more accurate experimental tests and numerical simulations.

Therefore, innovative methods and models using different and new materials and performances, such as unbonded fiber-reinforced elastomeric isolators, high-damping rubber bearings strengthened with glass fiber fabrics, quasi-zero stiffness isolation system, etc., have been introduced in recent years to introduce a new isolation system with better performance or improve the existing isolation systems to provide more performant, more convenient and economical solution for a wide range of structures and ground motion excitations (Nguyen & Guizani, 2021a; De Domenico et al., 2023; Mordini & Strauss, 2008; Ye et al., 2020). Other researchers carried out parametric studies to identify the optimal range of hysteretic properties of seismic isolation devices leading to batter compromise between the reduction of seismic forces and increase of seismic displacement, depending on the characteristics of the area's seismic records (Nguyen & Guizani, 2021b).

Among different pivotal parameters on bridges' structural responses, evidence from past earthquakes has indicated that the earthquake characteristics and the site conditions are two of the most critical parameters affecting the seismic performance of infrastructures (Roussis et al., 2003; Ucak & Tsopelas, 2008).

Ground motion records within 10-20 km distance to the ruptured fault are categorized as NF, while source-to-site distances of more than 20 km are classified as FF ground motions (Billah et al., 2013; Bray & Marek, 2004). Seismic responses of structures between NF and FF records differ considerably. Many research studies reported that NF pulse-like ground motions are more destructive to the structure than that ordinary ground motions (Jia et al., 2023; Jiang et al., 2020; Jiao et al., 2021; Mangalathu et al., 2019). NF records often have a higher PGV/PGA ratio. Frequently, they contain intense and long-period velocity pulses, which force the structure to behave in an inelastic range that may require much higher ductility demand and base shear than FF earthquakes, and the impulse effect may intensify the displacement of the isolation bearing. (Chouw & Hao, 2005; Ismail et al., 2014; W. I. Liao et al., 2000; Neethu & Das, 2019; Shen et al., 2004). Furthermore, in NF pulse-like ground motions, the pulse period (T_P) and peak pulse velocity (V_P), as critical parameters, show a significant influence on seismic responses of the structure (Yang et al., 2023; Zhong et al., 2022). Additionally, the variation of the period of structure to the pulse period has been found to show a strong correlation with the structural responses. Researchers showed that NF records particularly amplify the seismic responses of isolated bridges when the pulse period is close to the period of the structure (Yang et al., 2019; Zhong et al., 2022).

Local sites and SSI can also significantly influence the main characteristics of ground motions, such as amplitude, frequency content, and duration, impacting the seismic responses of isolated bridges. The extent of such influence depends on the dynamic characteristics of the bridge structure, the input ground motion characteristics, and the underlying soil's properties (Dezi et al., 2012; Tochaei et al., 2020).

In common seismic design practice of bridges, the structures are presumed to be fixed at their foundation where soil or (SSI) effects are ignored or considered separately. Misrepresenting results in an erroneous estimation of the seismic demand and the parameters governing the design of the isolation system and the bridge, especially where the underlying soil is soft (Chaudhary et al., 2001; Kulkarni & Jangid, 2003; Saritas & Hasgur, 2014; Stehmeyer & Rizos, 2008; Fatahi et al., 2014; Worku, 2014). Therefore, isolated bridges on softer soils are particularly vulnerable to severe damage due to underestimation of SSI effects, while isolation systems provide better performances on rocks or stiff soils (Alam & Bhuiyan, 2013b; Dicleli & Buddaram, 2006).

Despite many available research studies, there are no consistent, categorized, and definite results regarding the SSI effects on seismically isolated bridges. The reason could be because of the extensive scope of the study containing many details, varieties, and uncertainties in infrastructures and soil properties, so each researcher has tried to cover a related part to their interests and specialties; still, many details have remained uncovered.

Many studies have been conducted by modeling soil through linear springs and dashpots to investigate the SSI effects. Different results have been reported, such as higher isolation system drift due to SSI (Ates & Constantinou, 2011; Bi et al., 2011; Dezi et al., 2012; Stehmeyer & Rizos, 2008; Tongaonkar & Jangid, 2003; Ucak & Tsopelas, 2008) and higher base shears (Hoseini et al., 2018; Ucak & Tsopelas, 2008).

In contrast, some studies showed the beneficial aspect of SSI, causing a reduction in design costs and increasing the safety of the bridge (Tongaonkar & Jangid, 2003; Vlassis & Spyrakos, 2001) and reduction in the base shear was also reported (Tochaei et al., 2020; Vlassis & Spyrakos, 2001).

In addition, in some studies, researchers found that isolated bridges are less sensitive to SSI effects than conventional bridges (Dezi et al., 2012; Vlassis & Spyrakos, 2001). Comparison between linear and nonlinear modeling of springs and dashpots showed that in many situations, nonlinear behaviour for the soil model is an essential factor in properly reflecting the dynamic responses of the system (Soneji & Jangid, 2008). Furthermore, few studies concluded that SSI could be beneficial or detrimental and results in a higher or lower seismic response based on different factors such as structural elements, soil stratum properties, and ground motion characteristics (Betti et al., 1993; Carbonari et al., 2011; Jeremić et al., 2004; Mylonakis & Gazetas, 2000; Ucak & Tsopelas, 2008). Very limited studies involved comprehensive consideration of the direct method showing that linear modeling of isolators and the soil leads to incorrect evaluation of the structural behaviour, and in the case of conventional bridges, SSI has a significant effect on all dynamic responses and considering the soil effects is a crucial matter (Forcellini, 2017; Güllü & Jaf, 2016).

Aside from various conclusions drawn from the literature, most of these studies have focused on either ground motion characteristics or soil properties with or without isolation systems, with little attention paid to the combined effects of both factors on isolated bridges for prone areas. As a result, this research aims to look into the effects of multiple record characteristics, such as NF and FF, and their frequency content and the effect of different soil types on a bridge with and without the isolation system. Actual earthquake records are extracted from rock strata. Each record is passed through various soil properties using the direct method, allowing the seismic responses of the isolated bridge to be studied. Results are compared with responses of the simplified method recommended by Commentary on Canadian bridge code (CSA S6-19) to study the differences between these methods (Association, 2019; CSA, 2019).

Furthermore, this research aims to understand how soil and SSI affect the efficiency of isolation systems in different models. Reaching a more advanced comprehension of the responses of isolated bridges leads to better and more optimal bridge designs in future projects. Furthermore, this understanding allows for more precise and effective isolation strategies through the choice of more appropriate methods and models, when necessary, to catch the SSI effects. The following sections will present more details about the records, soil categories, and isolation systems.

3.3 Numerical modeling

3.3.1 Case study and modeling of the bridge

The selected case study bridge model is the typical three-span continuous concrete box girder deck highway bridge studied by Jangid (2003) (Tongaonkar & Jangid, 2003) and Elias (2017) (Elias & Matsagar, 2017), shown schematically in Figure 3.1. The bridge is symmetric with three equal spans of a box girder deck having a total length of 90 m, 20 m in width and 1.86 m height. The superstructure is supported by two concrete cylindrical single piers of 2.28 m diameter and two abutments with 10 m height above the natural ground level. The single piers and end abutments are supported by shallow foundations of 4 m by 4 m in the horizontal plane and a depth of 1 m. As shown in Figure 3.1, for the conventional design, the superstructure to the top of piers through a rigid connection, transmitting moments, and is on mobile, friction free bearings allowing rotation and displacement in the longitudinal direction. The bridge in this configuration has a fundamental period of vibration in the longitudinal direction of 0.54 s, and a damping ratio of 5% is assumed. Table 3.1 illustrates the geometric and material properties used to model the bridge based on the data presented in the reference studies (Elias & Matsagar, 2017; Tongaonkar & Jangid, 2003).

The bridge's deck and abutments are all straight, with zero skew, and the lateral flexibility of abutments is neglected. In the present study, the structural modeling of the bridge and NLTHA are performed using Abaqus software (ABAQUS, 2019). Beam-column elements are defined to model the deck, piers, and abutments. C3D8R solid elements are used for the foundations and the soil stratum. The strategy behind the seismic isolation is to reduce the seismic forces to or near the elastic capacity of the structure and to limit the inelastic deformations within the isolation devices (Tongaonkar & Jangid, 2003; Ucak & Tsopelas, 2008). Similarly, for the conventional bridge it is supposed that the bridge is classified as an essential bridge which is designed to remain essentially elastic under the design earthquake.

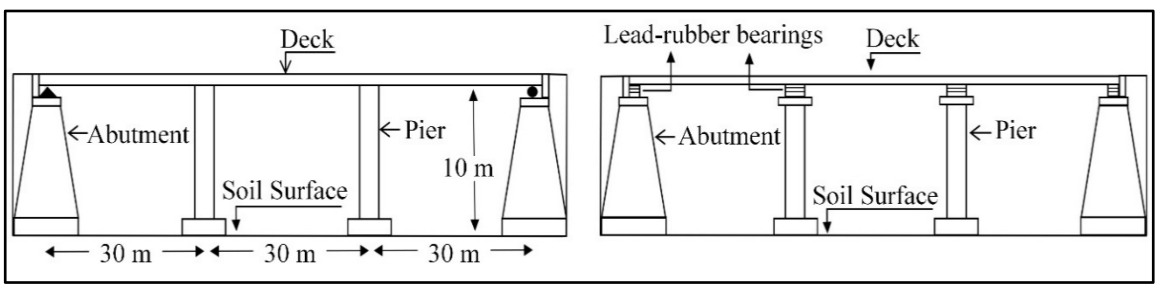

Figure 3.1 General elevation of the studied conventional (left) and isolated (right) bridge

Table 3.1 Material and dimension properties of the bridge

Properties of the Bridge	**Deck**	**Piers**
Cross-sectional areas (m^2)	15.6	1.767
Length or height (m)	3@30	10
Young’s modulus of elasticity (Gpa)	36	36
Mass density (kg/m^3)	2400	2400
Compressive Strength (Mpa)	30	30
Poisson Ratio	0.2	0.2

Consequently, the superstructure and piers are assumed to remain in the elastic state during seismic excitation for both cases of conventional and isolated bridges. For the isolated bridge, the superstructure is supported by seismic isolation bearing with low lateral flexibility, as described later, at all supports in the longitudinal direction, which is the direction investigated in this study.

For the transverse direction, the bearings at piers and abutments, when applicable, are defined as fixed for displacements with rotations free. Vertical supports/connections between the superstructure and foundation are infinitely rigid. For both cases of conventional and isolated bridges, the piers' bases are fixed in all translation directions and rotation, where the soil effect is not considered.

To validate the original model, a comparison of structural responses of the conventional bridge model and the results of reference papers for Northridge record (captured at La County fire station component with PGA=0.58g) is carried out, and results are presented in Table 3.2. Good agreements between the results, in terms of vibration period, base shear, and deck acceleration, are obtained with a difference lower than 5%. After validation of the model, as the bridge is assumed to be in Montreal, to consider the frost action, the foundation is considered to be at a depth of D = 1.8 m of the soil surface, and upper soil load is considered in analyses.

Table 3.2 Comparison of the responses with current study

Responses	**Jangid 2003, Elias 2017**	**Present study**	**Difference %**
Period (s)	0.53	0.54	1.85
Base Shear/W_{deck}	1.439	1.388	-3.54
Deck acceleration (g)	1.396	1.461	4.45

3.4 Isolation system

Considering the bridge is located in Montreal, as a moderate seismicity area, the isolation system is designed using the 6th generation hazard of earthquakes Canada (NRCAN, 2022) for an effective period of T= 2.5 s, an effective damping of 19% at the design displacement of 60.0 mm. The isolation properties are calculated based on the single-mode spectral analysis, and all the parameters are among the proposed domain by Nguyen and Guizani to design an optimal seismic isolation system (CSA, 2019; Nguyen & Guizani, 2021b).

The substructure is decoupled from the deck by lead rubber bearings, and the isolation system is lumped between the deck and substructure. Only the longitudinal direction is studied for implementing seismic isolation. Link elements with bilinear behaviour based on the multi-plastic model given by Abaqus are used to model the isolation system (ABAQUS, 2019). The global model of the isolated bridge and soil, used for the direct approach, as well as the bilinear force-displacement relation, used to represent the seismic isolation system (SIS) behaviour, are shown in Figure 3.2. The SIS hysteretic model parameters are presented in Table 3.3, where Q_d is the characteristic strength, K_d represents the post-elastic stiffness, K_u stands for the elastic stiffness, and K_{eff} is the effective stiffness at the maximum displacement in the isolation system, D_{max}.

3.5 Direct approach

3.5.1 Soil model and properties

Accounting for the effect of SSI, an elastic-perfectly plastic behaviour is assigned for the soil domain using the Mohr-Coulomb yield criterion (Labuz & Zang, 2012). The 8-node brick elements (C3D8) are applied to the soil deposit model as a rectangular shape of 130 (m) in length and 20 (m) in width. Three different non-liquefiable homogeneous soil profiles are adapted and studied as Rock, Soil-C, and Soil-D in this study based on the site classification in CSA (S6-19) (CSA, 2019).

In addition, considering the fact that most amplifications occur within the first 30 m of the soil profile, soil depth is considered to be 30 m (Rayhani & El Naggar, 2008; Tabatabaiefar & Fatahi, 2014). The characteristics of each soil type are presented in Table 3.4, where E is the Elastic modulus, ρ represents the density, C stands for the cohesion stress, ϑ is the Poisson's ratio, ø defines the friction angle, V_s is shear wave velocity, Ψ represents dilatancy and ξ is the damping ratio.

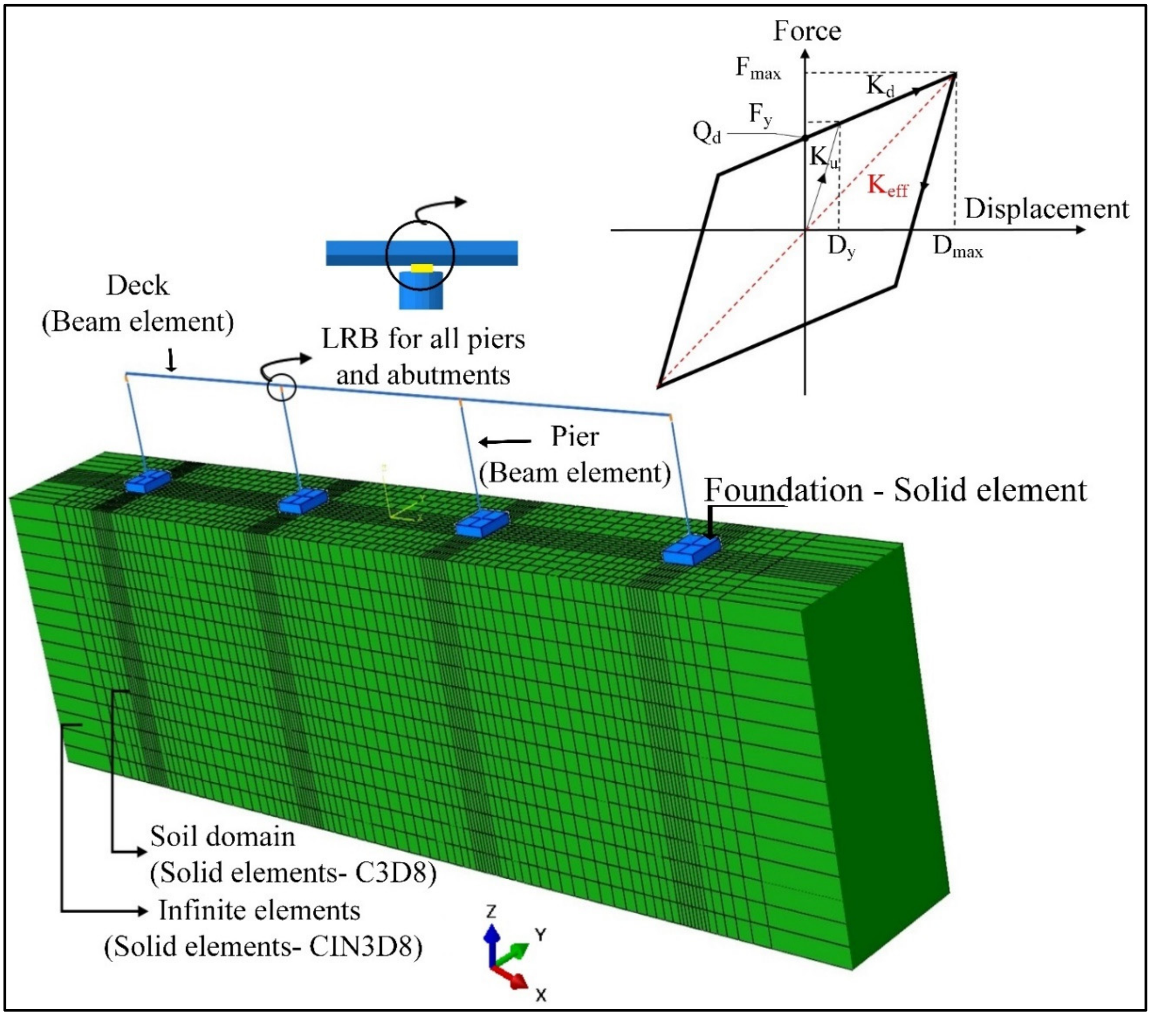

Figure 3.2 Isolated bridge model in the direct approach and bilinear force-displacement behaviour of SIS

Table 3.3 Isolation properties

Isolation system	**T (s)**	**K_{eff} (KN/m)**	**K_u (KN/m)**	**K_d (KN/m)**	**Q_d (KN)**	**D_{max} (mm)**
Piers	2.5	7,880	83,500	5,600	140	60
Abutments	2.5	4,000	30,000	2,800	72	60

Table 3.4 The mechanical properties of soils

Soil	**E (MPa)**	**ρ (kg/m^3)**	**υ**	**C (KPa)**	**Ø (°)**	**V_s (m/s)**	**ψ**	**ξ (%)**
Rock	24960	2600	0.2	25e3	48	2000	7	5
Soil-C	1323	2100	0.26	0	40	500	5	5
Soil-D	430	1900	0.32	0	35	300	4	5

3.5.2 Soil boundary conditions

Regular boundaries will cause the reflection of waves at the finite boundaries of the soil model, which superimpose with the other waves resulting in an inaccurate simulation of actual motions within the studied domain if the domain is not large enough (Liu & Jerry, 2003).

For this reason, the 3D solid continuum as CIN3D8 with 8-node linear, one-way infinite brick elements provided by Abaqus software are used in the longitudinal direction, which is the direction of the study, and fixed boundaries for the transverse direction with free rotations are used in this study. For realistic modeling of the bedrock, at the base of the model, the boundary condition is rigid, which is the most appropriate assumption (Tabatabaiefar et al., 2013).

Therefore, the earthquake acceleration records are directly applied to the grid points along the rigid base of the soil in the longitudinal direction. The surface-to-surface contact between the foundation and the soil surface is modeled as an interaction interfacial behaviour following the algorithm implemented by Abaqus (ABAQUS, 2019).

The interface stiffness values control the relative interface movement in the normal and tangential directions. Hard contact is used in the normal direction, and the penalty method is defined for tangential behaviour. It should be noted that the foundation rocking effect is not considered in this study as it typically implies non-acceptable damage levels for lifeline bridges with conventional design, except when explicit measures are undertaken to mitigate such damages. It is not expected to occur for efficiently base-isolated bridges.

In tangential behaviour, the reduction strength factor (R_{inter}=0.7) is implemented in the classical friction model, and this factor has been selected based on practical cases and suggested domain in the literature intending to calculate a final friction coefficient (μ) of 0.5 (Dehghanpoor et al., 2019; Khazaei et al., 2017; Kimmerling, 2002; Manual & Mechanics, 1986; Pando et al., 2006) according to:

$$\mu = R_{inter} \tan \emptyset_{soil} \qquad (3-1)$$

A mesh sensitivity process is carried out to select the final mesh size so that the tolerance is less than 1%, in terms of displacement and stresses at control stations for structural and soil elements. As shown in Figure 3.2, the final mesh pattern shows fine mesh sizes for important areas close to the bridge and large elements at lateral soil boundaries. In addition, all mesh sizes are within the range between 1/8 to 1/35 of Rayleigh wavelength, a suggested domain in the literature (Jesmani et al., 2012). Overall, 1300 elements for the bridge and 23520 C3D8R, and 480 CIN3D8 elements for the soil domain are used.

3.6 Simplified approach

The soil-foundation-structure system can be represented using simplified models of soil and foundation. Different methods of using springs or springs and dashpots equivalent to the soil model have been proposed to avoid the direct method's complications and shorten the analysis time. This study uses the Winkler spring computational model recommended in the commentary on Canadian bridge code (CSA S6-19) (Association, 2019; CSA, 2019) to simplify the soil behaviour.

As it is shown in Figure 3.3, to apply the simplified method, in the first step, site response analyses are conducted in the free-field foundation soil for all selected soil properties in order to determine earthquake time histories in the absence of the bridge structure using the computer program Deepsoil v6.1 (Hashash et al., 2016). In the second step, the stiffness of the springs is calculated using formulas presented in Table 3.5. Then, the springs are modeled at the base of the bridge for 3 translational and 3 rocking motions.

Finally, in the third step, extracted free-field earthquake records are applied to the base of the springs, and seismic responses of the bridge are obtained and studied from NLTHA. Table 3.5 shows the expressions for computing the springs' stiffnesses and the final stiffnesses values are presented in Table 3.6, where βx is the embedment factor, G is the shear modulus of the soil, L, B, d stand for the length, width, and height of the foundation footing, h represents depth to the centroid of effective sidewall contact, and D represents the total height of the soil from the bottom of the foundation.

Studies show that for small shear strain index (generally under 0.03%), defined as the ratio of input motion peak velocity to time-averaged shear-wave velocity in the top 30 m of the soil profile, equivalent linear and nonlinear analyses results are practically identical (Kim et al., 2016). Therefore, shear strain indexes are calculated for all records and soil deposits, and shown in Figure 3.4. It is shown that the differences between equivalent linear and nonlinear analyses become more pronounced for all records on Soil-C and Soil-D. Therefore, using the equivalent method is not acceptable for the current study. Despite such a fact, the simplified method is also conducted in this study, and all responses are compared with the direct method for the purpose of research and investigations.

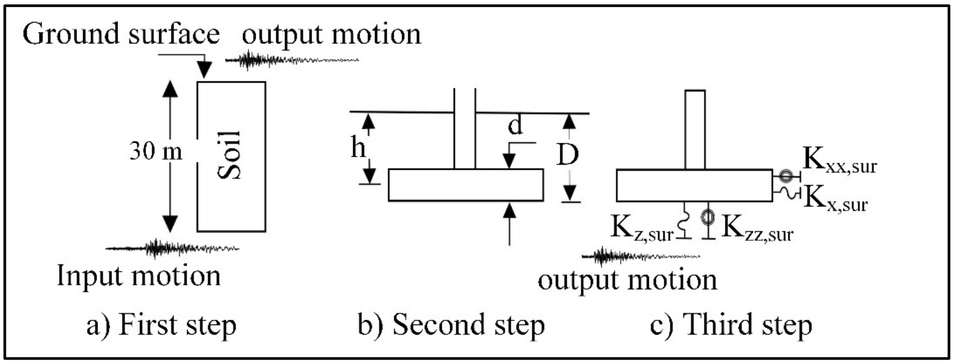

Figure 3.3 Steps to create simplified model

Table 3.5 Foundation compliance springs for embedded foundations, adapted from FEMA 356 (2000)

Direction	**Stiffness**
Translation along x-axis	$K_{x,sur} = \beta_x \frac{GB}{2-\vartheta}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 1.2\right]$ $\beta_x = \left(1 + 0.21\sqrt{\frac{D}{B}}\right)\cdot\left[1 + 1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$
Translation along y-axis	$K_{y,sur} = \beta_y \frac{GB}{2-\vartheta}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 0.4\frac{L}{B} + 0.8\right]$ $\beta_y = \beta_x$
Translation along z-axis	$K_{z,sur} = \beta_z \frac{GB}{1-\vartheta}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]$ $\beta_z = \left(1 + \frac{1}{21}\frac{D}{B}\left(2 + 2.6\frac{B}{L}\right)\right)\cdot\left[1 + 0.32\left(\frac{d(B+L)}{BL}\right)^{2/3}\right]$
Rocking about x-axis	$K_{xx,sur} = \beta_{xx} \frac{GB^3}{1-\vartheta}\left[0.4\left(\frac{L}{B}\right) + 0.1\right]$ $\beta_{xx} = 1 + 2.5\frac{d}{B}\left[1 + \frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right]$
Rocking about y-axis	$K_{yy,sur} = \beta_{yy} \frac{GB^3}{1-\vartheta}\left[0.47\left(\frac{L}{B}\right)^{2.4} + 0.034\right]$ $\beta_{yy} = 1 + 1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5 + 3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]$
Rocking about z-axis	$K_{zz,sur} = \beta_{zz} GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45} + 0.51\right]$ $\beta_{zz} = 1 + 2.6\left(1 + \frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}$

Table 3.6 Foundation stiffness

Foundation size (L × B × D)	Rock	Soil-C	Soil-D
	4.0 × 4.0 × 1	**4.0 × 4.0 × 1**	**4.0 × 4.0 × 1**
Kx (GN/m)	414.65	21.65	6.96
Ky (GN/m)	414.65	21.65	6.96
Kz (GN/m)	322.62	17.61	5.94
Kxx (MN-m/rad)	1,644.43	89.74	30.3
Kyy (GN-m/rad)	1,798.42	98.15	33.14
Kzz (MN-m/rad)	3,451.86	174.25	54.06

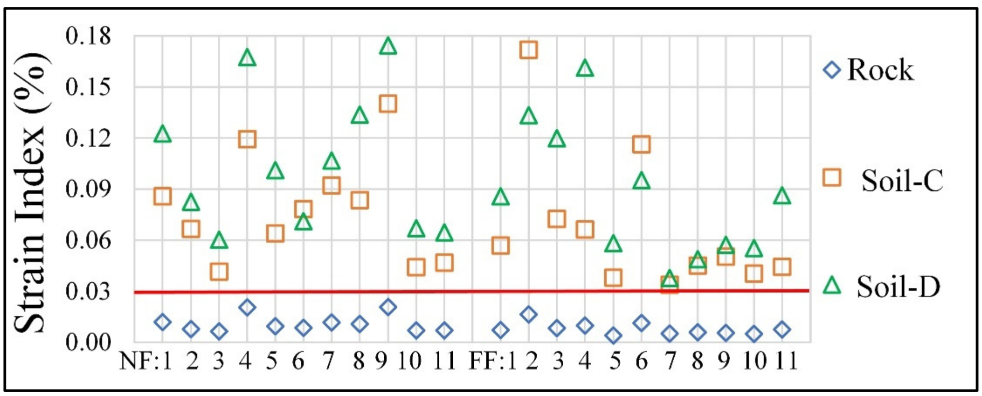

Figure 3.4 Strain index for different soil types

3.7 Seismic analyses

3.7.1 Earthquake record selection and calibration

As the bridge is assumed to be in Montreal, records are selected among moderate earthquakes with the magnitude 4-6 (Richter scale). In this study, 11 NF records with rupture fault distance within 20 (km) and 11 FF records with rupture fault distances more than 20 (km) captured on the rock are selected from the Pacific Earthquake Engineering Research (PEER) strong motion database (PEER, 2013).

The reason for extracting these records on rocks is that minor changes in the ground motions occur in rocks. Therefore, the earthquake ground motions applied at the soil base are closer to the original input ground motions released from their sources.

The second reason for choosing the mentioned records is related to studying the effects of NF and FF sources and to study the effect of the ruptured fault distance, and investigating the effect of the frequency content (PGA/PGV) of the records on the dynamic responses of the bridge with and without SSI effect. It is worth mentioning that all records are selected among crustal earthquakes in active tectonic regimes.

Because most earthquakes result from fault movement in the crust, there is a rich database in accordance with the objectives of this study. In addition, the strength of shaking at the surface from a deep earthquake is considerably less than the same crustal earthquake (U.S. Geological Survey). To compare the results, all records are scaled to 0.444g, which is the PGA associated with the uniform hazard design spectrum, 6th generation (CNB2020), recommended for Montreal for 2% probability of exceedance in 50 years, on class A (average rock) (Canadian Commission on & Fire, 2022; NRCAN, 2022). The details of the selected ground motions scaled to 0.444 (g) are given in Table 3.7, and spectral accelerations of the scaled NF and FF records are shown in Figure 3.5.

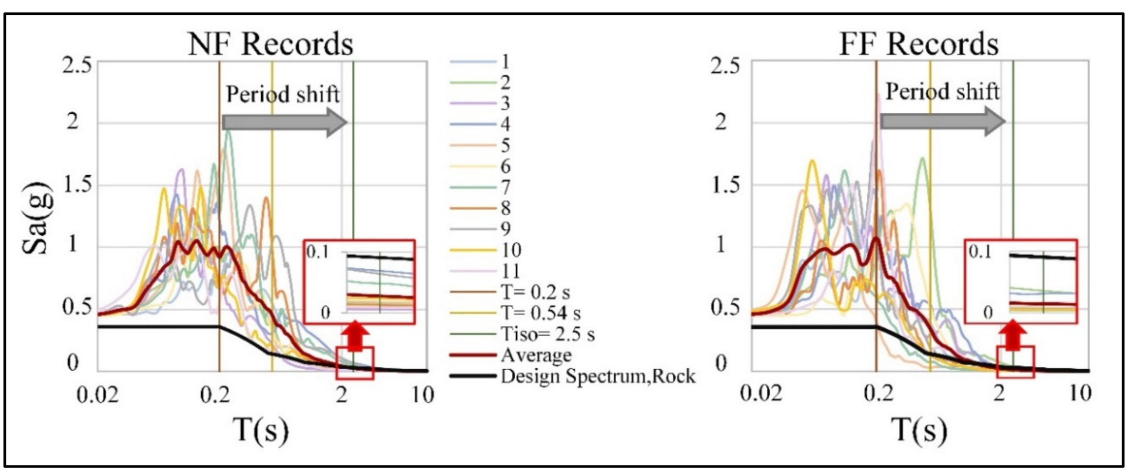

Figure 3.5 Spectral accelerations of the scaled NF and FF records log scale

Table 3.7 Earthquake records adopted in the analyses

ID	Earthquake, Station	Component	Mw	R_{rup} (km)	PGA/PGV (1/s)	Scale factor	Predominant period (s)
NF:1	Parkfield, Turkey Flat	36529270	6	5.3	19	1.8	0.26
2	30226086, Warm Springs Dam	N2122090	4	8.8	28	14.8	0.28
3	Hollister, Gilroy Array #1	A-G01247	5.1	10.5	34	3.1	0.1
4	Coyote Lake, Gilroy Array #1	G01320	5.7	10.7	11	3.8	0.08
5	San Francisco, Golden Gate	GGP100	5.3	11	23	4.7	0.22
6	21530368, Carmenet Vineyards	BKCVSHHE	4.5	12.1	26	12	0.18
7	Umbria, Gubbio	GBB090	5.6	15.7	19	6.6	0.24
8	Northridge, Wonderland Ave	WON095	5.3	17.1	21	7.9	0.48
9	Whittier Narrows, CIT Kresge	A-KRE090	6	18.1	11	4	0.36
10	Lytle Creek, Allen Ranch	CSM095	5.3	19.4	32	10.8	0.14
11	14151344, Pinon Flats	AZPFOHLE	5.2	19.6	32	1.9	0.12
FF:1	14095628, Cattani Ranch	CITEHHLE	5	20.6	31	17.8	0.26
2	Northridge, Griffith Park	GPO000	5.3	21.7	13	13.8	0.14
3	Whittier Narrows, Wonderland Av.	A-WON075	6	27.6	27	10.6	0.1
4	40204628, Mount Umunhum	NCJUMHNN	5.5	30.8	22	18.5	0.08
5	Anza, Keenwild Fire Station	0604A180	4.9	32.1	56	15.3	0.22
6	21530368, Hamilton Field	NHFHNE	4.5	35.1	19	15.9	0.18
7	RiviereDuLoup, Riviere-Ouelle	CN.A16.HHE	4.7	39	44	15.9	0.24
8	Sierra Madre, Vasquez Rocks	VAS090	5.6	39.8	38	4.2	0.48
9	Molise, Sannicandro	B-SCO000	5.7	51.3	40	12	0.36
10	ValDesBois, Innes Road_ ON	CN.ORIO.HHE	5.1	52.9	45	10.1	0.14
11	Saguenay, US.DCKY	US.DCKY.HHE	5.9	192	29	4.8	0.12
	NF Average	…..	5.3	13.5	23.2	…..	…..
	FF Average	…..	5.3	49.4	33.2	…..	…..

It should be mentioned that the interference of different source mechanisms, such as directivity effects and focal mechanisms (strike-slip, normal, or reversing faulting), is not considered during the selection of records. Additionally, the vertical component of the ground motions is not considered, and only the longitudinal direction is investigated in this study.

3.7.2 Analysis programme and procedure

All calibrated records are input at the base of the conventional and isolated bridge variants, first without considering the presence of soil where the base of the bridge is fixed and then with modeling the soil using the direct approach and simplified method.

The bridge variants are analyzed by NLTHA in Abaqus software, first for the static gravity dead load to obtain initial stress conditions and then for dynamic loading conditions. Dynamic loading is started by attaining the acceleration to the base of the model. The structural responses of NLTHA, including the maximum acceleration on top of the deck, the maximum displacement of the bridge deck and isolation system, and the maximum base shear, are studied as seismic demands. Results are discussed in the following sections.

3.8 Results and Discussion

SSI effects on the seismic responses are studied and discussed in the following sections by comparing seismic demands, due to NF and FF records, for the conventional and isolated bridge variants located on Rock, Soil-C, and Soil-D, using the direct and simplified methods.

3.8.1 Effect of earthquake characteristics and SSI on the acceleration responses

The spectral accelerations for the higher and lower PGA/PGV ratio in both NF and FF records and for the average of NF and FF records are shown in Figure 3.6. The Spectral acceleration on different soils and earthquake records indicate that the response spectra show amplification in Soil-C and Soil-D in the range of short periods. However, the extent of the amplification faded with increasing the period leading to a minor difference in the responses at long periods.

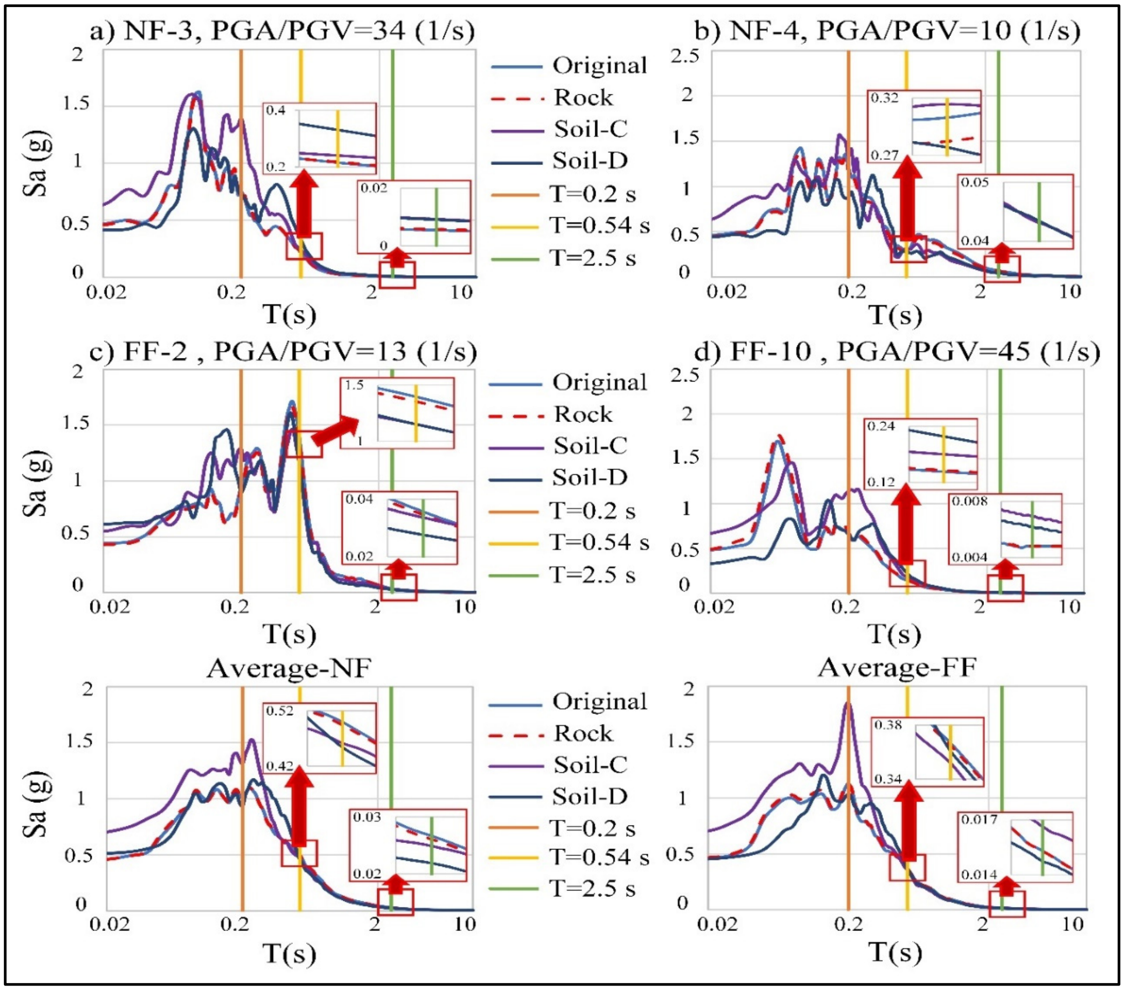

Figure 3.6 Spectral accelerations captured on different soils with the maximum and minimum PGA/PGV ratios for NF and FF and the average spectrum for all records

In addition, the amplification is observed at the predominant period of each earthquake record, making higher responses on the Soil-C and Soil-D in case of earthquakes with the period close to the conventional bridge. It is worth mentioning that the intensity and the duration of the amplification strongly depend on each individual record carrying its own characteristics and frequency content. Figure 3.7 shows the correlation of the maximum spectral acceleration response with the PGA/PGV ratio for different soil conditions for conventional and isolated bridge variants obtained from the direct and simplified approach. The common tendency of the responses is that the maximum acceleration responses in both conventional and isolated bridges decrease with increasing the PGA/PGV ratio.

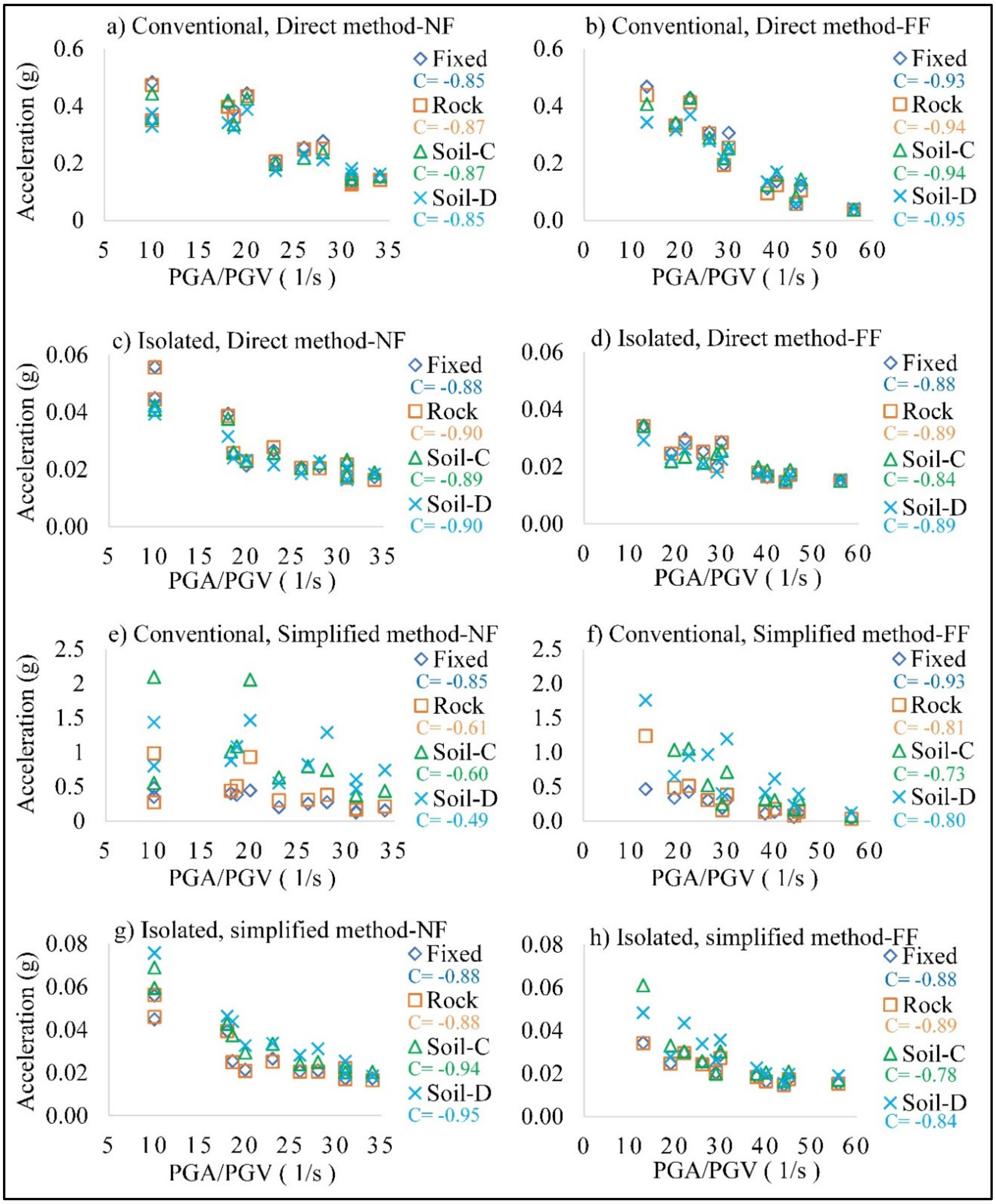

Figure 3.7 Absolute maximum acceleration responses vs. PGA/PGV ratios
(C= Correlation coefficient from Anova)

In conventional bridge and for the direct method, in the majority of earthquake records, the acceleration responses are decreasing from Rock to Soil-D, and they are less than the fixed base bridge by an average of 3%, 4%, and 11% for NF records and 6%, 1%, and 8% for FF records, on Rock, Soil-C, and Soil-D, respectively. The differences in responses of the fixed base bridge and the bridge on rock can be explained by the different behaviour of the boundary condition at the base of each bridge.

In the case of the isolated bridge and for the direct method, the responses are attenuated and less scattered despite the fluctuation in acceleration responses for different soils. On average, the acceleration responses are decreasing from the fixed-base bridge to softer soils by 1%, 4%, and 10% for Rock, Soil-C, and Soil-D for NF records and 1%, 2%, and 10% for FF records, respectively. In addition, NF records which mostly have a lower value of PGA/PGV, cause higher acceleration responses than FF records by an average of 30%, 32%, 25%, and 25% for the conventional bridge and 28%, 29%, 25%, and 26% for the isolated bridge in case of fixed-based, Rock, Soil-C, and Soil-D, respectively.

In the conventional bridge and for the simplified method, a significant increase in acceleration responses is observed compared to the fixed-base bridge in most cases. The average of the increasing trend in Rock is 46%, but it increases drastically up to about 200% (~ 3 times) on Soil-C and Soil-D for both NF and FF records.

On average, responses of the simplified method for the isolated bridge show a good agreement on the Rock with a difference of up to one percent, and the increasing trend of 23% and 40% for Soil-C and Soil-D, in NF records and 20% and 27% for Soil-C and Soil-D, in FF records, respectively. Besides, NF records cause higher acceleration responses than FF records by an average of 31%, 11%, and 32% for the conventional bridge and 28%, 30%, and 38% for the isolated bridge on Rock, Soil-C, and Soil-D, respectively.

For both conventional and isolated bridges, the PGA/PGV ratio of the records has an important effect on the maximum acceleration responses for all soil types, and the distance of the ruptured fault is not an effective factor. The general trend shows that on softer soils, when the period of the structure increases, in the direct method for both conventional and isolated bridges, there is a reduction in the acceleration responses on the inclusion of SSI, and the reduction is more pronounced for the conventional bridge. This trend is the opposite in the simplified method because no damping is defined in the system.

To study the effect of soil, all maximum acceleration responses are normalized by the responses of the fixed-base condition, and the results are shown in Figure 3.8. By this normalization, if the ratios are close to one, the soil effect is neutral, and SSI does not play a significant role in modifying the responses. In the case of a ratio of more than one, responses are amplified, and the effect of SSI is not positive. Conversely, when the ratio is negative, SSI plays a favorable role in modifying and reducing the responses.

In the conventional bridge and the direct method, for most of the NF records, SSI is favorable, and it decreases the acceleration on softer soils. In contrast, there is no constant trend in FF records, and SSI plays a positive and negative role in different records.

In the isolated bridge and the direct method under both NF and FF records, the ratio tends to be close to one or less than one in most records. Therefore, the presence of soil plays a neutral or favorable role in modifying the isolated bridges' acceleration responses, which agrees with the fact that isolated bridges are less sensitive to the SSI effects (Dezi et al., 2012).

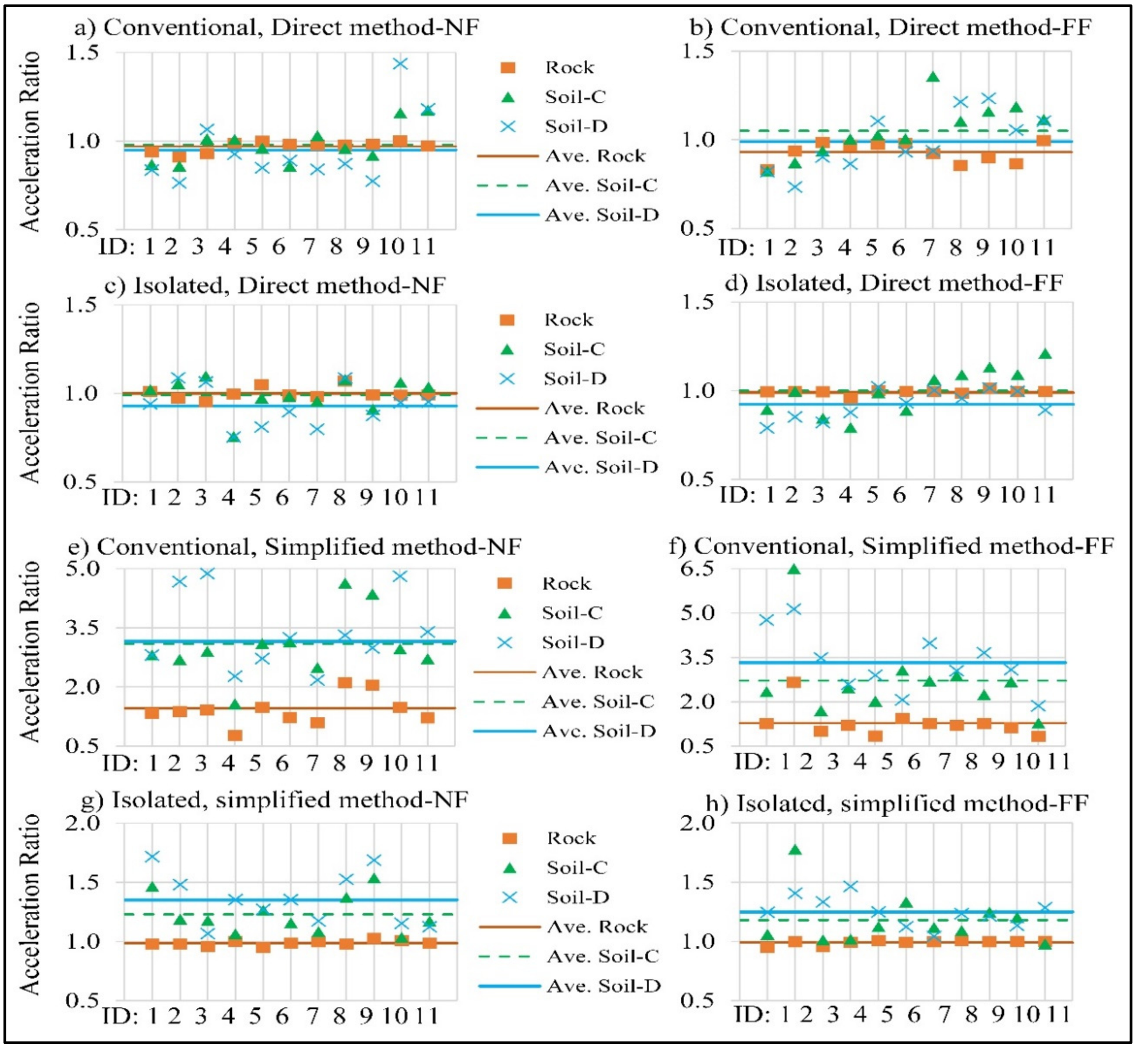

Figure 3.8 Acceleration ratio (SSI/Fixed-base)

In the simplified method using in this study and for both conventional and isolated bridges, SSI has a negative effect and amplifies the responses, while the increasing factor is more in conventional bridges. Table 3.8 shows the differences between the average dynamic responses in the simplified and direct methods for the conventional and isolated bridges. The percentage of the differences shows that the sensitivity of the conventional bridge to the selected method for Soil-C and Soil-D is more than the isolated bridge. At the same time, the isolation system controls the scattering of the responses.

Table 3.8 The difference between the average responses of the simplified method compared to the direct method

	NF			FF		
	Rock	**Soil-C**	**Soil-D**	**Rock**	**Soil-C**	**Soil-D**
Conventional Bridge:						
Acceleration (g)						
Direct method	0.28	0.28	0.26	0.21	0.23	0.21
Simplified method	0.43	0.92	0.92	0.33	0.74	0.7
Difference (%)	54	229	254	57	222	233
Base Shear (W_d)						
Direct method	0.274	0.277	0.26	0.211	0.229	0.209
Simplified method	0.437	0.932	0.934	0.333	0.708	0.711
Difference (%)	60	236	260	58	209	240
Deck drift (mm)						
Direct method	23	25.4	25.6	16.5	19.9	20
Simplified method	32	69.2	67.8	24.5	62.1	51.7
Difference (%)	39	172	166	49	211	158
Isolated Bridge:						
Acceleration (g)						
Direct method	0.029	0.027	0.025	0.022	0.022	0.02
Simplified method	0.028	0.035	0.039	0.021	0.027	0.028
Difference (%)	-1	29	54	-3	23	41
Base Shear (W_d)						
Direct method	0.028	0.029	0.022	0.023	0.023	0.017
Simplified method	0.022	0.026	0.025	0.019	0.019	0.018
Difference (%)	-21	-9	12	-18	-20	5
Isolation displacement, piers (mm)						
Direct method	34.2	43.6	48.3	20.5	33.6	37.8
Simplified method	33.4	47.6	56.6	19.9	32.5	33.4
Difference (%)	-2	9	17	-3	-3	-11
Isolation displacement, abutments (mm)						
Direct method	38.2	102.5	109.9	23.3	96.4	96.1
Simplified method	34.4	49.1	58.6	20.3	31.2	34.6
Difference (%)	-10	-52	-47	-13	-68	-64

Table 3.9 shows the comparison of site coefficients, F(T), in CSA (S6-19) with the average responses of this study with respect to the fact that in CSA (S6-19), soil C is considered as a reference with F(T)=1. Results demonstrate that in the conventional bridge, the site coefficients for rocks proposed by CSA (S6-19) could lead to underestimation of responses as the F(T) in the direct method is two times higher than the proposed value in CSA (S6-19).

On the other hand, the site coefficient in Soil-D is less than the factor suggested by CSA (S6-19) by an average of 21% and 19% for NF and FF, respectively. In the case of the simplified method, responses are in good agreement for Rock and Soil-D by the maximum difference of 4 %, respectively. It should be noted that in the isolated bridge, the responses of different soils in the direct method are almost the same for NF and FF records, and all the factors are close to one.

Table 3.9 Comparison of site coefficient F(T)

	Rock		**Soil-C**		**Soil-D**	
	NF	**FF**	**NF**	**FF**	**NF**	**FF**
Conventional Bridge:						
CSA (S6-19)	0.48	0.48	1	1	1.18	1.18
Direct method	1.01	0.9	1	1	0.93	0.95
Simplified method	0.46	0.49	1	1	1	1.33
Isolated Bridge:						
CSA (S6-19)	0.4	0.4	1	1	1.35	1.35
Direct method	1.02	1.01	1	1	0.93	0.93
Simplified method	0.82	0.86	1	1	1.1	1.09

The same pattern of underestimating the responses on Rock more than two times appears in the isolated bridge in both direct and simplified methods. In addition, the site coefficient in Soil-D is less than the factor suggested by CSA (S6-19) by an average of 31% and 19% for both direct and simplified methods, respectively. Therefore, the site coefficient proposed for Rock in CSA (S6-19) might lead to underestimating the responses. In contrast, the increasing factor on soft soils is conservative for both NF and FF records.

3.8.2 Effect of earthquake characteristics and SSI on the displacement responses

The maximum displacement responses and the effect of PGA/PGV on different soil conditions and bridges are shown in Figure 3.9. For the conventional bridge and direct method, the maximum deck drift is related to the earthquake records with the lowest PGA/PGV ratio in NF and FF records and earthquake records with the predominant period close to the conventional bridge such as NF:8 and FF:2 with the period of 0.48 s and 0.46 s, respectively. On average, the deck drift decreases from the fixed-base bridge to Rock by 7% and 5% in NF and FF records which could be because of the differences in their boundary conditions. With some exceptions, the whole pattern of the maximum deck drift is an increasing trend from Rock to softer soils in both NF and FF records. Compared to the fixed-base bridge, the maximum deck drift is increasing by 2% and 3% for Soil-C and Soil-D under NF records and by 15% and 16% increase for Soil-C and Soil-D under FF records, respectively.

In the conventional bridge and simplified methods, because there is no damping in the system and springs are acting in linear behaviour, the deck drift increases drastically, up to 3 times more than the fixed-base bridge for both NF and FF records. In addition, NF records cause higher deck displacement than FF records in the conventional bridge by an average of 44%, 40%, 27%, and 28% in the direct method and 44%, 30%, 11%, and 31% for the simplified method in the case of fixed-based, Rock, Soil-C, and Soil-D, respectively.

In the isolated bridge and direct method, the maximum isolator displacements on piers significantly increase from Rock to softer soil compared to the fixed-base bridge for both NF and FF records. On average, there is a 3%, 30%, and 45% increase in NF records and a 3%, 69%, and 90% increase in the direct method in FF records for Rock, Soil-C, and Soil-D, respectively.

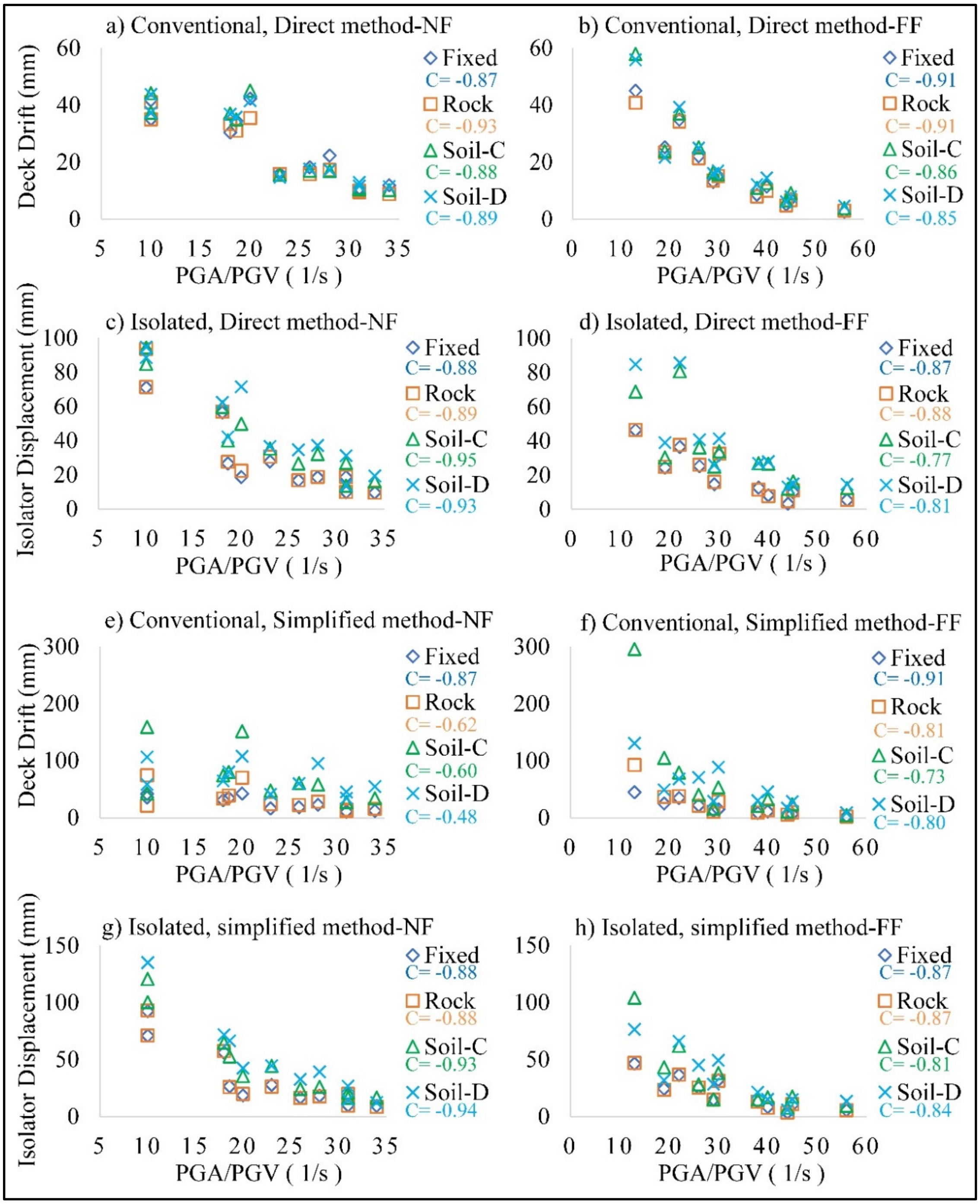

Figure 3.9 Absolute maximum displacement responses vs. PGA/PGV ratios

(C= Correlation coefficient from Anova)

In the isolated bridge and simplified method, the same increasing trend in isolator displacement is observed by the average of 1%, 43%, and 67% in NF records and 1%, 63%, and 68% in FF records for Rock, Soil-C, and Soil-D, respectively. It should be noted that the isolator displacement of two NF records with the lowest PGA/PGV ratio is higher than the designed displacement in all bridge cases, regardless of the presence of the soil in both methods.

In addition, the isolator displacements of NF:7 and NF:8 on Soil-D are higher than the designed displacement. In FF records, the same trend happens in the record with the lowest ratio of PGA/PGV in both methods. Additionally, the isolator displacements of FF:2 and FF:4 on Soil-C and Soil-D are higher than the designed displacement in both methods. The maximum isolator displacement for abutments shows a higher response than piers up to 3 times in both methods. The considerable difference between the isolator displacement of the piers and abutments shows the importance of considering the soil effect in the design stage to fulfill the displacement demand based on the records characteristics and soil category. Moreover, NF records cause higher isolator displacement than FF records by an average of 40%, 40%, 23%, and 22% for the direct method and 40%, 40%, 32%, and 41% for the simplified method in the case of fixed-based, Rock, Soil-C, and Soil-D, respectively.

The correlation between PGA/PGV ratios and the maximum displacement responses on both conventional and isolated bridges, as shown in Figure 3.9, indicates that the PGA/PGV ratio of the records has an important effect on the dynamic responses for both NF and FF records for all soil types and the distance of the ruptured fault is not an effective factor in displacement responses.

Furthermore, as on softer soils, the period of the structure increases; in the direct method for both conventional and isolated bridges, there is an increase in the displacement responses on the inclusion of SSI, and the increase is more pronounced for the isolated bridge under NF records. This trend is the same for the simplified method. The normalized displacement responses ratio presented in Figure 3.9 shows that soil's positivity or negativity effects strongly depend on the record characteristics in the conventional bridge and the direct method.

For the isolated bridge and the simplified method, the pattern is almost the same, showing that soil does not play a positive role, and it increases the isolator displacement. Considering the average responses, SSI plays an unfavorable role in the isolator displacement responses, and isolated bridges are sensitive to the SSI effects. Therefore, these results agree with previous studies mentioning that isolated bridges on soft soils have a more significant potential for severe damage, while the isolation systems perform better on rocks during earthquakes (Alam & Bhuiyan, 2013b; Dicleli & Buddaram, 2006).

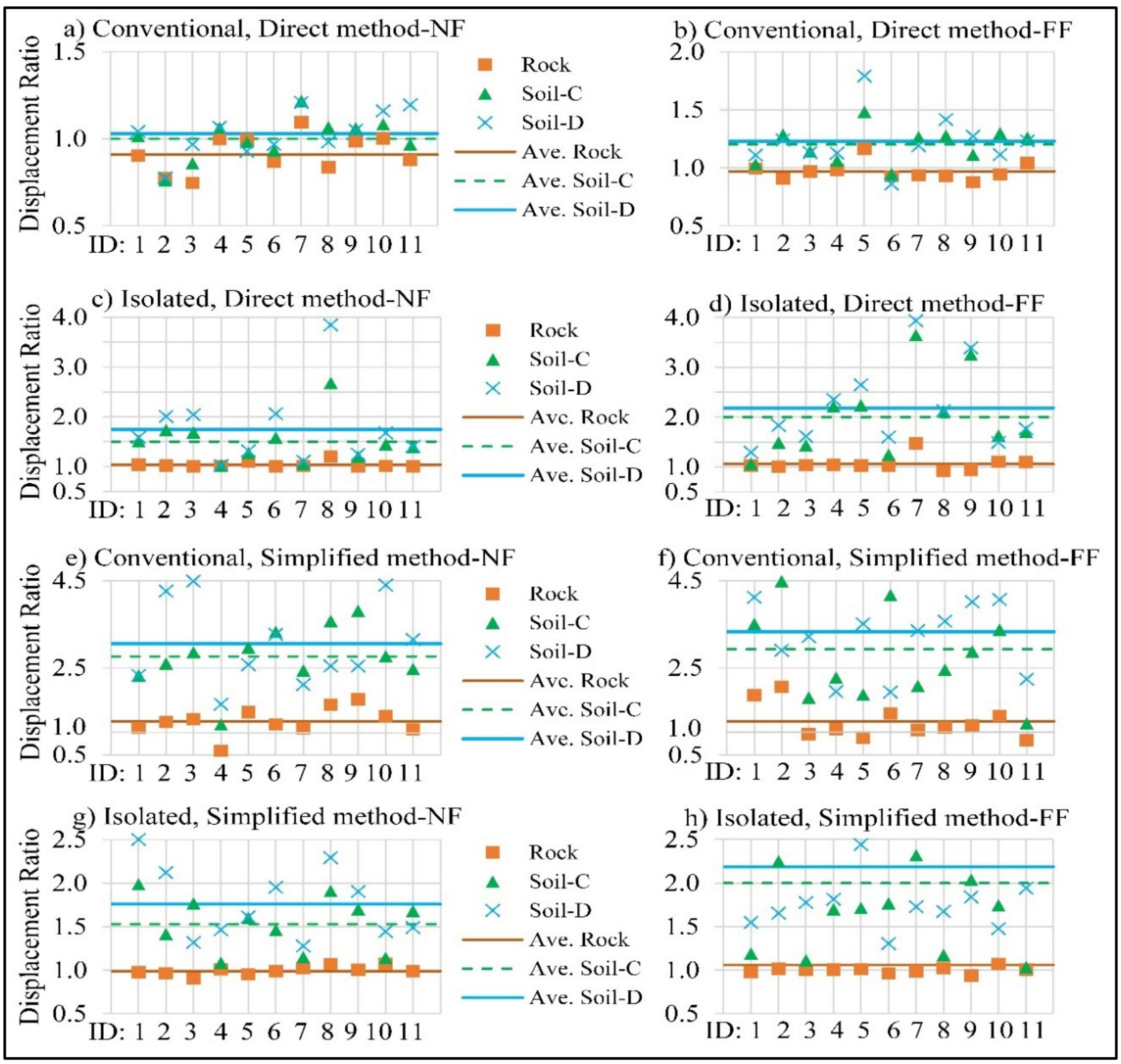

Figure 3.10 Displacement ratio (SSI/Fixed-base)

3.8.3 Effect of earthquake characteristics and SSI on the base shear responses

The maximum base shear responses and the effect of record characteristics on the responses are shown in Figure 3.11. In conventional bridges and the direct method, there is no constant trend for the maximum base shear responses, so in a few records, the diminishing of the responses is observed on softer soils, while in other records responses are increasing and are higher than fixed-based bridge depends on each individual record and its characteristics, especially in the case of Soil-C.

The maximum base share is related to the records with the lowest PGA/PGV and records with a predominant period close to the period of the conventional bridge (NF:8 with T_P=0.48 s and FF:2 with T_P=0.46 s). In the conventional bridge and the simplified method, as there is no damping in the system and also due to the linear behaviour of the springs, an increasing trend is observed from Rock to softer soils in all records.

Considering the average of the base shear responses, there is a reduction in responses of the direct method by 12%, 10%, and 16% for NF records and 10%, 2%, and 11% for FF records on Rock, Soil-C, and Soil-D compared to the fixed-base bridge, respectively. On the other hand, for the simplified method, an increase of 42%, 202%, and 202% is observed for both NF and FF records on Rock, Soil-C, and Soil-D compared to the fixed-base bridge, respectively. These observations are in agreement with the results of literature showing that depending on the details of the individual ground motions and their correlation to the dynamic properties of the pier and foundation, SSI could increase or decrease the base shear responses, and it should specifically be considered for design purposes as ignoring the observed increases in pier shear due to SSI will cause severe damage to the structure (Makris & Zhang, 2004; Ucak & Tsopelas, 2008). A comparison of responses of the conventional bridge for NF and FF records shows that NF records cause higher base shear responses compared to FF records by an average of 30%, almost in all cases. In addition, some individual records show an increasing trend in their base shear responses, up to 32% in NF records and 43% in FF records on Soil-D compared to the fixed-base bridge.

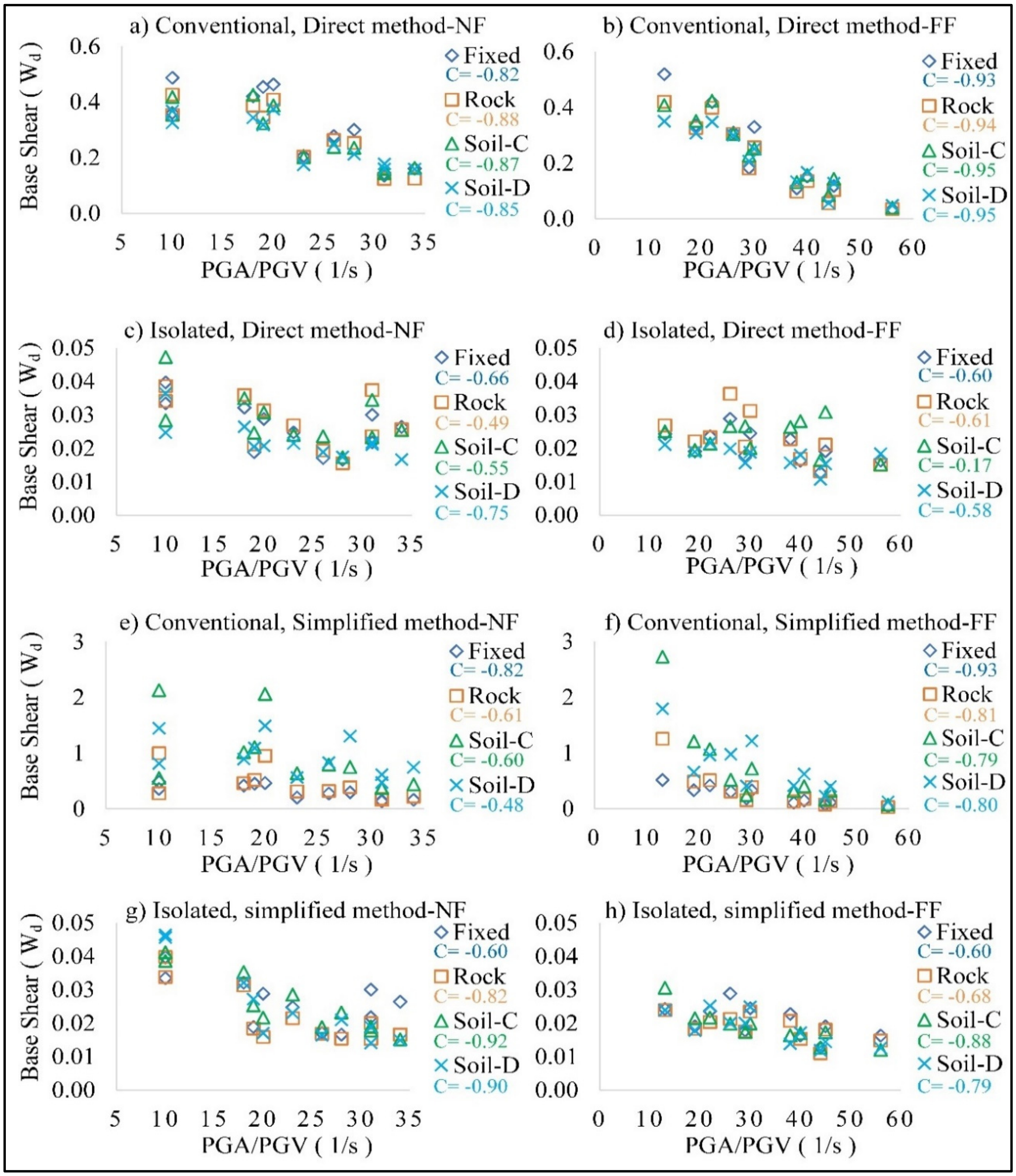

Figure 3.11 Absolute maximum base shear responses vs. PGA/PGV ratios

(C= Correlation coefficient from Anova)

In the isolated bridge and direct method, maximum base shear responses increase in most of the records on Rock and Soil-C for both NF and FF records. However, it shows a decreasing trend for Soil-D. Considering the average of the maximum base shear responses for NF and FF records show that the base shear is increasing on Rock and Soil-C by 7% and 9% for NF and 11% and 14% for FF, and then it reduces on Soil-D by 15% and 5% for NF and FF records, respectively.

In the isolated bridge and simplified method, there is a reduction of 15%, 2%, and 4% for NF records and 10%, 8%, and 10% for FF records on Rock, Soil-C, and Soil-D, respectively. In addition, NF records cause higher base shear than FF records in the isolated bridge by an average of 29%, 25%, 23%, and 27% in the direct method and 29%, 20%, 38%, and 37% in the simplified method. It should be mentioned that some individual records show an increase in their base shear responses up to 40% in NF records and 70% in FF records on Soil-C compared to the fixed-base bridge.

As shown in Figure 3.11, the maximum base shear responses in the conventional and isolated bridges, for both NF and FF records, are related to the record with the lowest PGA/PGV ratio regardless of the soil effect and decrease with the increasing ratio of PGA/PGV.

In addition, the effect of record frequency is less in the isolated bridge, especially for FF records. The normalized base shear ratio in Figure 3.12 shows that in the conventional bridge and direct method, soil has a positive role in reducing the base shear in the majority of NF records, while soil effect could be favorable or unfavorable in FF records, depending on their characteristics. Base shear responses in the simplified method show that the soil has a negative effect, and softer soils increase the base shear responses. In the isolated bridge and direct method, the base shear responses are more positive in Soil-D, while the trend is unclear for Soil-C. On the other hand, in the isolated bridge and simplified method, soil has an unfavorable effect on NF records, and soil effect is less pronounced for FF records and has almost a neutral effect.

In conclusion, the analysis of variance (ANOVA) was implemented to acquire the effect of PGA/PGV ratios and seismic responses of the bridges by calculating the linear correlation coefficient (shown in Figure 3.7, Figure 3.9, and Figure 3.11) and the probability of error (p-value). As a result, a strong correlation exists between PGA/PGV and maximum deck acceleration and the maximum base shear with the p-values much lower than 0.05 for all soil types and also for both approaches, direct and simplified methods, which is the threshold for the 95% confidence level.

However, depending on the analysis and modeling methods, the correlation between PGA/PGV is divergent for the maximum displacement responses. They are proven to be statistically significant only for Soil-C and Soil-D in the direct method and for fixed-base and rock in the simplified method.

It should be mentioned that in order to thoroughly investigate the effect of SSI on conventional bridges with different flexibility, a conventional bridge with the period of 0.2 (s) is modeled using the direct method, and the results are compared with the original conventional bridge with the period of 0.54 (s). As it is shown in spectral acceleration in Figure 3.6, there is a noticeable amplification in Soil-C at the period of 0.2 (s) in all records, and the responses of the stiffer bridge with the period of 0.2 (s), shown in Figure 3.13, is higher than the bridge with 0.54 (s) period except for NF:8 and FF:2 and it can be explained by the fact that the predominant period of the records are at the proximity of the period of the bridge and resonance is taking place in these records.

The maximum acceleration and base shear of the bridge with 0.2 (s) period is higher than the bridge with the period of 0.54 (s) by the average of 50% and 70% for NF and FF records. The deck drift of the stiffer conventional bridge is less than the bridge with 0.54 (s) by an average of 73% and 65% for NF and FF records. The responses of the stiffer conventional bridge are less sensitive to the PGA/PGV ratio. On average, softer soil (Soil-D) plays a positive role in decreasing the acceleration and base shear responses but increases displacement demand.

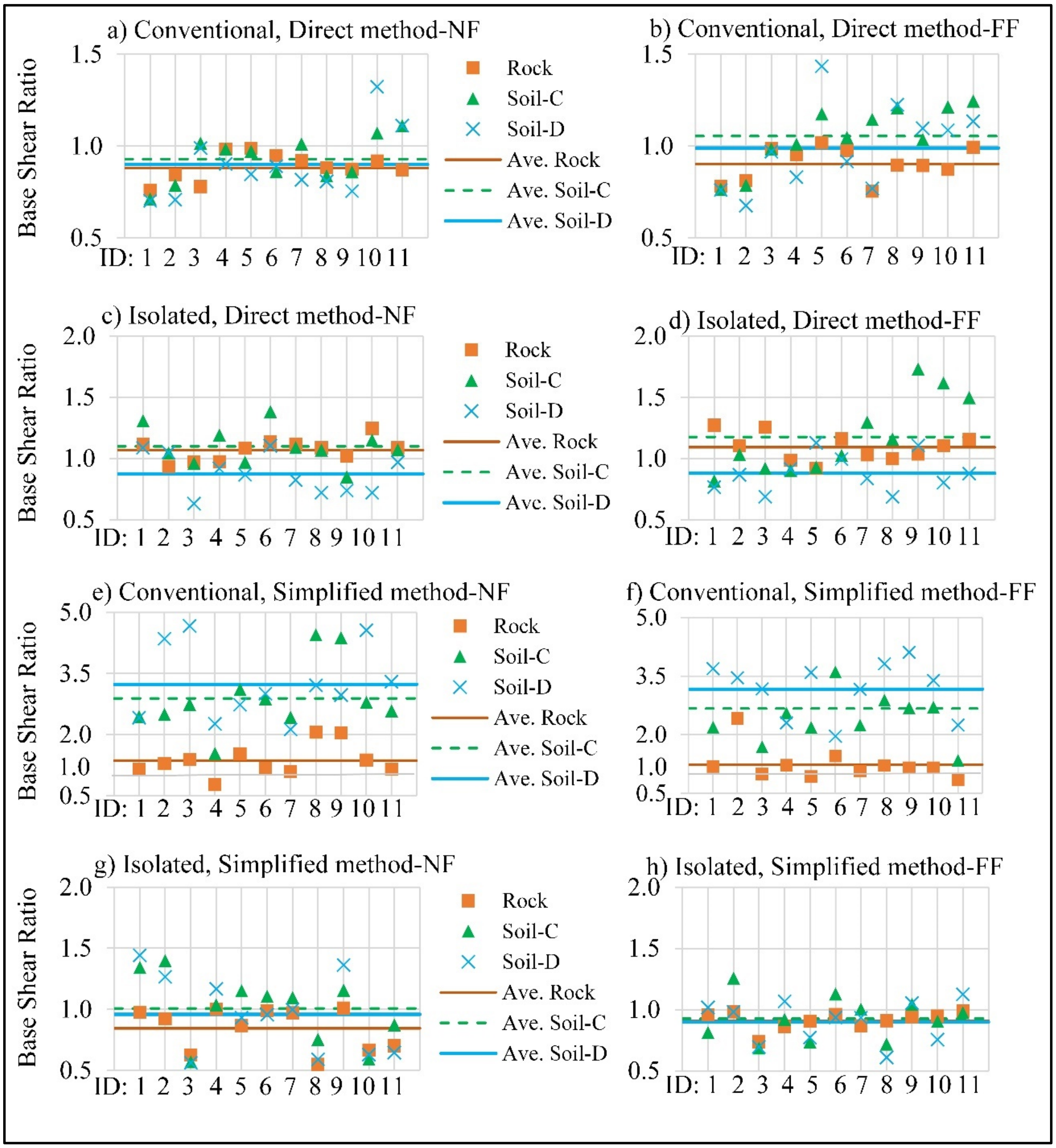

Figure 3.12 Base shear ratio (SSI/Fixed-base)

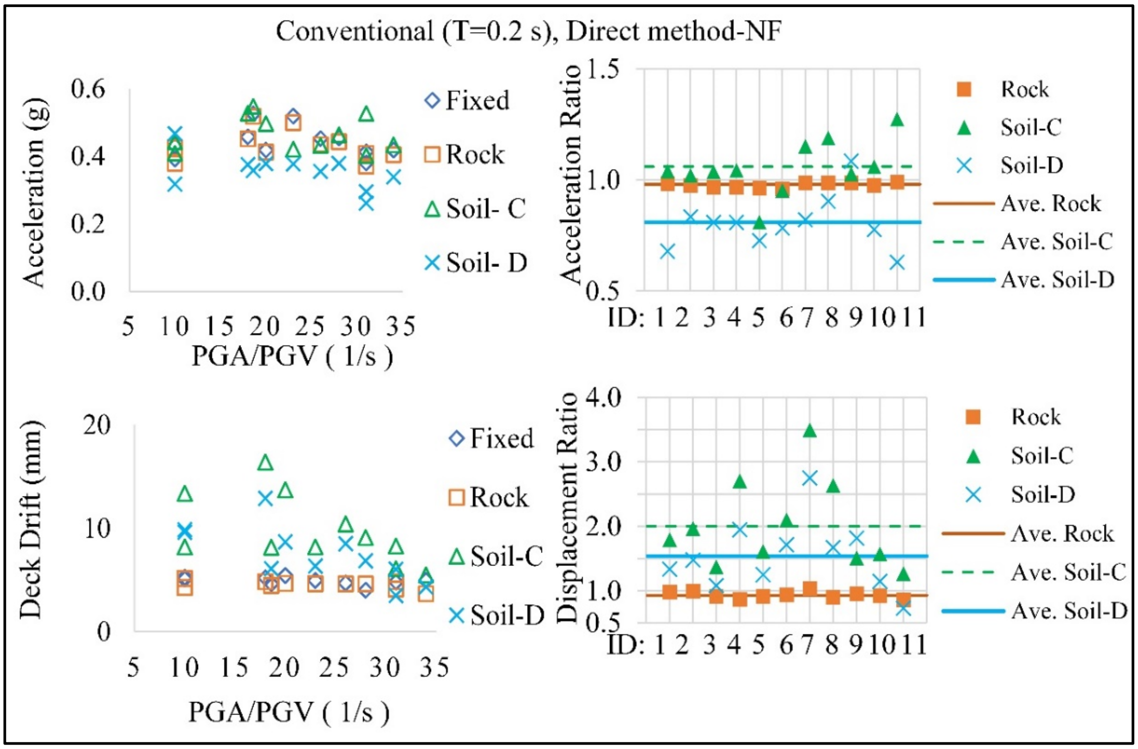

Figure 3.13 Responses of the conventional bridge with T=0.2 (s)

3.9 Conclusion

This paper studied the simultaneous effects of NF and FF record characteristics and SSI effect on a three-span bridge with and without the isolation system. Seismic responses of the fixed-base bridge are compared by considering the soil domain and SSI effects in the direct approach and simplified method. Three different soil properties representing Rock, Soil-C, and Soil-D have been selected. The role of soil characteristics has been evaluated by considering the bridge founded on different soil strata subjected to moderate NF and FF records. Responses of NLTHA lead to the following conclusions:

1. The fault distance does not play a decisive role in the dynamic responses of the bridge, and dynamic responses significantly depend on the low or high-frequency contents of the records, regardless of the soil type, so the lower ratios of PGA/PGV cause the higher dynamic responses, and they diminish with increasing this ratio.

2. With the same PGA, NF records which generally contain higher values of PGV, cause higher dynamic responses compared to FF records for both conventional and isolated bridges on different soils.

3. A considerable increase in the base shear response is observed in softer soils in a few individual records. For example, in the conventional bridge, there is a 43% increase in the maximum base share response on Soil-D for FF:5 compared to the fixed-base bridge (increasing by 0.015Wd, from 0.034Wd to 0.049Wd), while there is a 70% increase from the fixed-base isolated bridge to the isolated bridge on Soil-C (increasing by 0.012Wd, from 0.016Wd to 0.028Wd) in the case of FF:9. In fact, the base shear response is dominated by the uncertainty in the ground motion. Therefore, careful attention is needed at the design stage to anticipate the base shear demand depending on the frequency content, bridge condition and the underlying soil properties.

4. In the isolated bridge and direct method, the maximum isolator displacements happen in the records with the lowest ratio of PGA/PGV. However, they increase drastically from Rock to Soil-C so that the displacement demand goes beyond the designed displacement. Therefore, ignoring the effect of the flexibility of soil and the SSI effect will result in underestimating the displacement demand of the isolated bridge and the possibility of destruction in the isolation system in prone areas.

5. The isolator displacement demand at the abutments is higher than the piers by up to 3 times in some individual records. Therefore, it should be carefully designed considering the soil effects and the characteristics of the records.

6. As F(T) in the direct method is higher than the reduction factor proposed by CSA (S6-19) by the factor of 2.5, the site effect could lead to an underestimation of responses for rocks while it is conservative for soft soils.

7. Using the simplified method in this study should be alongside careful attention to the validity of using the equivalent linear method instead of the nonlinear meth-od. Because all the records on Soil-C and Soil-D were not eligible based on the limitation of the shear strain index (generally under 0.03%), and the responses were very scattered, especially in the conventional method. Therefore, the simplified method of using springs to represent the soil stratum is rather a simple approach to capture all the major mechanisms involved in soil, SSI, and characteristics of each earthquake ground motion.

It should be mentioned that regarding the specific objectives of the current research study and to limit computation and analysis efforts, this study did not consider many aspects such as spatial variation and non-coherence of the seismic motions, multi-directional seismic excitation, Analysis of base-isolated bridges at different levels of the hysteretic properties, notably at lower and upper bound values, the possibility of inelastic deformations within the bridge foundation units (piers and abutments) for both fixed-base design and isolated design, that could strongly impact the obtained results in different perspectives. Therefore, considering these aspects is worth further investigation.

CHAPTER 4

SOIL-STRUCTURE INTERACTION AND NEAR FAULT PULSE-LIKE EARTHQUAKES EFFECTS ON SEISMIC RESPONSES OF ISOLATED BRIDGES

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4.1 Abstract

Seismic isolation technology is an effective tool for mitigating seismic risk and improving structural performance during strong earthquakes. However, some parameters, such as earthquake and soil characteristics, influence and may reduce isolation technology's performance. This research aims to investigate the simultaneous effects of soil-structure interaction (SSI) and pulse-like earthquakes on the seismic responses of conventional and isolated bridges. Near-fault (NF) earthquakes with and without velocity pulses in their records are applied to the structure of a three-span bridge located in Vancouver (Canada), with and without considering the underlying soil. Using the direct method, three soil properties representing rock, stiff and medium soil are modeled by Abaqus software. Nonlinear time history analysis (NLTHA) is carried out, and structural responses regarding maximum deck acceleration, base shear, and displacement of the deck and the isolation systems are studied. Results demonstrate that pulse-type records cause higher seismic responses, and soil presence diminishes the negative effect of the pulse on the force demands. On average, and for the pulse-

like records, the softer soil reduces the acceleration by up to 30% and base shear responses by up to 25%, while increasing the displacement demand of conventional and isolated bridges by up to 80%. Therefore, careful attention should be paid to the isolation systems' design to prevent underestimating the displacement demand for pulse-like records, especially on softer soils. Responses of the different isolation systems demonstrate that the optimum design could provide the displacement demand for pulse-type records even on softer soils.

Keywords: Seismic isolation, Soil-Structure Interaction, Near-Fault records, Pulse-type records, Bridges.

4.2 Introduction

As natural disasters, strong earthquakes may cause devastating effects on seismic-prone areas. Seismic isolation systems are one of the rational and fundamental solutions for mitigating the effects of earthquakes with a significant positive effect on reducing the seismic responses of structures, as indicated by numerous post-earthquake in-field observations, experimental, and numerical research works (Tongaonkar & Jangid, 2003; Tsopelas et al., 1996).

This technology has proven a good performance even in the case of not considering all the effective parameters like NF effects for the prone areas; for instance, in the case of the Bolu Viaduct bridge, the isolation system suffered a complete failure and narrowly avoided the total collapse because of excessive superstructure movement in the Duzce Earthquake in 1999 as an NF earthquake which caused large displacements in the isolation system (Roussis et al., 2003).

Seismic isolation is based on reducing the fundamental structural vibration frequency to a value less than the predominant energy-containing frequencies of the earthquakes to decrease the seismic force demand to or near the elastic capacity of the structure; thereby, inelastic deformations within the structure will be obliterated or drastically diminished, and they take place in the isolation devices. The long-term advantage of these innovations is that they preserve the structure's serviceability following an earthquake, reducing the socio-economic losses and the cost of reconstruction (Guizani & Chaallal, 2011).

Serving as a crucial artery in transportation systems, bridges are one of the most critical infrastructures in today's modern society, especially in times of crisis, such as the period following a major earthquake. Therefore, it is required to consider all effective parameters at the design stage to ensure an adequate bridge design according to the target performance. Among different pivotal parameters, earthquake characteristics, and site conditions are two of the most critical parameters affecting the seismic performance of infrastructures (Ucak & Tsopelas, 2008).

Ground motion records close to the ruptured fault (within 20 km) are categorized as NF earthquakes (Bray & Marek, 2004). Seismic responses of structures between NF and FF records differ considerably. Many research studies reported that NF pulse-like ground motions are more destructive to the structure than that ordinary ground motions (Jia et al., 2023; Jiao et al., 2021). NF records often have a higher PGV/PGA ratio. Frequently, they contain intense and long-period velocity pulses, which force the structure to behave in an inelastic range that may require much higher ductility demand and base shear than FF earthquakes, and the impulse effect may intensify the displacement of the isolation bearing (Ismail et al., 2014; W. I. Liao et al., 2000; Neethu & Das, 2019).

In addition, NF records particularly amplify the seismic responses of isolated bridges when the pulse period is close to the period of the structure (Malhotra, 1999b; Shen et al., 2004) and hysteretic damping of the isolation systems might not be effective in dissipating the energy process in the first part of the pulse (Priestley et al., 2007), therefore the structure is prone to severe damage when the duration of the pulse is larger than the natural period of the structure (Anderson & Bertero, 1987). Consequently, the demand in the isolation system depends on the pulse duration and the ratio of pulse to the natural period of the structure (Chai & Loh, 1999) and in order to have an optimum isolation system, the characteristic strength (Q_d) of the isolation system, defined later, needs to be increased (Chai & Loh, 1999; Dicleli & Karalar, 2011).

The local site and SSI can also significantly influence the main characteristics of ground motions, such as amplitude, frequency content, and duration, and modify the seismic responses of isolated bridges. The extent of such influence depends on the dynamic characteristics of the bridge structure, the characteristics of the input ground motion, and the properties of the underlying soil (Chouw & Hao, 2008; Rayhani et al., 2008). Misrepresenting the soil effect could results in an erroneous estimation of the seismic demand and the parameters governing the design of the isolation system and the bridge especially where the underlying soil is soft (Chaudhary et al., 2001; Kulkarni & Jangid, 2003; Saritas & Hasgur, 2014; Stehmeyer & Rizos, 2008; Fatahi et al., 2014; Worku, 2014).

Aside from various conclusions drawn from the literature, little attention was paid to the effect of SSI and pulse-like earthquake records on the performance of the isolated bridges for prone areas. As a result, this research aims to look into the simultaneous effects of NF records with and without pulses in their velocity records and the effect of different soil properties on the bridge responses. Consequently, the efficiency of different isolation systems subjected to the above-mentioned situations will be investigated. The results will help to reach a more advanced comprehension of the responses of isolated bridges located on different soil strata. This understanding allows for more precise and effective isolation strategies by designing appropriate properties of the isolation systems in future projects for prone areas, when necessary, to catch the SSI and pulse effects.

4.3 Modeling of the case-study bridge

The selected case study bridge model is a typical three-span continuous deck highway bridge studied by Jangid et al. (2003), shown schematically in Figure 4.1 (Tongaonkar & Jangid, 2003). The bridge is symmetric, with three equal spans supported on two concrete single piers and abutments with a fundamental period of 0.54 s in the longitudinal direction and a damping ratio of 5%, for the conventional (fixed-base) bridge with zero skew. Table 4.1 illustrates the geometric and material properties of the bridge based on the data presented in the reference studies (Tongaonkar & Jangid, 2003).

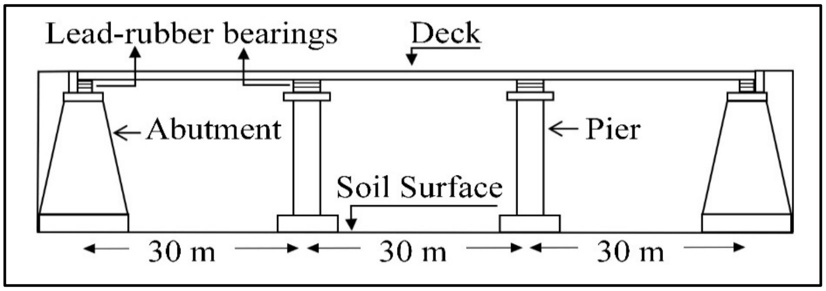

Figure 4.1 General elevation of the isolated bridge

Table 4.1 Material and dimension properties of the bridge

Properties of the Bridge	**Deck**	**Piers**
Cross-sectional areas (m2)	15.6	1.767
Length or height (m)	3@30	10
Modulus of elasticity (Gpa)	36	36
Mass density (kg/m3)	2400	2400
Compressive Strength (Mpa)	30	30
Poisson Ratio	0.2	0.2

In the present study, the structural modeling of the bridge and NLTHA are performed using Abaqus software (ABAQUS, 2019). Deck, piers, and abutments are modeled as Beam-column elements, and foundations and the soil stratum are modeled as solid elements. The superstructure, piers, and abutments are assumed to remain in the elastic state during seismic excitation for conventional and isolated bridges.

To validate the original model, a comparison of structural responses of the conventional bridge model and the results of reference papers for the Northridge record (captured at La County fire station component with PGA=0.58g) is carried out, and the results are presented in Table 4.2. Good agreements between the results, in terms of vibration period, base shear, and deck acceleration, are obtained with a difference lower than 5%.

Table 4.2 Comparison of the responses with current study

Responses	Jangid et al. (2003)	Present study	Difference (%)
Period (s)	0.53	0.54	1.85
Base Shear/W_{deck}	1.439	1.388	-3.54
Deck acceleration (g)	1.396	1.461	4.45

After validation of the model, the bridge is assumed to be in Vancouver, and the foundation is considered to be at a depth of D = 1.8 m of the soil surface.

4.4 Isolation system

Considering the bridge is located in Vancouver as a high seismicity area, three isolation systems as ISO-1 to ISO-3, are designed using the 6th generation seismic hazard model for Canada (NRCAN, 2022) for an effective period of T= 2.5 s. ISO-1 is calculated and designed based on the single-mode spectral analysis and spectral displacement demand for Vancouver. Based on the literature, earthquake records with low ratios of PGA/PGV, or earthquakes with pulses in their velocity records impose a larger strength and displacement demand (Dicleli & Karalar, 2011; Malhotra, 1999b), therefore, ISO-2 is designed with higher Q_d and displacement capacity (2 times) compared to ISO-1. Finally, ISO-3 is designed based on the proposed domain by Nguyen and Guizani (2021) to provide an optimal seismic isolation system for high seismicity areas with higher post-elastic stiffness and displacement capacity compared to ISO-2 to investigate the efficacy of the optimal design on dynamic responses of the bridge subjected to earthquakes with and without pulses in their velocity records (CSA, 2019; Nguyen & Guizani, 2021b).

The substructure is decoupled from the deck by lead rubber bearings, and the isolation system is lumped between the deck and substructure, and only the longitudinal direction is studied for implementing seismic isolation. Link elements with bilinear behaviour based on the multi-plastic model given by Abaqus are used to model the isolation system (ABAQUS, 2019). The global model of the isolated bridge and soil, and the bilinear force-displacement relation of the seismic isolation system (SIS), are shown in Figure 4.2 and the SIS hysteretic model parameters are presented in Table 4.3, where Q_d is the characteristic strength that is the force required at zero displacement, K_d represents the post-elastic stiffness, K_u stands for the elastic stiffness, K_{eff} is the effective stiffness at the maximum displacement in the isolation system, D_{max}, and an effective damping as β.

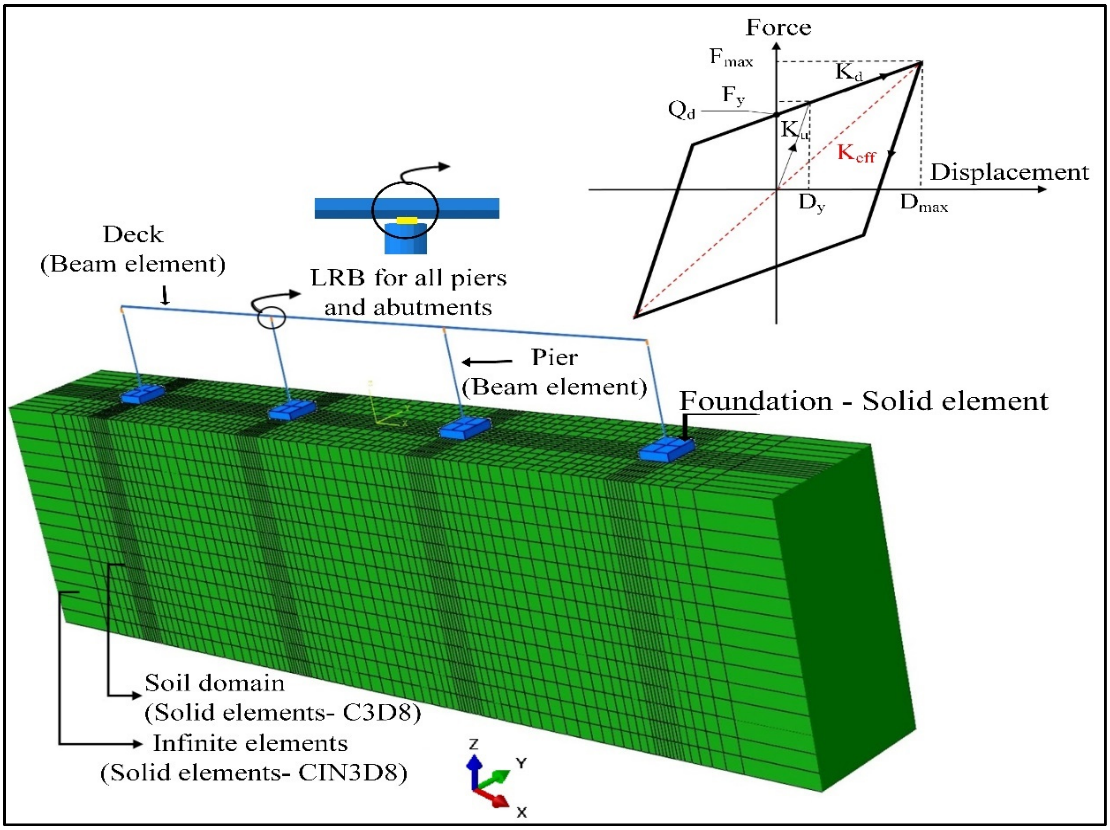

Figure 4.2 Isolated bridge model and bilinear force-displacement behaviour of SIS

Table 4.3 Isolation properties

ID	Location	T	K_{eff}	K_u	Q_d	K_d	D_{max}	β
		(s)	(kN/m)	(kN/m)	(kN)	(kN/m)	(mm)	%
ISO-1	Piers	2.5	7,750	228,250	450	3,240	100	35
	Abutments	2.5	3,550	101,500	200	1,550	100	35
ISO-2	Piers	2.5	7,750	453,250	900	3,240	200	35
	Abutments	2.5	3,550	202,000	400	1,550	200	35
ISO-3	Piers	2.5	7,895	455,700	900	5,700	400	20
	Abutments	2.5	3,775	202,800	400	2,800	400	20

4.5 Soil model and properties

Accounting for the effect of SSI, an elastic-perfectly plastic behaviour is assigned for the soil domain using the Mohr-Coulomb yield criterion (Labuz & Zang, 2012). The 8-node brick elements (C3D8) are applied to the soil deposit model as a rectangular shape of 130 (m) in length and 20 (m) in width. Three different non-liquefiable homogeneous soil profiles are adapted and studied as Rock, Soil-C (stiff soil), and Soil-D (medium soil) based on the site classification in CSA (S6-19) (CSA, 2019). In addition, considering the fact that most amplifications occur within the first 30 m of the soil profile, soil depth is considered to be 30 m (Rayhani & El Naggar, 2008). The characteristics of each soil type is presented in Table 4.4, where E is the Elastic modulus, ρ represents the density, C stands for the cohesion stress, ϑ is the Poisson's ratio, ø defines the friction angle, Vs is shear wave velocity, Ψ represents dilatancy, and ξ is the damping ratio.

Table 4.4 Mechanical properties of soils

Soil	**E (MPa)**	**ρ (kg/m3)**	**υ**	**C (KPa)**	**Ø (°)**	**V_s (m/s)**	**ψ**	**ξ (%)**
Rock	24960	2600	0.2	2.50E+04	48	2000	7	5
Soil-C	1323	2100	0.26	0	40	500	5	5
Soil-D	430	1900	0.32	0	35	300	4	5

To avoid the reflection of waves at the finite boundaries of the soil model, Infinite solid continuum as CIN3D8 with 8-node linear, as a one-way infinite brick element provided by Abaqus, are used in the longitudinal direction, which is the direction of the study, and fixed boundaries for the transverse direction with free rotations are used in this study. The earthquake acceleration records are directly applied to the grid points along the rigid base of the soil in the longitudinal direction.

The surface-to-surface contact between the foundation and the soil surface is modeled as an interaction interfacial behaviour following the algorithm implemented by Abaqus (ABAQUS, 2019). The interface stiffness values control the relative interface movement in the normal and tangential directions. Hard contact is used in the normal direction, and the nonlinear penalty method is defined for tangential behaviour. In tangential behaviour, based on practical cases and suggested domain in the literature intending to calculate a final friction coefficient (μ) of 0.5 is used (Dehghanpoor et al., 2019).

Overall, 1300 elements for the bridge and 23520 C3D8R, and 480 CIN3D8 elements for the soil domain are used. A mesh sensitivity analysis validated this choice (less than 1% tolerance, in terms of displacements and stresses at a selection of control stations within structure and soil domains).

4.6 Earthquake record selection and calibration

All earthquake records are selected among the historical strong earthquakes with the magnitude 6-7.5 (Richter scale). Four NF Pulse-like records and four NF records without pulse in their velocity records with rupture fault distance within 20 (km) captured on rocks are selected from the Pacific Earthquake Engineering Research (PEER) strong motion database (PEER, 2013).

The reason for extracting these records on rocks is that minor changes in the ground motions that occur in rocks. Therefore, the earthquake ground motions applied at the soil base are closer to the original input ground motions released from their sources.
The second reason for choosing the mentioned records is related to studying the effects of existing pulse on the dynamic responses of the conventional and isolated bridges with and without SSI effect.

To compare the results, all records are scaled to 0.32g, which is the PGA associated to the uniform hazard design spectrum ,6th generation (CNB2020), recommended for Vancouver for 2% probability of exceedance in 50 years, on a site class A (rock) (NRCAN, 2022). It should be mentioned that the interference of different source mechanisms, such as directivity effects, and focal mechanisms (strike-slip, normal or reversing faulting), is not considered during the selection of records.

Details of the selected ground motions given in Table 4.5 show that pulse-like records contain low PGA/PGV ratios, less than 10 (1/s), and higher PGD values. In comparison PGA/PGV for records without pulses are higher (more than 12 (1/s)) with lower PGD values. Furthermore, the spectral acceleration of records in Figure 4.3, shows that pulse-like records have higher responses in the vicinity of the period related to the conventional bridge and the high values of spectral acceleration continue even in long periods such as the isolated bridge's period. Although the period shift in the isolated bridge will move the structure to the low energy-containing frequencies of the earthquake records, the seismic force demand for the pulse-like records is still higher than the design spectrum and also higher than the records without pulses.

Table 4.5 NF earthquake records adopted in the analyses

ID	Earthquake	Station	Magnitude	R_{rup}	PA	PV	PD	PGA/PGV	T_p
			(Mw)	(km)	(g)	(cm/s)	(cm)	(1/s)	(s)
P-1	Kobe	Kobe University	6.9	0.9	0.32	63.9	18.3	4.9	1.49
P-2	Loma Prieta	Lexington Dam	6.9	5	0.32	74.5	23.7	4.2	1.57
P-3	Northridge	Pacoima Dam	6.7	7	0.32	31.7	4.7	9.9	0.59
P-4	Kocaeli	Gebze	7.5	10.9	0.32	72.2	67	4.3	5.99
NP-1	Parkfield	Turkey flat	6	5.3	0.32	16.5	2.8	19	NA
NP-2	Loma Prieta	Gilroy Array	6.9	9.6	0.32	25.8	5.7	12.2	NA
NP-3	Morgan Hill	Gilroy Array	6.2	14.9	0.32	9.7	1.4	32.5	NA
NP-4	Tottori	OKYH07	6.6	15.2	0.32	14.7	6.4	21.3	NA

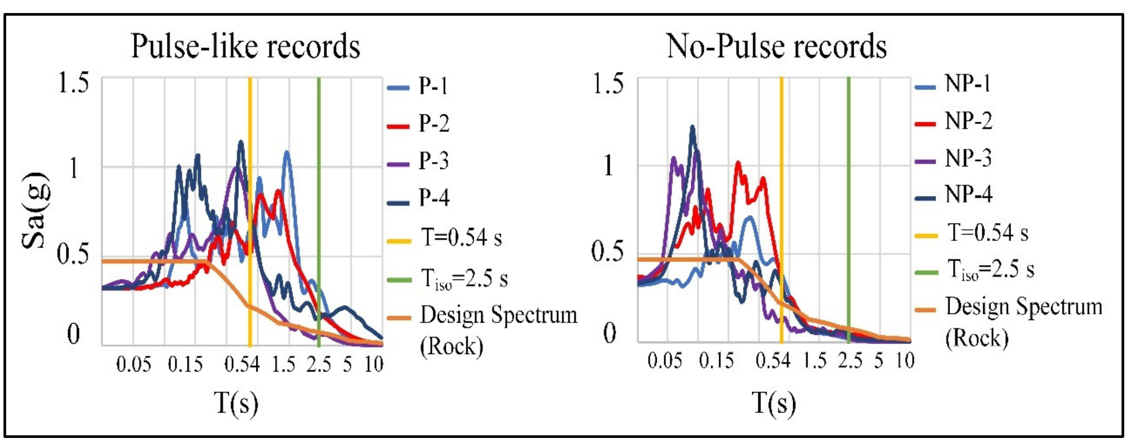

Figure 4.3 Spectral accelerations of the scaled records on Rock (class A), log scale

4.7 Analysis programme and procedure

All calibrated records are input at the base of the conventional and isolated bridge variants, first without considering the presence of the soil where the base of the bridge is fixed and then with modeling the soil using the direct approach. The bridge variants are analyzed by NLTHA in Abaqus software, first for the static gravity dead load to obtain initial stress conditions and then for dynamic loading conditions. The structural responses of NLTHA, including the maximum acceleration on top of the deck, base shear, and displacement of the bridge deck and isolation systems, are studied as seismic demands. Results are discussed in the following sections.

4.7.1 Effect of pulse-like records and SSI on the acceleration responses

The maximum acceleration in the conventional and isolated bridge, as shown in Figure 4.4, is higher in pulse-like records compared to records without pulses. On average, the maximum acceleration responses of the conventional bridge are higher by the factor of 2, 2, 1.6, and 1.6 for No-Soil, Rock, Soil-C, and Soil-D conditions. For the isolated bridge, this factor is 2, 2, 1.7, 1.6 for Iso-1, 1.4, 1.4, 1.1, 1.1 for Iso-2 and 1.6, 1.6, 1.3, and 1.3 for Iso-3, showing better control of the acceleration responses in ISO-2, and ISO-3, as the effect of the pulse is mitigated by reducing the differences between pulse-like records and records without pulses. In addition, the acceleration responses of the pulse-like records in conventional and isolated bridges show a decreasing trend on softer soil, while in records without pulses, the difference between the responses on different soil is not noticeable.

To study the effect of soil, all responses are normalized by the responses of the No-soil condition, and the results are shown in Figure 4.5. In the case of a ratio of more than one, responses are amplified, and the effect of SSI is unfavorable. In contrast, when the ratio is negative, SSI is favorable and reduces the responses.

As shown in Figure 4.5., soil has a noticeable positive effect in the case of pulse-like records by diminishing the pulse effect and reducing the acceleration responses from No-soil condition to Soil-D by the average of 30%, 27%, 21%, and 23% for the conventional, ISO-1, ISO-2, and ISO-3 bridge variants, respectively.

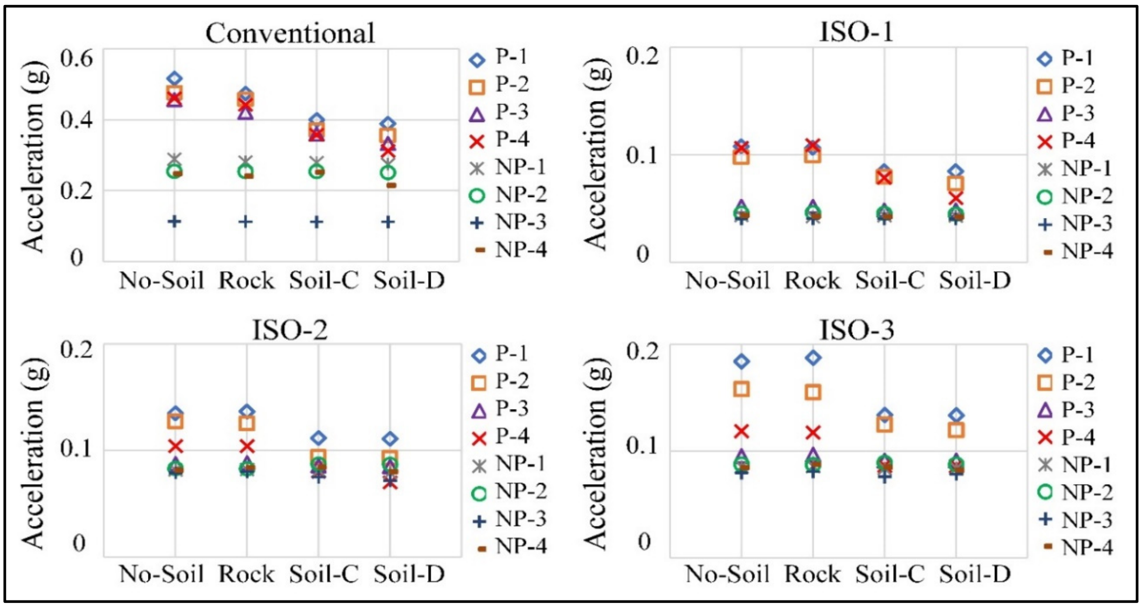

Figure 4.4 Maximum acceleration responses

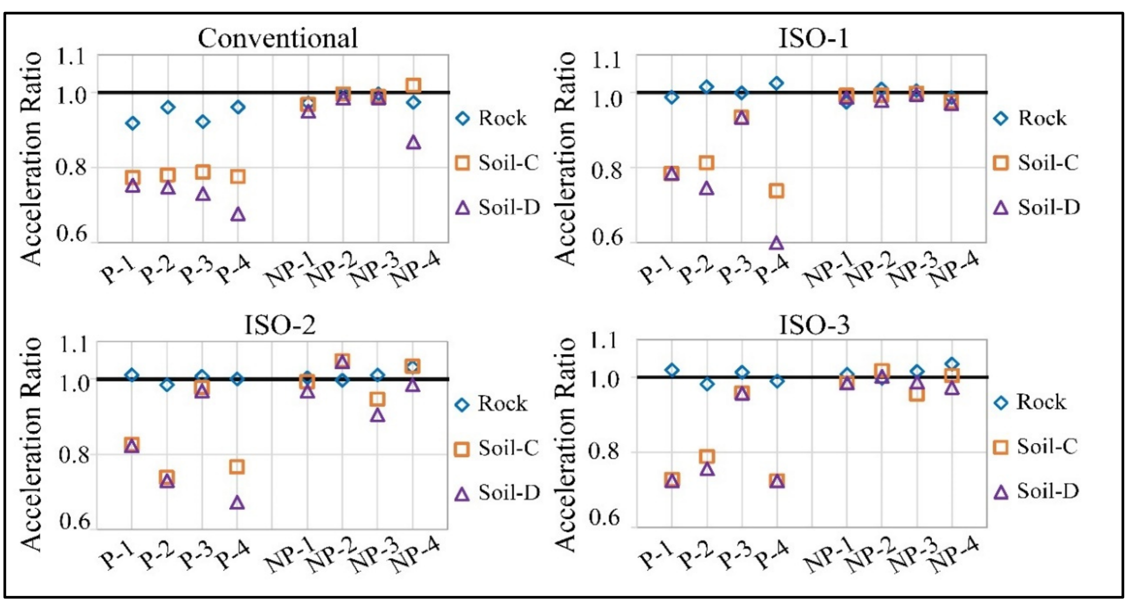

Figure 4.5 Acceleration ratio (SSI/Fixed-base)

In contrast, soil does not play an important role in amplifying or de-amplifying the acceleration responses in NF records without pulses. In the majority of the cases, the SSI effect is neutral, and the average difference between the soil-D and No-soil condition is 6%, 2%, 2%, and 1% for the conventional, SO-1, ISO-2, and ISO-3, respectively.

4.7.2 Effect of pulse-like records and SSI on the base shear responses

As it is shown in Figure 4.6, the maximum base shear responses have the same trend as the acceleration responses showing the higher responses for pulse-like records compared to no-pulse records.

On average the maximum base shear responses of the conventional bridge is higher in pulse-like records by the factor of 1.9, 1.9, 1.6 and 1.6 for No-Soil, Rock, Soil-C, and Soil-D conditions. For the isolated bridge this factor is 1.7, 1.5, 1.4, 1.4 for Iso-1, 1.3, 1.2, 1.2, 1.2 for Iso-2 and 1.5, 1.4, 1.3, and 1.3 for Iso-3.

Based on the normalized base shear responses shown in Figure 4.7, soil has a noticeable positive effect in most of pulse-like records and responses are reducing from Rock to Soil-D. On average, the base shear responses of the pulse-like records in both conventional and isolated bridges is reducing from No-soil condition to Soil-D by the average of 24%, 17%, 6%, and 12% for the conventional, SO-1, ISO-2, and ISO-3, respectively.

In contrast and for NF records without pulse, the soil effect is positive in reducing the responses of the conventional bridge by the average of 12%, but it plays either neutral or negative role in isolated bridges in most of the cases by increasing up to 8% in some records depending on the isolation system properties.

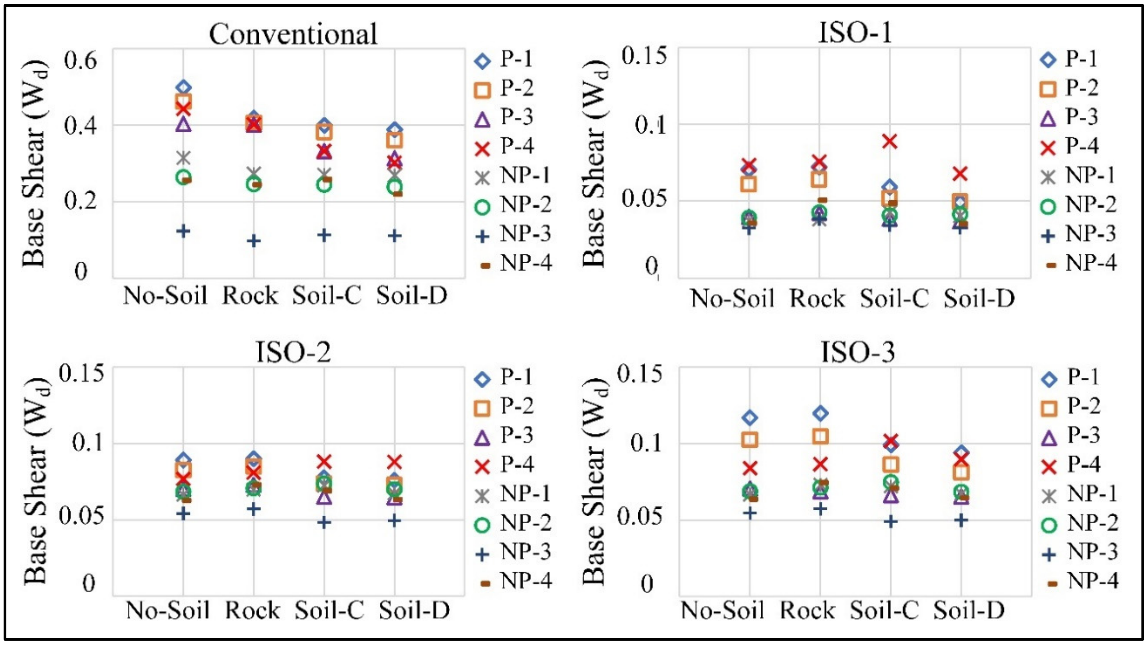

Figure 4.6 Maximum base shear responses

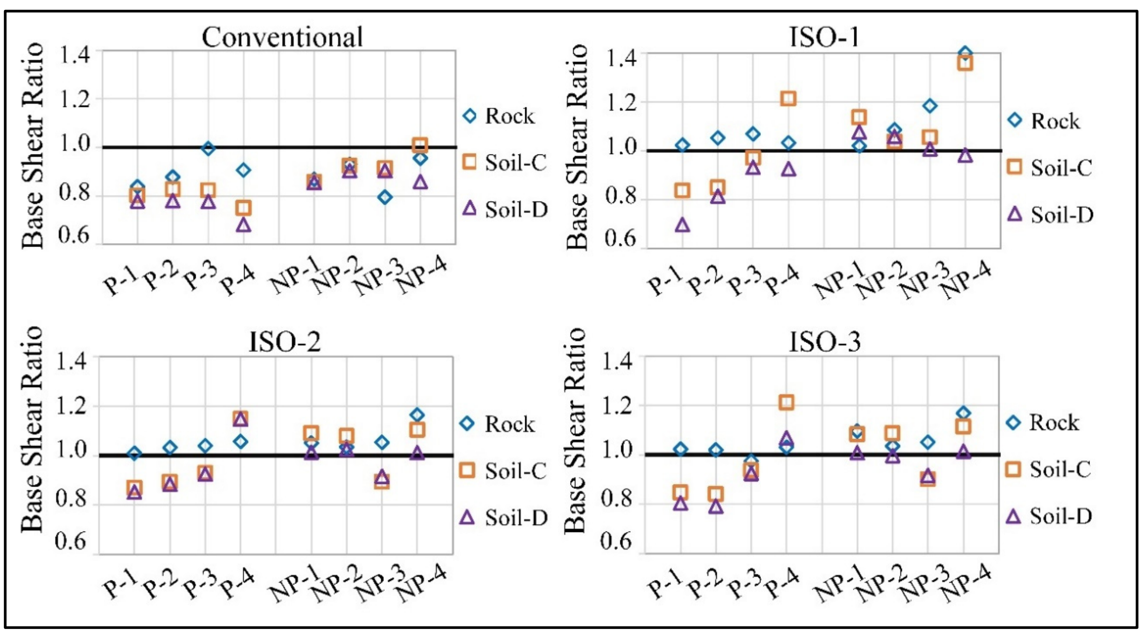

Figure 4.7 Base shear ratio (SSI/Fixed-base)

4.7.3 Effect of pulse-like records and SSI on the displacement responses

A higher displacement on top of the deck and in isolation systems is observed in pulse-like records for all bridges as it is shown in Figure 4.8. On average the displacement demand in pulse-type records is higher than records without pulses up to 3, 10, 8, and 8 times for the conventional, ISO-1, ISO-2, and ISO-3, respectively for all soils.

In the conventional bridge, the displacement responses are less scattered in records with no pulse and they show less sensitivity to SSI effect. In the isolated bridges, while all records without pulse show the displacement demand less than the designed displacement for all isolation systems confirming the effectiveness of these technology for strong earthquakes without pulse in their records, in most of the pulse-like records displacement demands are higher than the designed displacement in ISO-1. In ISO-2, Increasing the displacement capacity and the characteristic strength, reduces the number of earthquake records with higher displacement demand than the displacement capacity.

In ISO-3, which is an optimal design for strong seismicity areas with higher displacement capacity, characteristic strength, and post-elastic stiffness compared to ISO-1 and ISO-2, the displacement demand is less than the designed displacement in all pulse-like records, showing a need of special attention in the design of the isolation systems in high seismicity areas prone to pulse-like earthquake records. The normalized displacement ratio in Figure 4.9 shows that soil is a detrimental factor, increasing the displacement demand in pulse-like records up to 4 times and in records without pulses up to 2.5 times.

However, the effect of soil on the isolated bridges depends on the isolation system properties. On average, the displacement demand increases on Soil-D compared to No-soil condition by 55%, 40%, 77%, and 70% in pulse-like records and 10%, 85%, 85%, and 85% in records without pulses, for the conventional, ISO-1, ISO-2, and ISO-3, respectively.

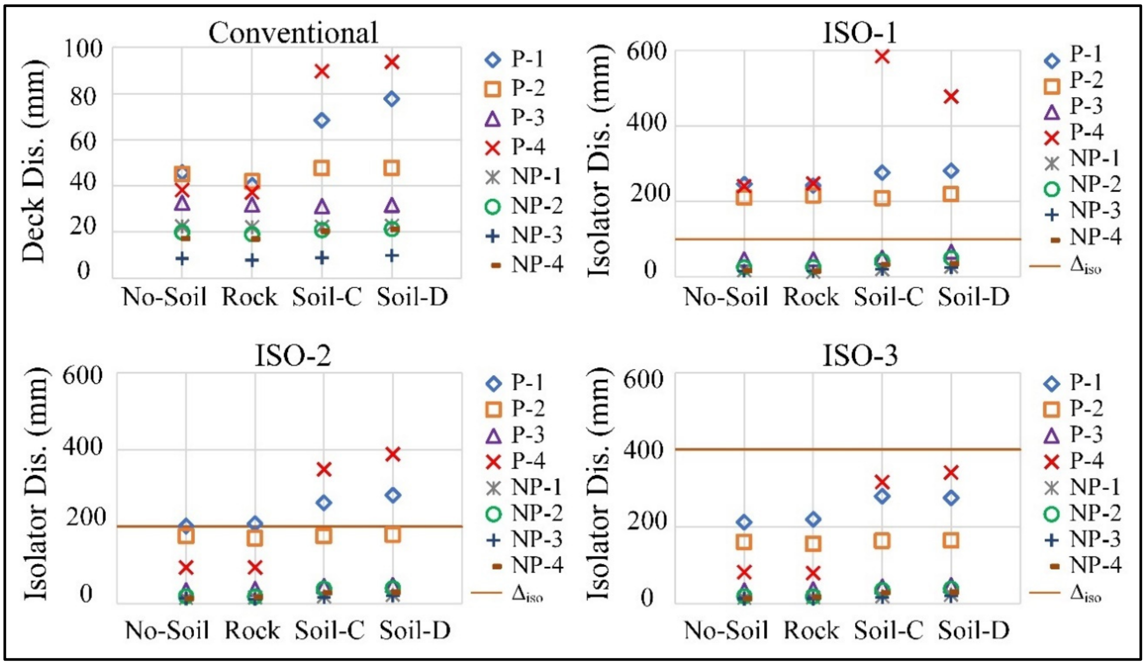

Figure 4.8 Maximum displacement responses

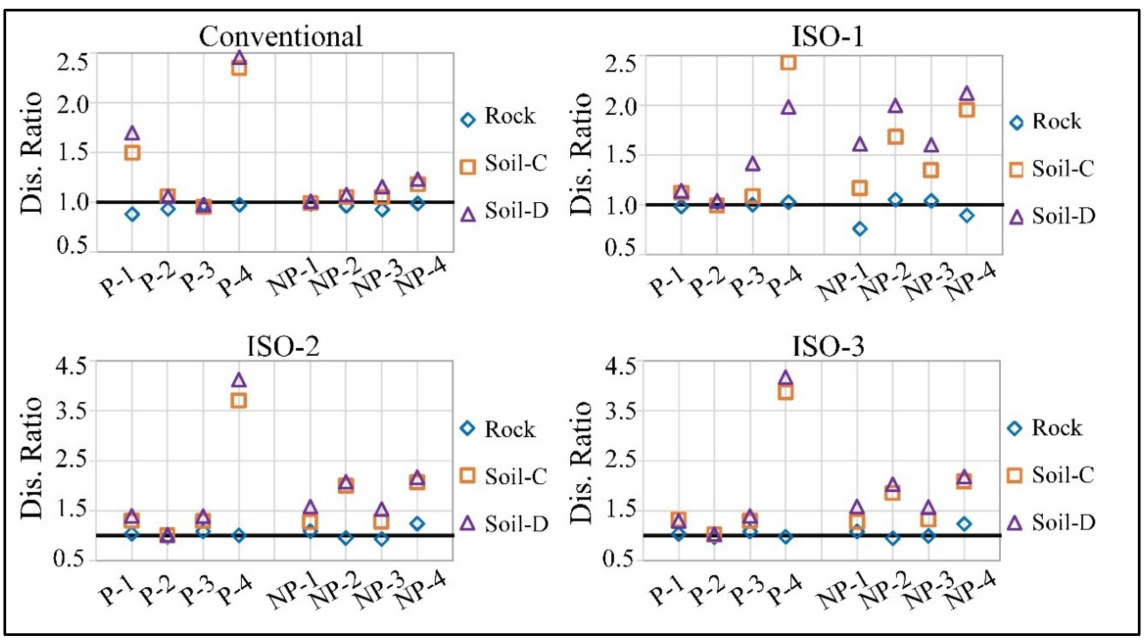

Figure 4.9 Displacement ratio (SSI/Fixed-base)

4.8 Discussion and Conclusion

This paper studied the simultaneous effects of NF earthquakes with and without pulses in their velocity records and the SSI effect on three-span conventional and isolated bridges. Seismic responses of the bridge without the presence of the soil are compared to those considering the SSI effects in the direct approach. Three different soil properties representing the rock, stiff and medium soil, have been selected. The role of soil characteristics has been evaluated by considering the bridge founded on different soil strata subjected to strong NF pulse-like and no pulse-like records.

Responses of NLTHA lead to the fact that pulse-like records cause higher dynamic responses in terms of force and displacement demand compared to records without pulses.
In addition, while considering that soil plays a positive role in pulse-type records by reducing the acceleration and base shear responses on softer soils, it does not show a notable effect in records without pulses.

Moreover, pulse-type records need higher displacement capacity in both conventional and isolated bridges, and the regular designing process of isolated bridges underestimates the displacement demand. Therefore, the optimum design of isolation systems is recommended for high seismicity areas to provide the displacement demand despite the fact that they attract higher forces compared to the common design process of isolation systems. Consequently, careful attention needs to be paid to designing the isolation systems on softer soils as the displacement demand could be two times more than the case of ignoring the soil effect.

CHAPTER 5

SEISMIC RESPONSE ASSESSMENT OF CONVENTIONAL AND BASE-ISOLATED BRIDGES UNDER STRONG NEAR-FAULT AND FAR-FIELD EARTHQUAKES CONSIDERING SOIL-STRUCTURE INTERACTION

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Paper ready for submission

5.1 Abstract

Seismic isolation system embodies one of the most effective approaches for the seismic protection of structures in earthquake-prone regions. However, earthquake inputs and soil characteristics are among the parameters that may influence and attenuate the effectiveness of this technology. This research investigates the simultaneous effects of soil-structure interaction and strong earthquakes in conventional and base-isolated bridges. Near-fault ground motion records with and without velocity pulses, and Far-field records with different frequency content are applied to a three-span bridge equipped with three different isolation systems. Bridges are simulated using the direct approach in Abaqus software and three non-liquefiable soil deposits categorized as hard rock, very dense soil, and stiff soil are studied. Nonlinear time history analyses are conducted, and dynamic responses regarding peak deck acceleration, base shear, and displacement on top of the deck for the conventional bridge and displacement across the isolation systems for the isolated bridge, under the calibrated records for the same PGA, are studied and results are compared with the bridge where the presence of the soil is ignored. Results of the study reveal that the bridge peak responses and soil effects are primarily influenced by the uncertainty in the ground motions and their frequency contents. Ratios of

peak ground acceleration to peak ground velocity significantly impact all dynamic responses. For the same PGA, records with PGA/PGV<12 cause higher dynamic responses in terms of force and displacement. In addition, pulse-like records cause higher responses concerning force demand, exhibiting increases of up to 50%, and displacement demand, with increases of up to 75% on average, compared to Near-fault records without pulses. Furthermore, special attention should be given to the base shear responses, as they are significantly amplified on softer soils, with increases of up to 45% observed in a few individual records. Moreover, results demonstrate that records with PGA/PGV<12 increase the displacement demand of conventional and isolated bridges by up to 2 times, and the displacement demands increase drastically on softer soils by up to 3.5 times. Therefore, meticulous consideration is needed in the design process of the isolation systems, particularly for the NF site, to ensure adequate displacement capacity.

Keywords: Bridges, Seismic isolation, Soil-Structure Interaction, Far-Field, Near-Fault, Earthquake characteristics.

5.2 Introduction

Seismic isolation of structures represents a logical and foundational approach to attenuating the destruction resulting from strong seismic activities. This technology has demonstrated a noteworthy beneficial impact in diminishing the seismic-induced damage and demands on infrastructures, as indicated by a multitude of experimental, numerical research studies, as well as observations conducted after earthquakes (Cardone et al., 2022; De Luca & Guidi, 2019; Tsopelas et al., 1996).

The underlying design strategy employed by seismic isolation involves reducing the seismic force demand to or near the elastic capacity of the structure. This serves to avert the damage linked to the inelastic deformation of the structure. To this end, it alters the frequencies of the primary vibration modes of the structure, shifting them to be less than the predominant energy-containing frequencies of seismic events. As a complement, the mechanism of the energy dissipation in the system increases the damping in the structure, therefore inelastic deformations within the structure are eliminated or significantly reduced, and they take place within the isolation system (De Domenico et al., 2020; Di Cesare et al., 2021).

The long-term benefit of such a design approach is the option of easy replacement to maintain the serviceability of the structure after an earthquake, reducing socioeconomic losses and costs associated with reconstruction (Billah & Todorov, 2019). In contemporary society, bridges stand as crucial components of infrastructure, with a pronounced susceptibility to seismic events, notably due to their low structural redundancy and particular mass distribution. Therefore, it is important to consider all effective factors at their seismic design and construction stages to ensure adequate seismic performance. In the literature, seismic response of bridges is noted to be influenced significantly by two pivotal factors: earthquake characteristics and Soil-Structure-Interaction (SSI) (Tochaei et al., 2020; Xie & DesRoches, 2019).

Ground motion records within 20 km of the ruptured fault are generally categorized as near-fault (NF) (Billah et al., 2013; Bray & Marek, 2004) and records collected at a distance more than 20 km are considered as far-field (FF) earthquakes. Typically, NF ground motions exhibit notable forward directional effects, effects related to hanging walls, distinct characteristics of strong vertical ground motion, and an impulse effect (Billah et al., 2013; Shao et al., 2021).

Many research studies have indicated that near-fault pulse-like earthquakes impose greater structural damage compared to ordinary earthquakes (Jia et al., 2023; Jiang et al., 2020; Jiao et al., 2021; Yang et al., 2023). Frequently, NF records contain long-period pulses in their velocity record. The presence of the intense long-period pulses forces the structure to act in inelastic ranges and consequently may have higher ductility and base shear demands compared to FF earthquakes (Ismail et al., 2014; W. I. Liao et al., 2000; Neethu & Das, 2019; Shen et al., 2004).

In this context, the correlation between the pulse period and the period of the structure displays an important role in the structural responses. Researchers have shown that when the period of the structure is close to the pulse period, NF records can significantly amplify the seismic responses of isolated bridges (Shen et al., 2004; Yang et al., 2019; Zhong et al., 2022).

Therefore, the hysteretic damping of the isolation systems might not be effective in dissipating the energy process in the first part of the pulse (Priestley et al., 2007) and the structure could be prone to severe damage (Anderson & Bertero, 1987). It was shown that for isolated bridges, the pulse effect causes a higher displacement demand of the isolation bearings and displacement demand depends on the duration of the pulse as well as the ratio between the pulse duration and the natural period of the structure (Chai & Loh, 1999; Ismail et al., 2014; Shen et al., 2004).

In order to have an effective isolation system, higher values of the main features of the isolation system, namely the post elastic stiffness, the characteristic strength, and the displacement capacity, defined later, are required (Dicleli & Karalar, 2011). SSI can also significantly modify the main characteristics of earthquake records, such as amplitude, duration, and frequency content depending on the dynamic characteristics of the structure, the characteristics of the input ground motions, and the properties of the soil (Chouw & Hao, 2008; Rayhani et al., 2008; Tochaei et al., 2020). Therefore, misrepresenting the soil effects could result in an erroneous assessment of the seismic demand of the bridges and the isolation systems, particularly in presence of softer soils (Chaudhary et al., 2001; Kulkarni & Jangid, 2003; Saritas & Hasgur, 2014; Stehmeyer & Rizos, 2008; Fatahi et al., 2014; Worku, 2014).

On the other hand, some investigations have revealed that isolated bridges exhibit a lower susceptibility to SSI effects compared to conventional bridges (Dezi et al., 2012; Vlassis & Spyrakos, 2001). Numerous research studies have been carried out using models that incorporate linear springs and dashpots to study the impacts of SSI. Diverse outcomes have been documented, including higher displacement in the isolation system due to SSI (Ates & Constantinou, 2011; Bi et al., 2011; Dezi et al., 2012; Stehmeyer & Rizos, 2008; Tochaei et al., 2020; Tongaonkar & Jangid, 2003; Ucak & Tsopelas, 2008) and higher base shears (Hoseini et al., 2018; Ucak & Tsopelas, 2008). At the same time, many investigations demonstrated the beneficial aspect of SSI, decreasing the force and displacements demands, increasing the bridge's safety, and reducing design costs (Tochaei et al., 2020; Tongaonkar & Jangid, 2003; Vlassis & Spyrakos, 2001).

Moreover, a limited number of studies have deduced that that SSI could be detrimental or beneficial, resulting in varying seismic demands, which can be either higher or lower responses. This variation is contingent upon a range of factors including structural characteristics, soil stratum properties, and ground motion characteristics (Betti et al., 1993; Carbonari et al., 2011; Jeremić et al., 2004; Mylonakis & Gazetas, 2000; Tochaei et al., 2020; Ucak & Tsopelas, 2008; Xie & DesRoches, 2019). Very limited studies carried out inclusive consideration of the direct method. These studies revealed that using linear models for both isolators and soil results in an inaccurate assessment of structural behaviour. Particularly for conventional bridges, SSI significantly impacts all dynamic responses. This underscores the crucial importance of correctly accounting for soil effects in seismic modeling and analysis (Forcellini, 2017; Güllü & Jaf, 2016).

Despite extensive research, no consistent conclusions exist about the impacts of SSI and earthquake characteristics on isolated bridges. This complexity arises from the diverse parameters, uncertainties in infrastructure, soil, and ground motions. Researchers' focus on specific aspects has left many parameters and interactions unexplored. The majority of these research studies have concentrated on either earthquake characteristics or SSI, primarily involving conventional bridges. However, there has been limited consideration of the combined impacts of both factors on both conventional and isolated bridges.

Consequently, this study seeks to explore the effects of various record characteristics, including NF records with and without velocity pulses, FF records, with various frequency contents. Additionally, the research aims to assess the impact of different soil types on both conventional and isolated bridges using a numerical parametric analysis.

Moreover, this research seeks to comprehend the impact of soil properties and SSI on the behaviour of isolation systems by studying different isolation main features. This helps in choosing better methods and models to consider SSI and earthquake effects for more effective isolation strategies. The following sections provide more details on the used bridge model, earthquake records, soil types, and isolation systems.

5.3 Studied parameters and numerical modeling

5.3.1 Case study bridge model

A typical three-span highway bridge studied by Jangid (2003) (Tongaonkar & Jangid, 2003) and Elias (2017) (Elias & Matsagar, 2017), shown in Figure 5.1, has been selected for this study. The fundamental period of vibration in this configuration in the longitudinal direction is 0.54 s with a damping ratio of 5% is assumed. In the present study, the bridge model and nonlinear-time history analyses (NLTHAs) are performed using Abaqus software (ABAQUS, 2019). The details of the conventional and isolated bridge model, boundary conditions, geometry, and verification of the model with the reference bridge could be found in the recently published study (Cheshmehkaboodi et al., 2023). In addition, Table 5.1 shows the material properties and geometric parameters used to model the bridge taken from the reference studies (ABAQUS, 2019; Elias & Matsagar, 2017; Tongaonkar & Jangid, 2003).

Beam-column element is assigned to model the deck, piers, and abutments, and C3D8R solid element is utilized for the soil domain and the foundations. As a common design for seismically isolated bridges, the superstructure and the piers remain within their elastic range. For the conventional bridge classified as an essential bridge, the same assumption is applied in the current study. Given the assumed location of the bridge in Canada, the inclusion of frost action necessitates the positioning of the foundation at a depth of D = 1.8 meters from the soil surface, and load associated with the upper soil is considered in analyses.

5.3.2 Isolation systems model, design and properties

The multi-plastic model given to link elements with bilinear behaviour is used in Abaqus to model the isolation system (ABAQUS, 2019). Link elements as representative of the isolation system decouple the substructure from the superstructure and only the longitudinal direction is considered for the study of employing seismic isolation. Assuming the bridge is located in Vancouver as a high seismicity area, three isolation systems are designed, using the 6th generation seismic hazard for Canada (NRCAN, 2022) with a target effective period T= 2.5 s. The properties of the isolation systems are calculated based on the single-mode spectral analysis, and named as ISO-1 to ISO-3. The design of ISO-1 is based on a target displacement within the isolation system of 100 mm for Vancouver location, considering a soil class A condition.

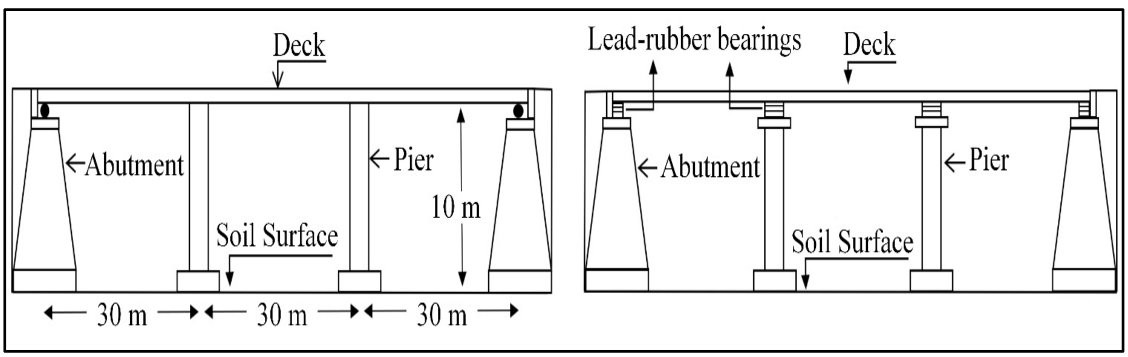

Figure 5.1 General elevation of the conventional (left side) and isolated (right side) bridge in this study

Table 5.1 Material and dimension properties of the bridge

Properties of the Bridge	Cross-section areas (m2)	Length or height (m)	Modulus of elasticity (Gpa)	Mass density (kg/m3)	Compressive Strength (Mpa)	Poisson Ratio
Deck	15.6	3@30	36	2400	30	0.2
Piers	1.767	10	36	2400	30	0.2

Considering the fact that earthquake records with a low ratio of PGA/PGV, or earthquakes with the pulse in their velocity records, impose a larger strength and displacement demand (Dicleli & Karalar, 2011; Malhotra, 1999b), therefore in ISO-2, a higher Q_d (2 times that of ISO-1) is specified, and the displacement capacity is set to be twice that based on the code.

With respect to T=2.5 s, the target for designing ISO-3 is the proposed domain of Q_d and K_d by Nguyen and Guizani (2021) to design an optimal seismic isolation system for Vancouver as a high seismicity area with higher post elastic stiffness (K_d), and displacement capacity (D_{max}) compared to ISO-2 and ISO-1 (CSA, 2019; Nguyen & Guizani, 2021b).

Figure 5.2 shows the global model including the isolated bridge and soil, and the bilinear force-displacement relation of the seismic isolation system (SIS), and Table 5.2 shows the SIS hysteretic parameters, where Q_d is the characteristic strength showing the force required at zero displacement, K_{eff} stands for the effective stiffness at D_{max} defined as the maximum displacement of the isolation system, K_u is the initial elastic stiffness, K_d represents the post-elastic stiffness, and β is the equivalent viscous damping ratio.

Table 5.2 Isolation properties

ID	Location	T	K_{eff}	K_u	Q_d	K_d	D_{max}	β
		(s)	(kN/m)	(kN/m)	(kN)	(kN/m)	(mm)	%
ISO-1	Piers	2.5	7,750	228,250	450	3,240	100	35
	Abutments	2.5	3,550	101,500	200	1,550	100	35
ISO-2	Piers	2.5	7,750	453,250	900	3,240	200	35
	Abutments	2.5	3,550	202,000	400	1,550	200	35
ISO-3	Piers	2.5	7,895	455,700	900	5,700	400	20
	Abutments	2.5	3,775	202,800	400	2,800	400	20

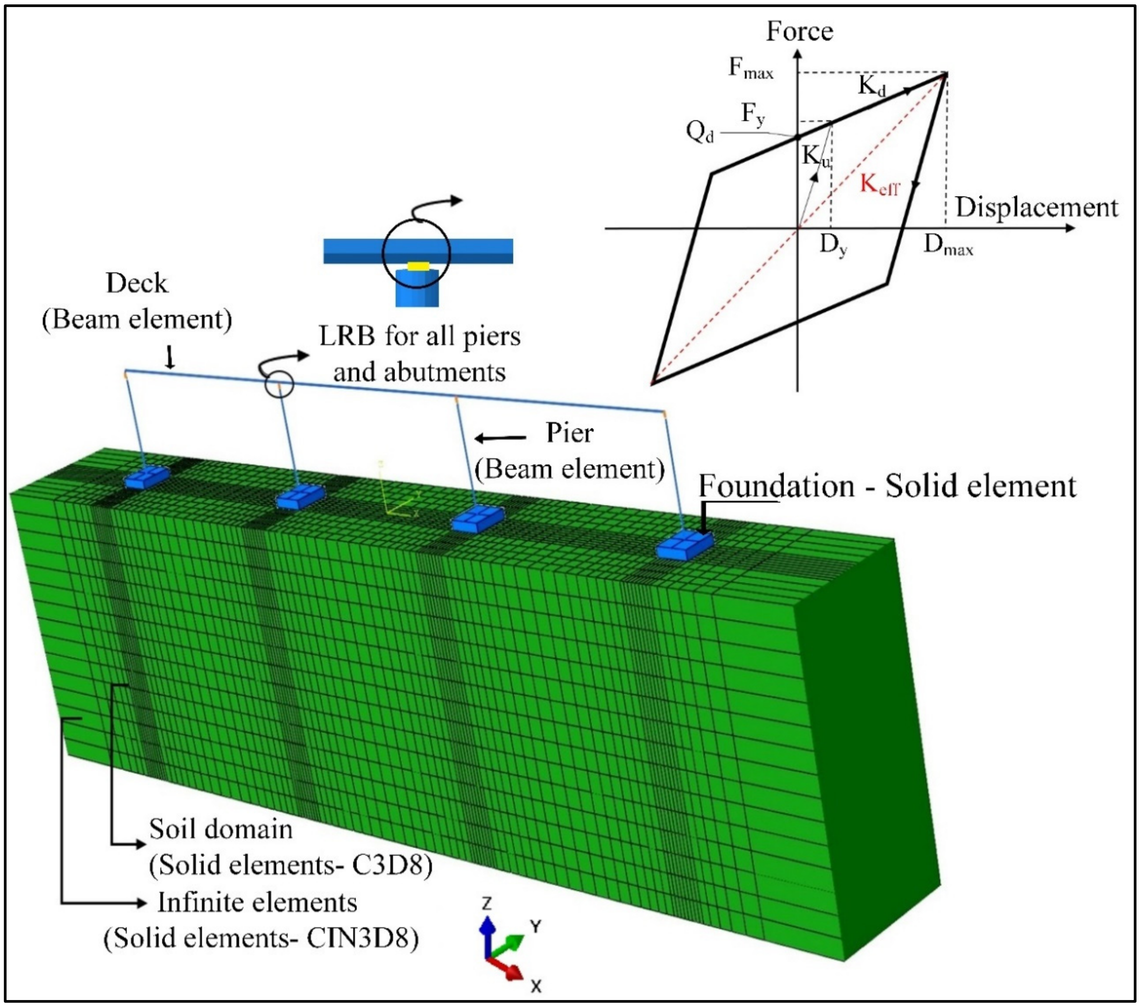

Figure 5.2 Isolated bridge model with soil and the bilinear force-displacement behaviour of SIS

5.3.3 Soil model and boundary conditions

A Mohr-Coulomb based constitutive model has been used in this study to simulate the soil medium and the nonlinear behaviour of the soil. The model uses Mohr-Coulomb criterion to derive the primary and envelope curve of soil response and is coupled with an elastic-perfectly plastic model to describe inelastic force-deformation relation, unloading and reloading branches, with no degradation. Such a relatively simple model is believed to catch efficiently, in terms of computation effort, the main features of soil hysteretic behaviour and has been employed by many researchers (e.g. Khazaei, Amiri, & Khalilpour, 2017, Ghandil & Behnamfar 2015, Rayhani and El Naggar 2008, Conniff and Kiousis 2007, among others) in modeling the dynamic SSI as a method to simulate soil behaviour under dynamic loads in soil-structure systems (Conniff & Kiousis, 2007; Khazaei et al., 2017; Rayhani & El Naggar, 2008; Tabatabaiefar & Fatahi, 2014). Mohr–Coulomb model requires commonly used and easy accessed soil parameters including friction angle, shear modulus, unit weight, cohesion intercept (Labuz & Zang, 2012; Rayhani et al., 2008).

The soil deposit model is represented as a rectangular area measuring 130 meters in length and 20 meters in width, utilizing 8-node brick elements (C3D8). Three distinct non-liquefiable uniform soil profiles; Rock, Soil-C (very dense soil), and Soil-D (stiff soil), are chosen for selection and analysis. These selections are according to the site classification provided in CSA (S6-19) (CSA, 2019). In addition, taking into account the observation that the majority of amplifications take place within the initial 30 meters of the soil profile, a soil depth of 30 meters is taken into consideration (Rayhani & El Naggar, 2008). Different soil properties used in this study are presented in Table 5.3, where ρ stands for the density, E represents the Elastic modulus, C is the cohesion stress, ø defines the friction angle, Vs is shear wave velocity, ϑ shows the Poisson's ratio, ξ represents the damping ratio, and Ψ is dilatancy.

In order to prevent wave reflection at the finite boundaries of the soil model, the study employs an infinite solid continuum represented by CIN3D8. This 8-node linear element, available through Abaqus, serves as a one-way infinite brick element in the longitudinal direction, which is the primary focus of the investigation. Additionally, fixed boundaries are implemented for the transverse direction, while allowing for free rotations within the scope of this study.

Table 5.3 Mechanical properties of soils

Soil	**E (MPa)**	**ρ (kg/m^3)**	**υ**	**C (KPa)**	**Ø (°)**	**V_s (m/s)**	**ψ**	**ξ (%)**
Rock	24960	2600	0.2	25e3	48	2000	7	5
Soil-C	1323	2100	0.26	0	40	500	5	5
Soil-D	430	1900	0.32	0	35	300	4	5

The contact and interaction between the soil surface and the foundation are simulated using surface-to-surface contact, utilizing the algorithm integrated within Abaqus (ABAQUS, 2019). The stiffness values of the interface govern the relative movement between interfaces in both the normal and tangential directions. Hard contact is defined for the normal direction, while the penalty method is employed for the tangential behaviour. A friction coefficient (μ) of 0.5 is used in tangential behaviour, based on the recommended range provided in the literature (Dehghanpoor et al., 2019; Khazaei et al., 2017; Kimmerling, 2002; Manual & Mechanics, 1986; Pando et al., 2006).

In total, the model employs 1300 elements for the bridge, along with 480 CIN3D8 and 23520 C3D8R elements for the soil domain. This selection was validated through a mesh sensitivity analysis, demonstrating accuracy with less than 1% tolerance in displacements and stresses at various control points within both the structure and soil domains.

5.3.4 Earthquake record selection and calibration

All earthquake records, presented in
Table 5.4, are selected among the real historical strong earthquakes with magnitude 6-7.5 (Richter scale). Seven NF pulse-like records, five NF records without pulse, all with the distance within 20 (km) from the ruptured fault, and 12 FF records, all captured on rocks, are used from the Pacific Earthquake Engineering Research (PEER) strong motion database (PEER, 2013). Additionally, eight NF records have PGA/PGV<12, and four contain PGA/PGV>12, while six FF records have PGA/PGV<12, and six contain PGA/PGV>12.

The reason behind choosing these records for rocks is the relatively subtle variations that occur in rock records. As a result, the earthquake records closely resemble the original ground motions originating from the source. The second reason for this selection is to study the difference between pulse-like records and records without pulses on the dynamic responses of the conventional and isolated bridges with and without SSI effects, and the third reason is related to the ruptured fault distance and its effect, as categorized as NF and FF records.

All records are scaled to 0.32g, which is the PGA recommended for Vancouver, for 2% probability of exceedance in 50 years, taken from the uniform hazard design spectrum, 6th generation (CNB2020) on a site class A (rock) (NRCAN, 2022). The locality of Vancouver is in a high seismic area, representative of the west coast of the north American continent. Details of the selected records in Table 5.4 show that pulse-like records contain low ratios of PGA/PGV<12 (s^{-1}), while most of the records without pulses have higher ratios of PGA/PGV (more than 12 (s^{-1})).

The spectral acceleration of records in Figure 5.3 shows that pulse-like records and those with PGA/PGV<12 have higher responses in the vicinity of the fundamental period of the conventional bridge compared to records without pulses and those with PGA/PGV>12. In addition, for PGA/PGV<12, the high values of spectral acceleration continue even in long periods, such as in the period range of the response of the isolated bridges (mainly in the range of 2 to 3 s). In fact, although the period shift in the isolated bridge will move the structure to the low energy-containing frequencies of the ground motion records, the seismic force demand for the records with PGA/PGV<12 in both NF and FF records is still higher than the design spectrum.

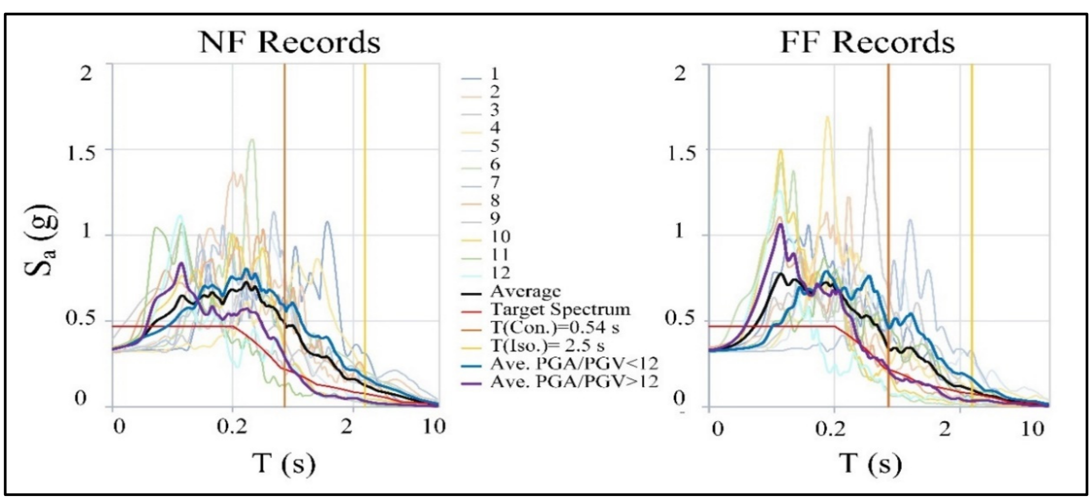

Figure 5.3 Spectral accelerations of the scaled records to (0.32g), on Rock (class A), log scale

Table 5.4 Earthquake records adopted in the analyses, scaled to PGA=0.32g

ID	Earthquake	Year	Station, component	Magnitude	R_{rup}	PGA/PGV	T_p
				Mw	km	1/s	S
N-1	Kobe	1995	Kobe University, RSN1108_KBU000	6.9	0.92	4.9	1.49
N-2	Tabas	1978	Tabas, RSN143_TAB-T1	7.4	2.05	6.9	6.19
N-3	Landers	1992	Lucerne, RSN879_LCN260	7.3	2.19	5.3	5.12
N-4	Loma Prieta	1989	Lexington Dam, RSN3548_LEX090	6.9	5.02	4.2	1.56
N-5	Northridge	1994	Pacoima Dam, RSN1050_PAC175	6.7	7.01	9.9	0.59
N-6	Kocaeli	1999	Izmit, RSN1161_GBZ270	7.5	7.21	5.9	5.37
N-7	Kocaeli	1999	Gebze, RSN1165_IZT090	7.5	10.92	4.3	5.99
N-8	Iwate	2008	IWT010, RSN5618_IWT010NS	6.9	16.27	10.2	NA
N-9	Parkfield	2004	Turkey flat, RSN4083_36529	6.0	5.29	19.0	NA
N-10	Loma Prieta	1989	Gilroy Array, RSN765_G01000	6.9	9.64	12.2	NA
N-11	Morgan Hill	1984	Gilroy Array, RSN455_G01	6.2	14.91	32.5	NA
N-12	Tottori	2000	OKYH07, RSN3925_OKYH07NS	6.6	15.23	21.3	NA
F-1	Northridge	1994	Wonderland Ave, RSN1011_WON095	6.7	20.29	13.1	NA
F-2	San Fernando	1971	Old Seismo Lab, RSN80_PSL180	6.6	21.5	16.0	NA
F-3	Northridge	1994	Vasquez Rocks Park, RSN1091_VAS000	6.7	23.64	8.4	NA
F-4	Duzce	1999	Lamont 1060, RSN1613_1060-E	7.1	25.88	7.8	NA
F-5	San Simeon	2003	Canyon Power Plant, RSN8167_DCPP247	6.5	37.97	5.1	NA
F-6	Niigata	2004	FKSH07, RSN4167_FKSH07EW	6.6	52.3	39.3	NA
F-7	Chi-Chi	1999	HWA003, RSN1257_HWA003-N	7.6	56.14	5.7	NA
F-8	Iwate	2008	IWTH17, RSN5649_IWTH17EW	6.9	72.44	27.8	NA
F-9	Loma Prieta	1989	Rincon Hill, RSN797_RIN000	6.9	74.14	11.3	NA
F-10	Chuetsuoki	2007	FKSH07, RSN5006_FKSH07EW	6.8	79.54	27.6	NA
F-11	San Fernando	1971	Allen Ranch, RSN59_CSM095	6.6	89.72	11.9	NA
F-12	Tottori	2000	HYG007, RSN3895_HYG007EW	6.6	99.64	29.9	NA
NF	Average	….	….	6.9	8.05	11.4	….
FF	Average	….	….	6.8	54.43	17.0	….

5.4 Analysis procedure

All the calibrated records are introduced as inputs at the base of both the conventional and isolated bridge configurations, in the horizontal longitudinal direction of the bridge. Initially, this is done without accounting for the presence of the soil, with the bridge base being fixed. Subsequently, the soil is modeled using a direct approach. First an initial static analysis is conducted under the gravity loads in order to obtain the initial stress and deformation states. Then, the earthquake records are applied to the base of the soil domain to obtain their dynamic response by NLTHAs. A time step sensitivity analysis is carried out to validate the convergence of the program to the correct solution.

The structural responses hence obtained by NLTHAs, including the maximum accelerations on top of the deck, the maximum base shears, and the maximum displacements of the bridge deck and within the seismic isolation units, for the isolated bridge, are studied. Results are discussed in the following sections.

5.4.1 Maximum acceleration responses and the effect of earthquake characteristics and SSI

The spectral accelerations for four NF and four FF records with different PGA/PGV ratios and the average of NF and FF records are shown in Figure 5.4. These spectral accelerations are captured at the surface of the soils. They indicate the amplification in Soil-C and Soil-D in the range of short periods. The amplification diminishes as the period increases, resulting in only a slight absolute difference in the responses for long periods, such isolated bridges. In both NF and FF records with PGA/PGV<12 ratios, the responses related to long periods are higher than those with PGA/PGV>12.

In addition, resonance is identified at the predominant period of the records, and the intensity and the duration of the amplifications related to the period or the soil effects strongly depend on each individual record that carries its own frequency content and characteristics.

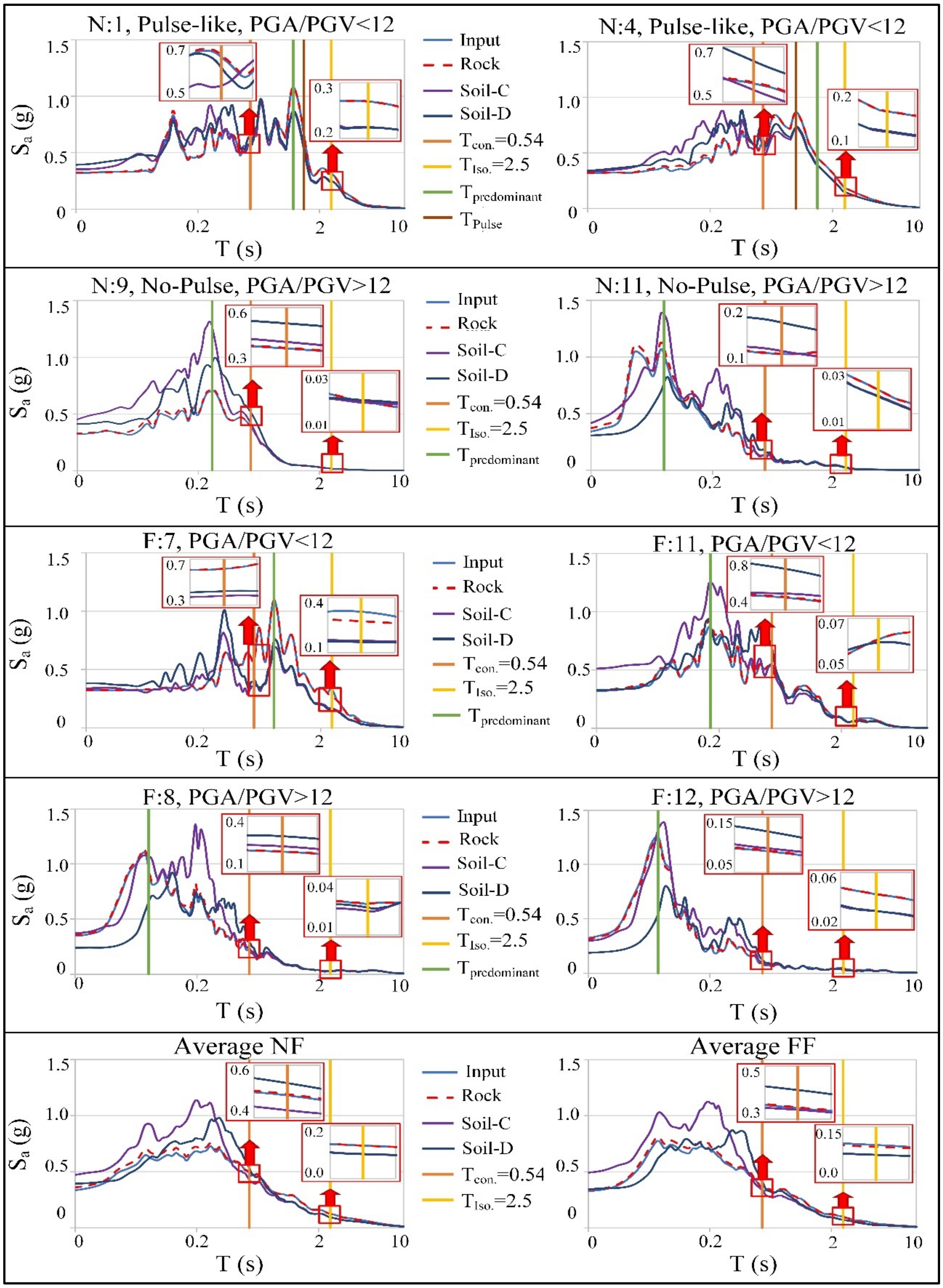

Figure 5.4 Spectral accelerations captured on different soils

Figure 5.5 shows the maximum acceleration response and its correlation with the PGA/PGV ratio for different soil conditions. The common trend observed in the responses is that the maximum acceleration response diminishes in both conventional and isolated bridges as the PGA/PGV ratio increases.

In the conventional bridge and for NF records, the maximum acceleration responses are related to the pulse-like records, all of which have a low ratio of PGA/PGV<12. On average, responses of pulse-like records are higher than records without pulses by 39, 37, 29, and 29 percent for No-soil, Rock, Soil-C, and Soil-D conditions, showing the reduction in differences on softer soils. Records with PGA/PGV<12 cause higher acceleration responses compared to records with PGA/PGV>12 by the average of 49, 47, 40, and 37 percent in NF records for No-Soil, Rock, Soil-C and Soil-D and 59, 58, 53, and 49 percent in FF records, respectively showing the key role of the frequency contents of records in seismic responses and also the effect of softer soils in reducing the responses.

Generally, the responses of NF records for the conventional bridge are higher than those of the FF records by an average of 22, 19, 18, and 18 percent for No-Soil, Rock, Soil-C, and Soil-D, respectively. The same trend of higher responses for pulse-like records and lower ratios of PGA/PGV is observed in the isolated bridge.

Additionally, in records with higher ratios of PGA/PGV>12, especially for FF zones, the responses are more attenuated and less distributed despite the fluctuation in a few cases on different soils.

For ISO-1, responses of pulse-like records in the NF zone are higher than those without pulses by an average of 49, 49, 35, and 31 percent for No-soil, Rock, Soil-C, and Soil-D conditions. NF records with PGA/PGV<12 show higher acceleration responses by an average of 53, 53, 39, and 36 percent compared to PGA/PGV>12, and for FF records are higher by 40, 39, 20, and 25 percent for No-soil, Rock, Soil-C, and Soil-D condition, respectively.

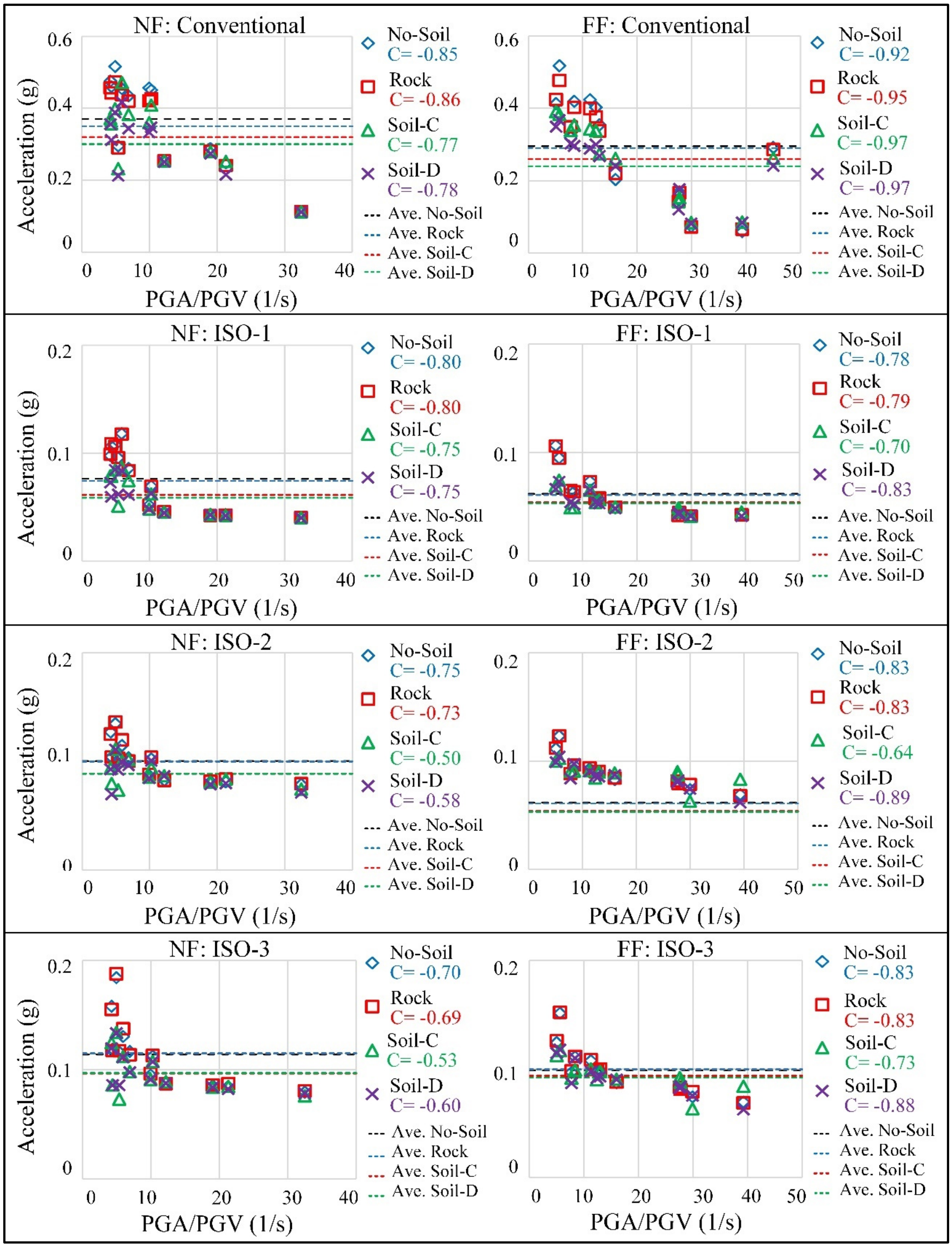

Figure 5.5 Absolute maximum acceleration responses at the deck versus PGA/PGV ratios, (C= Correlation coefficient from Anova)

For ISO-2, responses of pulse-like records in the NF zone are higher than those without pulses by an average of 23, 22, 9, and 9 percent for No-soil, Rock, Soil-C, and Soil-D conditions. On average, acceleration responses of NF records with PGA/PGV<12 are higher by 26, 25, 12, and 15 percent compared to PGA/PGV>12. This difference for FF records is 21, 20, 10, and 16 percent for No-soil, Rock, Soil-C, and Soil-D conditions, respectively.

For ISO-3, responses of pulse-like records in the NF zone are higher than records without pulses by the average of 31, 31, 15, and 15 percent for No-soil, Rock, Soil-C, and Soil-D conditions. On average, acceleration responses of NF records with PGA/PGV<12 are higher by 34, 34, 20, and 20 percent compared to PGA/PGV>12, and For FF records are 28, 28, 15, and 21 percent for No-soil, Rock, Soil-C, and Soil-D condition, respectively. On average, responses of NF records are higher than FF records by 14, 6, and 9 percent for ISO-1, ISO-2, and ISO-3.

To study the effect of SSI, the absolute values of maximum acceleration responses are normalized by those obtained for No-Soil condition responses, and the results are shown in Figure 5.6. Through this normalization process, when the ratios approach unity, the influence of soil effect is negligible, and the SSI does not substantially alter the responses. For ratios exceeding unity, the responses experience amplification, indicating a non-beneficial impact of SSI. Conversely, ratios lower than unity indicate a favorable role of SSI in modifying and attenuating the peak acceleration responses.

The results reveal that, in the case of the conventional bridge, SSI tends to be favorable for the majority of NF records, resulting in decreased peak acceleration on softer soil, except for a few specific instances. On the other hand, within FF records, a consistent pattern is absent, and SSI demonstrates both beneficial and adverse effects across a range of records characterized by varying PGA/PGV ratios.

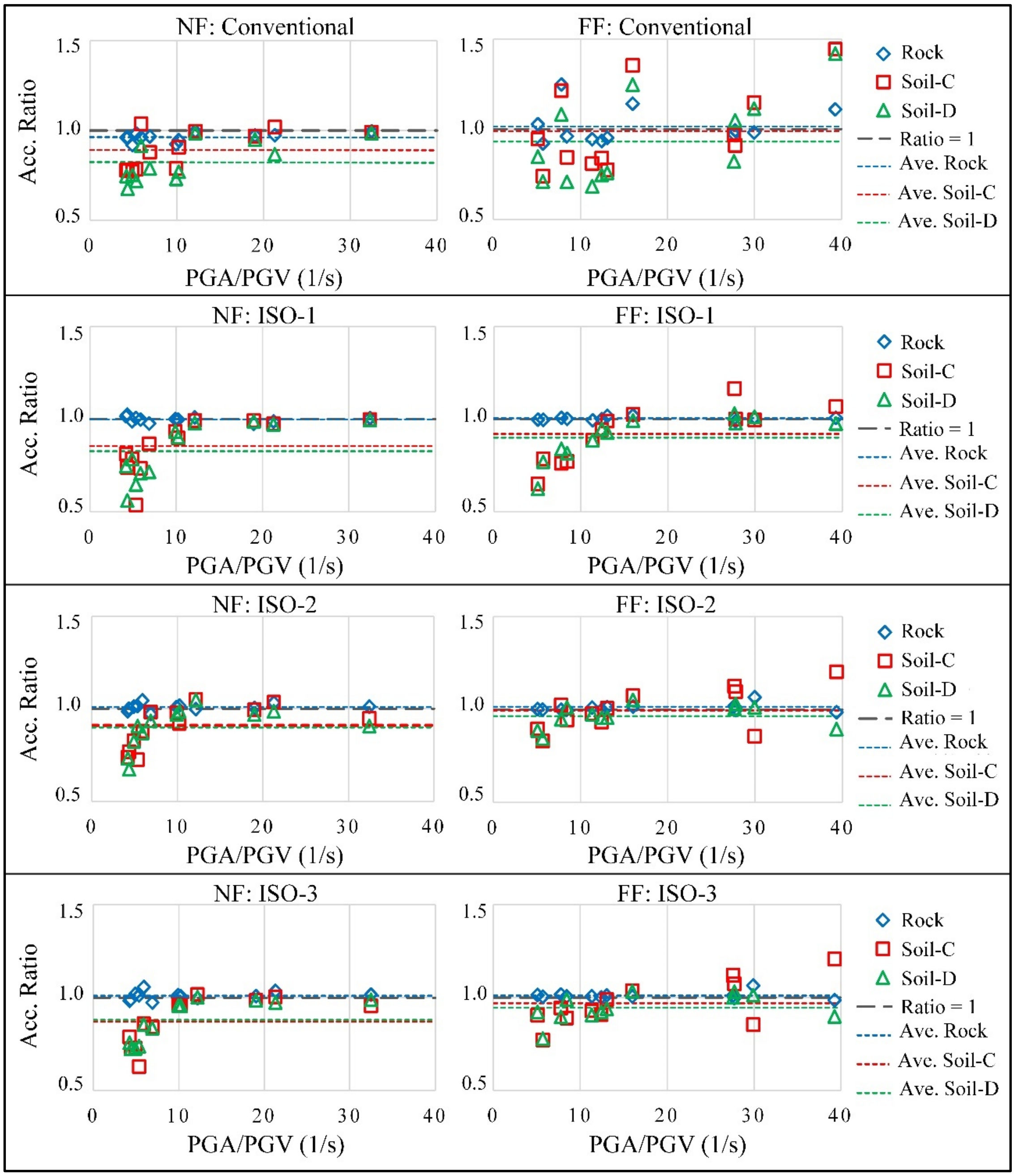

Figure 5.6 Normalized acceleration responses

In most cases, the conventional bridge's peak acceleration responses for No-soil conditions are slightly higher than Rock's. The slight difference in responses can be explained by the different behaviour of the boundary conditions defined at the base of each bridge in the case of the presence or absence of the soil. On average, from Rock to Soil-D, the maximum acceleration responses are reducing compared to No-Soil condition by 4%, 13%, and 20% for NF records, and 2%, 10%, and 17% for FF records, respectively.

In the isolated bridge and for both NF and FF zones, the ratio tends to be less than one or close to one in most records. Therefore, considering the soil plays a favorable or a neutral role in modifying the acceleration responses of the isolated bridges, which is in agreement with the fact that isolated bridges show less sensitivity to the SSI effects (Dezi et al., 2012). In addition, the reduction factor is more pronounced for pulse-like records in the NF zone and the records with lower PGA/PGV ratios for both conventional and isolated bridges so that the softer soils are de-amplifying the pulse effect and also the frequency content of the records with low PGA/PGV, and the distance of the ruptured fault is not a definite relative factor. This results are in agreement with the fact that softer soil can decrease the magnitude of the amplification or even cause de-amplification due to the hysteretic damping (Bolisetti, 2015).

Table 5.5 shows the comparison of site coefficients, F(T), in 6th Generation of hazard Canada (NRCAN, 2022), with the average responses of current study and Soil-C is considered as a reference with F(T)=1. Results demonstrate that in isolated and conventional bridges, the site coefficients for rocks might lead to underestimation of responses because the F(T) is two times higher than the recommended value in 6th Generation of hazard Canada. In the conventional and isolated bridges, the site coefficient in Soil-D is less than the factor recommended by 6th Generation of seismic hazard for Canada by an average of 10% for NF and 13% FF records for the conventional bridge and 30% for both NF and FF records for the isolated bridge. This indicates that the actual site factors for isolated bridges on soft soils may be conservative.

Table 5.5 Comparison of site coefficient, F(T)

F(T)	Rock		Stiff Soil		Soft Soil	
	NF	FF	NF	FF	NF	FF
Conventional Bridge:						
6th Generation of hazard Canada	0.54	0.54	1.00	1.00	1.40	1.40
Direct method	1.13	1.01	1.00	1.00	1.27	1.22
Isolated Bridge						
6th Generation of hazard Canada	0.59	0.59	1.00	1.00	1.41	1.41
Direct method	1.33	1.23	1.00	1.00	0.99	0.99

5.4.2 Maximum base shear responses and the effect of ground motion characteristics and SSI

Figure 5.7 shows the maximum base shear response and its correlation with PGA/PGV ratios for different scenarios. The general trend observed is that the peak base shear responses diminish in both conventional and isolated bridges as the PGA/PGV ratio increases, regardless of the distances from the fault rupture and the soil type.

In the conventional bridge and for NF records, the maximum base shear responses are related to the pulse-like records, which all have lower PGA/PGA ratios as well. On average, responses of pulse-like records are higher than records without pulses by 37, 35, 28, and 29 percent for No-soil, Rock, Soil-C, and Soil-D conditions, showing that on softer soils, the differences between responses of pulse-like and records without pulses are decreasing. Records with PGA/PGV<12 cause higher peak base shear responses compared to records with PGA/PGV>12 by the average of 45, 45, 38, and 37 percent in NF records and 89, 58, 43, and 49 percent in FF records, for No-Soil, Rock, Soil-C, and Soil-D, respectively, demonstrating the fundamental significance of frequency content in seismic responses, as well as highlighting the impact of softer soils in diminishing these peak force demands.

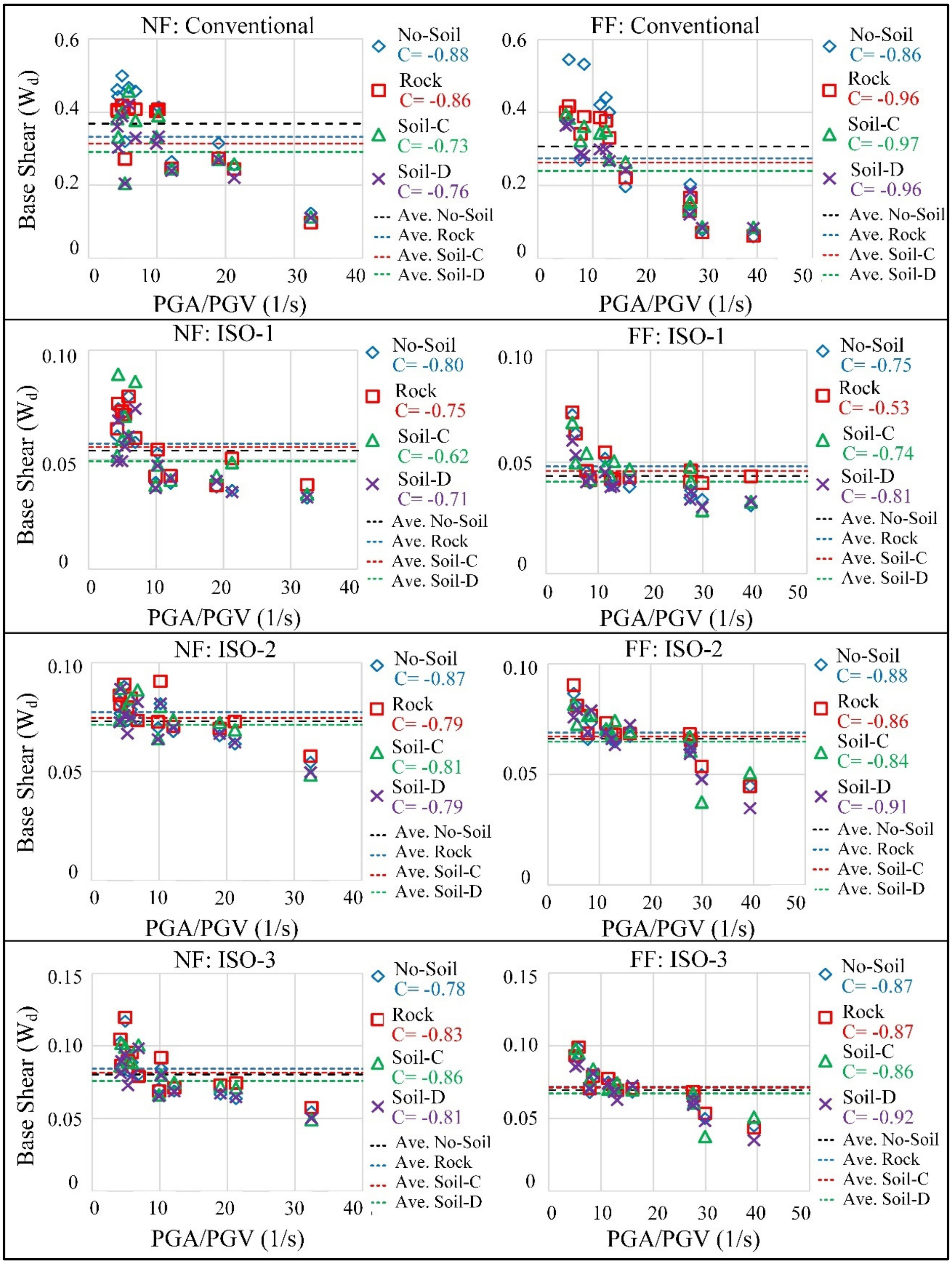

Figure 5.7 Absolute maximum base shear responses versus PGA/PGV ratios,

(C= Correlation coefficient from Anova)

In general, the responses of NF records are also higher than the FF records by an average of 14, 14, 13, and 15 percent for No-Soil, Rock, Soil-C, and Soil-D, respectively. The same trend of higher responses for pulse-like records and lower ratios of PGA/PGV is observed in the isolated bridge.

For ISO-1, responses of pulse-like records in the NF zone are higher than those without pulses by an average of 37, 34, 28, and 29 percent for No-soil, Rock, Soil-C, and Soil-D conditions. On average, base shear responses are higher for NF records with PGA/PGV<12 by 44, 44, 38, and 37 percent compared to PGA/PGV>12, and for FF records, the differences are 31, 20, 19, and 24 percent for No-soil, Rock, Soil-C, and Soil-D condition, respectively.

In NF records, base shear responses are also higher than the FF records by an average of 19, 16, 17, and 16 percent for No-Soil, Rock, Soil-C, and Soil-D, respectively. With the same trend for ISO-2, responses of pulse-like records in the NF zone are higher than records without pulses by an average of 14, 10, 13, and 12 percent for No-soil, Rock, Soil-C, and Soil-D conditions which indicates less differences in base shear responses of pulse-like and records without pulses compared to ISO-1 by the factor of 3. On average, base shear responses are higher in NF records with PGA/PGV<12 by 19, 18, 16, and 17 percent compared to PGA/PGV>12, and for FF records, the differences are 21, 20, 20, and 22 percent for No-soil, Rock, Soil-C, and Soil-D condition, respectively.

For ISO-3, responses of pulse-like records in the NF zone are higher than records without pulses by the average of 24, 20, 23, and 20 percent for No-soil, Rock, Soil-C, and Soil-D conditions. On average, base shear responses are higher in NF records with PGA/PGV<12 by 29, 25, 25, and 24 percent compared to PGA/PGV>12, and for FF records, the differences are 26, 24, 27, and 27 percent for No-soil, Rock, Soil-C, and Soil-D condition, respectively.

The normalized base shear ratios in Figure 5.8 show that based on the average lines, soil has a positive effect in reducing the base shear responses in NF records, for the conventional bridge. On the other hand, SSI might have a positive or negative effect in FF records, depending on each individual record's characteristics.

Despite the general trend of decreasing the maximum base shear responses from No-Soil condition to considering the presence of the soil, especially on softer soils, in the conventional bridge, there are few records in FF zones (F:2, 4, 6, 12), which show the increasing of base shear responses especially on Soil-C and Soil-D up to 45%.

In ISO-1, the base shear responses show a more positive role on Soil-D, while the trend is not clear for Soil-C, showing a significant increase in few records for both NF, (N:2, 7, 12), up to 50%, and FF, (F:1, 2, 4, 10, 11), records up to 40%. In ISO-2 and ISO-3, on average, the base shear responses show less sensitivity to the soil effects and are more neutral for different soils. However, with the same pattern as ISO-1, an increasing trend of base shear responses on Soil-C is observed in few records for NF, (N:2, 7, 12), up to 25%, and FF, (F:1, 2, 4, 10, 11), records up to 17% in both ISO-2 and ISO-3.

The observation of increasing base shear responses on a few earthquake records in both conventional and isolated bridges demonstrates that the base shear responses highly depend on the characteristics and frequency contents of each earthquake record. These findings align with existing literature outcomes, indicating that depending on the interaction between seismic motions and the dynamic properties of the structure, SSI can lead to varied effects on base shear responses, and it should especially be considered for design purposes as ignoring SSI might cause severe damage to the structure (Makris & Zhang, 2004; Ucak & Tsopelas, 2008). Importantly, we observed that the presence of soft soil, introduces a spatial variability of seismic ground motions at the base of piers and introduces a variability in the seismic demand of isolation units and piers taken individually.

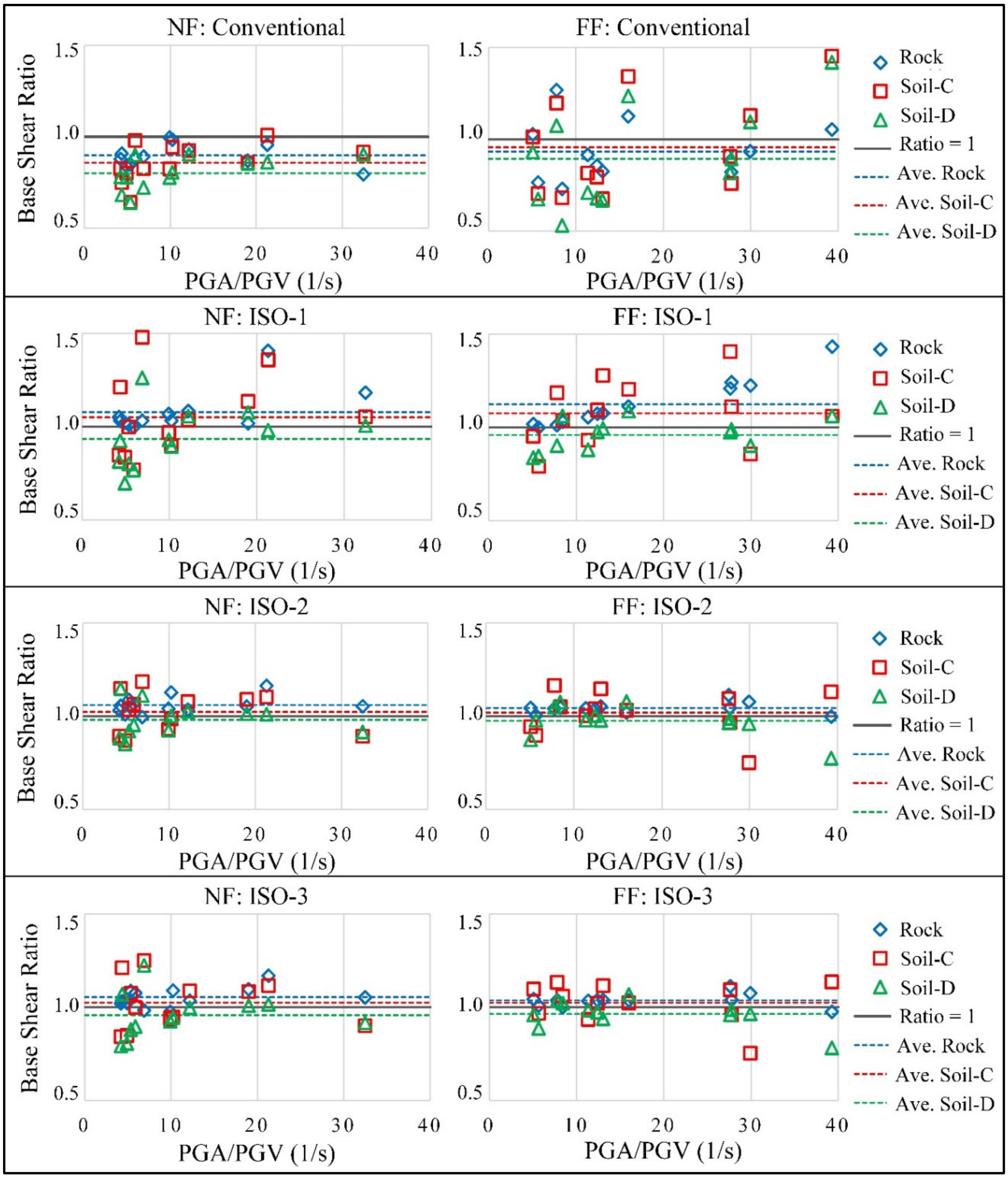

Figure 5.8 Normalized base shear responses

5.4.3 Seismic displacement demands and the effect of earthquake characteristics and SSI

The maximum displacement responses on top of the deck for the conventional bridge and within the isolation units for the isolated bridges on different soil conditions and its correlation with PGA/PGV ratios are shown in Figure 5.9. For the conventional bridge and NF zone, the maximum deck drift is related to the pulse-like earthquake records, and the records with the lowest PGA/PGV ratios for both NF and FF records. On average, the deck drift decreases from No-Soil conventional bridge to Rock by 5% and 2% in NF and FF records which can be explained by the differences in their defined boundary conditions.

With few exceptions, the common trend of the maximum deck drift shows an increasing trend from Rock to softer soils in both NF and FF records. In the NF zone, pulse-like records have higher deck displacement than records without pulses by 40, 40, 58, and 58 percent on average for No-Soil, Rock, Soil-C, and Soil-D, respectively.

For records with PGA/PGV<12, the increasing amount compared to the records with PGA/PGV>12 is 54, 52, 71, and 70% for NF records and 65, 67, 69, and 64% for FF records on No-Soil, Rock, Soil-C and Soil-D, respectively. While, the responses of NF records are higher than FF records by an average of 18, 14, 32, and 30% for the same situation, showing that the effect of PGA/PGV ratios which indicate the frequency content of the records are more predominant than the ruptured fault distance.

In the isolated bridge, the maximum isolator displacements are related to pulse-like records for NF zones and records with low ratios of PGA/PGV in both NF and FF zones. Except for a few cases, the maximum isolator displacements significantly increase on softer soils compared to Rock and No-Soil conditions.

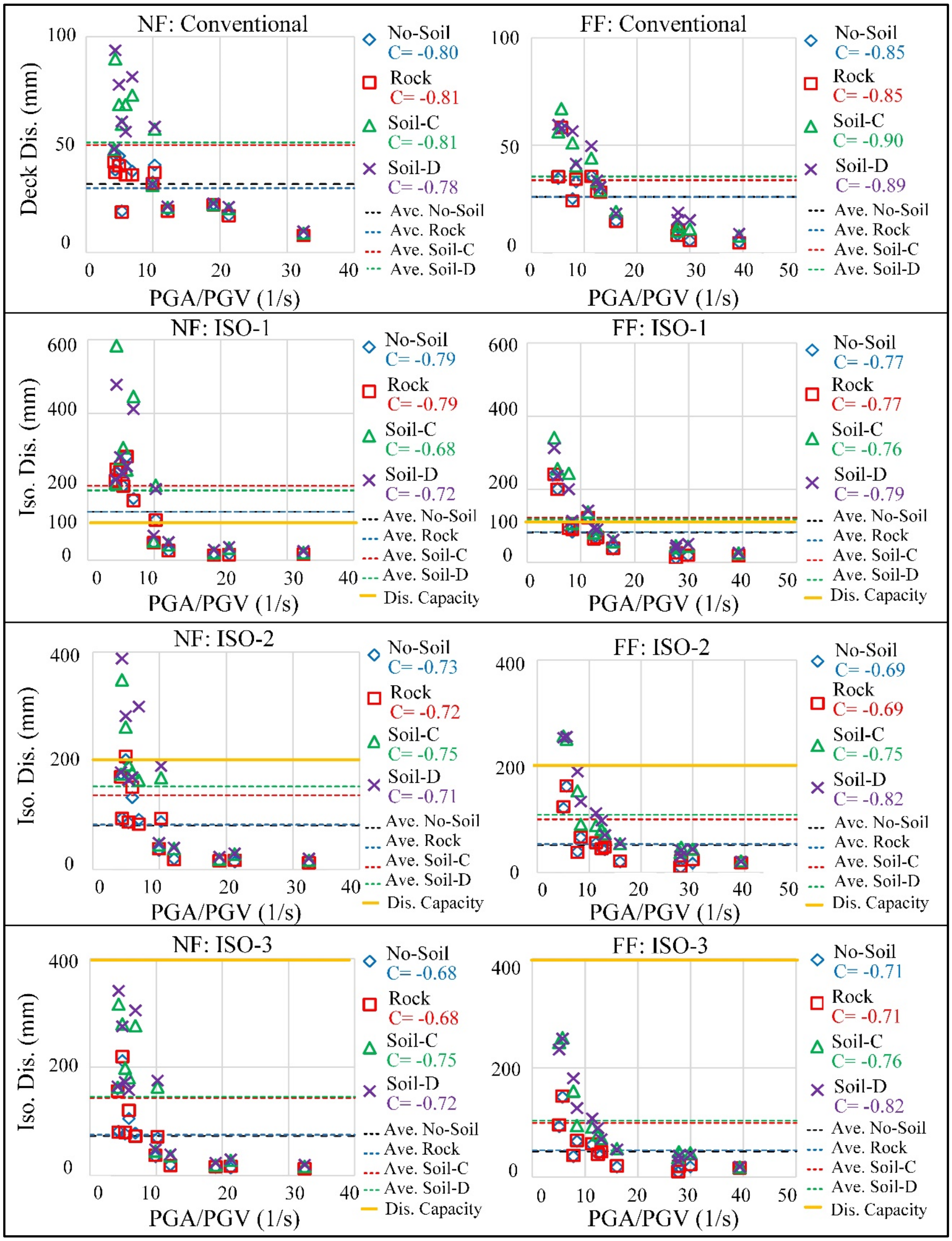

Figure 5.9 Absolute maximum displacement responses versus PGA/PGV ratios, (C= Correlation coefficient from Anova)

For ISO-1, displacement responses of pulse-like records in the NF zone are much higher by a factor of almost 5 compared to records without pulses. However, on average, for NF records with PGA/PGV<12, this factor increases up to 10 times compared to PGA/PGV>12, while for FF records, the responses of records with PGA/PGV<12 are almost 4 times higher than PGA/PGV>12.

The general trend shows an increase in the displacement demand on softer soils. As the displacement capacity of the ISO-1 is 100 mm, except for one NF pulse-like record (N:5), all pulse-like records and NF records with PGA/PGV<12 have a displacement demand more than the capacity, up to three (3) times in softer soils. In FF records, in four records with PGA/PGV<12; (F:3, 4, 5, 9), the displacement demand is higher than the displacement capacity, especially on softer soil.

For ISO-2 and ISO-3 with higher Q_d and displacement capacity compared to ISO-1, displacement responses of pulse-like records compared to those without pulses in the NF zone are higher by a factor of almost 4 for all soil types. In the NF zone and on average for records with PGA/PGV<12, this factor increases up to 7 times compared to PGA/PGV>12, while for FF records, the responses of records with PGA/PGV<12 are almost 4 times more than PGA/PGV>12.

Like ISO-1, the general trend for ISO-2 shows an increase in the displacement demand on softer soils. As the displacement capacity of the ISO-2 is 200 mm, two NF pulse-like records (N:1, 7), both with PGA/PGV<12 have a displacement demand more than the capacity, up to two (2) times in softer soils. In FF records, in two records with PGA/PGV<12; (F:5, 7), the displacement demand is higher than the displacement capacity, especially on softer soil.

As the displacement capacity of the ISO-3 is 400 mm, all the displacement responses in both NF and FF records are less than the displacement capacity. Moreover, a comparison of NF and FF records on the average observation shows that NF records have higher displacement demand than FF records, between 25 to 40 percent in all soil conditions.

Figure 5.9 indicates that the PGA/PGV ratio of the records has a significant effect on the dynamic responses for all soil types and records. Furthermore, as in softer soils, the structure's period increases more than in stiffer soils; for both conventional and isolated bridges, there is an increase in displacement responses when the SSI is included.

The normalized displacement responses ratio presented in Figure 5.10 shows that in the conventional bridge and on average, while soil plays a negative role and increases the displacement responses, soil’s positivity or negativity effects strongly depend on an individual record’s characteristics. For pulse-like and NF records with PGA/PGV<12, displacement on softer soils increases (up to 3 times in few records) and the effect of soil diminishes with increasing the PGA/PGV ratios.

For all isolation systems and under NF and FF records, soil does not play a positive role, and the isolator displacements increase from No-Soil condition to Soil-D up to 3.5 times. It is worth to mention that, in presence of soft soils, the isolation units as well the piers do not respond in phase, leading to a noticeable time shift of their maximum responses, which take place at different times and have significant differences as shown in Figure 5.11.

Considering the average maximum responses of both piers and their seismic isolation units, SSI has a negative effect on the isolator displacement responses, and isolated bridges are sensitive to the SSI effects regarding displacement demands. This negative effect is even more important and amplified when considering the individual maximum responses of piers and their isolation units. Therefore, these findings align with prior research indicating that isolated bridges on soft soils have a more significant potential for experiencing substantial damage, while the isolation systems show a better performance on rocks during earthquakes (Alam & Bhuiyan, 2013b; Dicleli & Buddaram, 2006).

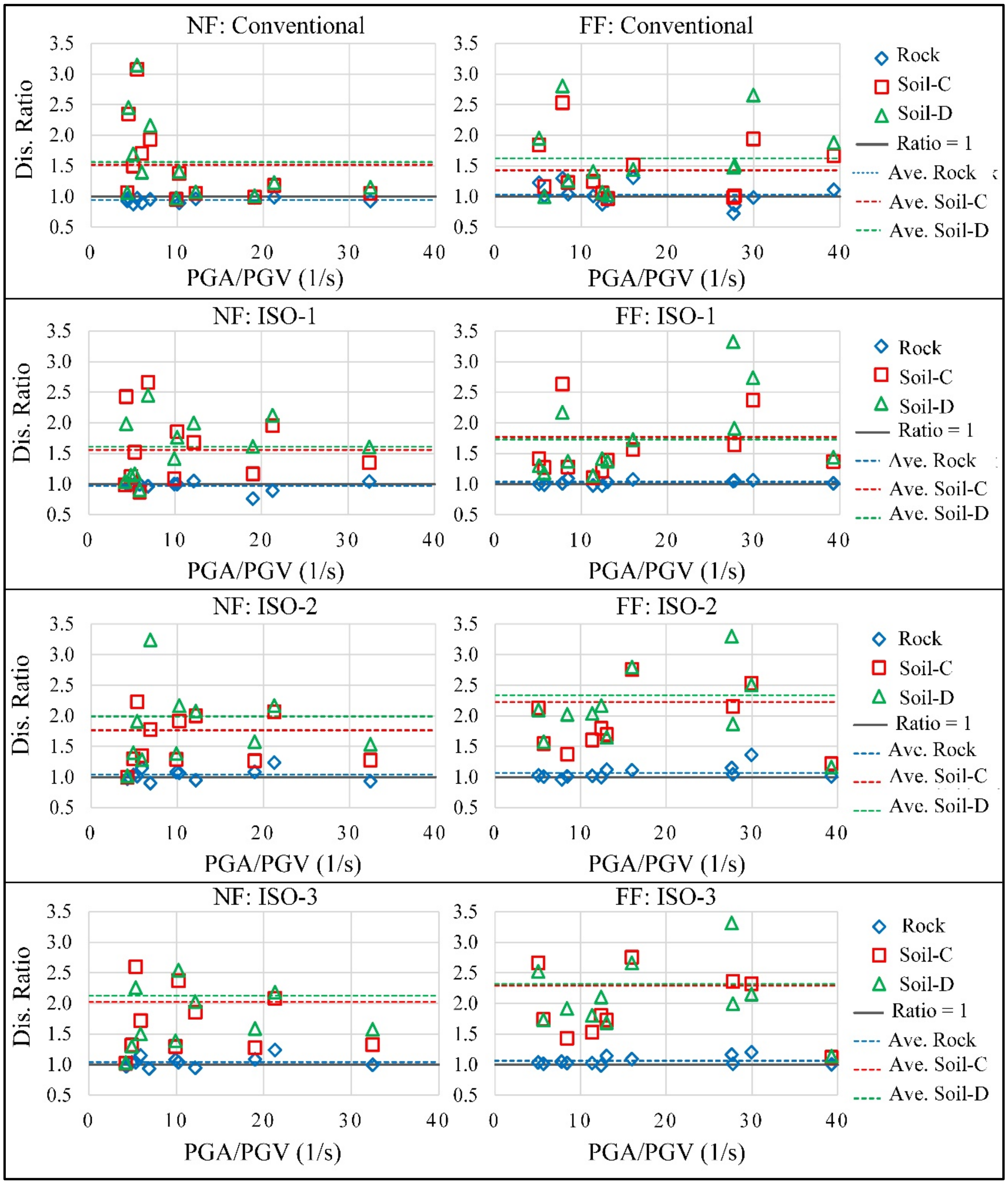

Figure 5.10 Normalized displacement responses

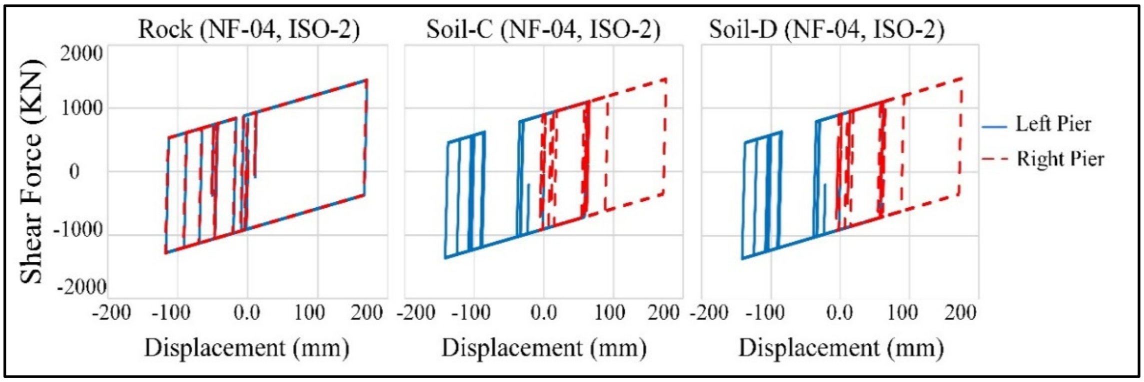

Figure 5.11 Isolation hysteresis loops on different soil types

In conclusion, the analysis of variance (ANOVA) was carried out to determine the effect of PGA/PGV ratios on the seismic responses of the conventional and isolated bridges on different soils. The linear correlation coefficient (shown in Figure 5.5, Figure 5.7, and Figure 5.9) and the probability of error (p-value) were calculated and results show that a strong correlation exists between PGA/PGV and the absolute maximum deck acceleration and base shear, as the p-values are much lower than 0.05 for all soil types and bridges, which shows 95% confidence level. The trend remains consistent for displacement responses and PGA/PGV in both conventional and isolated bridges, with a p-value less than 0.05, except for two cases of No-soil and Rock scenarios for the conventional bridge under FF records.

5.5 Discussion and Conclusion

This paper studied the effects of strong NF earthquakes with and without pulses in their velocity records and FF records simultaneously with SSI effects on a three-span bridge with and without isolation system. Seismic responses of the bridge without considering the presence of soil, as No-Soil condition, are compared with the presence of soil domain and SSI effects in the direct approach. Three different soil groups representing Rock, Soil-C, and Soil-D have been chosen. The role of SSI has been studied by considering the bridge founded on different soil strata subjected to strong NF and FF records. Responses of NLTHAs lead to the following conclusions:

1- The dynamic responses of the bridge depend strongly on the frequency contents of the records, regardless of the distance from the fault. Earthquake ground motions with low ratios of PGA/PGV result in higher seismic displacement and force demands, while these demands decrease as the PGA/PGV ratio increases. The ratio PGA/PGV governs the peak seismic demands and responses and is found to be the prominent parameter.

2- Generally, NF records have lower PGA/PGV ratios, resulting in higher seismic demands compared to FF records. On average, force demand is higher by up to 20 percent, and displacement demand is higher by up to 40 percent for both conventional and isolated bridges on various soils.

3- Pulse-like records, on average, result in higher dynamic responses in terms of force demand, up to 50%, and displacement demand, up to 75%, compared to NF records without pulses.

4- In conventional and isolated bridges, despite the overall decreasing trend of base shear responses on softer soils, there are a few records where a significant increase in base shear is observed in Soil-C and Soil-D. The increase amounts to up to 45% for the conventional bridge, 40% for ISO-1, and 25% for both ISO-2 and ISO-3, respectively. Therefore, careful attention is required during the design stage to anticipate the base shear demand, taking into account the frequency content, bridge condition, and underlying soil properties at the project's site.

5- In isolated bridges, the maximum displacements of the isolation units occur in records with a low ratio of PGA/PGV (<12), and the displacement responses significantly increase on softer soils, Soil-C and Soil-D in this study. In presence of an isolation system designed based on a target displacement within the isolation system of 100 mm for Vancouver location, with an effective period of 2.5sec ratio, such as ISO-1, the designed displacement is exceeded by the displacement demand for all records with PGA/PGV<12, regardless of being NF or FF records. For ISO-2, which is a system with higher Q_d and displacement capacity (two times more than ISO-1), still a few records on softer soils exhibit higher displacement demands beyond its displacement capacity in presence of NF and FF records. In ISO-3, designed following recent research by Nguyen and Guizani (2021) to design an optimal characteristics' range for high seismicity areas with higher Q_d, K_d, and displacement capacity, all records remain within the displacement capacity of the isolation system. Thus, disregarding the characteristics of the records, as well as the flexibility of the soil and SSI effects, will lead to underestimating the displacement demand of the isolated bridge and increasing the risk of destruction in the isolation system in vulnerable areas.

6- Considering the presence of underlying soil plays a positive role in pulse-type records and records with PGA/PGV<12 by reducing the acceleration and base shear responses, especially on softer soils. In contrast, the soil does not show a notable difference in force demand for records without pulses or with PGA/PGV>12.

7- The site coefficient F(T) obtained from the direct method is higher than the reduction factor recommended by 6th Generation of hazard Canada for Rock by a factor of 2, which could result in an underestimation of rock responses. However, for softer soils, the site coefficients of the Canadian code are conservative, when compared to those obtained by this research.

It is worth mentioning that the authors conducted a comparative research study, focusing on NF and FF earthquake records in moderate seismicity areas (Cheshmehkaboodi et al., 2023). Specifically, they targeted Montreal as a representative of medium seismicity areas with PGA=0.444g. By comparing the obtained results, it becomes evident that strong earthquakes result in higher dynamic responses, particularly in terms of displacement demand. The force demand is higher under strong records in high seismicity areas compared to moderate records for medium seismicity areas by factors of 1.3, 2.7, 3.5, and 4 for the conventional, ISO-1, ISO-2, and ISO-3 respectively. Additionally, on softer soils, the force demand decreases by factors of 1.15, 2.3, 3.4, and 3.5 for the same bridge types, thereby highlighting the isolated bridge's susceptibility to strong earthquakes, especially on softer soils.

Similarly, the displacement demand is higher under strong records compared to moderate records by factors of 1.3, 3.9, 2.4, and 2.18 for the conventional, ISO-1, ISO-2, and ISO-3 respectively. Notably, on softer soils, the displacement demand shows an increasing trend by factors of 2, 4.7, 3.2, and 3 for the respective bridge types, thus demonstrating the vulnerable nature of isolated bridges to strong earthquakes in high seismicity areas, particularly when constructed on soft soils. In addition, the study of dynamic responses shows consistent patterns in both moderate and strong earthquakes, emphasizing the influence of ground motion uncertainty and frequency contents on bridge performance and SSI. Low PGA/PGV ratios (<12) have a significant impact on all dynamic responses, resulting in higher force and displacement, regardless of the distance from the ruptured fault.

It should be mentioned that regarding the limitations and objectives of this study and to streamline computation and analysis efforts, many aspects were not considered such as multi-directional seismic excitation, different source mechanisms, spatial variation and non-coherence of the seismic motions, rocking of foundation, analysis of isolated bridges at different levels of the hysteretic properties, especially at upper bound and lower bound values, that could affect the obtained outcomes in different perspectives. Therefore, including these aspects is worth for further investigations.

CONCLUSION AND DISCUSSION

This research study was conducted to investigate the simultaneous effects of different earthquake ground motions and SSI on conventional and isolated bridges. To this end, two groups of earthquake records from NF and FF zones were selected among moderate and strong earthquake events. Strong NF earthquakes were categorized as pulse-like and without a pulse in their velocity records. In addition, both groups of records contain different frequency contents (PGA/PGV ratios). Furthermore, three soil properties were selected to study the underlying soil's effect, and NLTHA was conducted. Finally, the peak responses of acceleration, base shear, displacement of the deck, and displacement within the isolation systems for the conventional and isolated bridges were studied and compared with the case of ignoring the presence of the soil. The highlights of obtained results have been categorized and are as follows:

1- Effect of frequency contents of earthquake ground motions on dynamic responses (PGA/PGV ratio):

For both moderate and strong earthquakes, a strong correlation exists between the force responses and PGA/PGV ratios of the records. The maximum acceleration and base shear responses in both conventional and isolated bridges decrease with an increasing PGA/PGV ratio, emphasizing the significance of this ratio across various soil types and bridge designs.

In terms of displacement responses, including maximum deck drift and the displacement of the isolation system, a consistent correlation is observed between the PGA/PGV ratio and the responses. Higher responses are evident for the lowest ratios of PGA/PGV and pulse-like records. In isolated bridges, for both moderate and strong earthquakes, the displacement demand of records with PGA/PGV < 12 exceeds the displacement capacity of the isolation system designed based on the target displacement, regardless of soil type. This highlights the necessity for design considerations in isolated bridges to accommodate higher displacement and characteristic strength when the PGA/PGV ratio is less than 12 to meet the required demand.

2- Effect of the ruptured fault distance on dynamic responses (NF and FF records):

The results of moderate and strong NF and FF earthquake responses indicate that fault distance does not play a decisive role in the seismic responses of the bridge. Instead, dynamic responses are significantly influenced by the low or high-frequency contents of the records, irrespective of soil type.

Earthquake ground motions with low ratios of PGA/PGV lead to higher dynamic responses, and these responses decrease with an increasing PGA/PGV ratio for all records. Generally, NF records exhibit lower PGA/PGV ratios, resulting in higher dynamic responses compared to FF records. Furthermore, NF pulse-like records elicit higher dynamic responses in terms of force and displacement demands compared to NF records without pulses, necessitating higher displacement capacity for isolated bridges.

3- Effect of the selected model on dynamic responses (direct and simplified methods):

The difference between the direct and simplified (substructure) approaches is significant in the case of softer soils. Using the simplified method should be alongside careful attention to the validity of using the equivalent linear method as all the records captured on Soil-C and Soil-D were not eligible based on the limitation of the shear strain index (generally under 0.03%), therefore, the dynamic responses obtained from those cases were very scattered, especially in the conventional bridge. Consequently, the simplified method of using springs in this study to represent the soil stratum is a rather simple approach to capture all the major mechanisms involved in soil, SSI, and characteristics of each earthquake ground motion.

4- Soil properties, SSI and the dynamic responses:

Considering the average responses, in the majority of cases, SSI is generally favorable for both conventional and isolated bridges, by reducing force responses on softer soils compared to the rock, especially in pulse-type records and records with PGA/PGV<12. The differences in responses are diminished on softer soils in isolated bridges compared to conventional bridges, aligning with the observation that isolated bridges are less sensitive to SSI effects from the force responses aspect. However, the effect of SSI on isolated and conventional bridges is nuanced; the force demands in isolated bridges are much lower than those in conventional bridges.

It should be noted that depending on each earthquake record, a considerable increase in the base shear response might happen on softer soils for both conventional and isolated bridges and the uncertainty in the ground motion dominates the base shear response. Therefore, careful attention is needed at the design stage to anticipate the base shear demand depending on the frequency content, bridge condition, and underlying soil properties.

Furthermore, on softer soils, there is an increase in displacement responses for both conventional and isolated bridges when SSI is considered. The increase is more pronounced for the isolated bridge under records with low PGA/PGV ratios and pulse-type records. Therefore, soil plays a negative role in the displacement demand of the bridges. Thus, meticulous consideration of the site condition and the potential impact of soft soil, where applicable, as well as the level of seismicity and recorded characteristics of the area are crucial in designing isolation systems to avoid underestimating displacement demands and ensure sufficient displacement capacity. The Optimum design of isolation units can provide an adequate displacement capacity for Pulse-type records and records with low ratios of PGA/PGV even on softer soils, without a notable increase in force demand.

5- Site coefficient factor, F(T):

Analysis of bridge responses under both moderate and strong earthquakes reveals that the site coefficient factor, F(T), exceeds the recommended values by CSA(S6-19), and 6th Generation of hazard Canada for rocks. This discrepancy may lead to an underestimation of rock responses, indicating the need for reassessment and adjustment of design factors. Conversely, the factor proves to be conservative for soft soils. However further research and consideration of additional records and models is deemed necessary to this end.

6- Dynamic responses for moderate and strong earthquakes:

Despite using different PGAs in this study for medium and high seismicity areas (PGA=0.444g for Montreal as a representative of a medium seismicity area and PGA=0.32g for Vancouver as a representative of a high seismicity area), and considering the variations in their characteristics and frequency contents, it is observed that the force and displacement demand in strong earthquakes is higher than during moderate earthquakes in all bridges. These findings highlight the effect of frequency content and other characteristics of the earthquakes such as PGV and PGD, on the dynamic responses.

The comparison of responses between both groups demonstrates a consistent pattern, indicating that the performance of the bridge and soil-structure interaction (SSI) is primarily influenced by the ground motions and their frequency contents. Low ratios of PGA/PGV are found to significantly impact all dynamic responses, with records exhibiting PGA/PGV values less than 12 resulting in higher force and displacement responses, regardless of the distance associated with the ruptured fault.

RECOMMENDATIONS AND LIMITATIONS OF THIS STUDY

In this research study, a modified Mohr-Coulomb constitutive model was used as a soil failure criterion to simulate nonlinearity in soil components. The precise laboratory test results of the soil strata and a more comprehensive continuum modeling techniques approach that can accurately capture the cyclic response of structural components are recommended to further investigate the nonlinearity in soil components.

The assumption of non-liquefiable sandy soil has been used in this study. Therefore, the soil liquefaction phenomenon and other soil properties must be investigated. In addition, spatial variation and non-coherence of the seismic motions, multiple support excitations were not studied which are important aspects for long bridges in accidented sites.

The transverse direction and vertical component of the ground motions can affect the seismic response of the soil and bridge systems. Therefore, the significance of the response of the bridge in the transverse direction and vertical ground motions needs to be investigated.

The embedded foundation is the assumption of the modeling in this study, while the different foundation systems might lead to other results. Therefore, studying the simultaneous effects of different pile systems and record characteristics on isolated bridges will improve understanding of SSI and seismic responses of bridges in prone areas.

Simplified (substructure) method in this study consisted of only springs and the effect of damping or nonlinear springs and dashpots model was not studied which is worth further study to complete the results of the comparison.

The generic bilinear model of isolation system is used in this study, elaborated SI hysteretic models can add a complimentary understanding and result.

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