

POLYTECHNIQUE MONTRÉAL

affiliée à l'Université de Montréal

**Nonlinear Shear Stress Reduction Factor (r_d) for the Assessment of
Liquefaction Triggering in Eastern Canada for Induced Earthquakes**

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Ce mémoire intitulé :

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présenté par **Arvin YAZDANI DAMAVANDI**

en vue de l'obtention du diplôme de *Maîtrise ès sciences appliquées*

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DEDICATION

To Poune and my parents

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RÉSUMÉ

La liquéfaction des résidus miniers constitue une préoccupation majeure au sein de l'industrie minière. La liquéfaction peut se produire en raison d'un chargement statique ou dynamique, ce que l'on appelle la liquéfaction cyclique. La liquéfaction cyclique est généralement déclenchée par des tremblements de terre naturels ou induits. Cette étude se concentre sur la liquéfaction cyclique déclenchée par les tremblements de terre induits par les activités minières. Les tremblements de terre induits se produisent généralement à proximité des activités d'extraction minière souterraine avec une magnitude inférieure à six et peuvent potentiellement conduire à la défaillance des parcs à résidus. Il existe plusieurs approches pour évaluer la probabilité de déclenchement de la liquéfaction ; cependant, la méthode simplifiée initialement développée par Seed et Idriss (1971) est largement utilisée dans la pratique. Un point clé de la méthode simplifiée est la détermination de la demande sismique, qui est fonction des contraintes de cisaillement cycliques induites dans le profil du sol en raison du tremblement de terre. Puisque le sol se comporte comme un corps déformable, la méthode simplifiée introduit un facteur de réduction des contraintes de cisaillement non linéaire pour capturer la non-linéarité et déformabilité du sol. Plusieurs modèles, dont Idriss (1999), Youd et al. (2001) et Cetin et al. (2004), ont proposé des relations empiriques pour ce facteur de réduction ; cependant, tous ces modèles utilisent des analyses linéaires équivalentes et des enregistrements de provenance de régions sismiquement actives avec des magnitudes élevées et un contenu à basse fréquence, incompatibles avec les tremblements de terre induits par l'exploitation minière. Cette étude vise à remédier à cette limitation, en dérivant un nouveau facteur de réduction des contraintes de cisaillement non linéaire compatible avec les tremblements de terre induits par l'exploitation minière dans l'est du Canada en effectuant une analyse de réponse non linéaire du site.

Cette étude a choisi 169 mouvements de sol non mis à l'échelle de la base de données PEER, englobant à la fois les ensembles de données NGA-East et NGA-West2. La sélection s'est concentrée sur des séries temporelles caractérisées par des magnitudes relativement faibles et des distances proches afin de simuler au mieux les conditions sismiques induites. De plus, WSP Inc. a fourni des mesures in situ de la vitesse de propagation des ondes de cisaillement de 14 profils de résidus pour un site minier de l'ouest du Québec. À l'aide du logiciel DEEPSOIL, des analyses de réponse du site ont été effectuées pour déterminer la variation de la contrainte de cisaillement induite dans chaque profil, parallèlement à la détermination du PGA à la surface.

Sur la base des résultats de l'analyse DEEPSOIL et dans le cadre de la méthode simplifiée, le facteur de réduction des contraintes de cisaillement non linéaire a été calculé pour chaque profil de sol. Par la suite, pour formuler une relation de facteur de réduction, des analyses de régression multivariée non linéaire ont été exécutées, en adoptant les mêmes formulations proposées par Idriss (1999) et Cetin et al. (2004). Suite à ces analyses, quatre ensembles de paramètres d'ajustement ont été obtenus, chacun correspondant à un modèle spécifique (i.e. Idriss 1999 ou Cetin et al. 2004) et à une origine des mouvements de sol utilisés (sismiquement stable ou active). Des analyses résiduelles ont ensuite été effectuées sur les courbes ajustées pour déterminer les biais potentiels.

Les résultats de l'étude indiquent que les modèles actuels ont tendance à générer des facteurs de réduction trop conservateurs, principalement lorsqu'ils sont appliqués à des régions plus stables comme l'est du Canada. Les analyses résiduelles des nouvelles relations proposées ne révèlent aucun biais significatif, ce qui suggère que les deux nouvelles relations sont plus précises que les modèles existants. Par conséquent, ces nouvelles relations peuvent être appliquées en toute confiance aux parcs à résidus exposés à la sismicité induite par l'exploitation minière, se révélant efficaces dans les régions stables et sismiquement actives.

ABSTRACT

The liquefaction of tailings impoundments stands as a paramount concern within the mining industry. Liquefaction can occur due to static or dynamic loading, which the later so-called cyclic liquefaction. Cyclic liquefaction can be triggered by natural or induced earthquakes. This study focuses on the cyclic liquefaction that is triggered by induced earthquakes. Induced earthquakes commonly occur in proximity to the mining subsurface extraction activities with a magnitude lower than six and can potentially lead to failure of tailings storage facilities (TSFs). There are several approaches to assess the probability of liquefaction triggering; however, the simplified method originally developed by Seed and Idriss (1971) is widely used in practice. One key point in the simplified method is determining the seismic demand, which is a function of cyclic shear stress induced in the soil profile due to the earthquake. Since the soil behaves as a deformable body, the simplified method introduced a nonlinear shear stress reduction factor to capture the nonlinearity of the soil. Several models, including Idriss (1999), Youd et al. (2001), and Cetin et al. (2004), have proposed relationships for this reduction factor; however, they all used equivalent linear analysis and input motions from seismically active regions with high magnitudes and low frequency content, inconsistent with mining-induced earthquakes. This study aims to address this limitation, deriving a new nonlinear shear stress reduction factor compatible with mining-induced earthquakes in Eastern Canada by performing a nonlinear site response analysis.

This study chose 169 unscaled ground motions from the PEER database, encompassing both NGA-East and NGA-West2 datasets. The selection focused on time histories characterized by relatively low magnitudes and close distances to simulate induced seismic conditions closely. Furthermore, WSP Inc. provided shear wave velocity measurements from 14 tailings profiles at a Western Quebec mining site. Utilizing the DEEPSOIL software, site response analyses were carried out to ascertain the variation of induced shear stress across each profile under distinct ground motions, alongside the determination of the PGA at the surface.

Based on the DEEPSOIL analysis results within the simplified method framework, the nonlinear shear stress reduction factor was computed for each soil profile. Subsequently, to formulate a reduction factor relationship, nonlinear multivariate regression analyses were executed, adopting the same formulation proposed by Idriss (1999) and Cetin et al. (2004) models. Following these analyses, four sets of fitting parameters were obtained, each corresponding to a specific model (i.e.,

Idriss 1999 or Cetin et al. 2004) and the origin of the employed ground motions (seismically active and continental stable regions). Residual analyses were then performed on the fitted curves to determine the potential biases.

The study findings indicate that previous models exhibit a tendency to generate overly conservative reduction factors, mainly when applied in inherently more stable areas like Eastern Canada. Residual analyses of the proposed relationships revealed no significant bias, suggesting that both new relationships are more accurate than existing models for induced seismicity. Consequently, although the study's conclusions should be interpreted with caution given the use of data from a single gold mine and the limitations of the ground motion recordings used, these novel relationships can be applied to tailings impoundments exposed to mining-induced seismicity, proving effective in both stable and seismically active regions.

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LISTE OF SYMBOLS AND ABBREVIATIONS

1D	One dimension
AI	Arias intensity
a_{max}	Maximum horizontal acceleration at the ground surface
ARCS	Axis rotation and cubic spline
c'	Effective cohesion
C_σ	Coefficient of overburden correction
CDSS	Cyclic direct simple shear
CPT	Cone penetration test
CRR	Cyclic resistance ratio
CRR_M	Cyclic resistance ratio at a given earthquake magnitude
CSL	Critical state line
CSR	Cyclic stress ratio
D	Damping ratio
ξ	Damping ratio
D_{5-95}	Significant duration corresponding to 5 to 95 % of AI
D_M	Damping according to Masing rules
D_{min}	Small-strain damping ratio
D_r	Relative density
DSRA	Dynamic site response analysis
DSS	Direct simple shear
e	Void ratio
e_c	Void ratio after consolidation
EL	Equivalent linear

f	frequency
FC	Fines' content
FS	Factor of safety
γ	Shear strain
γ_{eff}	Effective shear strain
γ_d	Dry unit weight
γ_{max}	Maximum shear strain
γ_{ref}	Reference strain
γ_w	Unit weight of water
g	Acceleration due to gravity
G	Shear modulus
G_{max}	Maximum shear modulus or elastic shear modulus
GQ/H	General quadratic/hyperbolic
I_c	Soil classification index
K_0	In-situ stress ratio
K_α	Static shear stress correction factor
K_σ	Overburden correction factor
LL	Liquid limit
M	Earthquake moment magnitude
MASW	Multichannel analysis of surface waves
MR	Modulus reduction
MRD	Modulus reduction and damping
MSF	Magnitude scaling factor
$(N_1)_{60}$	Corrected standard penetration test blow count

$(N_1)_{60,cs}$	Equivalent clean sand blow count
NGA-East	Next Generation Attenuation Relationships for Central & Eastern North-America
NGA-West2	Next Generation Attenuation Relationships for Western US
NL	Nonlinear
OCR	Over consolidation ratio
ρ	Density
p_a	Atmospheric pressure
P_L	Probability of liquefaction
PEER	Pacific earthquake engineering research center
PI	Plasticity index
PGA	Peak ground acceleration
PGV	Peak ground velocity
PL	Plastic limit
PSA	Pseudo spectral acceleration
q_c	Tip resistance from CPT
q_{c1N}	Corrected CPT tip resistance
r_d	Nonlinear shear stress reduction factor
R_{jb}	Joyner-Boore distance
RSN	Record sequence number
σ_v	Total stress
σ'_0	Mean effective stress
σ'_v	Effective vertical stress
σ'_{vc}	Effective vertical consolidation stress
S_a	Spectral acceleration

SASW	Spectral analysis of surface waves test
SCPT	Seismic cone penetration test
SLS	Serviceability limit state
SPT	Standard penetration test
S_r	Saturation ratio
S_u	Undrained shear strength
τ	Shear stress
τ_{cyc}	Cyclic induced shear stress
τ_{max}	Maximum shear stress
T	Period
TSF	Tailing storage facility
u	Pore water pressure
ULS	Ultimate limit state
ϕ	Internal friction angle
ϕ'	Effective internal friction angle
V_s	Shear wave velocity
V_{s12}	Average shear wave velocity of the first 12 meters below surface
V_{s30}	Average shear wave velocity of the first 30 meters below surface
W_c	Water content

CHAPTER 1 INTRODUCTION

1.1 Context

Tailings are the residual waste materials produced during the milling operations in the mining industry, typically comprising fine-grained non-plastic particles. They are conventionally stored in tailings impoundments in saturated conditions due to being transported as a slurry (Vick 1990). Owing to their high saturation levels and relatively loose characteristics, which lead to contractive behavior, tailings impoundments are susceptible to liquefaction. Liquefaction commonly occurs when soil loses its static equilibrium due to static or dynamic loading, referred to as static and cyclic liquefaction, respectively. Earthquakes most often represent the critical factor in triggering cyclic liquefaction. The required earthquake typically originates from seismic faults due to the relative movements of tectonic plates or be caused by human activities. Such earthquakes are called induced earthquakes. Various industrial processes can potentially induce seismic events, with most of them associated with subsurface extraction and injection activities. In quarrying operations, the removal of overlying mass from the Earth results in a significant reduction in the effective stress around a fault. Effective stress plays a pivotal role in maintaining fault closure, so any reduction in effective stress subsequently reduces the required shear stress for fault failure and triggering an earthquake. According to Foulger et al. (2018), mining ranks second globally in terms of the number of induced earthquakes. These mining-induced earthquakes are typically shallow, and close to the mine's operation. Consequently, they pose a threat to the tailings storage facilities, even though their magnitude tends to remain lower than 6. The consequences of liquefaction failure in tailings impoundments can be catastrophic, resulting in substantial economic losses, environmental damage, and human casualties.

To minimize these losses, it is imperative to understand the mechanisms that trigger liquefaction in tailings dams and to assess the potential for its occurrence. The simplest way to assess the potential for liquefaction triggering is using the simplified method (Seed and Idriss 1971, Idriss and Boulanger 2008, Boulanger and Idriss 2014). This involves comparing seismic demand and resistance either by calculating a factor of safety or using a probabilistic method. The computation of the seismic demand relies on defining a depth-dependent reduction factor that defines how PGA and shear stress varies with depth. Building upon Seed and Idriss' work from 1971, several studies have further derived a relationship for calculating a reduction factor for seismic demand, such as

Idriss (1999), Youd et al. (2001), and Cetin et al. (2004). These studies compute reduction factors using equivalent linear analysis and input motions from seismically active regions with high magnitudes and low frequency content, inconsistent with mining-induced earthquakes. When comparing seismic events between Western and Eastern North America, it can be observed that the Western region experiences higher seismic activity, whereas the Eastern region is relatively more stable, and the motions from stable continental regions have a higher frequency content. The limitations of equivalent linear analysis are also now well-established (e.g. Kaklamanos et al. 2015), and the current methods are therefore not adapted to estimate the potential of liquefaction of tailings due to mining-induced earthquakes.

1.2 Objective

As elaborated in the preceding section, liquefaction emerges as a prominent factor contributing to the failure of tailings dams. However, a limited attention has been devoted to the potential of liquefaction due to mining-induced earthquakes. While various models have been developed for assessing the potential for liquefaction triggering for seismically active regions, they exhibit some limitations when applied to the assessment of liquefaction triggering in tailings impoundments located in Eastern Canada for low magnitude earthquakes. The main objective of this thesis is to provide new empirical relationships for the reduction factors r_d for low-magnitude events situated at short distances, typical of mining-induced events, for both high and low-frequency content motions. The secondary objectives of this thesis are as follows:

1. Select ground motion recordings representative of mining-induced earthquakes, with a low magnitude and recorded at a close distance to the fault rupture plane
2. Study the influence of different factors (e.g. PGA, Magnitude...) on the reduction factors, and investigate the effect of the frequency content of the input motions on the reduction factors.

To achieve these goals, fourteen tailings profiles have been used to perform nonlinear ground response analysis, and each of these profiles reflects realistic conditions from mine tailings impoundments located in Western Quebec. Approximately half of the utilized input ground motions in the simulations are selected from recorded earthquakes in stable regions located in Central and Eastern North America. Given the scarcity of recorded data in the regions of Central and Eastern North America, the remaining selected ground motions are sourced from a Western

North America database, specifically. In order to study induced seismicity conditions, only ground motions with low magnitude (i.e., $3 < M < 6$) and occurring at relatively short distances (i.e., $0 < r < 50$ km) are selected.

1.3 Methodology

The following steps have been followed to derive new relationships of reduction factors for assessing liquefaction triggering of tailings impoundments in Eastern Canada:

1. Time histories have been chosen from historical earthquakes that occurred in seismic region representative of Eastern Canada. These data were sourced from the NGA-East dataset (Goulet et al. 2021) within the PEER Ground Motion Database. Nevertheless, due to the scarcity of recorded earthquakes in stable regions like Central and Eastern North America, an additional set of ground motion data was obtained from the NGA-West2 dataset (Ancheta et al. 2014). This also allows to broaden the scope of the study, and investigate the influence of the frequency content on the reduction factor. The motions were selected to have low magnitudes and be recorded at short distances to align with potential induced seismic events in the mining sector.
2. To conduct ground response analysis, WSP Inc. has supplied shear wave velocity measurements from 14 undisclosed, actual tailings profiles located at a mining site in Western Quebec.
3. A MATLAB code from Zhang's master's thesis at Polytechnique Montreal in 2021 has been modified and used to interpret the dynamic characteristics of these unprocessed soil profiles. This code autonomously carries out the layering of each profile and calculates the essential properties necessary for subsequent analyses.
4. Nonlinear ground response analyses were performed using DEEPSOIL V7.0 (Hashash et al. 2017) on each soil profile. Ultimately, once the nonlinear site response analyses with DEEPSOIL were carried out, nonlinear shear stress reduction factor for each soil profile were computed.
5. A regression analysis of the reduction factor was performed to derive two new empirical relationships for reduction factors for induced-seismicity, for both high and low frequency motions. The influence of different factors on the reduction factor was also examined.

1.4 Organization of the thesis

This thesis is organized into five chapters, including an introduction as follows:

Chapter Two (Literature Review): delves into the relevant concepts and previous research efforts related to the subject matter, including topics such as the phenomenon of liquefaction, the procedure for assessing liquefaction triggering, site-specific response analysis, and an overview of the behavior of tailings.

Chapter Three (Methodology): outlines the necessary steps for deriving a new nonlinear shear stress reduction factor based on the results obtained through the nonlinear site response analysis conducted using DEEPSOIL. The selection of input ground motions and the profiles simulated are discussed.

Chapter Four (Results): presents the computed reduction factors based on the results of DEEPSOIL analyses, with respect to various variables such as depth, frequency content, magnitude and PGA, and discusses regression analysis method to establish a new relationship for the obtained reduction factor profile. The performance of the new relationships for the reduction factor is presented.

Chapter Five (Conclusions): provides a summary of the achievements of this thesis, highlights its limitations, and proposes recommendations for future research.

CHAPTER 2 LITERATURE REVIEW

2.1 Liquefaction overview

2.1.1 Liquefaction definition and mechanism

The definition of liquefaction provided by Idriss and Boulanger (2008) reads as follows:

"Liquefaction is observed when loose sand contracts under the cyclic loading that occurs during earthquake shaking. This cyclic loading has the capacity to transfer normal stress from the sand matrix to the pore water, especially if the soil is fully saturated and experiences restricted drainage while undergoing shaking. As a result, there is a reduction in the effective confining stress within the soil. This reduction leads to a loss of both strength and stiffness, ultimately contributing to deformations within the soil deposit."

The term liquefaction is typically used to refer to two different phenomena: flow liquefaction and cyclic mobility. Flow liquefaction occurs when static or dynamic loads disrupt the static equilibrium in a low residual strength soil. In this case, the residual strength usually becomes lower than the static shear stress. Flow liquefaction failures are frequently characterized by massive and rapid movements, which can have disastrous consequences. Cyclic mobility is caused by cyclic loading in soils with static shear stresses less than the soil strength. Deformations caused by cyclic mobility emerge gradually due to static and dynamic forces present during an earthquake. It typically results in lesser failures and restricted displacements. Lateral spreading is a common consequence of cyclic mobility.

For the occurrence of a liquefaction, three conditions need to be satisfied:

1. Saturation:

The soil subject to liquefaction must be fully or almost fully saturated with water. This requirement stems from the fact that if voids within the soil are occupied by air, a reduction in void ratio will occur upon cyclic loading instead of generating excess pore water pressure.

2. Position with respect to the Critical State Line:

Referring to the schematic diagram below, any soil positioned to the right of the Critical State Line (CSL) is categorized as a loose soil and will exhibit a contractive behavior. Conversely, soils positioned to the left of the CSL are dense soils and will exhibit dilative behavior. Under undrained cyclic loading, loose soils progressively approach a critical state with an increase in excess pore water pressure while maintaining a constant void ratio. This increase in pore water pressure results in the cancellation of the effective stress, and liquefaction. In contrast, dilative soils may experience an augmentation in strength during cyclic loading, attributed to negative pore water pressure generation.

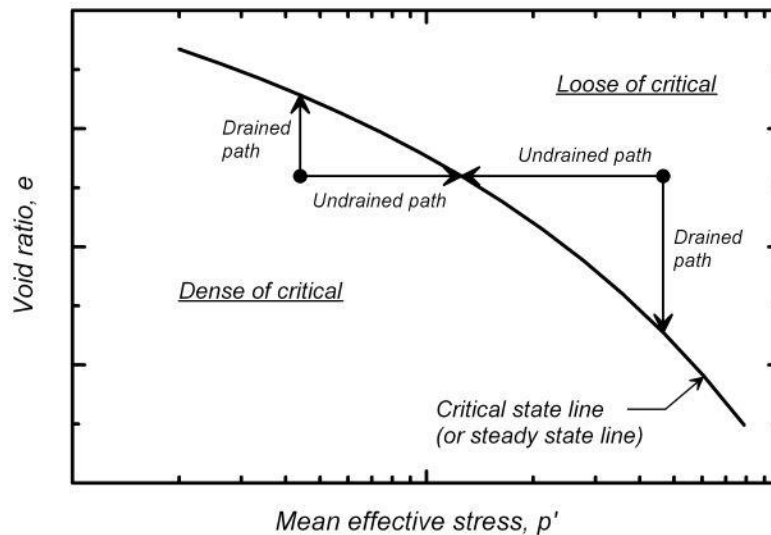


Figure 1 Stress paths for monotonic drained and undrained loading of saturated loose and dense sands (Idriss and Boulanger 2008)

3. Loading Rate:

As mentioned earlier, loose soils subjected to cyclic loading must follow an undrained path to generate excess pore water pressure. Hence, to achieve this state, the loading must be applied rapidly to the soil, preventing water drainage. Earthquakes inherently meet this condition.

2.1.2 Liquefaction susceptibility

Even when all the aforementioned prerequisites have been met, liquefaction can only occur in soils classified as liquefiable. From a geotechnical standpoint, liquefaction susceptibility primarily applies to granular soils. While cohesive materials like clays, peats, and plastic silts can also undergo substantial strains and produce notable pore water pressure during seismic events, it is advisable to employ the term "cyclic softening" for these cases due to the contrasting shear strength characteristics between cohesion-less and cohesive soils.

Numerous methods exist for assessing the susceptibility of soil to liquefaction. One such approach involves establishing empirical soil susceptibility criteria through their Atterberg limits, including the Liquid Limit (LL) and Plasticity Index (PI). This method classifies soils into two categories: "clay-like" and "sand-like". Given that clay-like soils are generally regarded as non-liquefiable, while sand-like soils are considered susceptible to liquefaction, this classification helps determine the liquefaction susceptibility of each distinct soil type. Two of these criteria are outlined herein:

1. Bray and Sancio (2006)

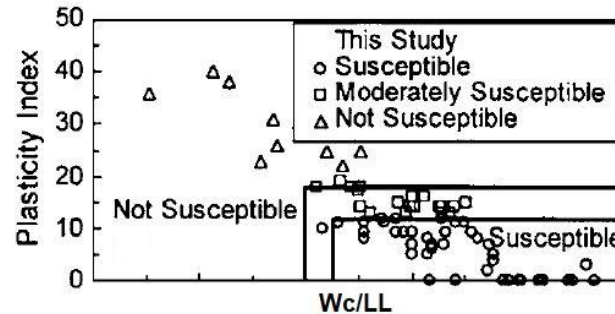


Figure 2 Susceptibility criteria by Bray and Sancio (2006)

Based on this criteria soils with $PI < 12$ and $W_c/LL > 0.85$ are susceptible to liquefaction, soils with $12 < PI < 20$ and $W_c/LL > 0.8$ are moderately susceptible to liquefaction, and soils with $PI > 12$ are not susceptible to liquefaction.

2. Boulanger and Idriss (2006)

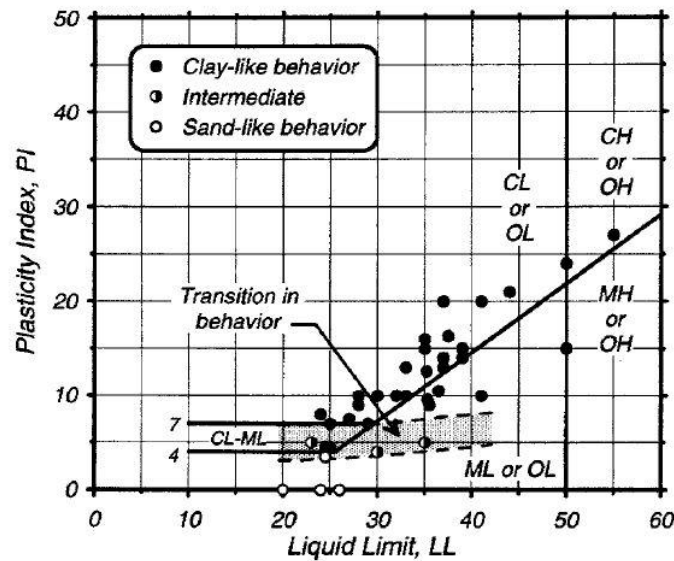


Figure 3 Susceptibility criteria by Boulanger and Idriss (2006)

According to this criteria soils with $PI > 7$ are clay-like and not susceptible to liquefaction and soils with $PI < 7$ are sand-like and susceptible to liquefaction.

2.1.3 Liquefaction triggering evaluation procedure

Numerous methodologies have been employed throughout the years to assess the likelihood of liquefaction occurrence. Among these methodologies, the stress-based framework holds a predominant position, being extensively utilized for both academic and practical purposes (Idriss and Boulanger 2008). Conversely, alternative approaches, including strain-based or energy-based methods, find less frequent application. The stress-based methodology is grounded in a sequence of three fundamental steps:

6. Characterizing seismic demand
7. Assessing cyclic strength of soil
8. Comparing seismic demand and cyclic strength of soil

2.1.3.1 Characterizing seismic demand

When a seismic event happens, the resultant horizontal movements induce cyclic stresses within the soil profile. Within the stress-based framework, the induced cyclic shear stress is compared against the soil's natural cyclic resistance. This forms the fundamental basis for assessing the potential for liquefaction occurrence. This relationship is graphically presented below, where the critical depth for the onset of liquefaction aligns with the point at which the cyclic shear stress exceeds the soil's threshold of cyclic resistance.

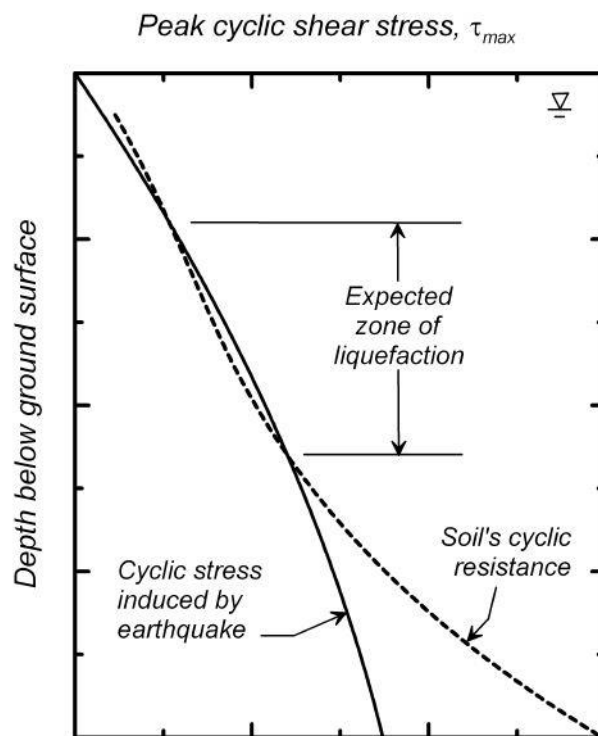


Figure 4 The projected liquefaction zone is ascertained through a comparison between the cyclic stresses induced by the earthquake and the cyclic resistances inherent in the soil (Idriss and Boulanger 2008)

The induced cyclic shear stress is normalized by the initial vertical effective stress to derive a CSR (Cyclic Shear Ratio), representing the solicitation.

Typically, two approaches exist to determine the CSR for a given soil. The first involves conducting site response analyses, while the second employs the simplified method originally proposed by Seed and Idriss (1971). While site response analyses offer heightened precision, they

necessitate the determination of dynamic properties for the entire soil profile down to the bedrock. The simplified method is more widely embraced, and its application is discussed in the subsequent section.

2.1.3.1.1 Simplified method by Seed and Idriss (1971)

As explained in previous section, when a soil column is subjected to seismic forces, the lateral displacements induce shear stresses within this column. If this soil column which extends to a depth of z from the ground surface (Fig. 5), was considered as a rigid body, the calculation for the maximum cyclic shear stress imposed on the column would be as follows:

$$(\tau_{max})_r = \frac{\gamma \cdot z}{g} a_{max} = \sigma_v \frac{a_{max}}{g} \quad \text{Equation 1}$$

where γ is the average total unit weight of the soil column, z is the depth of the soil column, g is gravity acceleration, σ_v is the total vertical stress at the bottom of the column and a_{max} (peak ground acceleration) is the maximum ground acceleration at the surface, often referred to as the PGA.

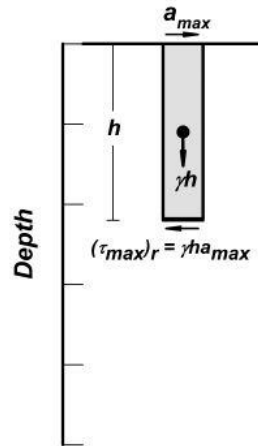


Figure 5 Schematic view of the soil column with depth of z and its maximum induced shear stress (Idriss and Boulanger 2008)

However, it is important to note that soil is deformable. Thus, in order to effectively apply the equation mentioned earlier, it is necessary to make the following correction demonstrated in Figure 6 to the deformable maximum shear stress:

$$(\tau_{max})_d = r_d(\tau_{max})_r \quad \text{Equation 2}$$

where r_d is the shear stress reduction factor.

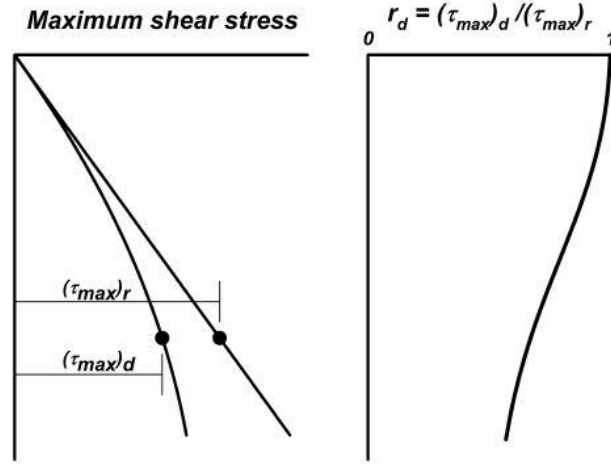


Figure 6 Rigid and deformable maximum shear stress in the soil column (Idriss and Boulanger 2008)

Given that earthquake ground motions are composed of multiple loading cycles with varying strengths, Seed and Idriss (1971) proposed an adjustment that involves utilizing an equivalent uniform value of induced cyclic shear stresses instead of the peak cyclic stress. For this purpose, an empirical factor of 0.65 was selected (i.e., $CSR = 0.65 \frac{\tau_{max}}{\sigma'_v}$).

2.1.3.1.2 Shear stress reduction factor (r_d)

The right-hand diagram in Figure 6 presents the typical shear stress reduction factor trend. This factor is one close to the ground surface, and it experiences a gradual decrease as the depth increases. For a specific soil profile, r_d can be computed by using one-dimensional Dynamic Site Response Analyses (DSRA). Several studies have conducted a series of site response analyses on a multitude of soil profiles and using a large set of input ground motions. By regressing the results of the simulations, an empirical relationship for determining the value of r_d can be derived. The following chapter presents three of the most commonly employed models.

1. Youd et al. (2001)

The use of r_d was originally established by Seed and Idriss (1971), and they presented charts to compute it. A few one-dimensional site response analyses were performed using an equivalent linear procedure with the SHAKE software (Schnabel et al. 1972). The selected profiles were a hundred feet tall sand columns above bedrock. The r_d range by Seed and Idriss is shown in Figure 7. The relation derived by Youd et al. (2001) was merely the mean value for the original r_d curve from Seed and Idriss (1971) study. The authors suggested their magnitude independent reduction factor relation as follows:

$$r_d = \frac{(1.000 - 0.4113 z^{0.5} + 0.04052z + 0.001753z^{1.5})}{(1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2)} \quad \text{Equation 3}$$

where z is the depth of the soil profile in meters.

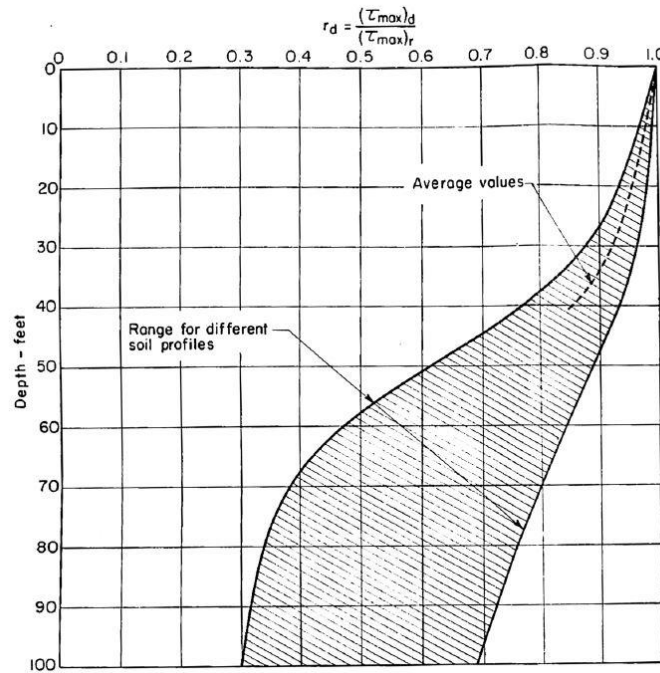


Figure 7 Suggested r_d by Seed and Idriss (1971)

2. Idriss (1999)

The original r_d by Seed and Idriss (1971) were considered magnitude independent. Further studies by Golesorkhi (1989) demonstrated the dependency of r_d on earthquake magnitude. Idriss (1999) built upon Golesorkhi (1989) and Idriss (1997) by conducting hundreds of equivalent linear site

specific response analyses. The r_d relation of Idriss (1999) as a function of moment magnitude is presented as follows:

$$r_d = \exp(\alpha(z) + \beta(z)M) \quad \text{Equation 4}$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \quad \text{Equation 5}$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \quad \text{Equation 6}$$

where z is the depth of the soil profile in meters, M is the moment magnitude of the earthquake and the values inside the sine functions are in radians. Idriss (1999) suggested using the relationship for depths up to 20 m, emphasizing on the high uncertainty at deep profiles.

The r_d curves by Idriss (1999) are shown in Figure 8 for different moment magnitudes along with the mean value of the original r_d by Seed and Idriss (1971). According to the figure the average value of the original r_d is compatible with the curve calculated by Idriss (1999) for a moment magnitude of 7.5.

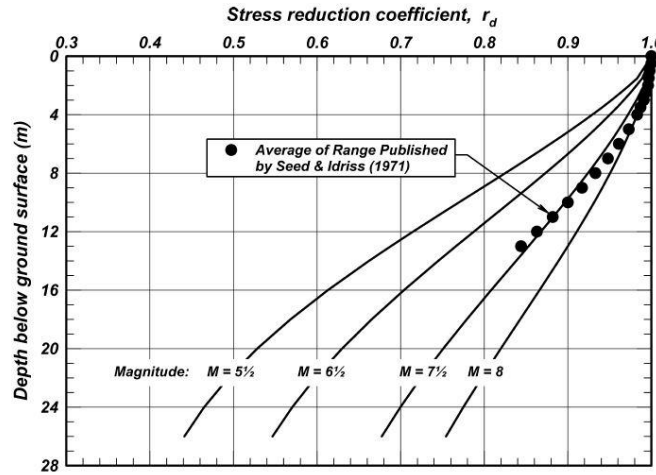


Figure 8 r_d vs. depth and moment magnitude from Idriss (1999)

3. Cetin et al. (2004)

Cetin et al. (2004) showed that the shear-stress reduction factor (r_d) also depends on the average shear wave velocity within the upper twelve meters of the profile (V_{s12}), which represents soil stiffness, as well as the peak ground acceleration (PGA). The suggested r_d relationship by Cetin et al. (2004) is outlined as follows:

$$r_d = \frac{1 + \frac{A}{B}}{1 + \frac{A}{C}} \quad \text{Equation 7}$$

$$A = -23.013 + 2.949PGA + 0.999M + 0.0525V_{s12} \quad \text{Equation 8}$$

$$B = 16.258 + 0.201e^D \quad \text{Equation 9}$$

$$C = 16.258 + 0.201e^E \quad \text{Equation 10}$$

$$D = 0.341(-z + 0.0785V_{s12} + 7.586) \quad \text{Equation 11}$$

$$E = 0.341(0.0785V_{s12} + 7.586) \quad \text{Equation 12}$$

These relations were obtained by regression of the results of over two thousand equivalent linear (EL) site-specific response analyses on various soil profiles across California. These profiles were subjected to a range of input ground motions carefully selected to span a broad spectrum of moment magnitudes (i.e., $5.7 < M < 8.1$). Due to the multiplicity of variables within this model, the variations of r_d with depth was visually represented across twelve figures. These figures correspond to various ranges of M , PGA , and V_{s12} which can be found in the original paper.

4. Perret et al. (2013)

Perret et al. (2013) establish a shear stress coefficient relationship that would align with the typical profiles and ground motions observed in Eastern Canada. To carry out this study, fourteen soil profiles were selected from the St. Lawrence River vicinity, and an additional four conceptualized soil profiles were adopted from the Idriss and Boulanger (2010) study. Since available motions in Eastern Canada are limited, they study only included recordings from the Saguenay earthquake from Quebec, and the Nahanni earthquake from the Northwest due to its geological and seismological resemblance to the conditions found in Eastern Canada. Additionally synthetic earthquake time histories were generated to simulate the natural ground motion patterns specific to Eastern Canada. In total 5,256 one-dimensional equivalent linear dynamic soil response (DSRA) analyses were conducted utilizing STRATA (Kotke and Rathje, 2009). The resulting r_d values obtained through DSRA are visually presented in Figure 9 below, showcasing their relationship with depth and offering a comparative perspective against values derived from current common models:

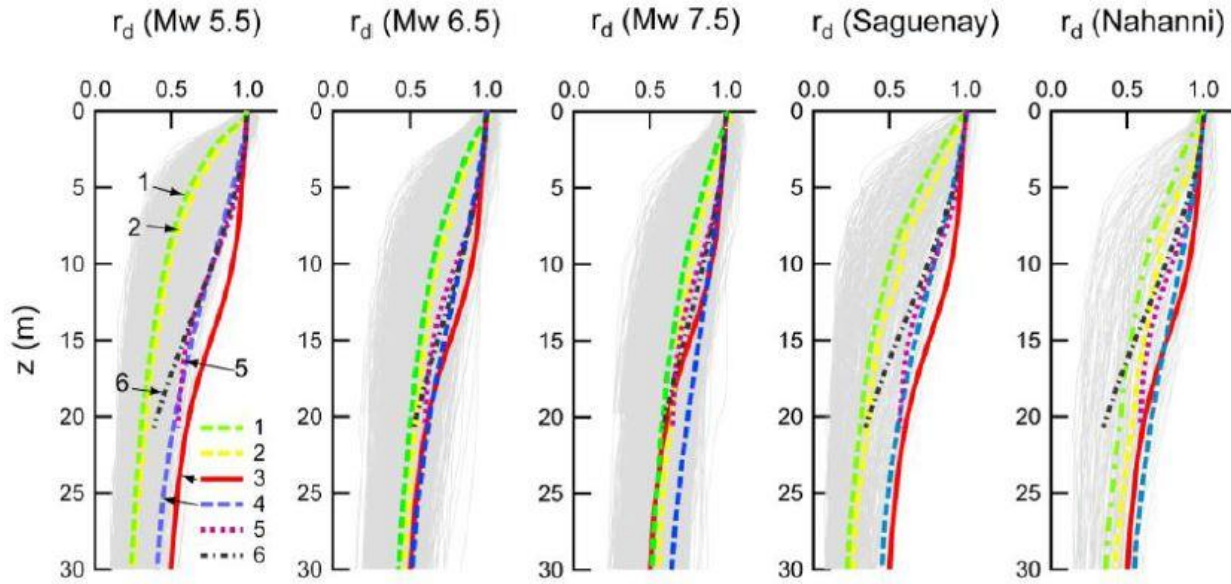


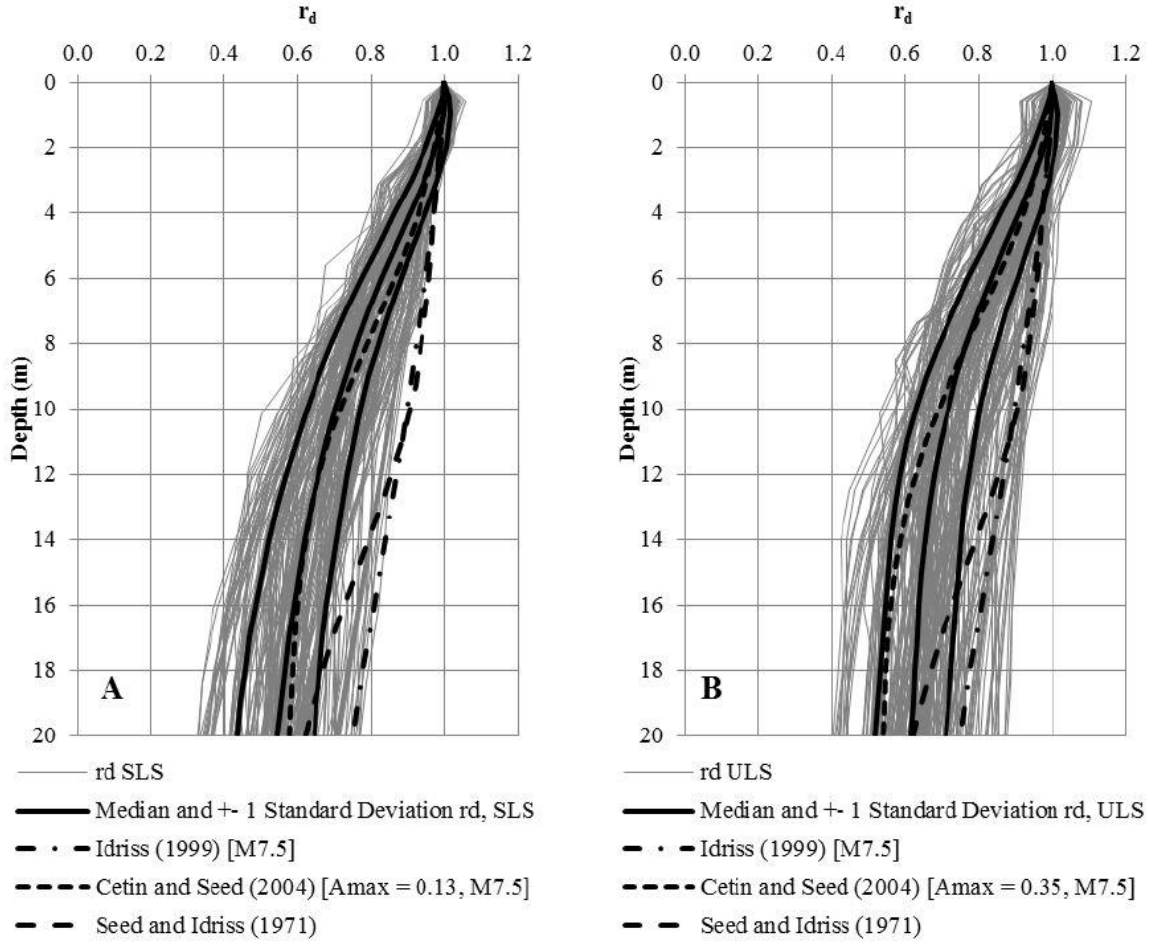
Figure 9 Comparison of r_d values determined from DSRA (light grey curves) and using existing models 1: mean; 2: 67th percentile; 3: Youd et al. 2001; 4: Idriss 1999; 5: Cetin et al. 2004; 6: Kishida et al. 2009 (Perret et al., 2013)

To establish the relationship, a nonlinear multivariate regression analysis was executed on the r_d values. This analysis retained the same functional forms and variables used in two existing models (i.e., Idriss 1999 and Cetin et al. 2004) while allowing for the adjustment of fitting parameters. The updated fitting parameters for the equations proposed by Idriss (1999) and Cetin et al. (2004) are provided in the original paper.

5. Dismuke (2014)

Dismuke (2014) to derive site-specific r_d values compatible with the geotechnical conditions and seismic hazards specific to Christchurch, New Zealand. For this study, two typical geologic models were developed to represent geologic conditions in Christchurch. Shear wave velocity profiles for these models were acquired through the multichannel analysis of surface waves (MASW) surveys conducted across Christchurch. Regarding ground motions, the design ground motions were categorized into two groups (i.e., serviceability limit state (SLS) and ultimate limit state (ULS) design ground motions). Twelve ground motions were selected from the PEER (2013) dataset. 528 one-dimensional nonlinear site response analyses were executed employing DEEPSOIL (Hashash et al., 2012) to derive the shear reduction factor. Additionally, a series of equivalent linear site response analyses were conducted using STRATA. The calculated r_d values obtained from the site

response analyses are displayed in Figure 10 below, providing a comparison with the r_d values acquired using existing relationships:



The r_d relationships with respect to depth for both serviceability limit state (SLS) and ultimate limit state (ULS) ground motion scenarios, as proposed by Dismuke (2014), are outlined below:

Figure 10 r_d computed for Christchurch for (A) SLS input ground motions and (B) ULS input ground motions.

$$r_{d,SLS} = -8.017E - 8z^6 + 5.729E - 6z^5 - 1.634E - 4z^4 + 2.345E - 3z^3 - 1.637E - 2z^2 + 1.375E - 2z + 1 \quad \text{Equation 13}$$

$$r_{d,ULS} = -4.475E - 08z^6 + 3.420E - 06z^5 - 1.082E - 04z^4 + 1.755E - 03z^3 - 1.344E - 02z^2 + 9.049E - 03z + 1 \quad \text{Equation 14}$$

2.1.3.2 Assessing cyclic strength of soil

According to Idriss and Boulanger (2008) cyclic resistance relationships are commonly formulated based on in-situ test results including CPT and SPT. Multiple earthquake case histories that encountered either liquefaction or non-liquefaction scenarios have been compiled. For each case, its corresponding CSR has been graphed against field measurements like $(N_1)_{60}$ or q_c . As depicted in Figure 11 below, a curve segregates data points associated with historical liquefaction and those unaffected by it. This boundary curve serves as the foundational CRR reference.

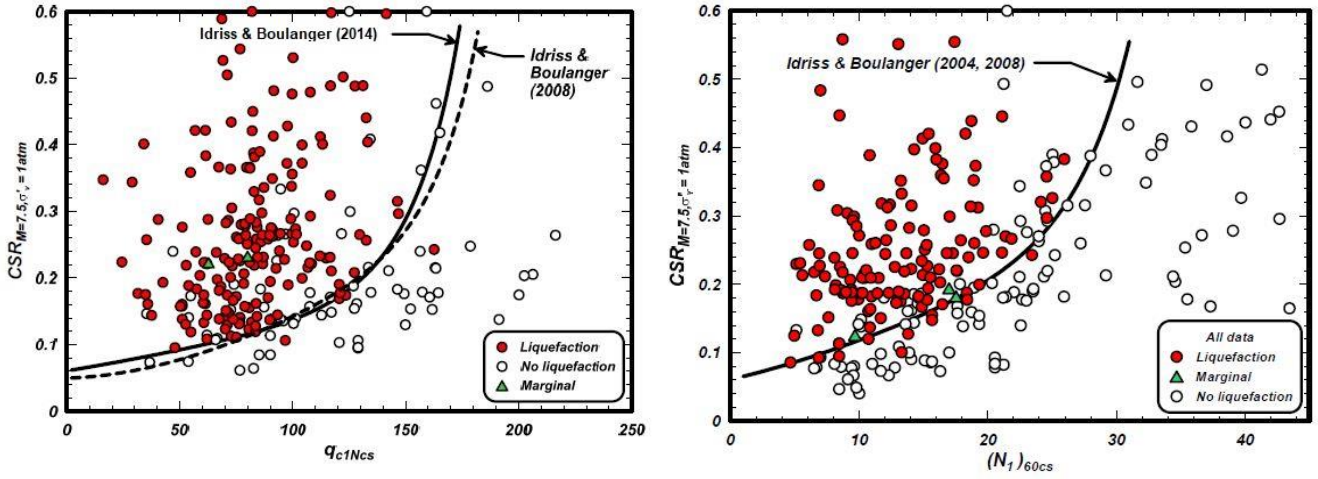


Figure 11 CPT and SPT case history databases of liquefaction (Idriss and Boulanger 2014)

The reference curve of CRR from Idriss and Boulanger (2014) is defined under reference conditions, confining pressure equal to the atmospheric pressure, zero static shear stress, and an earthquake magnitude of 7.5. Therefore, the reference CRR needs to be adjusted to account for the specific conditions of each particular scenario, as outlined below.

$$CRR_{\sigma', \alpha, M} = CRR_{\sigma'=P_a, \alpha=0, M=7.5} \times K_{\sigma} \times K_{\alpha} \times MSF \quad \text{Equation 15}$$

where K_{σ} is the overburden correction factor, K_{α} is the static shear correction factor, and MSF is the magnitude scaling factor, as further detailed in the subsequent sections.

2.1.3.2.1 Overburden correction factor, K_{σ}

Seed (1983) suggested using the overburden correction factor to normalize the CRR by considering the effective overburden stress. K_σ defines as follows:

$$K_\sigma = \frac{CRR_{\sigma'}}{CRR_{\sigma'=P_a}} \quad \text{Equation 16}$$

where $CRR_{\sigma'}$ is the cyclic resistance of soil under a certain effective stress of σ' , and $CRR_{\sigma'=P_a}$ is the cyclic resistance of the same soil under an effective stress equivalent to atmospheric pressure. Boulanger (2003) proposed an equation for K_σ as follows:

$$K_\sigma = 1 - C_\sigma \ln \left(\frac{\sigma'_v}{P_a} \right) \leq 1.1 \quad \text{Equation 17}$$

where based on Boulanger and Idriss (2004), C_σ can be determined as a function of the relative density of sand as follows:

$$C_\sigma = \frac{1}{18.9 - 17.3D_R} \leq 0.3 \quad \text{Equation 18}$$

It is worth mentioning that Boulanger and Idriss (2004) introduces additional relations for the calculation of C_σ based on the overburden corrected penetration resistances (i.e., $(N_1)_{60}$ and q_{c1N}).

2.1.3.2.2 Static shear stress correction factor, K_α

The presence of static shear stresses, which can occur in slopes or beneath foundations, can affect the soil's CRR. To address this influence, Seed (1983) proposed the use of K_α factor, in order to normalize the CRR by considering the effects of static shear stresses. K_α is defined as follows:

$$K_\alpha = \frac{CRR_\alpha}{CRR_{\alpha=0}} \quad \text{Equation 19}$$

where $\alpha = \frac{\tau_{static}}{\sigma'_{v0}}$, CRR_α is the cyclic resistance ratio of a soil under a particular value of α , and $CRR_{\alpha=0}$ is cyclic resistance ratio of the same soil with $\alpha = 0$. It is important to note that the impact of static shear stresses is negligible in the case of dealing with level or nearly level ground site conditions. However, relevant relationships can be found for inclined scenarios in Idriss and Boulanger (2003).

2.1.3.2.3 Magnitude scaling factor, MSF

According to Seed et al. (1975), the number of loading cycles, which is correlated to the earthquake magnitude, can impact CRR. To account for this effect, a magnitude scaling factor has been recommended to normalize the CRR with magnitude. The fundamental definition of MSF is as follows:

$$MSF = \frac{CRR_M}{CRR_{M=7.5}} \quad \text{Equation 20}$$

where CRR_M is the cyclic resistance ratio of a soil under an earthquake with specific magnitude of M , and $CRR_{M=7.5}$ is the cyclic resistance ratio of the same soil under an earthquake with magnitude of 7.5. Idriss (1999) established a relationship for MSF, which is presented as follows:

$$MSF = 6.9 \exp\left(\frac{-M}{4}\right) - 0.058 \leq 1.8 \quad \text{Equation 21}$$

2.1.3.3 Comparing seismic demand and cyclic strength of soil

Ultimately, the assessment of liquefaction triggering hinges on the comparison between the seismic demand, characterized by the Cyclic Stress Ratio (CSR), and the soil's resistance to liquefaction, represented by the Cyclic Resistance Ratio (CRR). Two approaches exist for this purpose.

1. Deterministic method:

In a deterministic approach a factor of safety will be computed as follows:

$$FS = \frac{CRR_{\sigma', \alpha, M}}{CSR} \quad \text{Equation 22}$$

when $FS > 1.4$, the occurrence of liquefaction is not expected, while for $1.1 < FS < 1.4$, there is a low possibility of liquefaction for stiff soils and a high possibility of liquefaction for soft soils, and in the case of $FS < 1.1$ the occurrence of liquefaction is expected.

2. Probabilistic method:

Numerous studies have been carried out to develop probabilistic correlations for the triggering of liquefaction in cohesionless soils based on SPT and CPT data. However, a common limitation in many of these studies is the simultaneous consideration of total uncertainty in assessing the case history recordings. In other words, they tend to combine the uncertainty in the triggering model (i.e., model uncertainty) and the uncertainty in the in-situ measurements (i.e., measurement uncertainty). Idriss and Boulanger (2010, 2014) employed a maximum likelihood method to propose SPT and CPT-based relationships that allow for separate consideration of model and measurement uncertainties. The conditional probability of liquefaction using SPT and CPT data can be computed as follows

$$P_L((N_1)_{60cs}, CSR_{M=7.5, \sigma'_v=1 atm}) \quad \text{Equation 23}$$

$$= \Phi \left[- \frac{\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.67 - \ln(CSR_{M=7.5, \sigma'_v=1 atm})}{\sigma_{\ln(R)}} \right]$$

$$P_L(q_{c1Ncs}, CSR_{M=7.5, \sigma'_v=1 atm}) \quad \text{Equation 24}$$

$$= \Phi \left[- \frac{\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000}\right)^2 - \left(\frac{q_{c1Ncs}}{140}\right)^3 + \left(\frac{q_{c1Ncs}}{137}\right)^4 - 2.60 - \ln(CSR_{M=7.5, \sigma'_v=1 atm})}{\sigma_{\ln(R)}} \right]$$

where Φ is the standard normal cumulative probability function and $\sigma_{\ln(R)}$ is the standard deviation on the CRR, and is taken as 0.2.

The probability triggering curves equal to 15, 50, and 85% solely with model uncertainty are illustrated in Figure 12 and 13, respectively based on SPT and CPT measurements.

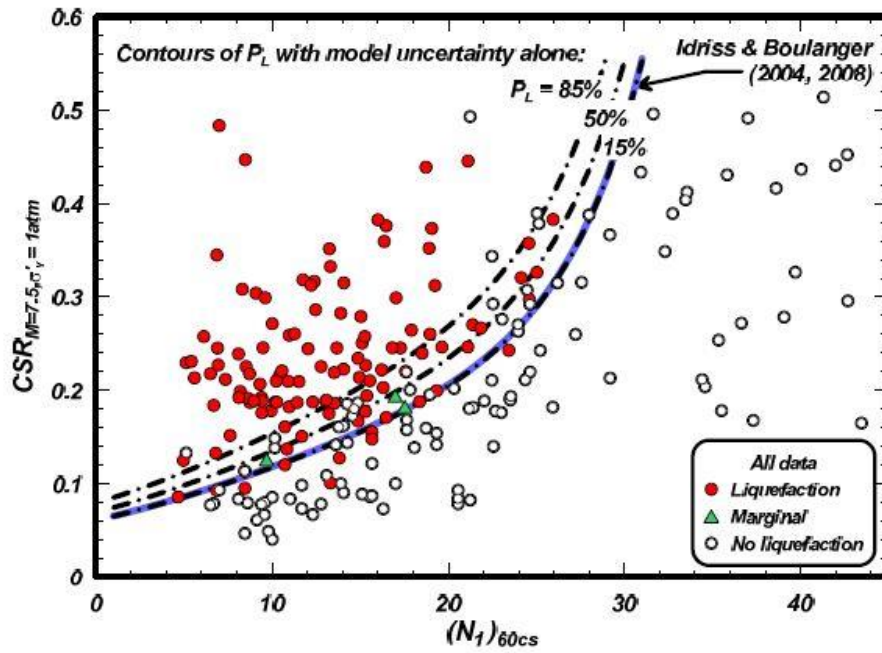


Figure 12 Curves of $CRR_{M=7.5, \sigma'_v=1 atm}$ with $(N_1)_{60cs}$ for probabilities of liquefaction of 15%, 50%, and 85% (Idriss and Boulanger 2010)

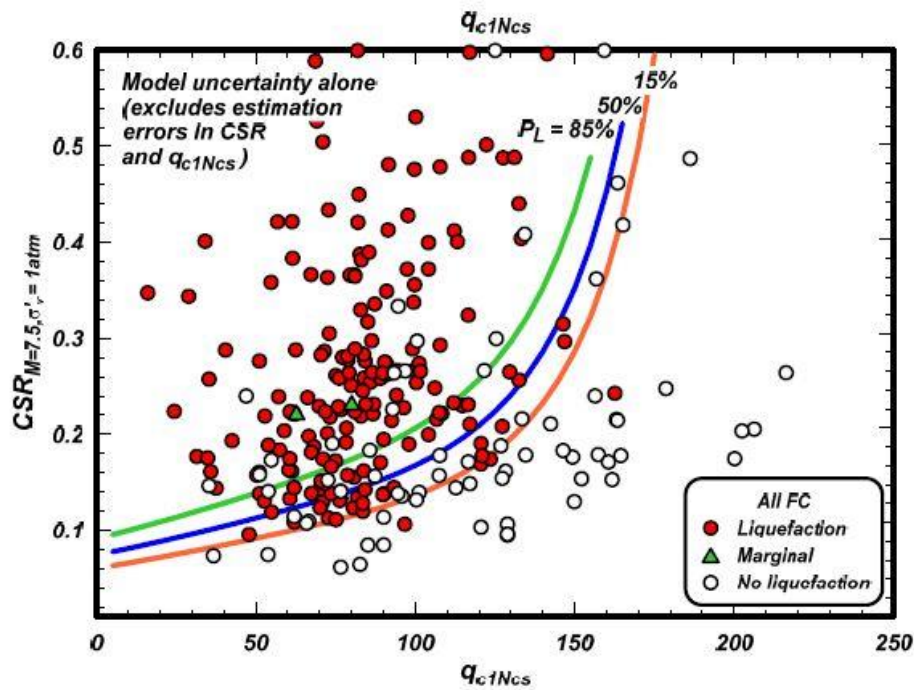


Figure 13 Curves of $CRR_{M=7.5, \sigma'_v=1 atm}$ with q_{c1Ncs} for probabilities of liquefaction of 15%, 50%, and 85% (Idriss and Boulanger 2014)

2.2 Site-specific ground response analysis

As the seismic waves propagate from the bedrock through a soil profile toward the ground surface they may experience amplification or attenuation due to the impedance contrast between the bedrock and the soil deposit. Ground response analysis investigates the propagation of seismic waves through a soil profile through the simulation of the soil's seismic response by utilizing the dynamic characteristics of the soil and the earthquake.

During an earthquake, shear waves emitted from the fault rupture, spread in all directions. Nevertheless, owing to the dissimilar shear wave velocities in shallow, softer layers compared to the stiffer layers beneath, horizontal waves encounter reflection and refraction as they propagate. As a result, shear waves frequently undergo significant bending, approaching a nearly vertical trajectory by the time they reach the ground surface. This behavior is exemplified in Figure 14 (Kramer 1996). According to this phenomenon, ground response analyses can confidently be conducted in a one-dimensional framework in practical scenarios. While the possibility of performing two and three-dimensional site response analyses exists, these techniques are infrequently used in practice.

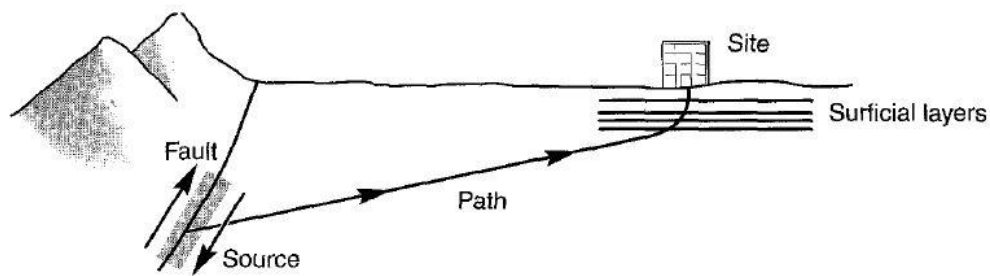


Figure 14 Nearly vertical wave propagation in close proximity to the ground surface (Kramer 1996)

One-dimensional (1D) ground response analysis, takes into account site-specific parameters to evaluate the propagation of seismic waves through a soil column that comprises various layers, each characterized by distinct mechanical properties. This configuration is depicted in Figure 15 below.

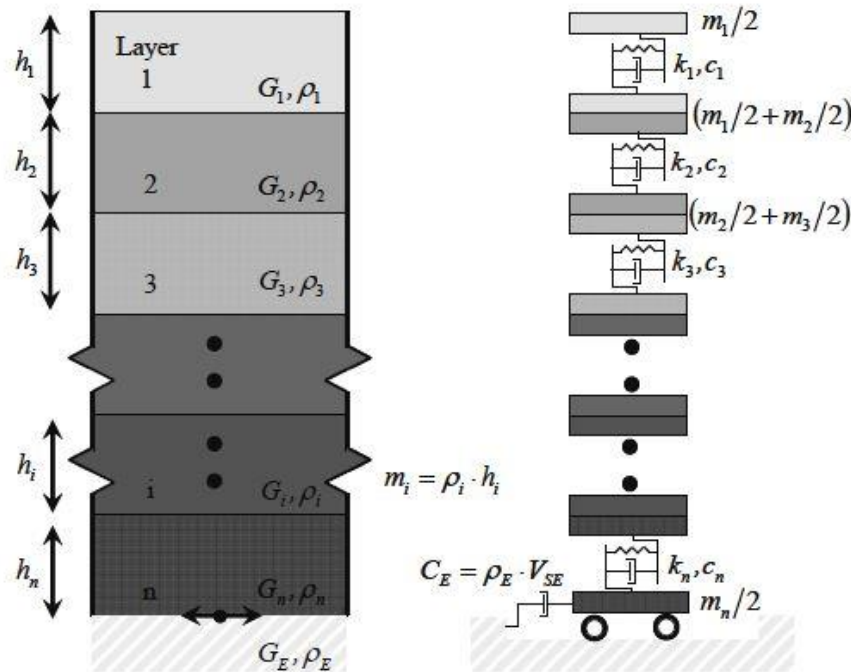


Figure 15 One-dimensional (1D) model of a soil profile being subjected to horizontal shaking at its base by a vertically propagating shear wave (Hashash et al. 2010)

2.2.1 Dynamic soil properties

The essential dynamic characteristics for conducting site response analyses are:

1. Soil shear modulus (G)
2. Soil density (ρ)
3. Soil damping ratio (D)

2.2.1.1 Shear modulus and shear wave velocity

The shear modulus, denoted as G , embodies the soil's shear stiffness. The secant shear modulus (G) is the slope of the line originating from the origin towards any point along the backbone curve as illustrated in Figure 16. The maximum shear modulus (G_{max}) is the initial slope of the backbone curve at $\gamma = 0$ (i.e., shear modulus at small shear strains where the soil behavior is elastic). This can be calculated using the subsequent formula:

$$G_{max} = \rho \cdot V_s^2 \quad \text{Equation 25}$$

where ρ is the soil density and V_s is the shear wave velocity.

Several methods exist for acquiring the shear wave velocity profile of a specific soil deposit, including:

1. Conducting a seismic cone penetration test (SCPT) directly at the site.
2. Employing surface wave techniques like spectral analysis of surface waves test (SASW) or multichannel analysis of surface waves test (MASW) on the field.
3. Carrying out shear wave velocity measurements within a laboratory setting.

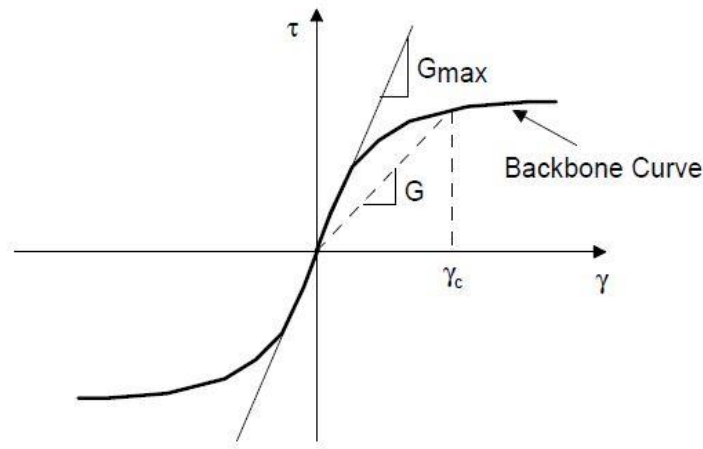


Figure 16 A sample backbone curve (Stewart et al. 2008)

2.2.1.2 Modulus reduction and damping (MRD) curves

Soils exhibit nonlinear behavior during cyclic loading, resulting in a decrease in stiffness and an increase of soil damping as the strain amplitude increases. The alteration of shear modulus with respect to shear strain is commonly defined using a normalized modulus reduction curve. The (G/G_{max}) ratio can be acquired directly from the backbone curve or through the execution of cyclic tests at varying (γ) . The hysteretic damping ratio (D) quantifies the ratio of energy dissipated to the maximum stored strain energy within each loading cycle, as a function of shear strain. Illustrated in Figure 17 below, the dissipated energy across a loading cycle is visually represented by the enclosed red region within the hysteresis loop (A), while the maximum retained strain energy is depicted by the blue triangular region (B).

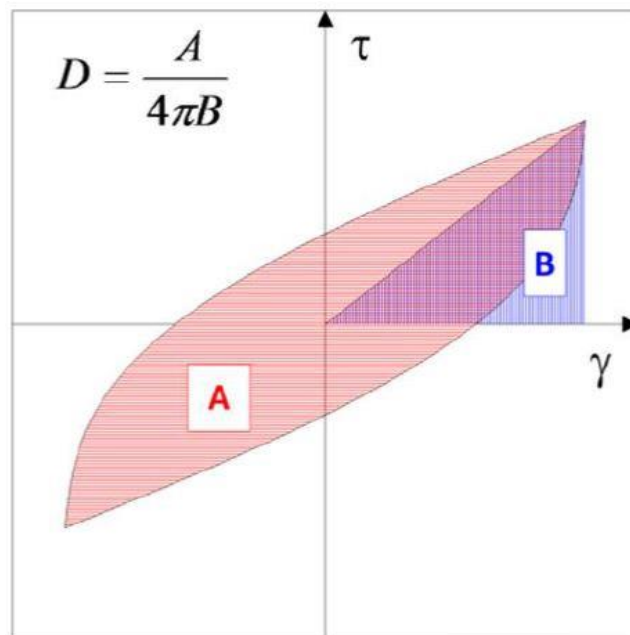


Figure 17 Damping ratio over one loading cycle (Yniesta et al. 2017)

An example of modulus reduction (G/G_{max}) and damping (D) curves is provided in Figure 18.

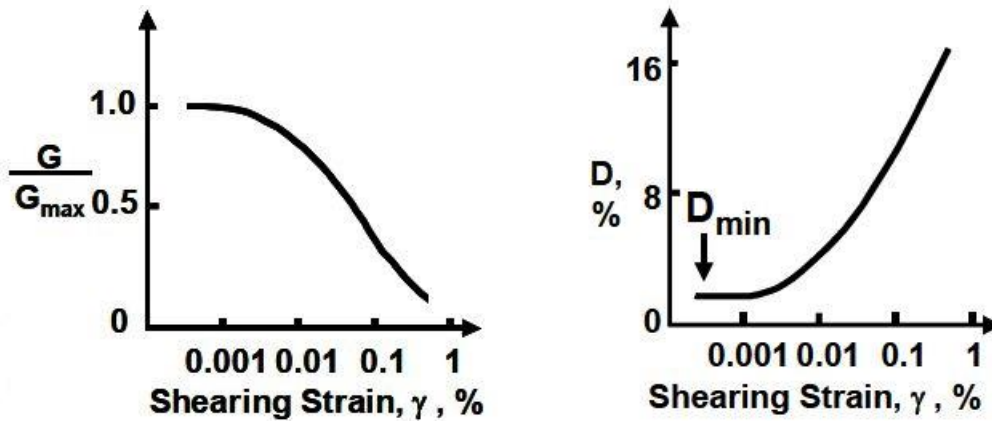


Figure 18 Modulus reduction (G/G_{max}) and damping (D) curves (Darendeli 2001)

It is also noteworthy to mention that, as shown in Figure 18 above, the damping ratio at small strains, where soil behavior remains nearly linear, is termed the small-strain damping ratio (D_{min}). The MRD curves can be derived either through laboratory tests or empirical relationships. However, due to the high costs involved in conducting laboratory experiments and the limited accessibility of testing equipment in many cases, the more common approach is to rely on empirical relationships.

2.2.1.2.1 Darendeli's (2001) empirical relationships

One prominent empirical model, widely employed in practical scenarios, was formulated by Darendeli (2001) and is applicable to silts, clays, and sands. The equations representing Darendeli's (2001) modulus reduction curve and hysteretic damping curve in relation to shear strain are presented as follows:

Modulus reduction curve:

$$\frac{G(\gamma)}{G_{max}} = \frac{1}{1 + \left(\frac{\gamma}{\gamma_r}\right)^\alpha} \quad \text{Equation 26}$$

where $\alpha = 0.92$ and γ_r is the pseudo reference strain

$$\gamma_r(\%) = (\varphi_1 + \varphi_2 \cdot PI \cdot OCR^{\varphi_3}) \cdot \left(\frac{\sigma'_0}{P_a} \right)^{\varphi_4} \quad \text{Equation 27}$$

where PI is plasticity index, OCR is over consolidation ratio, σ'_0 is mean effective stress, P_a is atmospheric pressure and φ are regression coefficients. Darendeli (2001) has presented varying φ values for different soil types in his research.

Hysteretic damping ratio curve:

$$D(\gamma) = D_{min} + b \cdot D_M(\gamma) \cdot \left(\frac{G(\gamma)}{G_{max}} \right)^{0.1} \quad \text{Equation 28}$$

where

$$D_{min} = (\varphi_6 + \varphi_7 \cdot PI \cdot OCR^{\varphi_8}) \cdot \left(\frac{\sigma'_0}{P_a} \right)^{\varphi_9} \cdot (1 + \varphi_{10} \cdot \ln(freq)) \quad \text{Equation 29}$$

$$b = \varphi_{11} + \varphi_{12} \cdot \ln(N) \quad [N = \text{number of cycle}, 10] \quad \text{Equation 30}$$

$$freq = 1.0 \text{ Hz}$$

$D_M(\gamma)$ is the hysteretic damping based on Masing rules (1926) and calculated as follows:

$$D_M(\gamma) = c_1 \cdot D_{M,\alpha=1} + c_2 \cdot D_{M,\alpha=1}^2 + c_3 \cdot D_{M,\alpha=1}^3 \quad \text{Equation 31}$$

$$D_{M,\alpha=1}(\gamma)[\%] = \frac{100}{\pi} \left[4 \cdot \frac{\gamma - \gamma_r \cdot \ln\left(\frac{\gamma - \gamma_r}{\gamma_r}\right)}{\frac{\gamma^2}{\gamma + \gamma_r}} - 2 \right] \quad \text{Equation 32}$$

$$c_1 = 0.2523 + 1.8618 \cdot \alpha - 1.1143 \cdot \alpha^2 \quad \text{Equation 33}$$

$$c_2 = -0.0095 - 0.0710 \cdot \alpha + 0.0805 \cdot \alpha^2 \quad \text{Equation 34}$$

$$c_3 = 0.0003 + 0.0002 \cdot \alpha - 0.0005 \cdot \alpha^2 \quad \text{Equation 35}$$

2.2.2 Site response analysis approaches

Three major categories of stress-strain models are employed for conducting site response analyses, as outlined below:

1. Linear model
2. Equivalent linear (EL) model
3. Nonlinear (NL) model

2.2.2.1 Linear analysis

The linear analysis method is the most basic approach for site-specific analysis due to its assumption that the dynamic properties of the soil profile remain constant during cyclic loading. This method solves the wave equation for a 1-D elastic system to assess the transmission of shear waves through a soil column. Despite its quick and straightforward application, this method is not widely accepted in practice due to its unrealistic assumption of elastic behavior for the nonlinear cyclic response of the soil.

2.2.2.2 Equivalent linear (EL) analysis

The fundamental drawback of the linear method's hypothesis is its assumption of a constant shear modulus (G) and damping ratio (D) throughout the duration of an earthquake. This assumption proves problematic due to the nonlinear nature of soil behavior, where both G and D undergo changes in response to time and strain amplitude. Addressing this issue, the equivalent linear (EL) method (Seed and Idriss 1969) provides a superior solution by acknowledging the nonlinear soil response during cyclic loading. Within this framework, G and D values are chosen from MRD curves, corresponding to specific degrees of nonlinearity. This selection accounts for the fluctuations in shear modulus and damping ratio during the course of seismic shaking. The efficiency of the EL analysis stems from its ability to be rapidly solved within the frequency domain.

The EL method employs an iterative procedure to determine suitable values for shear modulus and damping ratio that align with the average dynamic soil response during an earthquake. Each iteration encompasses four fundamental steps, outlined as follows:

1. Assuming initial values for shear modulus (G) and damping ratio (D) for each soil layer.
2. Conducting a linear site response analysis employing the assumed G and D values. This analysis yields the maximum shear strain (γ_{max}) for each soil layer based on the relationship between shear strain and time (γ_t).
3. Computing the effective shear strain (γ_{eff}) using the γ_{max} obtained in the previous step, using the following equation.

$$\gamma_{eff} = \frac{M - 1}{10} \cdot \gamma_{max} \quad \text{Equation 36}$$

where M is the earthquake magnitude.

4. Updating the shear modulus (G) and damping ratio (D) values to align with the calculated, γ_{eff} ensuring consistency between the analysis and the observed response.

This iterative procedure must be reiterated until the adjusted estimations for shear modulus (G) and damping ratio (D) in a new iteration converge within a given tolerance with those from a prior iteration, or the set maximum number of iterations is reached. Ultimately, the linear analysis outcomes obtained in the final iteration serve as the results derived from the equivalent linear analysis.

Although the EL analysis is still widely used in practice due to its simplicity compared to NL analysis, it has its limitations. In a recent study by Kaklamanos et al. (2015), the accuracy of employing the nonlinear, equivalent linear, and linear methods for site response analysis was assessed. As illustrated in Figure 19 below, a residual analysis of the PSA (at $T=0.1s$) results derived by these three methods reveals that the linear method consistently yields highly biased and unrealistic results. Furthermore, the EL method's results are less accurate and exhibit significant bias in the range of large strains (i.e., $>0.05\%$) in comparison to the outcomes from the NL method. Therefore, for increased accuracy in computations, it is recommended to perform NL analysis instead of EL analysis when dealing with large strains or highly nonlinear soils.

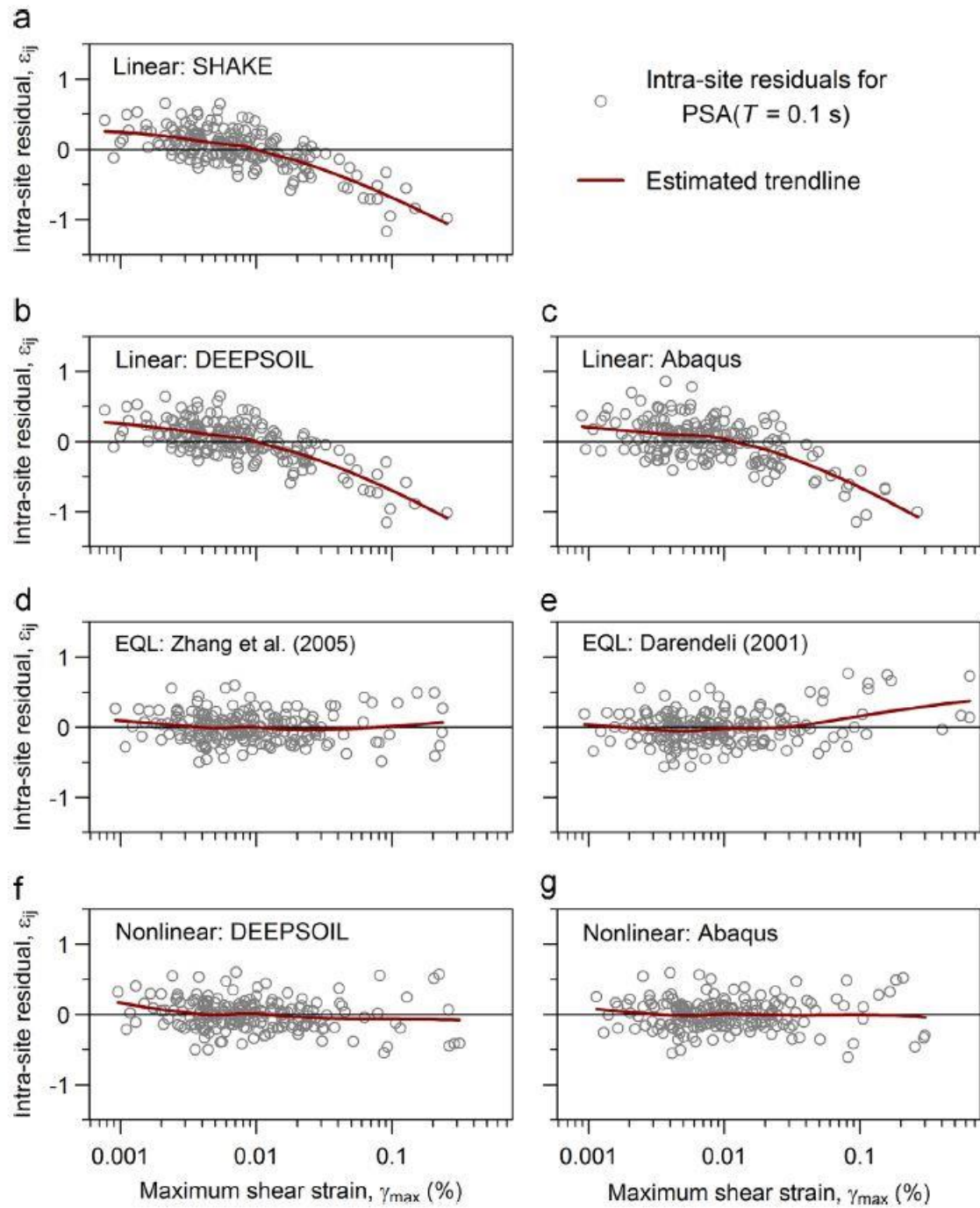


Figure 19 Intra-site residuals (ε_{ij}) for PSA at $T=0.1$ s, versus the maximum shear strain (γ_{max}) using different site response models (Kaklamanos et al. 2015)

2.2.2.3 Nonlinear (NL) analysis

The nonlinear (NL) method stands out as the most accurate approach in site response analysis due to its comprehensive consideration of the nonlinear soil behavior throughout the entire cyclic loading duration. The nonlinear method is performed in the time domain, which means solving the equation of motion at each time step as follows:

$$[M] \cdot \{\ddot{u}\} + [C] \cdot \{\dot{u}\} + [K] \cdot \{u\} = -[M] \cdot \{I\} \cdot \ddot{u}_g \quad \text{Equation 37}$$

where M , C , and K are the mass, damping, and stiffness matrices respectively, $\ddot{u}$, $\dot{u}$, and u are the soil acceleration, velocity, and displacement vectors respectively, $\ddot{u}_g$ is the base acceleration, and I is the identity vector.

Given that nonlinear (NL) analysis is conducted in the time domain and the stiffness matrix of soil changes over time due to cyclic loading, it requires a constitutive model to update the stiffness matrix at each time step. In 1-D ground response analysis, a typical constitutive model generally comprises two sets of equations that delineate the soil's shear stress response in relation to its shear strain, as outlined below:

1. A backbone curve that represents the initial loading conditions, and can be computed based on a Modulus Reduction (MR) curve,
2. A set of unloading/reloading rules, which control the shape of unloading and reloading loops, and illustrates the hysteresis behavior of soils.

Typically, constitutive models for 1D simulations necessitate inputs such as soil density (ρ), maximum shear modulus (G_{max}), modulus reduction curve ($G(\gamma)/G_{max}$), and damping ratio curve (D). Generally, these initial MRD curves are derived from empirical relationships like those proposed by Darendeli (2001). Subsequently, a constitutive model is employed to fit these curves, enabling their utilization within the nonlinear analysis.

2.2.2.3.1 Masing rules

The initial constitutive model, comprising a collection of unloading/reloading rules to characterize the cyclic behavior of soil, was provided by Masing (1926). The expanded Masing rules by Vucetic (1990) are presented below and illustrated in Figure 20.

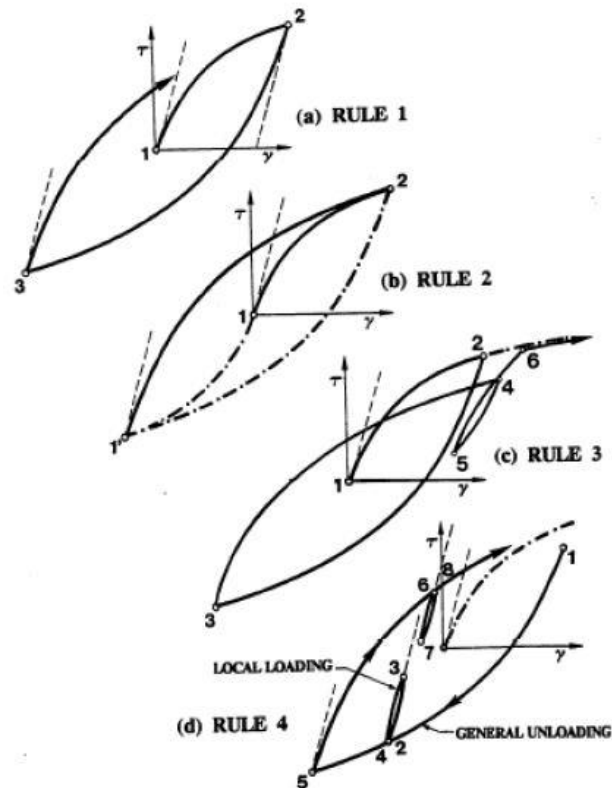


Figure 20 Extended Masing rules Vucetic (1990)

1. During initial loading, the stress-strain curve follows the backbone curve;
2. In the event of strain reversal, the unloading or reloading curve mirrors the backbone curve but scaled up by a factor of 2;
3. If the unloading or reloading curve exceeds the maximum past strain and intersects the backbone curve, it follows the backbone curve until the next stress reversal;
4. If the unloading or reloading curve crosses an unloading or reloading curve from a previous cycle, it follows the curve of that previous cycle.

While the Masing rules adequately capture the cyclic behavior of soils within medium strain levels, they exhibit limitations when applied to small and large strains. The Masing rules show zero damping at small strains due to the linear backbone curve, leading to unrealistic behavior. Consequently, the incorporation of the concept of viscous damping, described in section 2.2.2.3.2,

becomes essential to address this issue at small strain levels. Conversely, the Masing rules for large strains tend to overestimate damping significantly, as demonstrated in Figure 21.

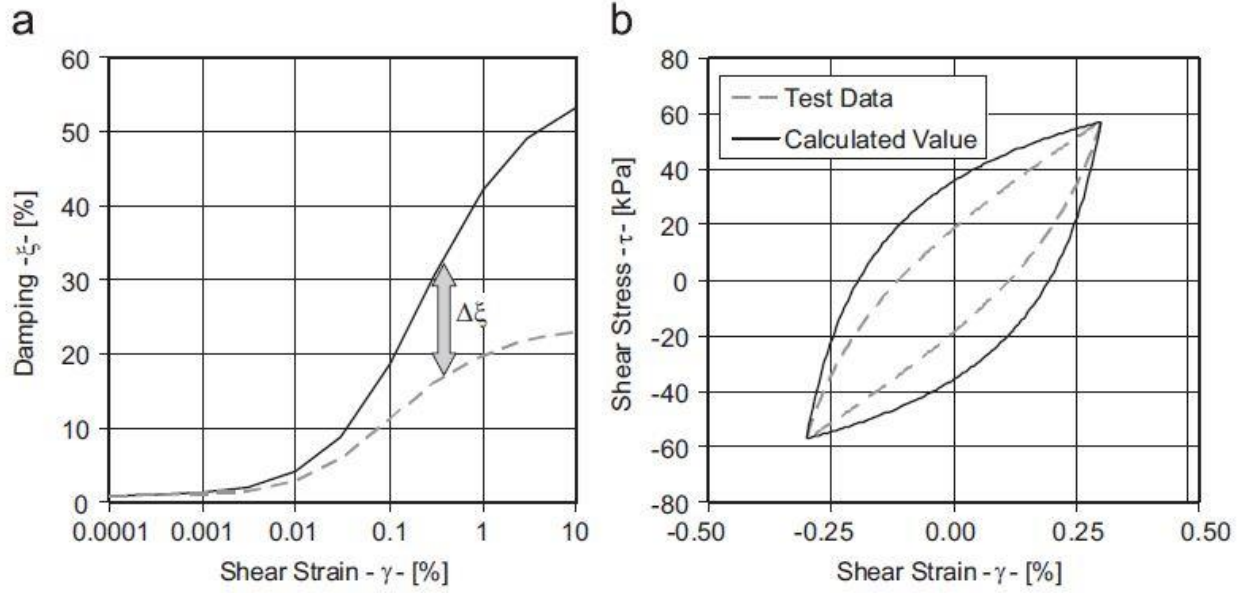


Figure 21 Overestimation of hysteretic damping using Masing rules (a) Damping curve (b) Hysteretic loop (Phillips and Hashash 2009)

2.2.2.3.2 Viscous damping

Since the soil cyclic behavior exhibits linearity at small strains, nonlinear constitutive models often fall short of accurately representing the hysteric damping within this range. To address this limitation, it becomes crucial to adopt a formulation for viscous damping to simulate damping at small strain levels. In general, the damping introduced by Rayleigh and Lindsay (1945) has been commonly employed to characterize damping within small strains. This approach proposes a damping matrix $[C]$ by combining elements from the mass and stiffness matrices.

$$[C] = a_0[M] + a_1[K] \quad \text{Equation 38}$$

where $[M]$ is mass matrix, $[K]$ is stiffness matrix, and a_0 and a_1 are scalar values. For two frequency modes, a_0 and a_1 can be calculated as follows:

$$\begin{bmatrix} \xi_i \\ \xi_j \end{bmatrix} = \frac{1}{4\pi} \begin{bmatrix} \frac{1}{f_i} & f_i \\ \frac{1}{f_j} & f_j \end{bmatrix} \begin{Bmatrix} a_0 \\ a_1 \end{Bmatrix} \quad \text{Equation 39}$$

where ξ_i and ξ_j are damping ratios corresponding to f_i and f_j frequencies, respectively. The precision of employing various frequency modes is illustrated in Figure 22. This comparison shows that using simplified damping (i.e., one frequency mode) should be avoided in practical applications. The approach of utilizing four frequency modes, as recommended by Park and Hashash (2008), yields greater accuracy at moderate frequencies compared to using two frequency modes. Conversely, two frequency modes exhibit superior performance at high frequencies compared to the four-frequency mode approach.

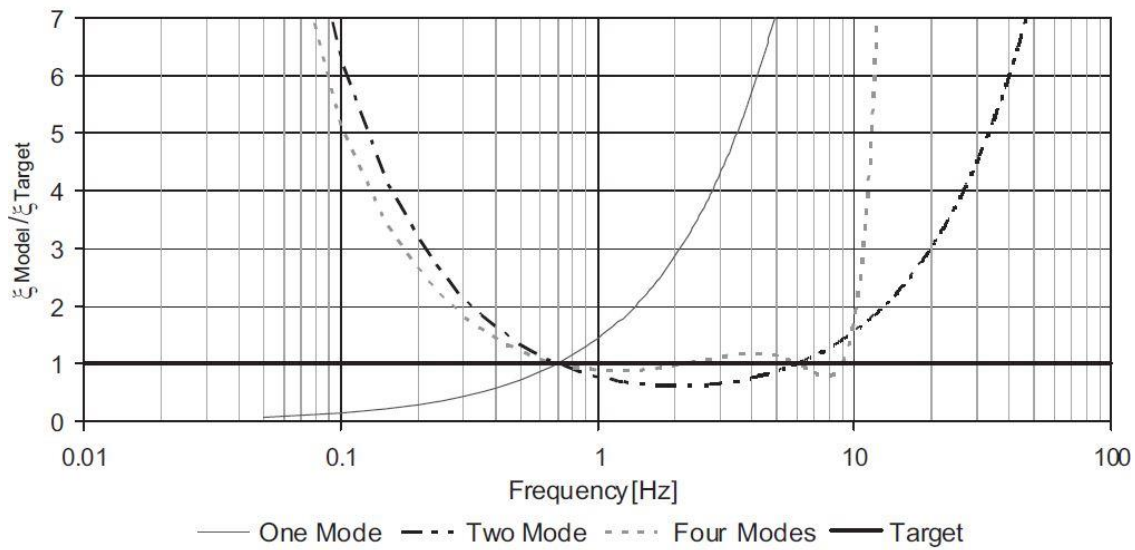


Figure 22 Effective damping for one, two and four modes Rayleigh formulation (Phillips and Hashash, 2009)

The original method by Rayleigh and Lindsay (1945) is frequency dependent. However, in a later study by Phillips and Hashash (2009), which was built upon the research of Liu and Gorman (1995), a frequency-independent Rayleigh damping formulation was proposed. This formulation is designed to match a target damping ratio across the entire frequency spectrum, and it aligns more consistently with experimental results at small strains. This approach has been implemented within the DEEPSOIL software as well.

2.2.2.3.3 Hyperbolic model

The Masing rules (1926) consist of loading and unloading rules and did not provide a functional form for the backbone curve. Subsequently, a study by Hardin and Drnevich (1972) was initially proposed a hyperbolic model to define the backbone curve as follows:

$$\tau = \frac{G_{max} \cdot \gamma}{1 + \frac{\gamma}{\gamma_{ref}}} \quad \text{Equation 40}$$

where reference strain (γ_{ref}) is the ratio of the shear stress at failure (τ_{max}) to the maximum shear modulus (G_{max}).

Subsequent studies have been conducted using the reference strain concept to develop constitutive models. Many of these models are variations of the hyperbolic model, including two of the most commonly used models for site response analyses in practice: the MKZ model by Matasovic (1993) and the GQ/H model by Groholski et al. (2016). It is worth mentioning that the GQ/H model has an advantage over the MKZ model in its capability to match the target shear strength at large shear strains. As illustrated in Figure 23, while the MR curves of the MKZ and GQ/H models align well with the reference curve from Darendeli (2001) at low strains, the GQ/H model provides a significantly better fit between the implied shear strength and the maximum shear strength.

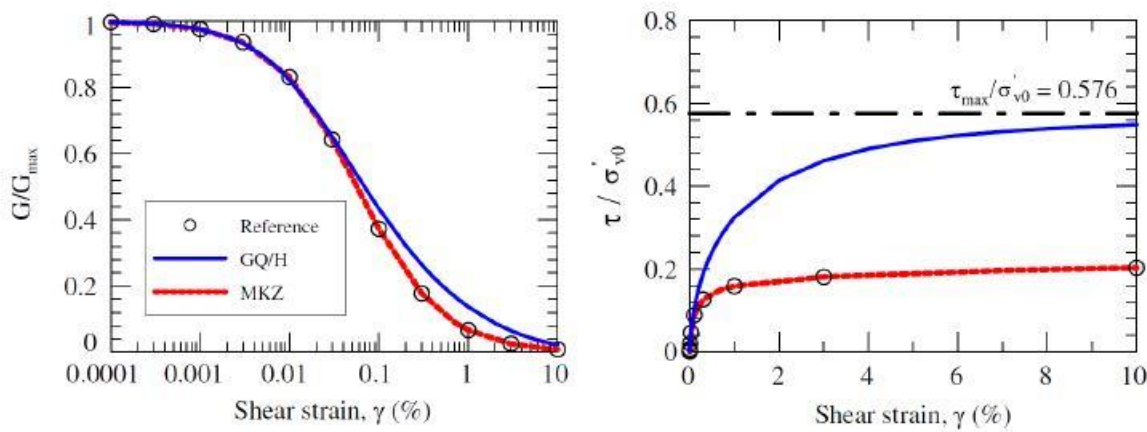


Figure 23 Superiority of the GQ/H model over the MKZ model (Groholski et al. 2016)

2.2.2.3.4 ARCS model

The reliability of outcomes in nonlinear seismic site response analysis relies significantly on the chosen model's capability to capture the nonlinearity in soil precisely (Groholski et al. 2016). Hence, the ARCS (Axis Rotation and Cubic Spline) model, a one-dimensional nonlinear stress-strain model derived by Yniesta et al. (2017), has been selected as the constitutive model for this project. This model has the capacity to fit various input MRD curves precisely. Regarding the

modulus reduction curve, this model employs a cubic spline fitting method applied to the backbone curve, and for the hysteretic damping curve, it employs a coordinate transformation approach guided by a novel unloading and reloading rules, which ensures an accurate fit of the damping curve. As showcased in Figure 24, the comparison drawn among the ARCS model and two commonly employed models, the GQ/H model used in DEEPSOIL and the PressureIndependentMultiYield (PIMY) Model in OpenSees, reveals the ARCS model's superior ability to predict backbone and damping curves, contrasted to the GQ/H model.

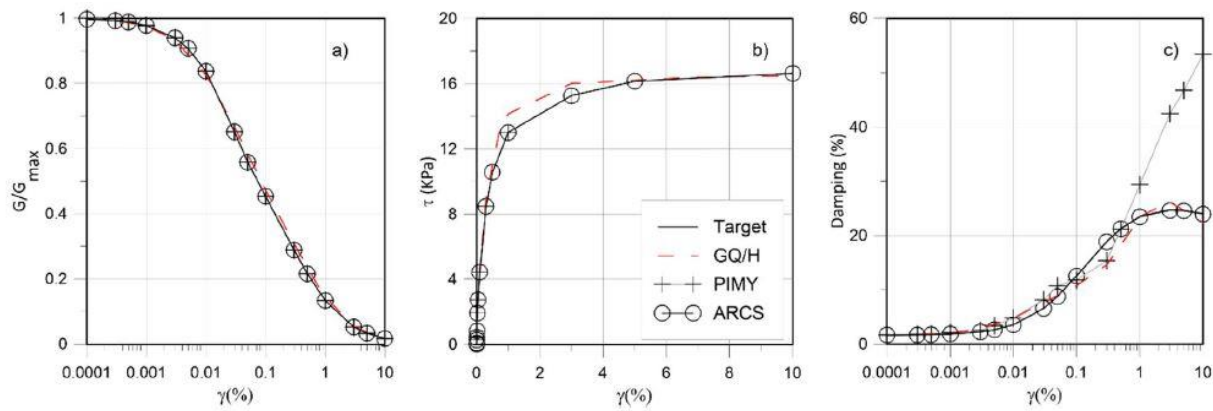


Figure 24 Modulus reduction (a), backbone (b) and damping ratio (c) curves predictions of different models (Yniesta et al. 2017)

The utilization of this model comes with several advantages compared to existing models. Firstly, unlike the commonly used hyperbolic model, it is not constrained by a specific form of backbone curve; secondly, unlike models developed based on Masing rules, it avoids the issue of over-estimating damping at high strains; thirdly, its computational efficiency is superior to existing models, as it does not require a viscous damping formulation for small-strain damping; and lastly, it has been successfully incorporated into the DEEPSOIL software, making it attractive for conducting 1-D ground response analysis using DEEPSOIL.

2.3 Tailings overview

Tailings are fine-grained waste products of mining operations which are produced during extraction of the valuable elements from an ore by milling procedures and deposited in slurry form in a tailings impoundment. The physical characteristics of tailings are highly dependent on various factors, such as the chemical composition of the parent rock; consequently, variations in properties such as size and plasticity index can be observed among different types of tailings. However, tailings from metal mining typically consist of non-plastic fine particles (i.e., less than 0.08 mm) with low hydraulic conductivity (Vick 1990, Aubertin et al. 1996, Mbonimpa et al. 2002, Bussière 2007, James et al. 2011). Tailing impoundments, often pose a substantial risk of potential debris flow (Vick 1990, Wei et al. 2013, Wang et al. 2022). In accordance with the necessary conditions for liquefaction, non-plastic saturated tailings, when deposited as slurries, tend to possess a cohesionless and contractive behavior, thereby making them susceptible to liquefaction. Moreover, as Rico et al. (2008) highlighted, liquefaction is the second major factor contributing to the failure of tailing impoundments. Numerous studies have been carried out on the mine tailings from Western Quebec over the years. Given the utilization of similar tailings profiles in this study, the subsequent sections present some of the critical properties highlighted in these research endeavors.

2.3.1 Geotechnical characterization of mine tailings from Western Quebec

Contreras (2022) presents, the grain size distribution curves of tailings (as illustrated in Figure 25) derived based on several sampling from mine sites located in Western Quebec during 2010s. Table 1 reports the fines' content (<0.002 mm), soil classification, and relative density of mine tailings from Western Quebec, based on the obtained distribution curves mentioned above. Additionally, the conducted Atterberg limits' test on these mine sites reveals that the plastic and liquid limits (i.e., PL and LL) of the tailings exhibit a non-plastic behavior (Contreras 2022).

It is also noteworthy to mention that several studies on mine tailings from Western Quebec, such as those conducted by Bolduc (2012), Contreras (2013), and Golder (2014), reported dry unit weights (γ_d) of 17.02, 10.29-17.36, and 19.5 kN/m^3 , respectively (Opris 2017).

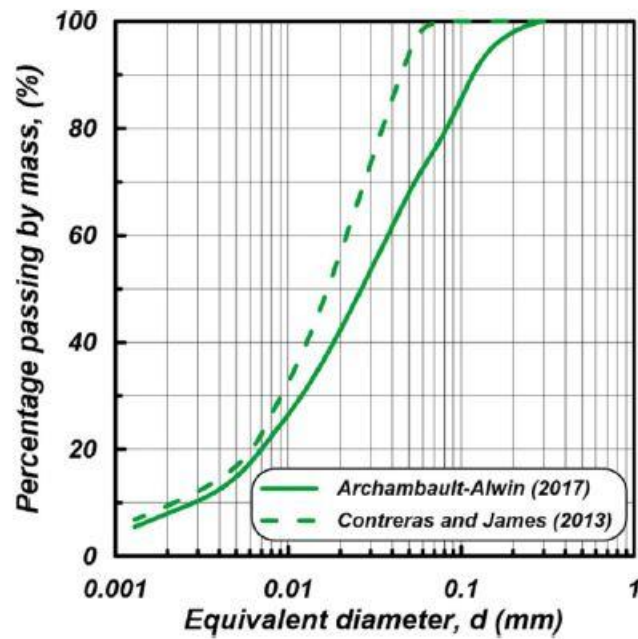


Figure 25 Grain-size distribution of the tailings from mine sites in Western Quebec, retrieved from Contreras (2022).

Table 1 Geotechnical properties of mine tailings from Western Quebec (Contreras 2022)

	Geotechnical characterization of tailings from mine sites located in Western Quebec
FC (%)	91-71
Soil classification (based on USCS)	ML (non-plastic silt)
D_r	2.73

2.3.2 Strength characterization of mine tailings from Western Quebec

James et al. (2011) performed five direct simple shear (DSS) tests on consolidated drained tailings samples obtained from mine pits in Western Quebec to assess their drained shear strength. The

initial vertical stress for these tests ranged from 50 to 300 kPa and following instructions by D3080-98 (ASTM 1998) for drained testing in fine-grained soils, the shear strain rate was set to 2.54 mm/min. The results of direct shear testing are summarized in Table 2 and visually presented in Figure 26. A linear fit to the test results reveals an effective internal friction angle (ϕ') of 36.6° and an effective cohesion (c') of 0 kPa. It is also worth mentioning that according to the results of direct shear tests, all of the samples exhibit contractive behavior throughout the experiment.

Table 2 Summary of direct shear test results (James et al. 2011)

Test	σ'_{vc} (kPa)	e_c (void ratio after consolidation)	τ_{max} (kPa)	$\phi'(^{\circ})$
DS01	103.19	0.701	74.03	35.7
DS02	199.56	0.581	153.02	37.5
DS03	299.28	0.641	204.44	34.3
DS04	54.97	0.723	43.37	38.3
DS05	103.19	0.683	78.37	37.2

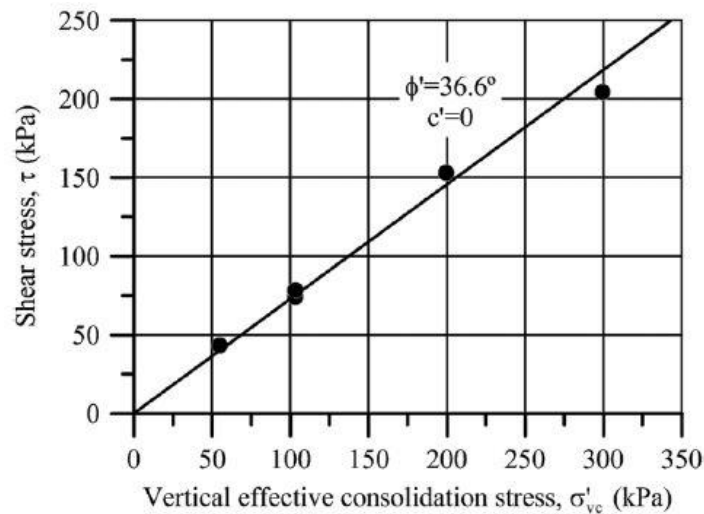


Figure 26 Results of direct shear tests (James et al. 2011)

2.3.3 Dynamic characterization of mine tailings from Western Quebec

Subsequent to direct shear testing, James et al. (2011) also investigate the dynamic characteristics of the same mine tailings using Cyclic Direct Simple Shear (CDSS) testing. The cyclic shear ratio and modulus reduction curves of tailings are obtained through laboratory testing, and illustrated in Figure 27 and 28 respectively. Comparing the trends derived by James et al. (2011) with previous studies reveals that their CSR and MR curves exhibited significant parallels with the sands analyzed by Seed et al. (1984), Sivathayalan (1994), and Sriskandakumar (2004).

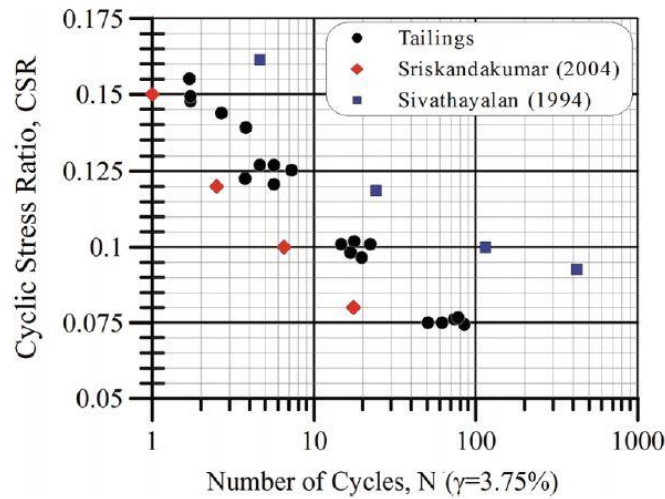


Figure 27 CSR of tailings and Fraser River sand (James et al. 2011)

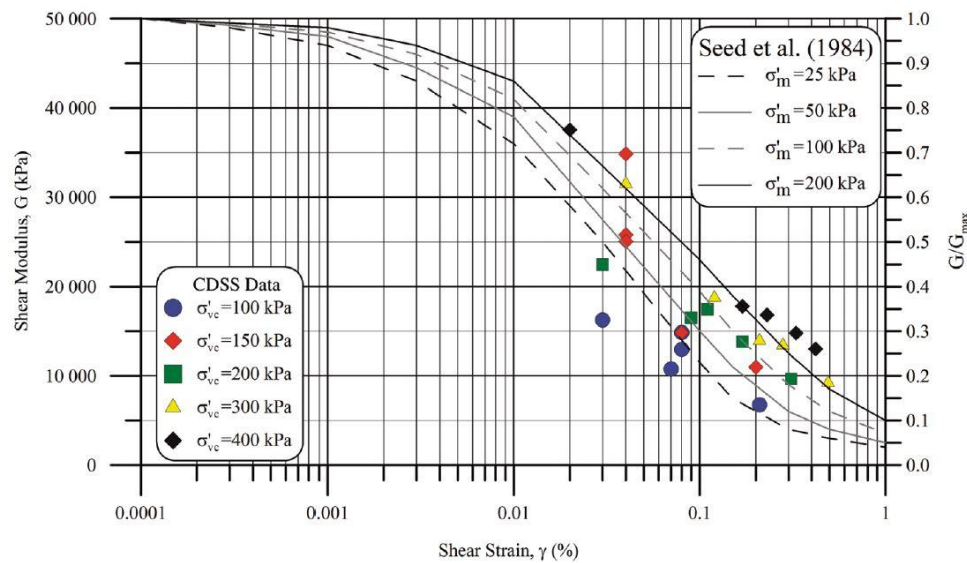


Figure 28 Shear modulus of tailings and MR curves of sands (James et al. 2011)

Another recent cyclic assessment on mine tailings from Western Quebec carried out by Contreras (2022), to calibrate the PM4Sand model (Boulanger and Ziotopoulou 2017). Contreras (2022) compiled test data from multiple studies, including recordings of seismic cone penetration test (SCPTu) conducted by ConeTec 2016 and Opris (2017) on the tailings impoundments from a mine site located in Western Quebec. The obtained shear wave velocity profiles are illustrated in Figure 29. It is noteworthy to mention that, Contreras (2022) attributes the unusual high values of shear wave velocity that observed on some profiles to presence of ice lenses.

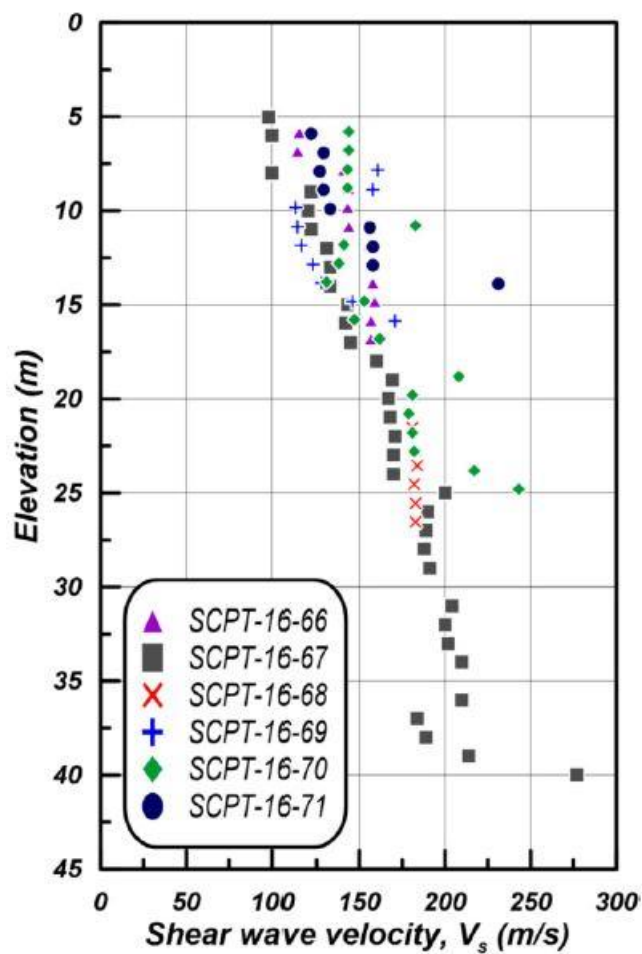


Figure 29 Shear wave velocity profile obtained from field testing (Contreras 2022)

Furthermore, the calibrated model by Contreras (2022) produces the curves shown in Figure 30. This shows that the MRD (modulus reduction and damping) curves for mine tailings from Western Quebec align well with prior studies for cohesion-less soils, such as relationships derived by Seed et al. (1986) and EPRI (1993).

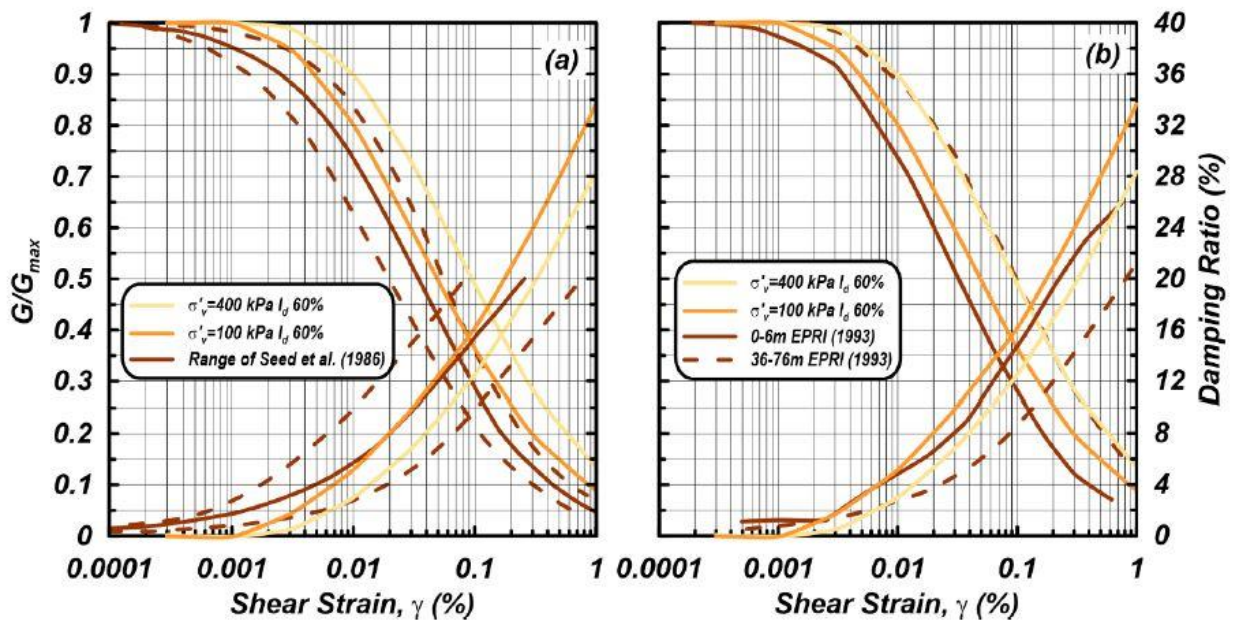


Figure 30 Empirical MRD curves compared with modelled PM4Sand response, based on the relationships proposed by a) Seed et al. (1986), and b) EPRI (1993) retrieved from Contreras (2022)

CHAPTER 3 METHODOLOGY

3.1 Introduction

The goal of this study is to formulate a new nonlinear shear stress reduction factor (r_d) equation suitable for the study of liquefaction in tailings impoundments due to mine-induced seismicity. To do so, the first step is to select adequate ground motion recordings from previous earthquakes. Then, in-situ strength measurements in the tailings impoundments must be collected to gather a representative set of soil profiles. Nonlinear (NL) site-specific response analyses need to be conducted using the ground motion data on the soil profiles obtained in the first two steps. The nonlinear 1D ground response analyses allow to obtain a shear stress reduction factor profile obtained based on the maximum shear stress for the realistic deformable condition of the soil profile (τ_d), and the Peak Ground Acceleration (PGA) at the surface, which is used to calculate the maximum shear stress for the rigid condition of the soil column (τ_r). Finally, the results are analyzed and a multivariate regression analysis is used to derive new equations. In the present chapter, a comprehensive explanation of the procedures involved in each step for deriving the r_d is provided.

3.2 Ground motion selection

Given that actual induced earthquakes recordings were unavailable for this study, ground motions were selected from the PEER Ground Motion Database to represent the mining-induced earthquakes which are characterized with magnitudes ranging from 3 to 6 (Foulger et al. 2018). Since this study is investigating the potential of liquefaction in tailings impoundments, these are expected to be at a short distance of the source, and consequently a maximum distance from the source of 50 km is selected. Note that the distance to the source of the earthquake used in this study is the Joyner-Boore distance (R_{jb}) which is the closest distance to the surface projection of the rupture plane. These recordings are selected from both NGA-East (Goulet et al. 2021) and NGA-West2 (Ancheta et al. 2014) in order to represent the local seismic conditions in Quebec and seismic conditions typical of Western Canada. Their seismicity differs by their rate of occurrence, the maximum magnitude, and the frequency content of their motions. Motions from Western Canada are typical of active seismic regions, with lower frequency content and higher magnitude than motions in Eastern Canada which is a continental stable region. Consequently, 169 unscaled ground motions were selected, comprising 82 from NGA-East and 87 from NGA-West2. It is worth noting

that, for earthquakes from NGA-East, both horizontal components of each recording (i.e., H1 and H2) were included, whereas, for earthquakes from NGA-West2, only one horizontal time history was utilized (i.e., H1). Table 3 and 4, presented below, provide a summary of the record sequence number (RSN), seismic event name, recording station, event year, magnitude, peak ground acceleration (PGA), and distance from the source (R_{jb}) for each ground motion selected from NGA-East and NGA-West2 respectively.

Table 3 Selected ground motions from NGA-East

RSN	Earthquake Name	Year	Station Name	Magnitude	R _j (km)	PGA (g)
1199-H1	LaMalbaie	2003	Ste-Mathilde_ QC	3.53	1.01	0.025
1199-H2	LaMalbaie	2003	Ste-Mathilde_ QC	3.53	1.01	0.030
1563-H1	PortHope	2004	Wesleyville_ ON	3.12	30.78	0.003
1563-H2	PortHope	2004	Wesleyville_ ON	3.12	30.78	0.003
1681-H1	RiviereDuLoup	2005	Riviere-Ouelle_ QC	4.65	36.53	0.034
1681-H2	RiviereDuLoup	2005	Riviere-Ouelle_ QC	4.65	36.53	0.022
1688-H1	RiviereDuLoup	2005	St-Simeon_ QC	4.65	13.96	0.045
1688-H2	RiviereDuLoup	2005	St-Simeon_ QC	4.65	13.96	0.040
1771-H1	RiviereDuLoup	2005	HQ.CHAR	4.65	39.44	0.028
1771-H2	RiviereDuLoup	2005	HQ.CHAR	4.65	39.44	0.031
2126-H1	BaieStPaul	2006	St-Roch-des-Aulnaies_ QC	3.72	25.54	0.003
2126-H2	BaieStPaul	2006	St-Roch-des-Aulnaies_ QC	3.72	25.54	0.004
2127-H1	BaieStPaul	2006	Riviere-Ouelle_ QC	3.72	36.77	0.006
2127-H2	BaieStPaul	2006	Riviere-Ouelle_ QC	3.72	36.77	0.012
2389-H1	Marston	2006	Stahl Farm_ MO	3.41	16.78	0.004
2389-H2	Marston	2006	Stahl Farm_ MO	3.41	16.78	0.004
2749-H1	MtCarmel	2008	Olney Central College Olney_ IL	4.64	32.81	0.042
2749-H2	MtCarmel	2008	Olney Central College Olney_ IL	4.64	32.81	0.036
2841-H1	MtCarmel	2008	Olney Central College Olney_ IL	4.03	37.22	0.018
2841-H2	MtCarmel	2008	Olney Central College Olney_ IL	4.03	37.22	0.013
2941-H1	MtCarmel	2008	Olney Central College Olney_ IL	3.75	37.04	0.004
2941-H2	MtCarmel	2008	Olney Central College Olney_ IL	3.75	37.04	0.004
3054-H1	RiviereDuLoup	2008	Riviere-Ouelle_ QC	3.57	35.89	0.002
3054-H2	RiviereDuLoup	2008	Riviere-Ouelle_ QC	3.57	35.89	0.002
3415-H1	Jones	2010	Guthrie_ OK	3.84	35.37	0.002
3415-H2	Jones	2010	Guthrie_ OK	3.84	35.37	0.002
3492-H1	Lincoln	2010	Wilshire Boulevard; Harrah	4.18	39.84	0.004
3492-H2	Lincoln	2010	Wilshire Boulevard; Harrah	4.18	39.84	0.004
3829-H1	Whiting	2010	Hickman_ KY	3.4	29.53	0.003
3829-H2	Whiting	2010	Hickman_ KY	3.4	29.53	0.004
3843-H1	Whiting	2010	Stahl Farm_ MO	3.4	37.6	0.001
3843-H2	Whiting	2010	Stahl Farm_ MO	3.4	37.6	0.001
3847-H1	Whiting	2010	Penman_ Portageville_ MO	3.4	44.49	0.001
3847-H2	Whiting	2010	Penman_ Portageville_ MO	3.4	44.49	0.002
4027-H1	ValDesBois	2010	Orleans; Innes Road_ ON	5.1	49.27	0.030
4027-H2	ValDesBois	2010	Orleans; Innes Road_ ON	5.1	49.27	0.049
4731-H1	Slaughterville	2010	Jones High School	4.36	39.65	0.008
4731-H2	Slaughterville	2010	Jones High School	4.36	39.65	0.012
4736-H1	Slaughterville	2010	Choctaw High School	4.36	30.51	0.013
4736-H2	Slaughterville	2010	Choctaw High School	4.36	30.51	0.013
4738-H1	Slaughterville	2010	Spencer City Hall	4.36	34.21	0.007
4738-H2	Slaughterville	2010	Spencer City Hall	4.36	34.21	0.007
5242-H1	Guy	2010	X102 Temporary station_ Greenbriar_ AR	3.86	11.36	0.017
5242-H2	Guy	2010	X102 Temporary station_ Greenbriar_ AR	3.86	11.36	0.010
5243-H1	Guy	2010	X201 Temporary Station_ Greenbriar_ AR	3.86	3.73	0.048
5243-H2	Guy	2010	X201 Temporary Station_ Greenbriar_ AR	3.86	3.73	0.032
5714-H1	Arcadia	2010	Jones High School	3.96	8.21	0.023
5714-H2	Arcadia	2010	Jones High School	3.96	8.21	0.034
5718-H1	Arcadia	2010	Luther Middle School	3.96	5.57	0.032
5718-H2	Arcadia	2010	Luther Middle School	3.96	5.57	0.037
5720-H1	Arcadia	2010	GS.OK009	3.96	16.46	0.008
5720-H2	Arcadia	2010	GS.OK009	3.96	16.46	0.004
6011-H1	BethelAcres	2010	Tecumseh_ OK	3.23	28.62	0.0003
6011-H2	BethelAcres	2010	Tecumseh_ OK	3.23	28.62	0.001
6569-H1	Guy	2010	X102 Temporary station_ Greenbriar_ AR	3.9	7.03	0.044
6569-H2	Guy	2010	X102 Temporary station_ Greenbriar_ AR	3.9	7.03	0.023
6570-H1	Guy	2010	X201 Temporary Station_ Greenbriar_ AR	3.9	4.62	0.009
6570-H2	Guy	2010	X201 Temporary Station_ Greenbriar_ AR	3.9	4.62	0.022
6934-H1	Greenbrier	2011	Wooly Hollow	4.68	5.25	0.040

Table 3 Selected ground motions from NGA-East (cont'd)

RSN	Earthquake Name	Year	Station Name	Magnitude	R _{jb} (km)	PGA (g)
6934-H2	Greenbrier	2011	Wooly Hollow	4.68	5.25	0.050
8202-H1	Hawkesbury	2011	Alfred_ ON	3.59	26.03	0.008
8202-H2	Hawkesbury	2011	Alfred_ ON	3.59	26.03	0.007
8277-H1	Charlevoix	2001	Riviere-Ouelle_ QC	3.6	21.05	0.003
8277-H2	Charlevoix	2001	Riviere-Ouelle_ QC	3.6	21.05	0.004
8278-H1	Charlevoix	2001	St-Andre_ QC	3.6	17.88	0.004
8278-H2	Charlevoix	2001	St-Andre_ QC	3.6	17.88	0.002
8303-H1	BaieStPaul	2002	St-Roch-des-Aulnaies_ QC	3.24	25.64	0.006
8303-H2	BaieStPaul	2002	St-Roch-des-Aulnaies_ QC	3.24	25.64	0.003
8571-H1	Mineral	2011	SE.NANPP	5.74	18.53	0.104
8571-H2	Mineral	2011	SE.NANPP	5.74	18.53	0.265
9069-H1	Mineral	2011	NQ.Q793	3.97	9.42	0.022
9069-H2	Mineral	2011	NQ.Q793	3.97	9.42	0.035
9935-H1	Sparks	2011	Meyer Ranch_ Chandler_ OK	4.73	23.78	0.014
9935-H2	Sparks	2011	Meyer Ranch_ Chandler_ OK	4.73	23.78	0.010
10055-H1	Sparks	2011	Wilshire Boulevard; Harrah	5.68	38.85	0.028
10055-H2	Sparks	2011	Wilshire Boulevard; Harrah	5.68	38.85	0.031
10056-H1	Sparks	2011	Luther Middle School	5.68	39.84	0.027
10056-H2	Sparks	2011	Luther Middle School	5.68	39.84	0.022
10058-H1	Sparks	2011	GS.OK010	5.68	42	0.042
10058-H2	Sparks	2011	GS.OK010	5.68	42	0.032
10683-H1	Comal	2011	Kenedy_ TX	4.71	32.54	0.008
10683-H2	Comal	2011	Kenedy_ TX	4.71	32.54	0.011

Table 4 Selected ground motions from NGA-West2

RSN	Earthquake Name	Year	Station Name	Magnitude	R _{jb} (km)	PGA (g)
1-H1	Helena_ Montana	1935	Carroll College	6	2.07	0.164
2-H1	Helena_ Montana	1935	Helena Fed Bldg	6	2.09	0.048
4-H1	Imperial Valley	1938	El Centro Array #9	5	32.44	0.015
10-H1	Imperial Valley	1951	El Centro Array #9	5.6	24.58	0.031
16-H1	Northern Calif	1952	Ferndale City Hall	5.2	42.69	0.054
18-H1	Imperial Valley	1953	El Centro Array #9	5.5	15.11	0.007
19-H1	Central Calif	1954	Hollister City Hall	5.3	25.11	0.049
21-H1	Imperial Valley	1955	El Centro Array #9	5.4	13.78	0.051
23-H1	San Francisco	1957	Golden Gate Park	5.28	9.74	0.087
24-H1	Central Calif	1960	Hollister City Hall	5	7.28	0.023
26-H1	Hollister	1961	Hollister City Hall	5.6	19.55	0.059
27-H1	Hollister	1961	Hollister City Hall	5.5	17.2	0.059
34-H1	Northern Calif	1967	Ferndale City Hall	5.6	27.36	0.254
35-H1	Northern Calif	1967	Hollister City Hall	5.2	37.11	0.017
42-H1	Lytle Creek	1970	Cedar Springs Pumphouse	5.33	21.33	0.061
43-H1	Lytle Creek	1970	Cedar Springs_ Allen Ranch	5.33	17.4	0.042
45-H1	Lytle Creek	1970	Devil's Canyon	5.33	18.39	0.175
50-H1	Lytle Creek	1970	Wrightwood - 6074 Park Dr	5.33	10.7	0.145
96-H1	Managua_ Nicaragua	1972	Managua_ ESSO	5.2	4.33	0.263
97-H1	Point Mugu	1973	Port Hueneme	5.65	15.48	0.128
98-H1	Hollister	1974	Gilroy Array #1	5.14	9.99	0.101
99-H1	Hollister	1974	Hollister City Hall	5.14	8.85	0.094
100-H1	Hollister	1974	San Juan Bautista_ 24 Polk St	5.14	8.56	0.047
101-H1	Northern Calif	1975	Cape Mendocino	5.2	28.73	0.118
102-H1	Northern Calif	1975	Ferndale City Hall	5.2	8.2	0.091
103-H1	Northern Calif	1975	Petrolia_ General Store	5.2	28.48	0.169
106-H1	Oroville	1975	Oroville Seismograph Station	5.89	7.79	0.089
107-H1	Oroville	1975	Oroville Airport	4.79	13.55	0.040
108-H1	Oroville	1975	Up & Down Cafe (OR1)	4.79	12.07	0.031
109-H1	Oroville	1975	Medical Center	4.37	9.22	0.079

Table 4 Selected ground motions from NGA-West2 (cont'd)

RSN	Earthquake Name	Year	Station Name	Magnitude	R _b (km)	PGA (g)
110-H1	Oroville	1975	Oroville Airport	4.37	13.37	0.020
111-H1	Oroville	1975	Up & Down Cafe (OR1)	4.37	11	0.073
112-H1	Oroville	1975	Broadbeck Residence	4.7	5.95	0.160
113-H1	Oroville	1975	DWR Garage	4.7	0.1	0.113
116-H1	Oroville	1975	Nelson Ranch (OR7)	4.7	0.77	0.104
119-H1	Oroville	1975	Summit Ave (OR6)	4.7	4.79	0.103
127-H1	Friuli_ Italy	1976	Buia	5.5	10.56	0.045
128-H1	Friuli_ Italy	1976	Forgaria Cornino	5.5	16.33	0.098
129-H1	Friuli_ Italy	1976	San Rocco	5.5	16.26	0.033
130-H1	Friuli_ Italy	1976	Buia	5.91	10.99	0.110
131-H1	Friuli_ Italy	1976	Codroipo	5.91	41.37	0.030
132-H1	Friuli_ Italy	1976	Forgaria Cornino	5.91	14.65	0.262
133-H1	Friuli_ Italy	1976	San Rocco	5.91	14.37	0.060
134-H1	Izmir_ Turkey	1977	Izmir	5.3	0.74	0.426
135-H1	Santa Barbara	1978	Cachuma Dam Toe	5.92	23.75	0.073
136-H1	Santa Barbara	1978	Santa Barbara Courthouse	5.92	0.1	0.102
144-H1	Dursunbey_ Turkey	1979	Dursunbey	5.34	5.57	0.225
145-H1	Coyote Lake	1979	Coyote Lake Dam - Southwest Abutment	5.74	5.3	0.141
148-H1	Coyote Lake	1979	Gilroy Array #3	5.74	6.75	0.255
149-H1	Coyote Lake	1979	Gilroy Array #4	5.74	4.79	0.233
150-H1	Coyote Lake	1979	Gilroy Array #6	5.74	0.42	0.422
155-H1	Norcia_ Italy	1979	Bevagna	5.9	31.43	0.036
156-H1	Norcia_ Italy	1979	Cascia	5.9	1.41	0.157
157-H1	Norcia_ Italy	1979	Spoletto	5.9	13.21	0.044
204-H1	Imperial Valley	1979	El Centro Array #6	5.01	7.4	0.160
205-H1	Imperial Valley	1979	El Centro Array #7	5.01	7.32	0.141
207-H1	Imperial Valley	1979	El Centro Differential Array	5.01	7.87	0.149
208-H1	Imperial Valley	1979	Holtville Post Office	5.01	7.69	0.115
209-H1	Imperial Valley	1979	Westmorland Fire Sta	5.62	9.39	0.151
211-H1	Livermore	1980	Antioch - 510 G St	5.8	13.38	0.042
212-H1	Livermore	1980	Del Valle Dam (Toe)	5.8	23.92	0.130
214-H1	Livermore	1980	San Ramon - Eastman Kodak	5.8	15.19	0.150
215-H1	Livermore	1980	San Ramon Fire Station	5.8	15.84	0.055
219-H1	Livermore	1980	Del Valle Dam (Toe)	5.42	10.03	0.039
221-H1	Livermore	1980	Livermore - Fagundas Ranch	5.42	0.79	0.255
222-H1	Livermore	1980	Livermore - Morgan Terr Park	5.42	7.94	0.184
224-H1	Livermore	1980	San Ramon Fire Station	5.42	19.09	0.046
225-H1	Anza (Horse Canyon)	1980	Anza - Pinyon Flat	5.19	12.24	0.100
17794-H1	9163314	2000	Santa Monica Fire St	3.08	4.87	0.083
17871-H1	9907437	2003	LB - Orange Ave	3.14	2.95	0.018
17986-H1	10488077	2009	Owens Lake	3.05	0.85	0.002
17996-H1	10488997	2009	Owens Lake	3.07	2.21	0.002
18654-H1	51210970	2008	Mt. Pleasant High School	3.2	9.69	0.001
18667-H1	51210970	2008	Mt. Hamilton Road	3.2	6.31	0.004
17024-H1	9083372	1999	Santa Ana	3.45	8.04	0.018
17062-H1	9089885	1999	Univ Southern Ca	3.43	5.78	0.003
17443-H1	51177794	2007	Anderson Dam	3.4	10.1	0.001
17799-H1	14000872	2003	Calstate Bakersfield	3.28	27.8	0.001
17823-H1	10004657	2004	Saddle Peak Fire Camp 8	3.25	8.74	0.003
14802-H1	21339029	2004	Laurel Hill	3.58	15.79	0.011
15074-H1	10065241	2004	Santa Barbara	3.52	16.91	0.001
4258-H1	Ancona_ Italy	1972	Ancona-Palombina	3.9	9.97	0.100
4264-H1	Ancona_ Italy	1972	Ancona-Palombina	4	1.31	0.410
4289-H1	Umbria_ Italy	1979	Norcia	3.7	4.11	0.017
12476-H1	21266207	2003	Dublin - Fire Station	4	2.1	0.010
12815-H1	9128775	1999	Highland_ Baseline	3.81	19.4	0.002
13046-H1	21305648	2003	Haviland Hall_ Berkeley_ CA_ USA	4	4.16	0.015

3.2.1 Ground motion recordings moment magnitude distribution

The histograms below illustrate the magnitude distribution of the motions from NGA-East, NGA-West2, and the combined set of motions, respectively. Additionally, Figure 34 displays the magnitude distribution of the selected ground motion recordings with their corresponding distance.

As illustrated in Figures 31 and 32, the majority of Eastern ground motions selected tend to have lower magnitudes, while most of Western ground motions fall within the 5 to 6 magnitude range. The higher average magnitude of ground motions selected from NGA-West2, in contrast to those chosen from NGA-East, is attributed to the heightened seismic activity in Western North America, as opposed to the relatively more stable seismic activity observed in Central and Eastern North America. However, Figure 33 shows that the combination of ground motions sourced from NGA-East and NGA-West2 exhibits a nearly normal distribution within the moment magnitude range of 3 to 6 in the context of this study.

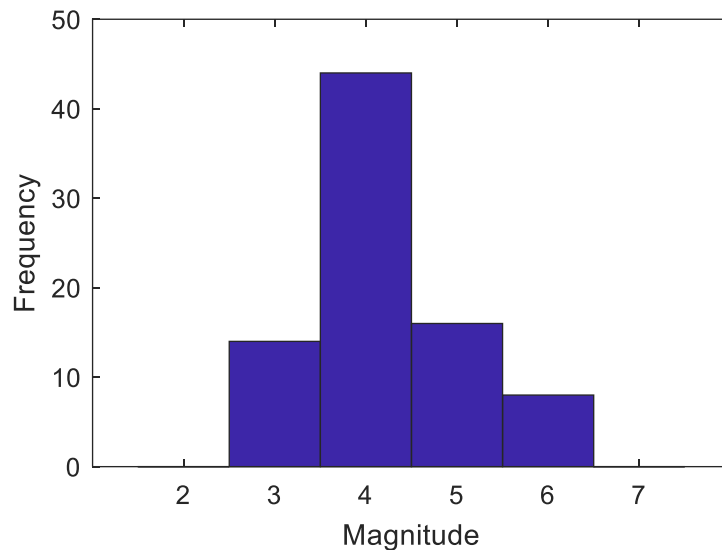


Figure 31 Magnitude distribution of selected ground motions from NGA-East

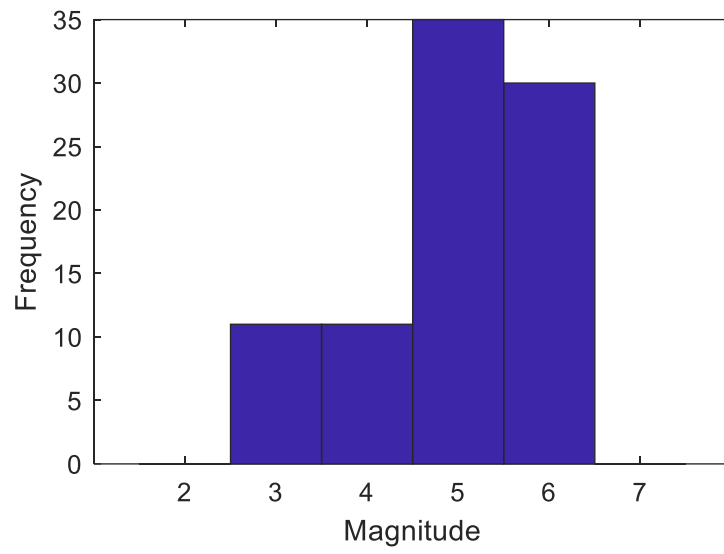


Figure 32 Magnitude distribution of selected ground motions from NGA-West2

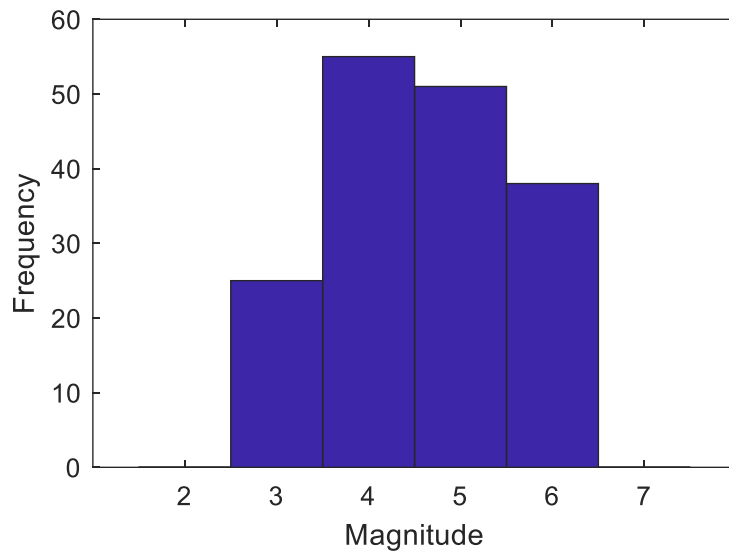


Figure 33 Magnitude distribution of all selected ground motions

Figure 34 shows that the majority of ground motions were captured within a distance span of 10 to 50 kilometers from the fault rupture. Nevertheless, the ground motions from NGA-West2 exhibit a wider range of distances from the seismic source in comparison to those obtained from NGA-East. In fact, several of the NGA-West2 recordings were captured at distances of less than 10 kilometers from the fault rupture plane, while only a handful of motions from NGA-East were.

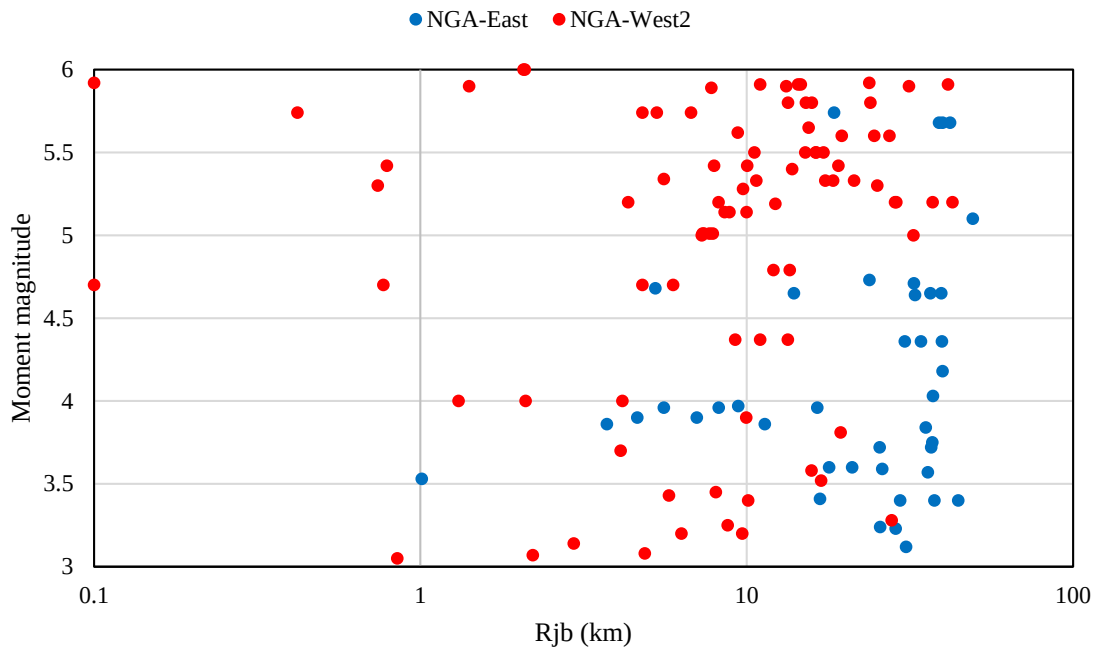


Figure 34 Distribution of moment magnitude with respect to distance

3.2.2 Ground motion recordings peak ground acceleration and velocity distributions

The values of peak ground acceleration (PGA) and peak ground velocity (PGV) for the selected ground motions vary from 0.0003g to 0.43g and 0.00008 m/s to 0.44 m/s, respectively. Figure 35 displays the distribution of PGV in relation to PGA. It shows that PGV and PGA of selected ground motions have a linear relationship. Also, ground motions selected from NGA-West2 exhibit a slightly higher PGV to PGA ratio when compared to those from NGA-East. Furthermore, Figures 36 presents the distribution of PGA and PGV as a function of magnitude. As expected there is a noticeable linear correlation between both PGA and PGV with moment magnitude. Additionally,

it is evident that the average moment magnitude of ground motions derived from NGA-West2 exceeds that of those from NGA-East subsequently, the corresponding average values of PGA and PGV for NGA-West2 are also higher than those for NGA-East.

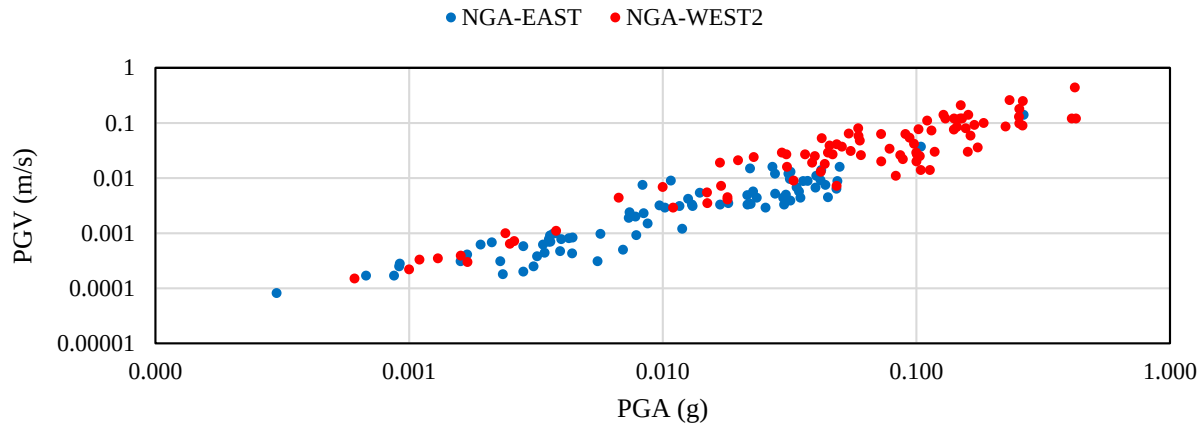


Figure 35 PGV in relation to PGA

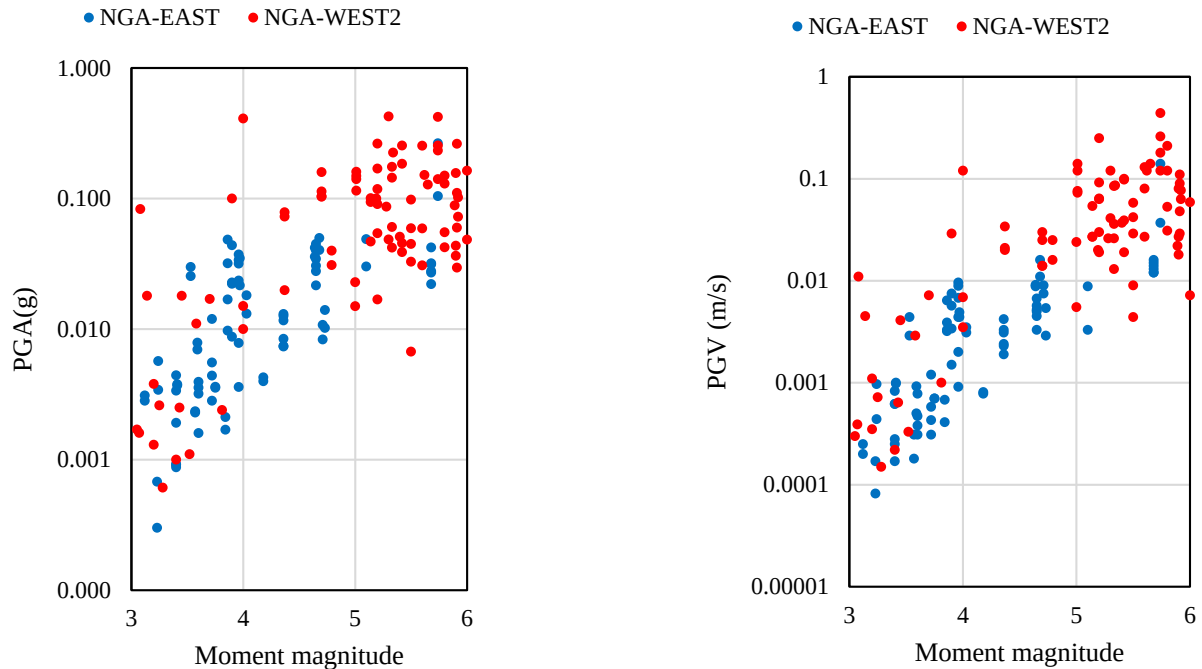


Figure 36 Distribution of PGA and PGV with moment magnitude

3.2.3 Distributions of ground motion recordings Arias Intensity and significant duration

The Arias Intensity (AI) is an additional metric for assessing the intensity of the selected ground motions. It is an indicator of cumulative energy and is calculated as follows:

$$AI = \frac{\pi}{2g} \int_0^{t_{max}} a(t)^2 dt \quad \text{Equation 41}$$

Where a and t are acceleration and time, and g is gravity acceleration.

It is noteworthy that in this research, the AI for each earthquake was computed by taking the geometric mean of both horizontal components (i.e., H1 and H2). The distribution of AI in relation with moment magnitude is represented in Figure 37 below.

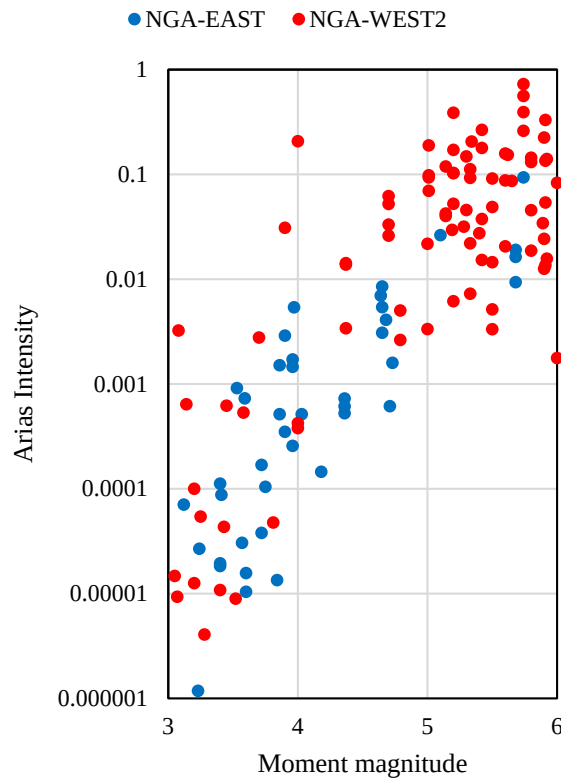


Figure 37 Arias intensity in relation with Moment magnitude

Moreover, the significant duration of each earthquake (i.e., D_{5-95}), representing the time needed to go from 5% to 95% of AI_{max} , was calculated for each motion. Figure 38, provided below, illustrates the distribution of D_{5-95} with moment magnitude.

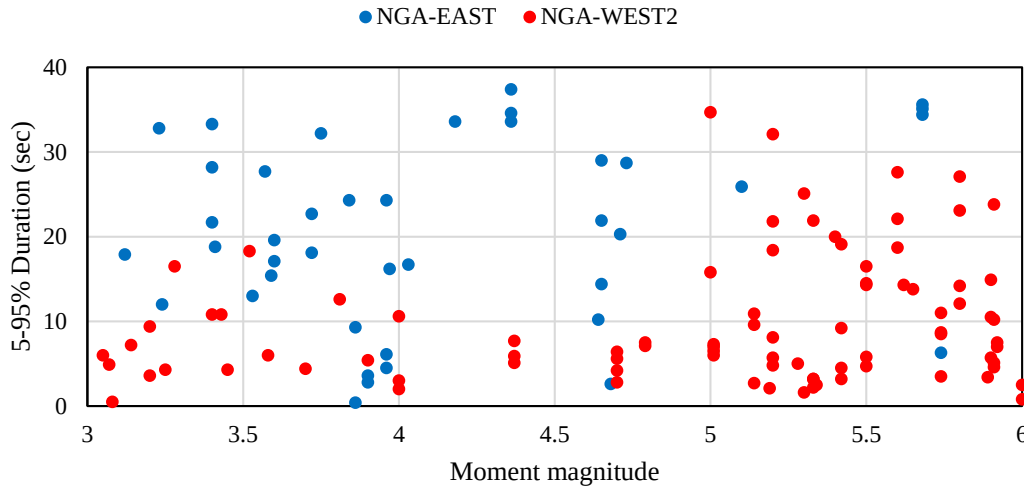


Figure 38 5-95% Distribution of significant duration with respect to moment magnitude

3.2.4 Distribution of spectral acceleration

The spectral acceleration (S_a) of the chosen ground motions at periods of 0.1, 0.5, 1, and 2 second are plotted in Figure 39, respectively, alongside their corresponding moment magnitudes.

As expected, given that the earthquakes selected from NGA-West2, originating from a more seismically active region, tend to have higher magnitudes when compared to those selected from NGA-East, all of the corresponding intensity parameters, such as PGA, PGV, AI, and PSA, are higher for NGA-West2 ground motions than NGA-East at higher values.

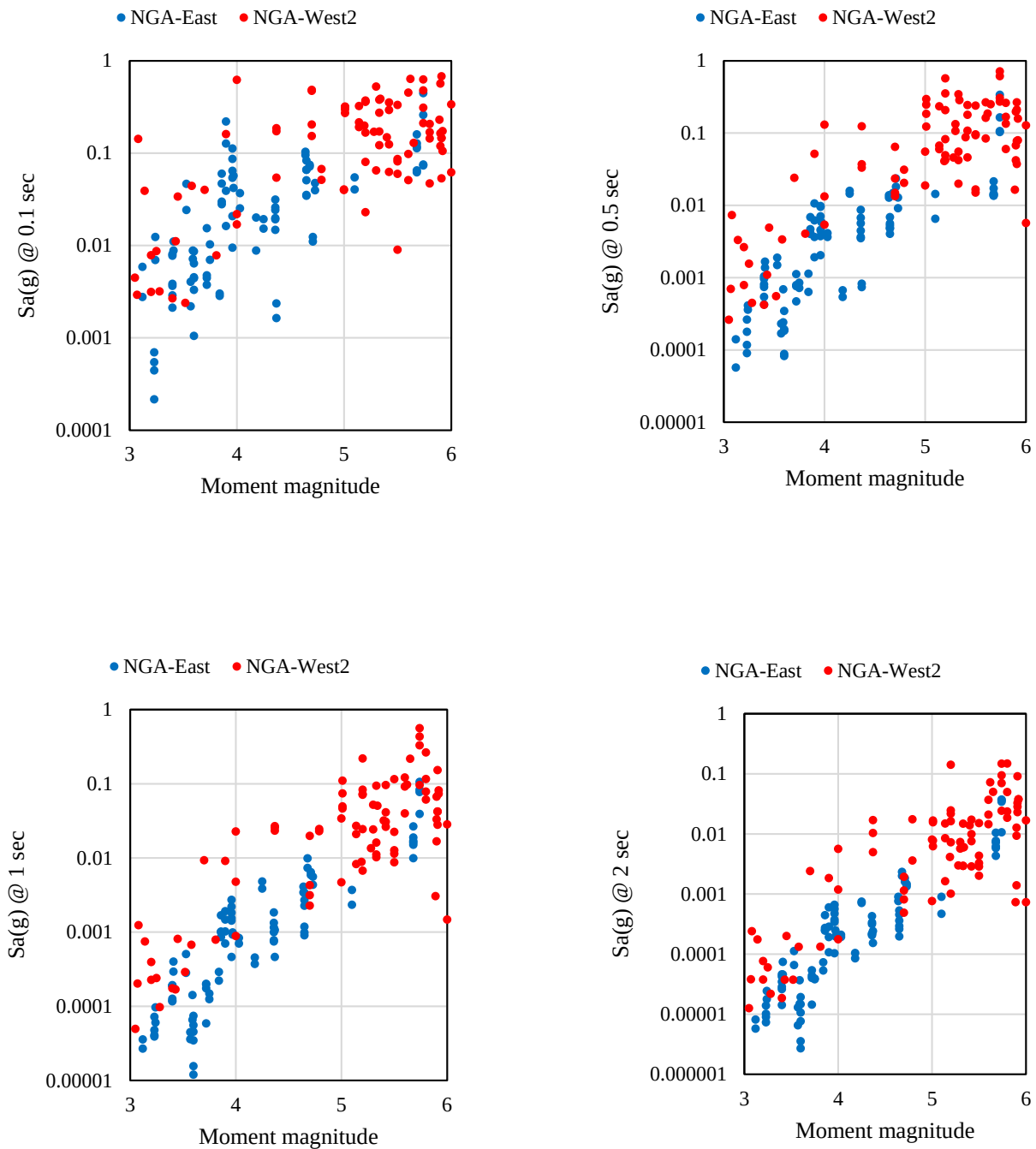


Figure 39 S_a at 0.1, 0.5, 1, and 2 second as a function of moment magnitude

3.3 Selected soil profiles

3.3.1 Available field data

For this study, shear wave velocity recordings of 14 anonymous soil profiles from a mine site in Western Quebec have been provided by WSP Inc. The shear wave velocity through each soil profile was measured at 1 meter intervals. Available V_s profiles with respect to depth are illustrated in Figure 40 below.

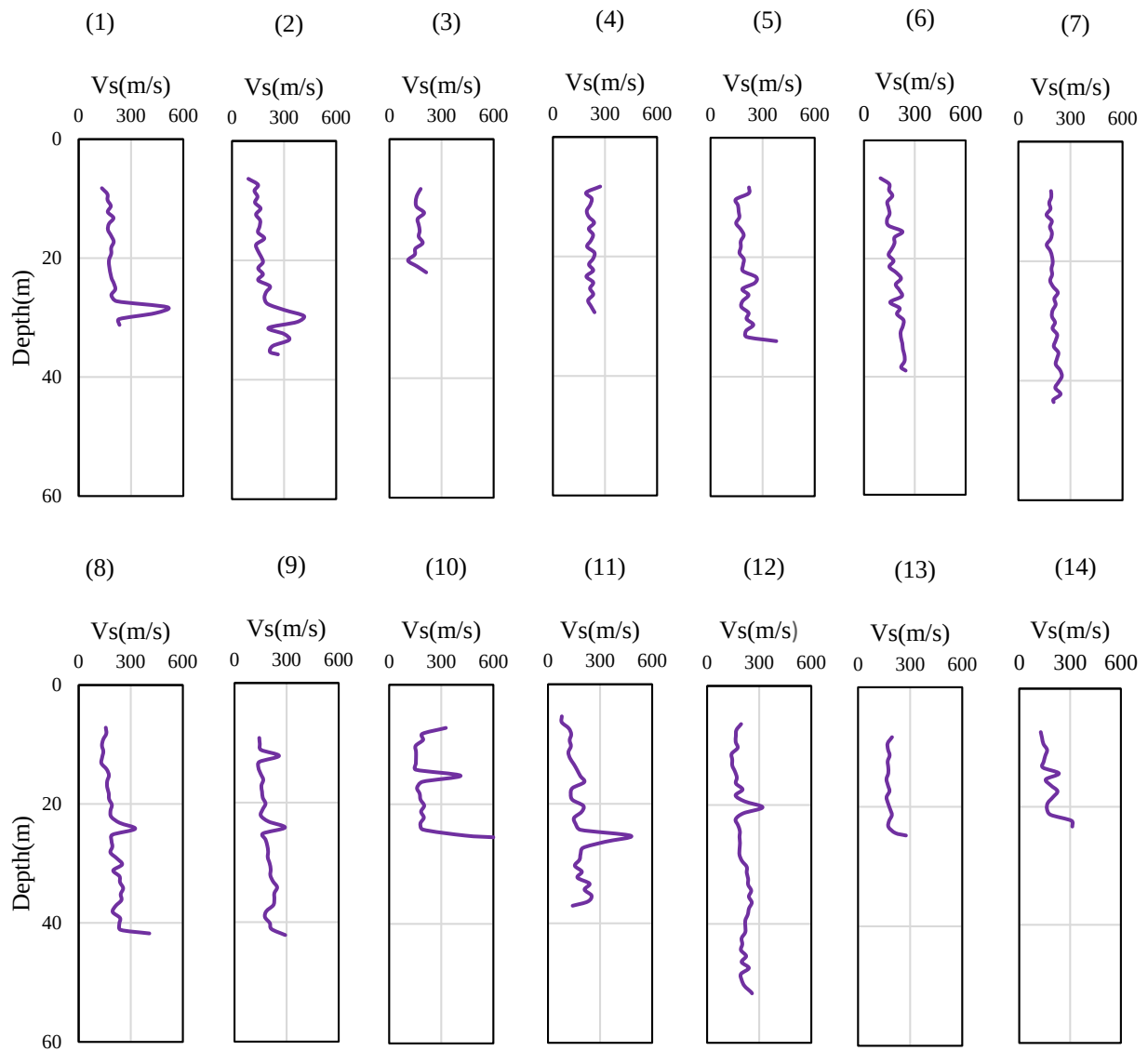


Figure 40 Shear wave velocity profiles based on SCPT recordings

As shown in Figure 40 above, the shear wave velocity was not recorded over the initial couple of meters near the surface. This lack of data stems from the fact that a layer of waste rock is typically deposited on top of the tailings impoundments. Consequently, during penetration tests (e.g., seismic cone penetration test (SCPT)) on these profiles, the first meters of waste rock are drilled out and once the tip of the cone and its probe reach the tailings beneath them, the measurement begins. Thus, in this project, every given profile is assumed to consist of two distinct material layers: the waste rock layer from the surface to the depth of the first recorded data and the tailings layer extending from that point to the bedrock. A schematic representation of a given soil profile is presented in Figure 41 below.

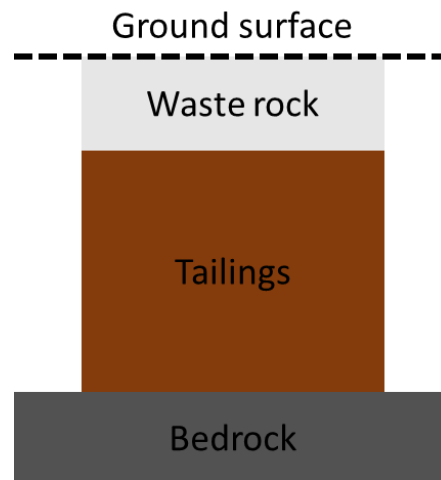


Figure 41 Schematic view of soil profiles

3.3.2 Primary properties of soil profiles

The main characteristics of each profile including total depth, thickness of waste rock layer, thickness of tailings layer, natural frequency, natural period, $V_{s,12}$, and $V_{s,30}$ are summarized in Table 5 below. Notably, for the profiles with total depths less than 30 meters, the average shear wave velocity throughout the entire profile up to the bedrock is considered equivalent to $V_{s,30}$.

Table 5 Summary of profiles' properties

Profile	Total depth (m)	Thickness of waste rock (m)	Thickness of tailings (m)	Natural frequency (Hz)	Natural period (sec)	$V_{s,12}$ (m/s)	$V_{s,30}$ (m/s)
1	32	7	25	1.74	0.57	232	214
2	36	3	33	1.31	0.76	170	177
3	22	6	16	2.20	0.45	238	194
4	29	6	23	2.06	0.48	272	238
5	34	6	28	1.56	0.64	245	208
6	39	4	35	1.22	0.82	185	181
7	44	6	38	1.24	0.80	246	211
8	42	6	36	1.22	0.82	205	194
9	42	7	35	1.21	0.83	248	197
10	25	6	19	2.14	0.47	240	214
11	37	3	34	1.13	0.88	140	161
12	52	3	49	1	0.99	207	194
13	25	6	19	2.11	0.47	247	206
14	24	6	18	2.14	0.47	207	200

3.3.3 Assumptive physical properties

In order to import the soil profiles to DEEPSOIL to perform nonlinear analysis, additional physical properties of each soil profile must be determined. Due to limited field data of soil profiles, some of these characteristics are derived from practical experience or previous studies as follows:

1. Water table position: As tailings are typically deposited as saturated slurry in practical applications, the boundary dividing the waste rock and tailings layer is considered the water table's position in this study. In other words, the tailings layer is assumed to be fully saturated. While it is likely that the water table height would be partially in the waste rocks, this assumption has a limited effect on the results of the analysis.
2. Plasticity index (PI) and Over consolidation ratio (OCR): Since tailings are characterized as nonplastic fine-grained materials and waste rock consists of crushed large stone particles, both layers are assumed to be cohesionless in this study. Consequently, the plasticity index for both materials is considered to be zero, and their over consolidation ratio is assumed to be equal to one.
3. Unit weight (γ): Based on the values employed in practical applications by the mining team at WSP Inc., the unit weight of waste rock is considered to be 20 kN/m³, while the unit weight of tailings is considered to be 17.5 kN/m³ based on site investigation report prepared by ConeTec Investigations Ltd. and provided by WSP Inc.
4. Friction angle (ϕ): According to Azam et al. (2020), the friction angle of waste rock is typically within the range of 37° to 39°. For the purposes of this study, the mean value (i.e., 38°) is adopted. Regarding the friction angle of tailings, various studies have been conducted to characterize tailings properties in Western Quebec, including studies by Poncelet (2012), Contreras (2013), Golder (2014), Grimard and James (2017), and Archambault-Alwin (2017). These studies have suggested a range of friction angles between 35° and 38.7°. However, it has been generally observed that a 35-degree angle is more representative (Opris 2017). Therefore, a friction angle of 35° is selected for tailings in this project.
5. Shear wave velocity (V_s): As all available data for shear wave velocity is situated within the tailings layer, an average value of 350 m/s for the shear wave velocity has been considered through the waste rock layer in accordance with current practices at WSP Inc.

Table 6 below summarizes the assumptive properties of the given soil profiles in this study.

Table 6 Summary of assumptive physical properties of soil profiles

	Waste rock	Tailings
$S_r(\%)$	0	100
PI	0	0
OCR	1	1
$\gamma(kN/m^3)$	20	17.5
$\phi(^{\circ})$	38	35
$V_s(m/s)$	350	Retrieved from recordings

3.3.4 Shear strength

Given the unit weight of each material layer and the water table depth, the vertical total stress, pore water pressure, and effective stress can be computed along the soil profile using Terzaghi (1925, 1936) equations. To determine the mean effective stress, the coefficient of earth pressure at rest (K_o) is estimated using empirical correlations derived from the work of Kulhawy and Mayne (1990) as follows:

$$K_o = (1 - \sin \phi')(OCR)^{\sin \phi'} \quad \text{Equation 42}$$

since OCR is considered equal to one for this study, K_o is equal to $1 - \sin \phi'$. Hence, the mean effective stress is calculated as follows:

$$\text{mean effective stress} = \sigma' \cdot \frac{1 + 2K_o}{3} \quad \text{Equation 43}$$

The shear strength (τ_{max}) in both waste rock and tailings layers, which exhibit cohesionless behavior, is calculated using the Mohr-Coulomb relations, as outlined below:

$$\tau_{max} = \sigma' \cdot \tan \phi \quad \text{Equation 44}$$

3.3.5 Interpretation of soil profiles to use in DEEPSOIL

The site response analysis software which has been chosen for this study, is DEEPSOIL V7.0 (Hashash et al. 2017). During the execution of a nonlinear analysis using DEEPSOIL, the software's runtime is directly influenced by the number of layers defined for a given soil profile. A higher number of layers results in larger stiffness and mass matrices. To reduce the runtime, the approach developed by Zhang 2021 is used. In this approach, the soil layers defined by the CPT results are grouped to reduce the total number of layers for each profile effectively. Moreover, Zhang provided a MATLAB code to automatically calculate the dynamic properties of each layer and ultimately export an Excel worksheet file consisting of thickness, unit weight, shear wave velocity, minimum damping value, strength adjusted modulus reduction curves, and damping curves of each layer to use in DEEPSOIL's "Soil Profile Definition" step. It is noteworthy to mention that Zhang's framework, since developed based on different sets of in-situ measurements and soil profiles as input, has undergone specific modifications to ensure its compatibility with the data utilized in this study. Firstly, Zhang 2021 utilized continuous CPT recordings throughout the entire soil profile depth, leading to the initial layering of profiles based on these recordings. However, as outlined in section 3.3.1, in this study, the first meters of each profile were drilled out and recordings for the waste rock were unavailable. Therefore, each profile was primarily layered into two sections of waste rock and tailings. Secondly, Zhang 2021 categorized its layers into two layers of sandy and clayey by computing their classification index (I_c). Subsequently, two separate sets of empirical relationships were employed to determine the basic properties of each soil layer. However, since the soil profiles for this study consist of tailings and waste rock material and the empirical relationships were developed for sand and clay, as mentioned in section 3.3.3, some physical properties of the layers have been retrieved from practical experience or previous studies. Thirdly, in Zhang's (2021) work, the utilization of limited data points from SCPT in the field prompted the need for calibration. This involved aligning these data points with empirical CPT-based relationships estimating the shear wave velocity (V_s) to enhance accuracy in computations. In contrast, in this study, SCPT data were available for the entire depth of each profile, obviating the need for calibration. Fourthly, Zhang (2021) undertook the calibration of undrained shear strength (S_u) for clayey layers using field vane shear measurements, aiming to rectify potential overestimation in the estimated values. However, as this study did not involve clayey layers, such calibration was unnecessary.

3.3.5.1 Grouping of soil layers

To maintain compatibility with the Lysmer criterion (Kuhlemeyer and Lysmer, 1973) and to avoid introducing artificial damping, it is essential that the grouping process adheres to the specified maximum allowable thickness for each layer. This ensures that the earthquake propagation through the soil profile remains accurate without a loss at higher frequency:

$$\Delta H < \frac{V_{s,minimum}/f}{10} \quad \text{Equation 45}$$

where a maximum frequency of $f = 25 \text{ Hz}$ is utilized for this study to ensure that the maximum layer thickness does not exceed one-tenth of the smallest wavelength (i.e., $V_{s,minimum}/f$) expected to propagate through the soil profile.

3.3.5.2 Calculating dynamic properties of new soil layers

For each group layer, the empirical relationships by Darendeli (2001) were employed to obtain the modulus reduction (MR) curve, minimum damping (D_{min}), and hysteretic damping curve ($D(\gamma)$) as a function of PI , OCR , and K_o as explained in section 2.2.1.2.1. Following the definition of these curves, the strength adjustment procedure by Yee et al. (2013) is applied. This adjustment is required due to the possible disparity between the implied shear strengths generated by the Darendeli curves and the maximum shear strengths of the individual groups.

3.4 Performing nonlinear site response analysis using DEEPSOIL

In this study, in order to perform a nonlinear site-specific response analysis, the software DEEPSOIL V7.0 (Hashash et al. 2017), which is widely used in both academic and practical applications, has been utilized. This software is particularly developed for 1D soil response analysis and possesses the capability to conduct linear, equivalent linear (EL), and nonlinear (NL) analyses. The process of executing a nonlinear analysis with DEEPSOIL comprises six steps as follows:

1. Analysis type definition
2. Soil type definition
3. Input motion selection
4. Viscous/small-strain damping definition
5. Analysis control definition
6. Results

Each of these steps above is further elaborated in the following sections.

3.4.1 Analysis type definition

The nonlinear analysis method was chosen, using total stress instead of effective stress. The preference for total stress over effective stress was made to be consistent with previous studies (e.g. Idriss 1999), and based on previous studies that have shown that total stress analysis yield more accurate surface motion predictions than effective stress analysis, even when a liquefiable layer is present (Bessette and Yniesta 2021). The final stage in this step involves the selection of the constitutive soil model. The ARCS model (Yniesta et al. 2017) has been selected for this study due to its precise performance to fit the input MRD curves obtained through Darendeli (2001) empirical relationships and adjusted by Yee et al. (2013) model, as explained before in section 3.3.5.2. Since the ARCS model is not a default soil model in DEEPSOIL, it has been implemented into the software as a user-defined model in dynamic link library (*dll*) format.

3.4.2 Soil type definition

For the second step, it is necessary to specify the soil layers, including their properties, the bedrock characteristics, and the water table level for each profile. Concerning the properties of soil layers and the water table level, all the required data for each layer are provided in the Excel file generated from the modified Zhang's (2021) code. These data can be directly employed in the "Advanced table view" section. As an example, the layer properties for one of the profiles (i.e., profile 2) are presented in Tables 7, 8, 9, and 10 below. Table 7 indicates the layer at which the water table is positioned, the layer number, thickness, unit weight, shear wave velocity, and minimum damping for each layer. Tables 8 and 9, on the other hand, consist of 15 data points for the modulus reduction (MR1-MR15) and damping (D1-D15) curves, corresponding to the 15 shear strain points (g1-g15) as outlined in Table 10.

Table 7 Layers' properties of soil profile no. 2

Water table at top of layer	Layer number	Thickness(m)	Unit weight(kN/m ³)	V _s (m/s)	D _{min} (%)
...	...	...	...	...	...
9	58	0.375	17.5	158	0.75
9	59	0.375	17.5	168	0.75
9	60	0.375	17.5	176	0.75
...	...	...	...	...	...

Table 8 Layers' MR curve values of soil profile no. 2

MR1	MR2	MR3	MR4	MR5	MR6	MR7	MR8	MR9	MR10	MR11	MR12	MR13	MR14	MR15
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
0.995	0.991	0.981	0.960	0.919	0.842	0.719	0.625	0.554	0.469	0.357	0.232	0.130	0.064	0.030
0.995	0.991	0.981	0.960	0.919	0.843	0.716	0.590	0.507	0.424	0.320	0.208	0.116	0.058	0.027
0.995	0.991	0.981	0.960	0.920	0.844	0.717	0.571	0.478	0.394	0.296	0.192	0.107	0.053	0.025
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

Table 9 Layers' Damping curve values of soil profile no. 2

D1 (%)	D2 (%)	D3 (%)	D4 (%)	D5 (%)	D6 (%)	D7 (%)	D8 (%)	D9 (%)	D10 (%)	D11 (%)	D12 (%)	D13 (%)	D14 (%)	D15 (%)
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
0.035	0.080	0.181	0.408	0.902	1.932	3.882	7.018	11.015	14.936	17.934	19.674	20.247	19.962	19.159
0.035	0.079	0.180	0.406	0.898	1.923	3.867	6.996	10.99	14.915	17.920	19.668	20.247	19.966	19.165
0.035	0.079	0.179	0.404	0.894	1.915	3.852	6.975	10.96	14.894	17.906	19.661	20.246	19.969	19.170
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

Table 10 Layers' shear strain values of soil profile no. 2

g1 (%)	g2 (%)	g3 (%)	g4 (%)	g5 (%)	g6 (%)	g7 (%)	g8 (%)	g9 (%)	g10 (%)	g11 (%)	g12 (%)	g13 (%)	g14 (%)	g15 (%)
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
0.0001	0.0002	0.0005	0.001	0.003	0.006	0.014	0.03	0.07	0.16	0.37	0.85	1.93	4.4	10
0.0001	0.0002	0.0005	0.001	0.003	0.006	0.014	0.03	0.07	0.16	0.37	0.85	1.93	4.4	10
0.0001	0.0002	0.0005	0.001	0.003	0.006	0.014	0.03	0.07	0.16	0.32	0.85	1.93	4.4	10
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

Regarding the bedrock properties, a uniform elastic Halfspace bedrock is considered for all of the soil profiles. The characteristics of the bedrock are as follows:

1. Shear wave velocity: 1300 m/s (based on current practices at WSP Inc. in Western Quebec)
2. Unit weight: 25 kN/m³ (based on Motazedian et al. 2010)
3. Damping ratio: 2% (based on Hashash et al. 2017)

3.4.3 Input motion selection

For the third step, the 169 ground motions selected from the PEER Ground Motion Database (Ancheta et al. 2014, Goulet et al. 2021) are chosen as input motion. It is noteworthy to mention that the downloaded time histories from both NGA-East and NGA-West2 databases are in “AT2” format, and in order to be interpretable by DEEPSOIL, they have to first be converted to “txt” format.

3.4.4 Viscous/small-strain damping definition

For the fourth step, the frequency independent method by Phillips and Hashash (2009) was selected over Rayleigh damping model by Park and Hashash (2008) for the “Damping matrix type” in order to capture the viscous damping at small strains. Using frequency independent model in collaboration with the ARCS model will mitigate the potential numerical fluctuation through the analyses. It is worth noting that the damping matrix has not been updated at each time step in this study because updating damping with time will significantly increase the run time for each profile, while providing little improvement in terms of accuracy.

3.4.5 Analysis control definition

For the final step, some criterion has to be determined to solve the equation of motion. As suggested by DEEPSOIL, for "Step control", the flexible strain increment option is selected with a maximum strain increment set at 0.005%. The next choice pertains to the "Time integration scheme", where the implicit Newmark Beta method has been chosen. Lastly, for the "Time-history interpolation method", the linear interpolation within the time domain has been chosen.

3.4.6 Results

In the end, the results of a nonlinear analysis performed on the given soil profile are generated as separate files per each utilized ground motion. These files are formatted in "db3", which is an SQLite database file. To access these files and interpret their data for subsequent analyses, the software SQLiteStudio has been employed. It is worth emphasizing that DEEPSOIL allows the retrieval of results in either Text or Excel file formats. However, using these formats remarkably increases the runtime of further computations.

As an example the results of the nonlinear site response analysis conducted on soil profile #2 using ground motion RSN 1-H1 are presented below. These include the PGA profile, maximum shear stress profile, response spectrum at the surface, and Fourier amplitude spectrum at the surface.

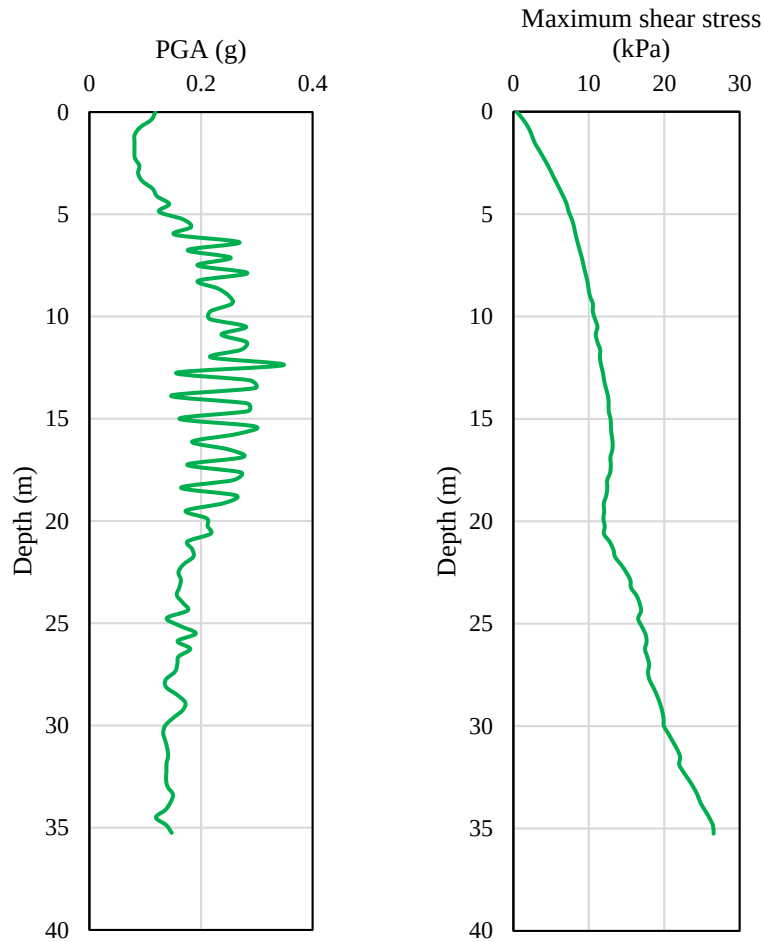


Figure 42 PGA and maximum shear stress profile of soil profile #2 using ground motion RSN 1-H1

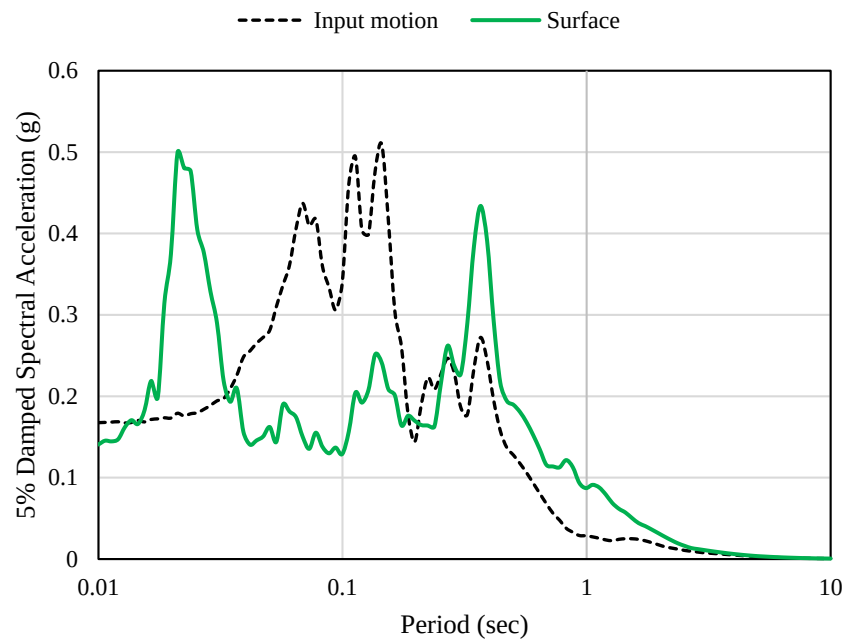


Figure 43 S_a at the surface for soil profile #2 and ground motion RSN 1-H1 in contrast with original input motion

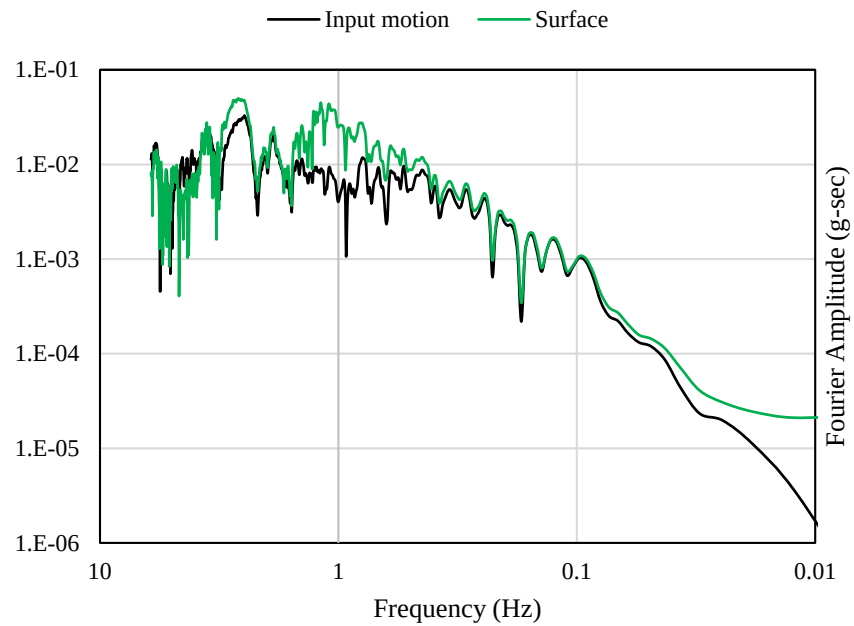


Figure 44 FAS at the surface for soil profile #2 and ground motion RSN 1-H1 in contrast with original input motion

CHAPTER 4 DERIVATION OF A NEW REDUCTION FACTOR

Upon the completion of the nonlinear site response analysis carried out with DEEPSOIL, it is possible to calculate the nonlinear shear stress reduction factor (r_d) for each soil profile. As described in section 2.1.3.1.1, utilizing the simplified method by Seed and Idriss (1971), r_d has been defined as the ratio between the maximum shear stress for a deformable solid and the maximum shear stress for a rigid body (i.e., $r_d = (\tau_{max})_d / (\tau_{max})_r$). The procedure for determining $(\tau_{max})_r$, $(\tau_{max})_d$, and r_d is presented in the subsequent sections.

4.1 Maximum shear stress under rigid conditions ($\tau_{max})_r$

To calculate the maximum shear stress at any given depth for a rigid soil column according to section 2.1.3.1.1 (i.e., $(\tau_{max})_r = \sigma_v \frac{PGA}{g}$), two key variables must be defined initially: 1) the total stress at that specific depth (σ_v), and 2) the Peak Ground Acceleration (PGA) at the surface. The latter can be extracted from the results of a nonlinear site response analysis conducted using DEEPSOIL. The total stress is computed based on the unit weight of the layer and the layers above, the thickness of the layers, and the position of the water table.

4.2 Maximum shear stress under deformable condition ($\tau_{max})_d$

The maximum shear stress under realistic deformable conditions can also be extracted from nonlinear site response analysis results. DEEPSOIL provides the ratio between effective stress and maximum shear stress (i.e., $\frac{\sigma'_v}{\tau_{max}}$) for each layer in the “MAX_STRESS_RATIO” column within the “PROFILES” table of results. The shear stress within this ratio represents the actual shear stress induced in the soil due to the propagation of seismic waves through a realistic deformable soil profile. By multiplying this ratio with the values in the “EFFECTIVE_STRESS” column within the same table, $(\tau_{max})_d$ will be obtained.

4.3 Nonlinear shear stress reduction factor (r_d)

Ultimately, once $(\tau_{max})_r$ and $(\tau_{max})_d$ are determined for a particular layer, r_d is computed for the corresponding depth of that layer. This procedure must be repeated for all layers to derive the r_d values through the entire soil profile. It is crucial to emphasize that DEEPSOIL generates the results

of nonlinear analysis for each ground motion individually. Therefore, to obtain all the r_d values for a specific soil profile, this procedure must be iterated once for each specific ground motion.

4.4 Reduction factors (r_d) for each soil profile

Mean values of r_d along with one standard deviation computed based on the results of the nonlinear site response analyses for each soil profile using the whole set of ground motions selected from both the NGA-East and NGA-West2 data sets are plotted in Figure 35 as a function of depth. They are plotted along r_d obtained by three existing models which were described in section 2.1.3.1.2: i.e., Youd et al. (2001), Idriss (1999), and Cetin et al. (2004). Note that each plot contains 169 curves (i.e., gray lines), each of which corresponds to a distinct r_d profile derived from a specific ground motion.

As illustrated in Figure 45 below, the existing models consistently yield relatively conservative r_d values when compared to the actual values obtained from nonlinear ground response analysis. Note that a higher reduction factor is associated with a higher shear stress, thus a lower factor of safety. Additionally, it is noteworthy that, although in some cases, for instance the soil profile 11, Cetin et al. (2004) shows more accurate estimation compared to other models, for the majority of the profiles, the r_d profiles derived from Idriss (1999) exhibit the closest fit to the mean curve in comparison to other models. In contrast, Youd et al. (2001) consistently produces the least accurate match to the actual mean curve. It is worth mentioning that since Youd et al. (2001) solely takes into account reduction factor as a function of depth and similarly, Idriss (1999) only considers reduction factor as a function of depth and magnitude, consequently they would have uniform trends across all fourteen profiles. It is critical to state that the functionality of the existing models used in this study is limited to depths of approximately 20 m, which results in less accurate fits for r_d values beyond this depth when compared to the actual r_d values. For instance, as illustrated in Figure 1 for profile 12, the corresponding curve to Idriss (1999) tends to move backward when applied to depths exceeding 30m. This unrealistic behavior arises due to the limitation discussed earlier in existing models.

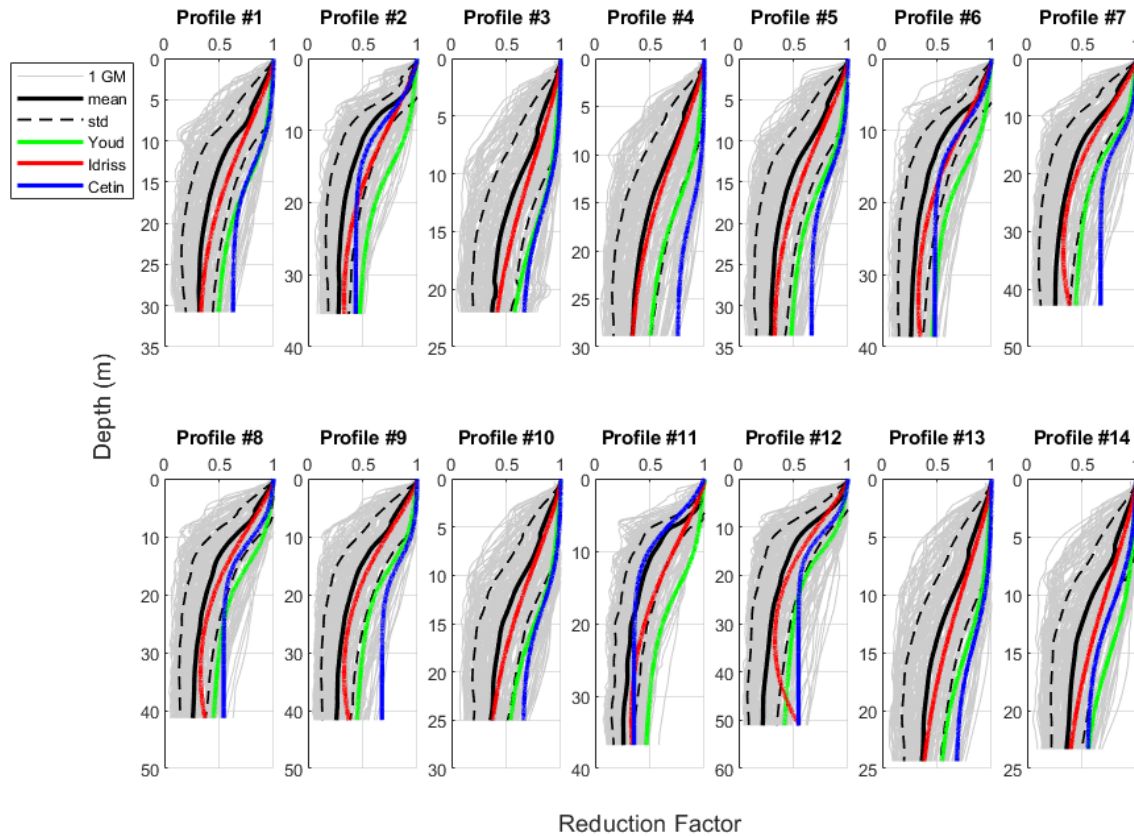


Figure 45 Reduction factor for each soil profile

4.5 Analysis of r_d for all fourteen soil profiles

To enhance the comprehensive presentation of the results of nonlinear site response analysis in this study, the mean value of computed r_d values along with one standard deviation for the entire set of fourteen soil profiles, have been aggregated together in Figure 46 as a function of depth in contrast with the reduction factor obtained from existing models. It is also worth mentioning that to unify the depth for all fourteen profiles, the depth is limited to 20 m in the illustration below, which is also consistent with the limitations of the existing models which are not suggested to be used at depths greater than 20 m.

As presented in Figure 46 below, similar to the individual profiles, the existing models exhibit more conservative trends when compared to the actual values obtained from nonlinear site response analysis. While all three models yield conservative estimates, Idriss (1999) follows the trend closest

to the mean value, whereas Youd et al. (2001) presents the most conservative and least accurate trend. Cetin et al. (2004) mostly provides values that lie between those of Idriss (1999) and Youd et al. (2001), though closer to Youd et al. (2001) overall.

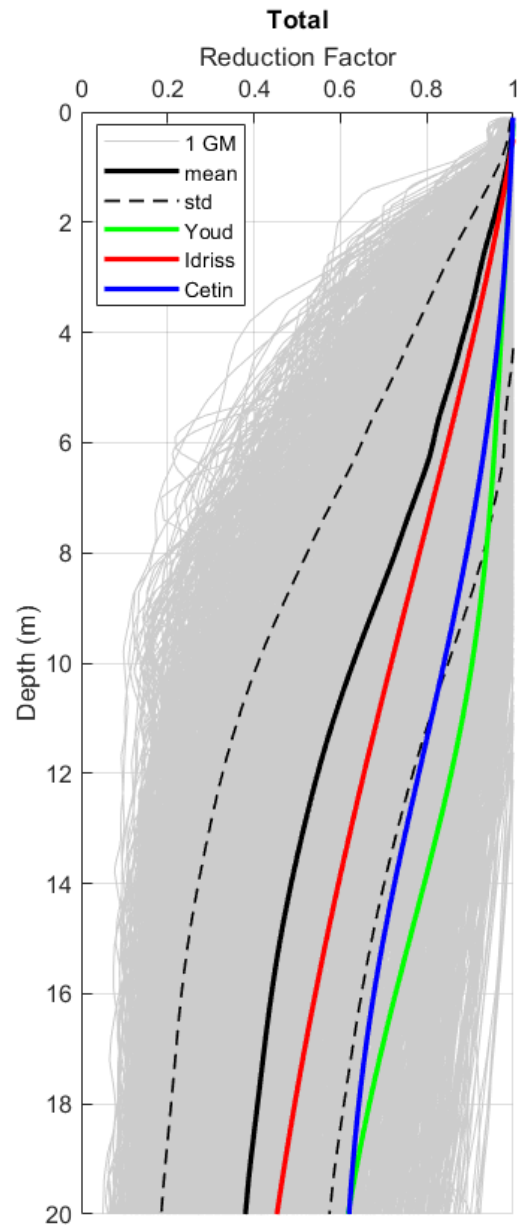


Figure 46 Variation of reduction factor with depth for all of the fourteen soil profiles

4.6 Analysis of r_d profiles for Eastern and Western motions

The utilized ground motions for this project were sourced from the PEER Ground Motion Database, which includes data from both NGA-East (Goulet et al. 2021) and NGA-West2 (Ancheta et al. 2014) datasets. The ground motions from Eastern Canada typically have a higher frequency content. Hence, in order to better capture the effect of frequency, the r_d profiles have been separated into East and West. The East group comprises r_d values computed using ground motions from the NGA-East database, as listed in Table 3, while the West group comprises r_d values computed using ground motions from the NGA-West2 database, as listed in Table 4. The corresponding aggregated r_d profiles for East and West are illustrated in Figure 47 in contrast with the reduction factor obtained from existing models.

As illustrated in Figure 47, when considering plots for both the Eastern and Western motions, the calculated values of r_d from existing models still tend to be more conservative in comparison to the r_d values derived from NL site response analysis. The trend that best aligns with the data is attributed to Idriss (1999), while the Youd et al. (2001) model stands out as the most conservative one, on the other hand the Cetin et al. (2004) model falls in between. Moreover, for Western motions the r_d profiles derived from existing models display a closer agreement with the mean profile obtained through NL site response analysis, while for Eastern motions the discrepancy is greater. To illustrate this point, consider the r_d profile from Idriss (1999), which closely aligns with the mean trend for the Western motions, a level of alignment not mirrored for the Eastern motions. As previously mentioned, this marked differentiation between the East and West profiles is attributed to the fact that the existing models were developed based on ground motions originating from Western North America. Consequently, the results produced by these models demonstrate greater accuracy when using ground motions from the West instead of the East.

Another noteworthy aspect to highlight is the comparatively lower mean values of r_d profiles for the Eastern motions in comparison to those for the Western motions. For instance, at a depth of 10m, the mean value of the reduction factor for the Eastern motions is approximately 0.6, whereas it is around 0.8 for the Western motions. This trend persists across all depths. This observation underscores that, based on the results derived from NL site response analysis, the reduction factor obtained using ground motions from NGA-West2 is typically more conservative when compared to utilizing ground motions from NGA-East at a given depth.

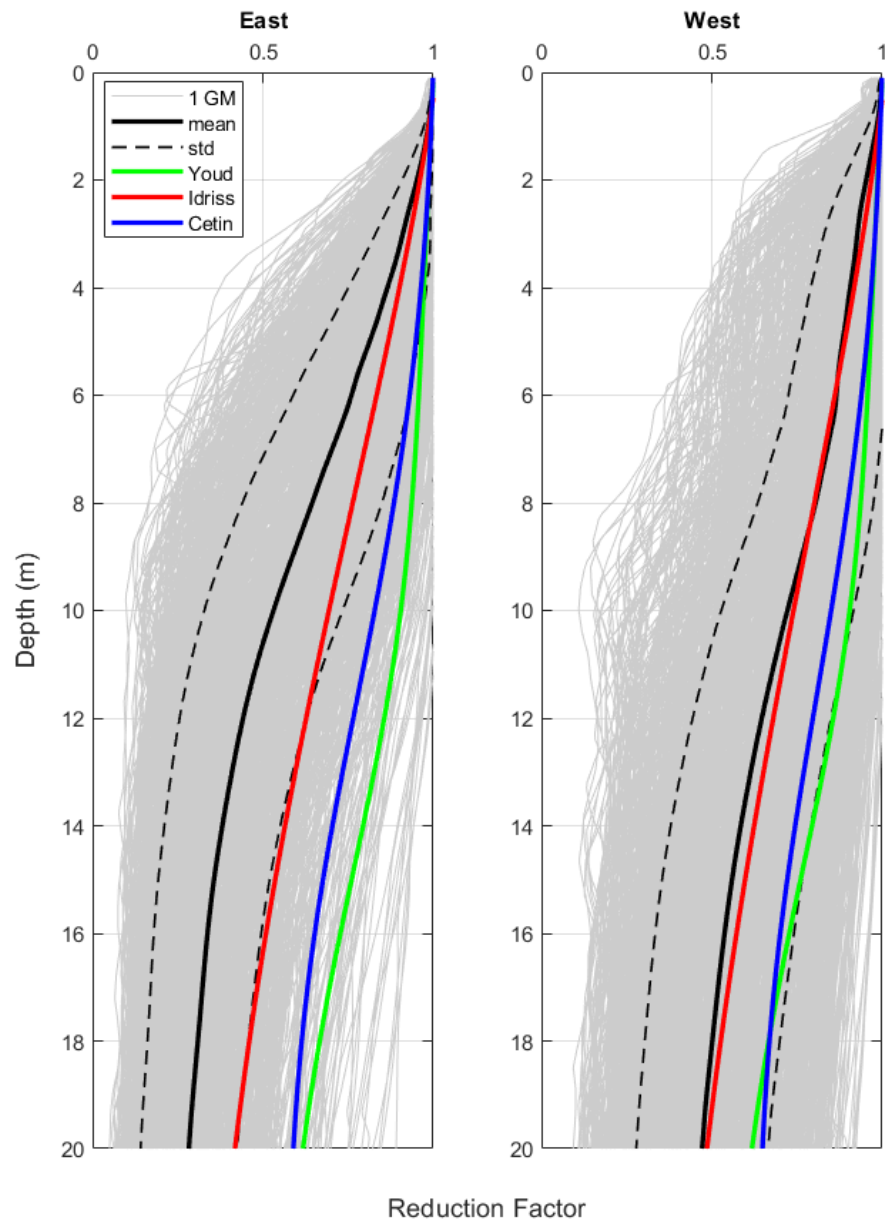


Figure 47 Variation of reduction factor with depth for all fourteen soil profiles separated based on region of utilized ground motions

4.7 Effect of magnitude on r_d

Golesorkhi (1989) studied the impact of several factors on r_d and established a significant dependency of r_d to earthquake magnitude. As a result, subsequent models (e.g., Idriss 1999 and Cetin et al. 2004) proposed relationships that accounted for earthquake magnitude as a key factor influencing the variation of r_d . In order to determine the relationship between r_d and magnitude, for all of the fourteen profiles, at four various depths of tailings (i.e., 0.5, 1, 2, and 4 meters below the interface between the superficial layer of waste rocks and the tailings) the variation of the reduction factor is plotted with the corresponding magnitude in Figures 48 and 49. It is important to note that, as in the previous section, to highlight the distinctions between the ground motions sourced from the NGA-East and NGA-West2 datasets, the data points have been separated into two distinct plots, respectively for Eastern and Western motions, thus further highlighting the effect of seismicity and frequency content.

As shown in Figures 48 and 49, a linear trend is fitted to the data points at each depth to provide a more accurate representation of the relationship between the reduction factor and magnitude. This linear fit for both Eastern and Western motions reveals an ascending relationship between reduction factors and magnitude, similar to the observation by Cetin et al. (2004).

Additionally, as anticipated, for both seismic conditions, the reduction factors at greater depths have lower mean values in comparison to those at shallower depths. Conversely, the linear fit of the reduction factor with magnitude at greater depths has a more significant gradient than shallower depths.

It is also worth mentioning that, by comparing the curve inclination between the two plots, it becomes apparent that the variation rate of reduction factor with magnitude for the Eastern motions is greater than for the Western motions.

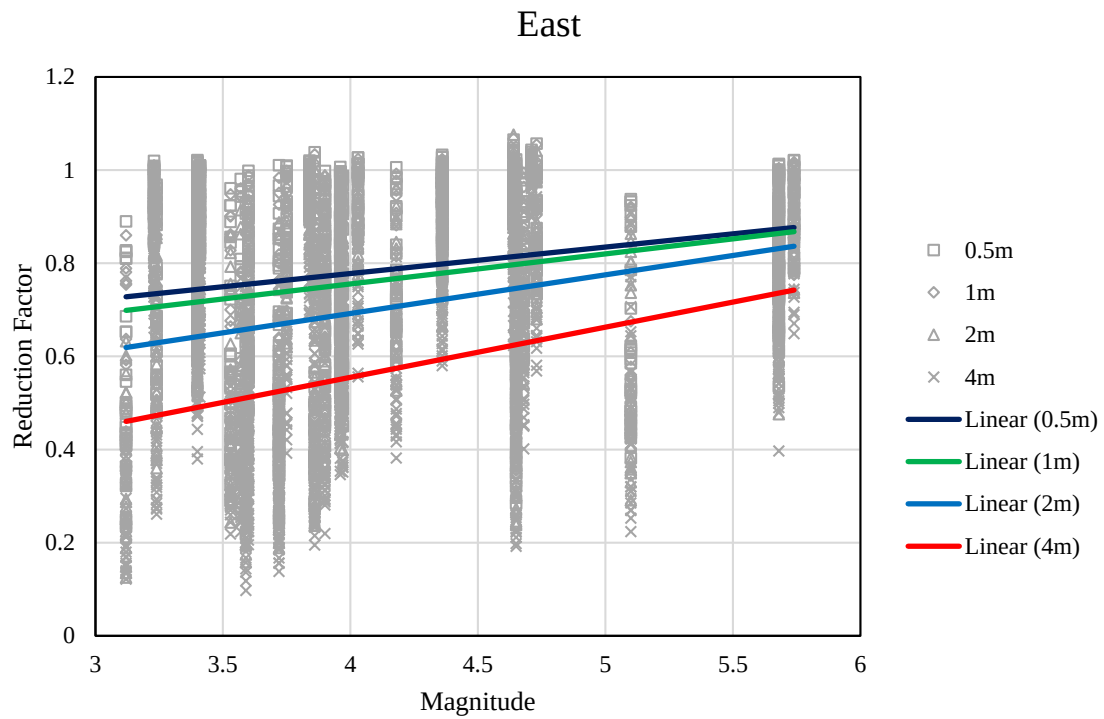


Figure 48 Variation of the reduction factors with moment magnitude for the Eastern motions

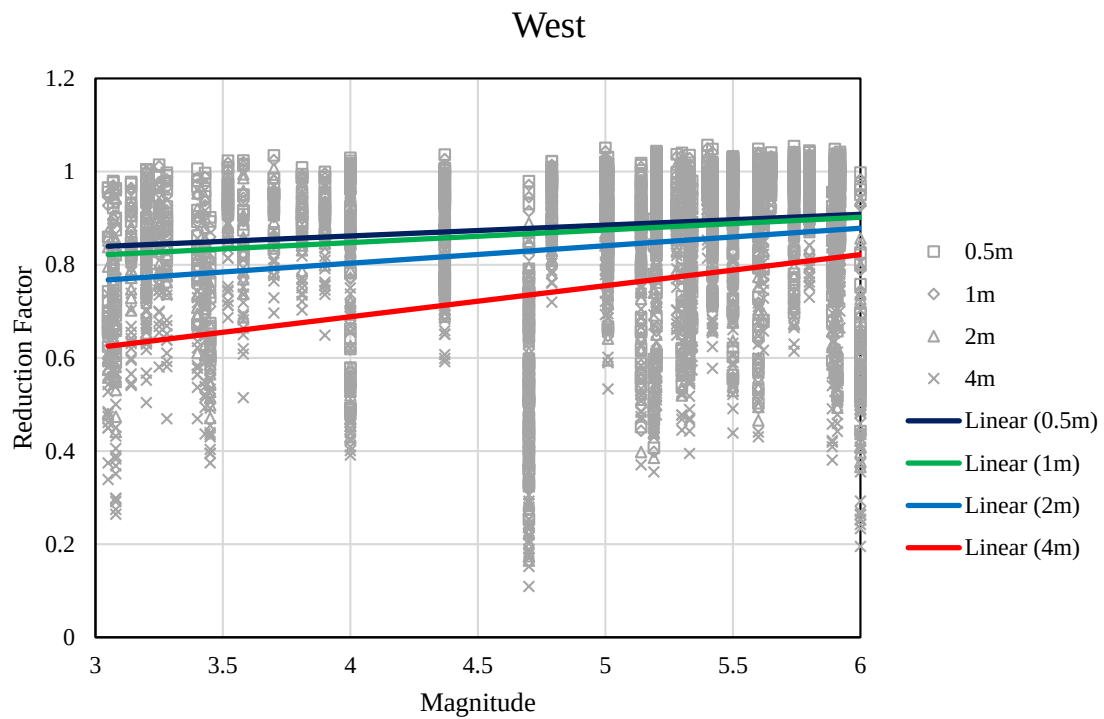


Figure 49 Variation of the reduction factors with moment magnitude for the Western motions

4.8 Effect of PGA on r_d

Cetin et al. (2004) stated that r_d depends on the same variables that can affect ground response analysis. They categorize these variables into two general distinct groups: 1) site condition properties and 2) ground motion characteristics. Additionally, they decided to solely consider moment magnitude and PGA among ground motion characteristics for simplicity. Similarly, in this study, the PGA was selected to study the effect of ground motion intensity. For all of the fourteen profiles at four specific depths within the tailings part of profiles (i.e., 0.5, 1, 2, and 4 meters), the reduction factor is plotted with the corresponding PGA in Figures 50 and 51. It is worth noting that the data points are divided into two plots, respectively corresponded to the Eastern and Western motions, to underscore the distinctions between the ground motions obtained from these two different regions.

Figures 50 and 51 are displayed as semi-logarithmic plots, therefore, a logarithmic trend is fitted to the relevant data points for each depth to describe the relationship between reduction factors and PGA. The logarithmic fit for both Eastern and Western motions, indicates a descending trend between reduction factor and PGA, similar to the suggested trend by Cetin et al. (2004). Furthermore, as seen in the previous section, reduction factors exhibit a descending trend with increasing depth. It is also noteworthy to mention that, unlike the relationship between reduction factor and magnitude, for both Eastern and Western motions the variation gradient of reduction factor with PGA, is greater at shallower depths in contrast to deeper ones.

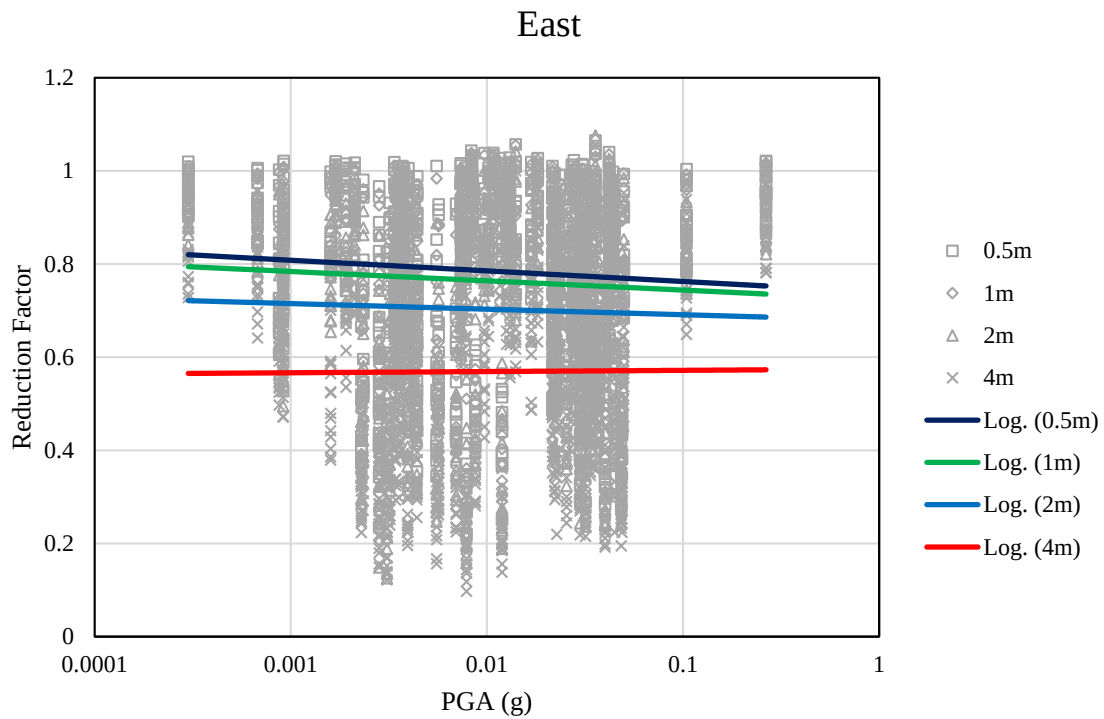


Figure 50 Variation of the reduction factor with PGA for the Eastern motions

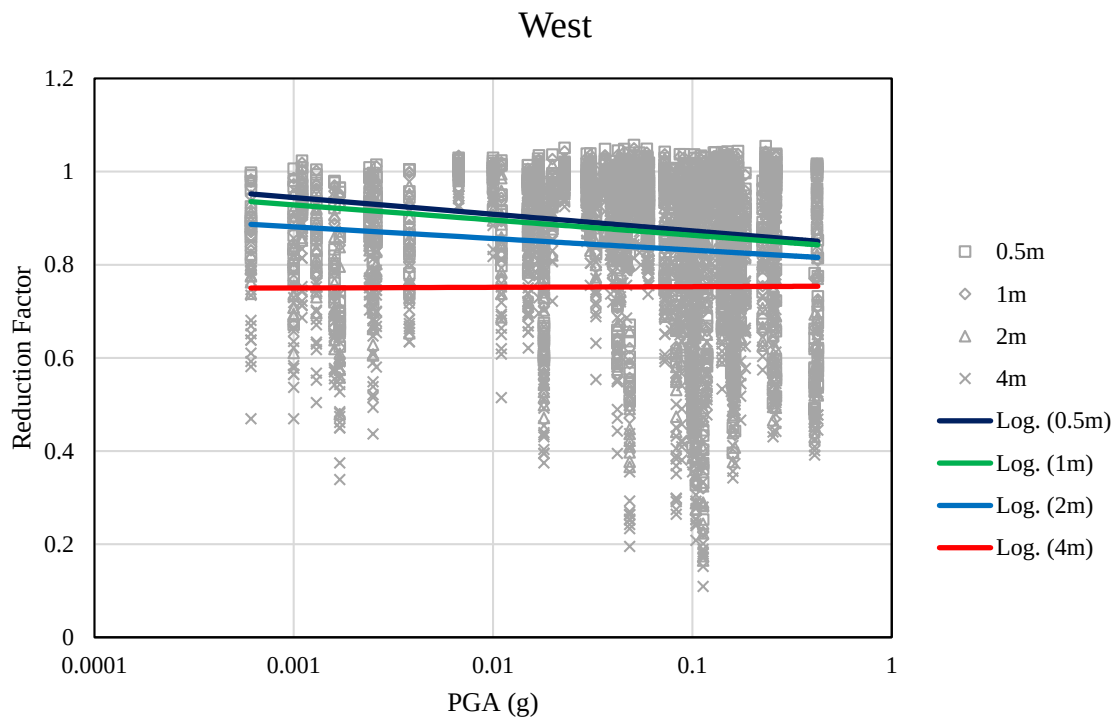


Figure 51 Variation of the reduction factor with PGA for the Western motions

4.9 Proposed equations for r_d

In this study, the results of the NL site response analysis are curve-fitted to derive a new r_d relationship by employing a nonlinear multivariate regression analysis. Given that Idriss (1999) and Cetin et al. (2004) models are commonly used in practice, the equations of these two models are considered for the regression analysis. Furthermore, it is worth mentioning that to capture dissimilarities in frequency content between Eastern and Western ground motions, each model is curve-fitted for Eastern and Western motions separately, leading to the derivation of two separate equations. The proposed r_d relationship by Idriss (1999) is presented as follows:

$$r_d = \exp(\alpha(z) + \beta(z)M) \quad \text{Equation 46}$$

$$\alpha(z) = a_1 + a_2 \sin\left(\frac{z}{a_3} + a_4\right) \quad \text{Equation 47}$$

$$\beta(z) = a_5 + a_6 \sin\left(\frac{z}{a_7} + a_8\right) \quad \text{Equation 48}$$

where z is the depth of the soil profile in meters, M is the moment magnitude of ground motion, $a_1, a_2, a_3, a_4, a_5, a_6, a_7$, and a_8 are fitting parameters. In addition the suggested equation by Cetin et al. (2004) for r_d is outlined as follows:

$$r_d = \frac{1 + \frac{A}{B}}{1 + \frac{A}{C}} \quad \text{Equation 49}$$

$$A = a_1 + a_2 PGA + a_3 M + a_4 V_{s12} \quad \text{Equation 50}$$

$$B = a_5 + a_6 e^D \quad \text{Equation 51}$$

$$C = a_5 + a_6 e^E \quad \text{Equation 52}$$

$$D = a_7(-z + a_8 V_{s12} + a_9) \quad \text{Equation 53}$$

$$E = a_7(a_8 V_{s12} + a_9) \quad \text{Equation 54}$$

where V_{s12} is the average shear wave velocity within the upper twelve meters of the profile, $a_1, a_2, a_3, a_4, a_5, a_6, a_7, a_8$, and a_9 are fitting parameters.

The equation proposed by Youd et al. (2001) (section 2.1.3.1.2), was excluded from consideration due to its limitations in capturing the relationship between r_d and earthquake magnitude and intensity.

Tables 11 and 12 showcase the newly determined fitting parameters resulting from regression analysis for Idriss (1999) and Cetin et al. (2004) equations alongside their original values, respectively.

Table 11 Fitting parameters for Idriss (1999)

	Idriss (1999) equation							
Fitting parameters	a_1	a_2	a_3	a_4	a_5	a_6	a_7	a_8
Original values	-1.012	-1.126	11.73	5.133	0.106	0.118	11.28	5.142
East	0.354	-0.397	7.349	11.609	0.083	-0.0003	-0.206	9.421
West	0.506	-0.524	5.862	4.831	0.050	0.057	5.181	4.722

Table 12 Fitting parameters for Cetin et al. (2004)

	Cetin et al. (2004) equation								
Fitting parameters	a_1	a_2	a_3	a_4	a_5	a_6	a_7	a_8	a_9
Original values	-23.013	2.949	0.999	0.0525	16.258	0.201	0.341	0.0785	7.586
East	-22.741	-11.107	2.419	0.006	15.780	5.427	0.295	0.008	9.182
West	-22.529	-11.707	2.311	0.015	14.684	7.0970	0.224	0.008	10.468

The updated curves of Idriss (1999) and Cetin et al. (2004) are presented in Figure 52 in comparison with the mean values of r_d , plus and minus one standard deviation, and the original curves of Idriss (1999) and Cetin et al. (2004).

As presented in Figure 52, for the Eastern motions both updated curves exhibit an excellent fit with the mean value curve, and since there is a noticeable disparity between the original model and mean value curves, the updated curves provide a more accurate estimate of the mean r_d . Similarly, for the Western motions, the updated curves also demonstrate a perfect fit with the mean value curve, mirroring the situation for the Eastern motions. Also, as mentioned before, the original curve by Idriss (1999) is capable of deriving a proper approximation of the mean r_d trend for the Western motions, therefore although there may be slight variations between the original Idriss (1999) curve and the mean r_d curve at certain depths, both the updated curves and the original Idriss (1999) curve effectively capture the mean r_d trend for the Western motions.

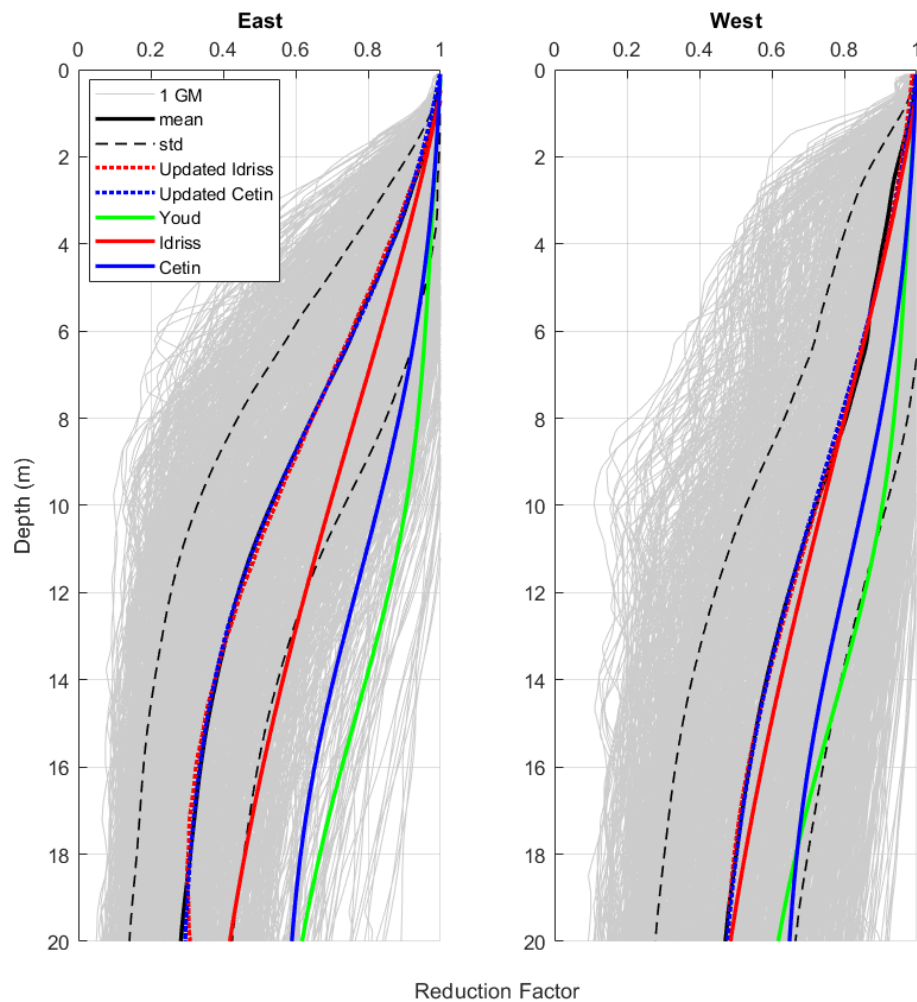


Figure 52 Updated curves of Idriss (1999) and Cetin et al. (2004) in comparison with the mean values of reduction factor and original curves of Youd et al. (2001), Idriss (1999), and Cetin et al. (2004)

4.10 Residual analysis

To assess the disparities between the r_d values obtained through NL site response analysis and those from the fitted curves, and potential biases induced by the empirical relationship, one approach is to use a residual analysis. Residuals can be calculated as follows:

$$\text{Residual} = \ln \left(\frac{r_d \text{ from NL site response analysis}}{r_d \text{ from fitted curve}} \right) \quad \text{Equation 55}$$

A positive residual signifies that the NL site response analysis has yielded a reduction factor greater than the r_d value obtained from the fitted curve, and the latter is under predicting the r_d . Conversely, a negative residual value would indicate that the fitted curve predicts a greater r_d value compared to the reduction factor derived from the NL site response analysis.

To evaluate any potential bias, residuals of the updated Idriss (1999) and the updated Cetin et al. (2004) curves for Eastern and Western motions are plotted against PGA at the surface, input PGA, PGV, moment magnitude, and spectral periods in the following sections. Data points are binned, and the mean value of residuals plus and minus with one standard deviation in each bin is plotted.

4.10.1 Residual against PGA at the surface

As demonstrated in Figure 53, for the Eastern motions, the updated curves do not display any significant bias across the entire spectrum of surface PGA values. In the upper and lower ranges, where there is a limited number of data points, both models yield reduction factors that are less conservative in comparison to the actual values obtained from NL site response analysis. Due to the limited dataset at high and low PGA, the trends are deemed acceptable.

Conversely, as presented in Figure 54 for the Western motions the updated curves exhibit a consistently, unbiased trend across the entire spectrum of surface PGA values, surpassing the performance observed for the Eastern motions. However, it is important to note that the updated Cetin et al. (2004) curve tends to yield slightly less conservative reduction factors when compared to the actual values obtained from NL site response analysis for the Western motions, especially at high PGA. Therefore, for the Western motions, utilizing the updated Idriss (1999) curve holds a slight advantage over the updated Cetin et al. (2004) curve, especially dealing with high values of surface PGA.

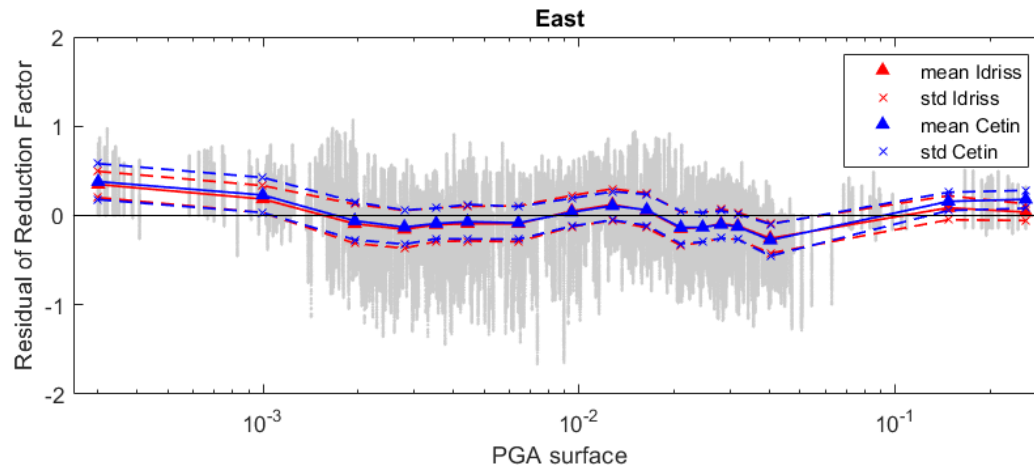


Figure 53 Residual of reduction factor with respect to PGA at the surface for the updated Idriss and cetin curves for the Eastern motions

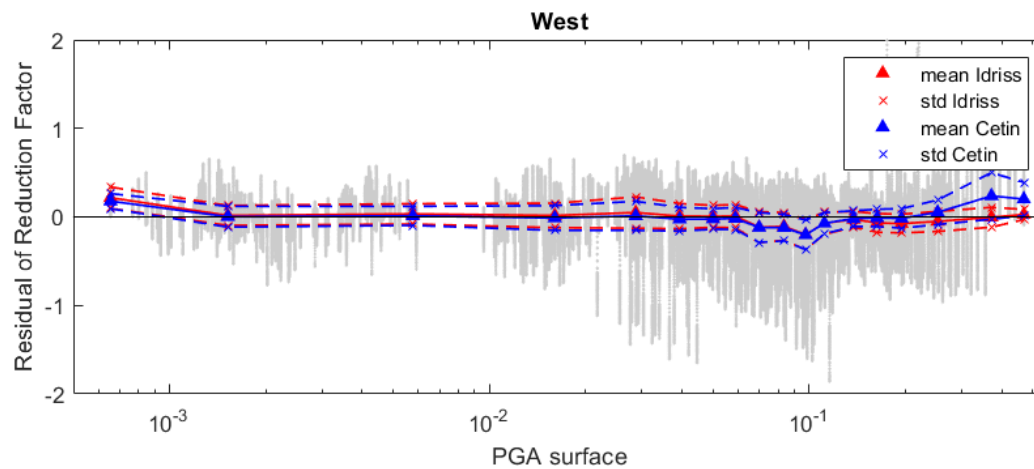


Figure 54 Residual of reduction factor with respect to PGA at the surface for the updated Idriss and Cetin curves for the Western motions

4.10.2 Residual against input PGA

As shown in Figures 55 and 56, no substantial bias is observed in any of the updated curves when plotting the residuals against surface PGA. Nevertheless, for both Eastern and Western motions, for lower values of input PGA, both updated curves tend to yield slightly less conservative reduction factors when compared to the actual values obtained from nonlinear site response analysis, similarly to what was discussed for the surface PGA. On the other hand, for the Western motions, for higher values of input PGA, both updated curves exhibit relatively more conservative reduction factors in contrast to the reduction factors derived from nonlinear site response analysis.

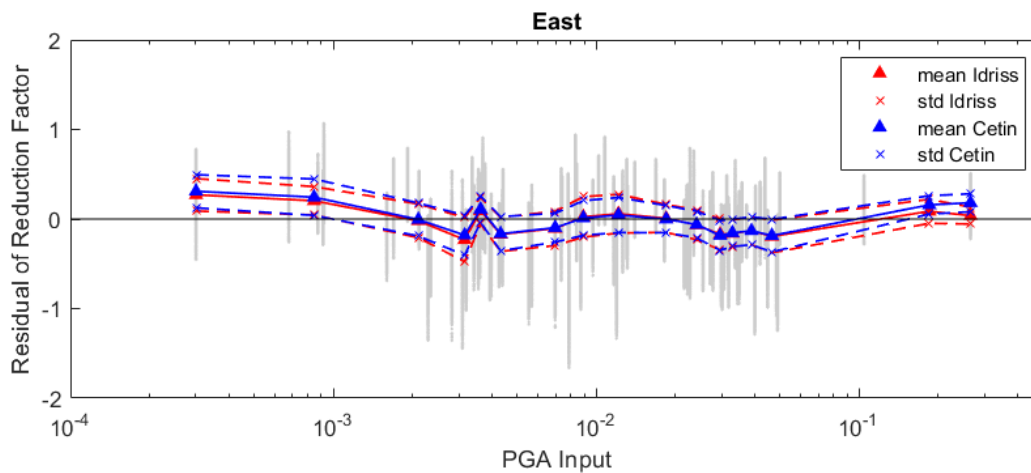


Figure 55 Residual of reduction factor with respect to input PGA for the updated Idriss and Cetin curves for the Eastern motions

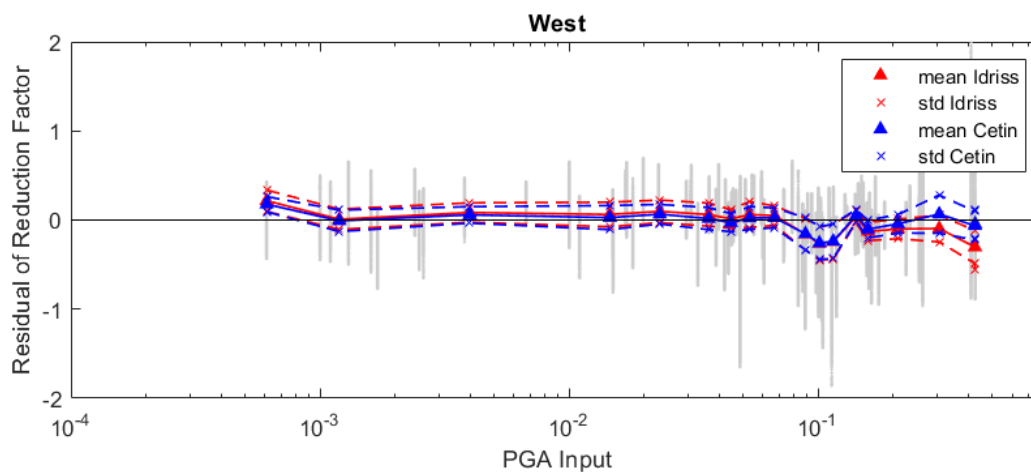


Figure 56 Residual of reduction factor with respect to input PGA for the updated Idriss and Cetin curves for the Western motions

4.10.3 Residual against PGV

Based on Figures 57 and 58, the updated curves exhibit no bias for either Eastern or Western motions. However, for more intense motions the same comments can be made, as the updated Cetin curve demonstrates is slightly less conservative when compared to the nonlinear site response analysis results for both Eastern and Western motions. In contrast, for the lower range of PGV for both the Eastern and Western motions, both updated curves yield less conservative reduction factors.

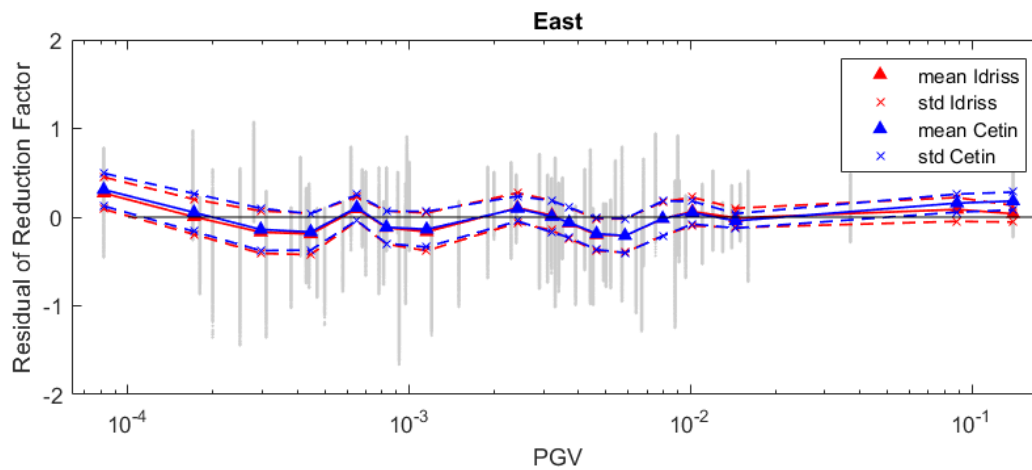


Figure 57 Residual of reduction factor with respect to PGV for the updated Idriss and Cetin curves for the Eastern motions

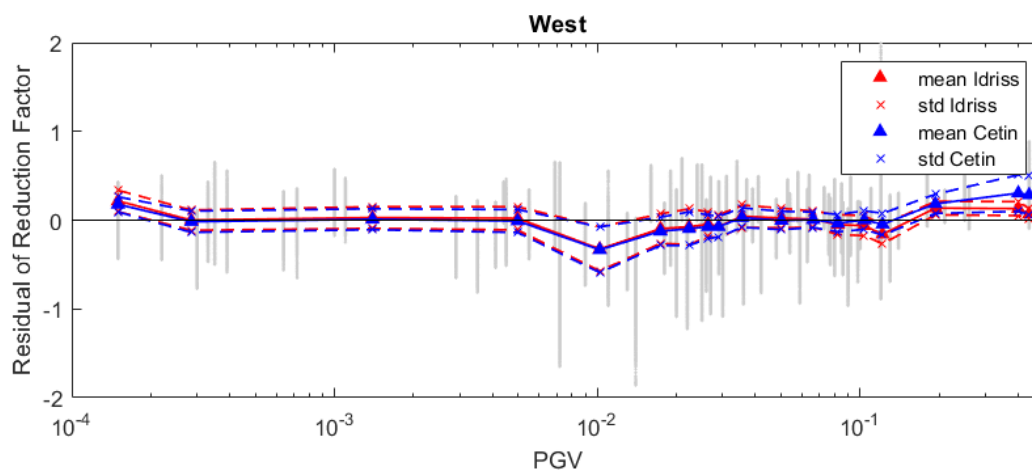


Figure 58 Residual of reduction factor with respect to PGV for the updated Idriss and Cetin curves for the Western motions

4.10.4 Residual against magnitude

As illustrated in Figures 59 and 60, no significant bias with magnitude was observed for both models under both seismic conditions. However, the residuals show significant fluctuations around the zero line for the Eastern motions compared to Western motions, indicating a more accurate model for the Western motions across the entire range of magnitudes.

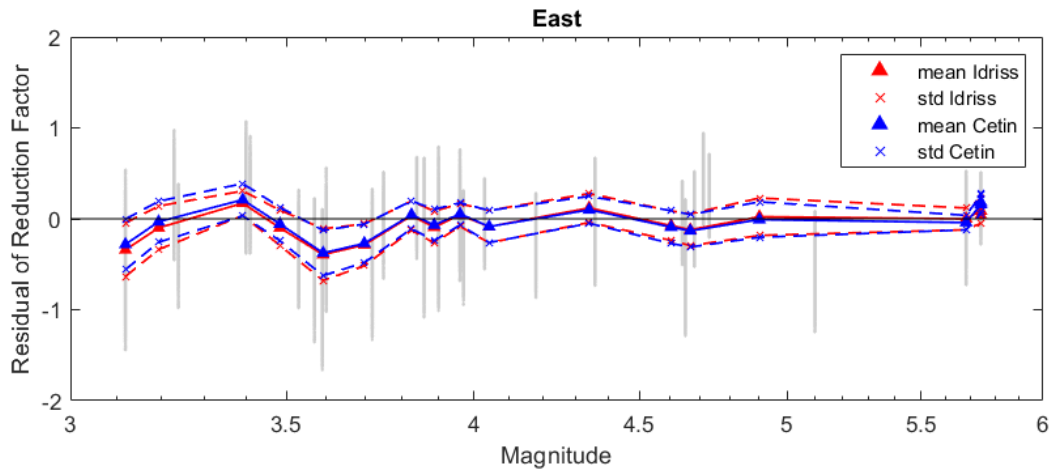


Figure 59 Residual of reduction factor with respect to magnitude for the updated Idriss and Cetin curves for the Eastern motions

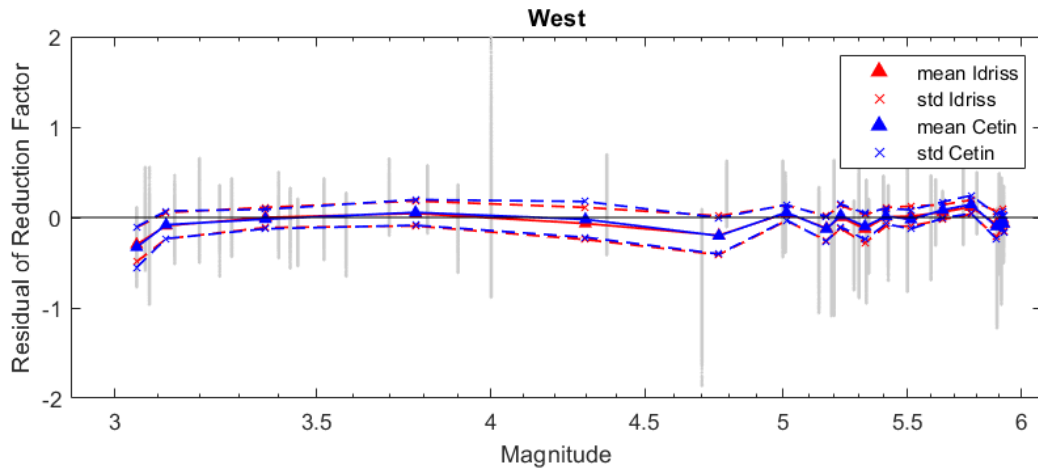


Figure 60 Residual of reduction factor with respect to magnitude for the updated Idriss and Cetin curves for the Western motions

4.10.5 Residual against S_a at 0.1 s

As presented in Figures 61 and 62, for the Eastern motions, both updated curves demonstrate unbiased behavior for midrange values of spectral acceleration at 0.1 s. For both low and high ranges of S_a at 0.1s, they generate relatively less conservative reduction factors compared to values obtained from nonlinear site response analysis. Conversely, for the Western motions, no significant bias is observed, and both updated curves align perfectly with the actual values of reduction factors. However, for the high range of S_a , the updated curves yield more conservative values when compared to the results from nonlinear site response analysis.

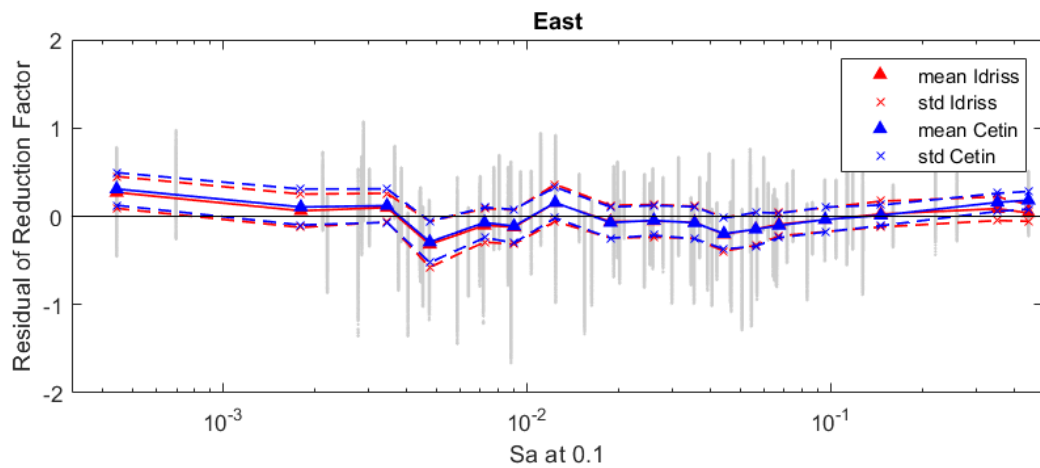


Figure 61 Residual of reduction factor with respect to S_a at 0.1s for the updated Idriss and Cetin curves for the Eastern motions

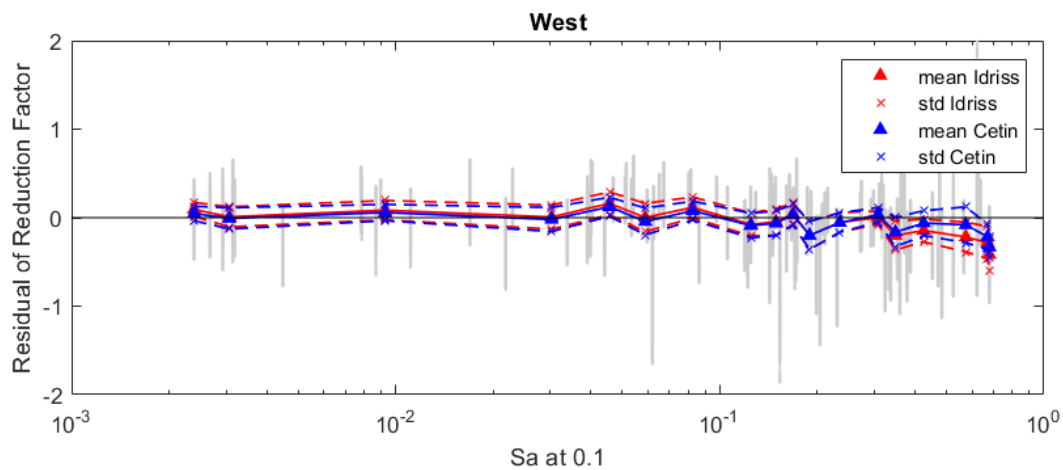


Figure 62 Residual of reduction factor with respect to S_a at 0.1s for the updated Idriss and Cetin curves for the Western motions

4.10.6 Residual against S_a at 1 s

As shown in Figures 63 and 64, overall no significant bias is observed in both seismic conditions. However, for Eastern motions, a slightly upward trend is showing indicating a potential bias with S_a at 1s. However, since the frequency content of the motions is likely to be weak at such spectral periods for Eastern motions, this bias is unlikely to affect the liquefaction analysis. For the Western motions, for large and small values of spectral acceleration at 1 second, both updated curves generate slightly more and less conservative reduction factors, respectively, comparing to those obtained from NL site response analysis.

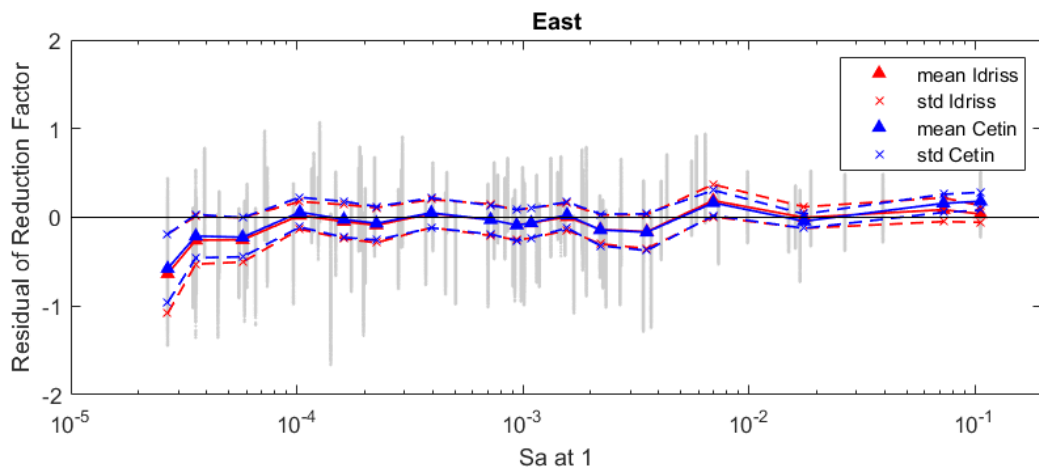


Figure 63 Residual of reduction factor with respect to S_a at 1s for the updated Idriss and Cetin curves for the Eastern motions

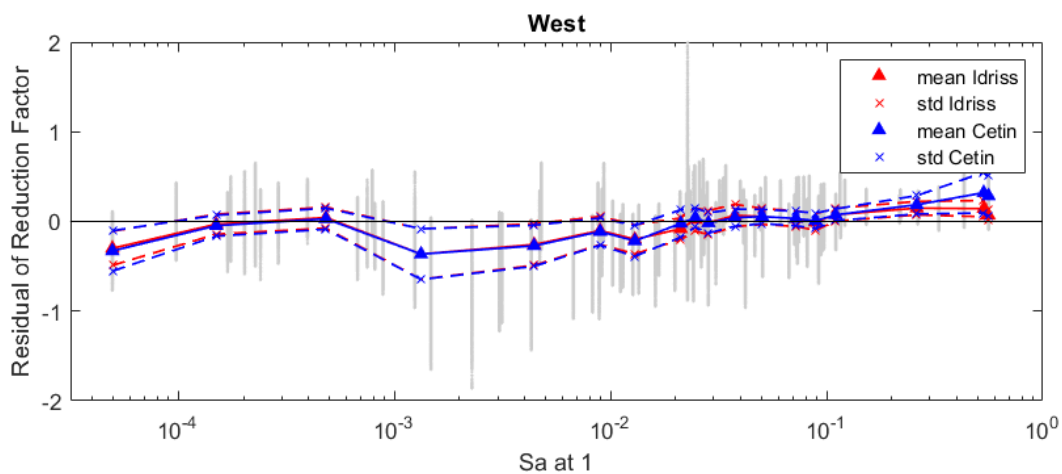


Figure 64 Residual of reduction factor with respect to S_a at 1s for the updated Idriss and Cetin curves for the Western motions

CHAPTER 5 CONCLUSIONS

5.1 Summary

The simplified method originally developed by Seed and Idriss (1971) is the most widely used in practice for assessing the probability of liquefaction triggering. One of the critical steps in this procedure is to determine the seismic demand which is equal to the cyclic shear stress induced in the soil profile by an earthquake divided by the confining pressure. The cyclic shear stress can be estimated using site-specific ground response analysis, or by computing the shear stress based on the surface acceleration. Given that the soil behaves as a deformable body rather than a rigid one, this simplified method introduces a non-linear shear stress reduction factor to describe the non-linear reduction in shear stress with depth. Numerous studies have proposed empirical relationships for this reduction factor for seismically active regions, by using the results of Equivalent Linear ground response analysis using different ground motions and soil profiles. With the liquefaction failure of tailings impoundments emerging as a prominent concern in the mining industry, the present study aimed to develop a reduction factor relationship to be used to assess the potential of liquefaction triggering in tailings impoundments for mining-induced earthquakes.

A set of nonlinear site response analyses have been conducted on 14 anonymous tailings profiles from a gold mine located in Western Quebec provided by WSP Inc., in order to derive a new nonlinear shear stress reduction factor for tailings impoundments. The shear wave velocity profiles were processed using a MATLAB code developed by Zhang at Polytechnique Montreal in 2021, in order to obtain the required soil properties for a nonlinear site response analysis. 169 unscaled ground motions were selected from the PEER database from both NGA-East and NGA-West2. All these ground motions were picked among time histories with relatively low magnitudes and close distances to be similar to induced seismic conditions. As a result, 2,366 site response analyses were executed using DEEPSOIL software in order to determine the variation of induced shear stress across each profile under each ground motion and the PGA at the surface.

From the results of the DEEPSOIL analysis within the simplified method framework, the nonlinear shear stress reduction factor was computed across each soil profile. Then, the computed reduction factor aggregated for whole sets of profiles and plotted with respect to depth alongside reduction factors computed by existing models such as Idriss (1999), Youd et al. (2001), and Cetin et al. (2004). Afterward, nonlinear multivariate regression analyses were performed in order to establish

a relationship for the reduction factor by considering the same formulation as Idriss (1999) and Cetin et al. (2004) models with adjustable fitting parameters. As a result of these analyses, four distinct sets of fitting parameters were obtained, each corresponding to one model (i.e., Idriss 1999 or Cetin et al. 2004) and different seismic regions, i.e., stable continental and seismically active. A set of residual analyses were conducted to investigate any potential biases. The study's results come with limitations as they are based on empirical relationships and data collected solely from a single gold mine. However, the findings can still be valuable in the context of liquefaction risks faced by tailings impoundments due to mining-induced earthquakes.

5.2 Conclusions

This study showed that the existing models tend to produce excessively conservative results when applied to more stable regions such as Eastern Canada. The same trend was observed for the motions from a seismically active region, but with the Idriss model being only slightly conservative. The new relationships allowed for a more precise estimate and can be used for tailings impoundments subject to mining-induced seismicity in both stable and seismically active regions. The results of the residual analyses revealed that all four relationships provide a good fit, with no significant bias observed with respect to the surface and input PGA, PGV, and moment magnitude. The analysis showed that both the updated Idriss (1999) and Cetin et al. (2004) curves are equally suitable for use with Eastern ground motions. For Western ground motions, it was found that the updated Idriss (1999) curve exhibits a slightly superior fit when compared to the Cetin et al. (2004) curve.

5.3 Limitations

For this study, no laboratory testing results were available, necessitating the use of empirical relationships, assumptions based on fundamental material behavior, and data obtained from common mining practices within the WSP Inc. team or related literature to determine specific properties of the given tailings and waste rock for subsequent analysis. Nevertheless, conducting laboratory tests and deriving material specific properties such as modulus reduction and damping curves would lead to more precise NL site response analysis results.

Furthermore, it is important to note that this thesis conducted nonlinear analysis using total stress in DEEPSOIL and did not account for excess pore water pressure generation. This choice was

driven by uncertainties surrounding the reliability of effective stress models in ground response analysis software like DEEPSOIL.

5.4 Future Work

Since the tailings profiles assessed in this research were chosen from a single gold mine, it raises the potential for future research endeavors that undertake a similar nonlinear, site-specific response analysis on tailings profiles sourced from several mines. By comparing the corresponding reduction factor relationships for diverse tailings, it can be determined whether alterations in the source rock's metal composition and the mining process significantly impact the outcomes.

In addition, it may be beneficial for future research initiatives to undertake nonlinear analysis while incorporating effective stress. Consequently, a comparison of the obtained results with those detailed in this study can be conducted to ascertain whether there are any advantages in utilizing effective stress over total stress with existing models.

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