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**Site-Specific Seismic Nonlinear Ground Analysis on Sample Soil Profiles in
Eastern Canada**

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Mémoire

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

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RÉSUMÉ

Parmi les pratiques actuelles de l'industrie dans l'est du Canada, les facteurs d'amplification trouvés dans le Code national du bâtiment de 2015 (NBCC 2015) sont couramment utilisés pour déterminer l'amplification des ondes sismiques à travers un dépôt de sol mou en cas de tremblement de terre. Lorsqu'une analyse spécifique au site est requise, l'analyse équivalente linéaire est couramment utilisée. Cette dernière est une procédure itérative qui consiste à réaliser des simulations linéaires avec des propriétés de sols constantes, prises d'un ensemble de courbes de réduction du module de cisaillement et d'amortissement à un niveau de déformation donné, jusqu'à ce que le niveau de déformation obtenu dans l'analyse corresponde au niveau des données d'entrée. En revanche, l'analyse non-linéaire de la propagation des ondes sismiques est une approche plus robuste et plus fiable, capable de représenter les variations des propriétés dynamiques durant l'analyse, mais elle est présentement rarement utilisée dans l'est du Canada à cause sa complexité.

Ce projet de recherche vise dans un premier temps à créer un code MATLAB pour faciliter l'utilisation de DEEPSOIL, un logiciel informatique d'analyse non-linéaire bien établi dans la recherche, sur la base de données géotechniques. Selon des données disponibles de CPT, d'essais scissométriques et de vitesse des ondes de cisaillement, le code MATLAB génère automatiquement des tableaux de couches de sols avec leurs propriétés géotechniques de base et leurs propriétés dynamiques, telles que les courbes de module de réduction (corrigées pour la résistance au cisaillement) et d'amortissement. Ces tableaux peuvent ensuite être directement importés dans DEEPSOIL afin de réduire considérablement le temps nécessaire pour la définition des couches de sol.

Dans ce projet, à l'aide du code MATLAB, des analyses non-linéaires ont été efficacement effectuées pour 33 profils de sol fournis par WSP. À partir de ces analyses, des facteurs d'amplification (AF) du sol ont été calculés et comparés à ceux trouvés dans NBCC 2015. Les résultats des analyses sont classés dans trois cas différents, dépendamment des niveaux de similarité entre les facteurs d'amplification du code et des simulations.

Les résultats montrent que les analyses non-linéaires ont quelquefois donné des AF soit assez similaires, soit significativement inférieures aux AF du NBCC 2015. Cependant, pour la majorité des profils de sol analysés, les AF des analyses non-linéaires étaient significativement plus élevés

que ceux du NBCC 2015 autour de la période naturelle du site, où les AF des analyses non-linéaires forment un pic aigu qui n'est pas représenté dans le NBCC 2015. Ainsi, se fier uniquement au NBCC 2015 pour prédire l'amplification des ondes sismiques dans un dépôt de sol peut soit mener à des risques, car les AF du NBCC 2015 ne sont pas toujours sécuritaires à la période naturelle du site, ou bien mener à une conception inutilement trop sécuritaire.

ABSTRACT

Among current industry practices in Eastern Canada, amplification factors found in the National Building Code of Canada 2015 (NBCC 2015) are commonly used to determine the amplification of seismic waves in a soft soil deposit in the event of an earthquake. When a site-specific analysis is warranted, the equivalent linear ground response analysis has been commonly used. The latter is an iterative procedure that consists of performing linear analysis with constant dynamic soil properties, that are picked from a set of modulus reduction and damping curve at a given shear strain level, until the strains from the analysis are consistent with the input values. On the other hand, the nonlinear ground response analysis is a more robust and reliable approach, able to represent the variations of dynamic properties during the analysis, but it is currently seldomly used in Eastern Canada due to its complexity.

This research project first aims to create a MATLAB code to facilitate the use of DEEPSOIL, a well-established nonlinear analysis computer software, based on available geotechnical data. Based on available CPT, vane shear and shear wave velocity data, the MATLAB code automatically generates a table of soil layers with corresponding basic geotechnical properties and dynamic soil properties, such as the modulus reduction (corrected for the shear strength) and damping curves, which can then be directly imported into DEEPSOIL in order to greatly reduce the time required for the definition of soil layers.

In this project, with the help of the MATLAB code, nonlinear analysis were efficiently performed for 33 soil profiles provided by WSP. From these analyses, soil amplification factors (AF) were calculated and compared to those obtained from the NBCC 2015. The results of the analysis are classified in three different cases depending on the level of similarity between the AF from the NBCC 2015 and the nonlinear simulations.

The results show that the nonlinear analysis occasionally yielded AF either fairly similar to or significantly lower than AF from the NBCC 2015. However, for the majority of the soil profiles analysed, the AF from the nonlinear analysis were significantly higher than those from the NBCC 2015 around the natural site period, where the AF from the former present a sharp peak that is not represented in the latter. Thus, relying only on the NBCC 2015 to predict soil amplification of

seismic waves can either pose a design risk, since the AF from the NBCC 2015 may not always be conservative at the natural site period, or lead to unnecessary conservative design.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS	III
RÉSUMÉ.....	IV
ABSTRACT	VI
TABLE OF CONTENTS	VIII
LIST OF TABLES	XI
LIST OF FIGURES.....	XII
LIST OF SYMBOLS AND ABBREVIATIONS.....	XV
LIST OF APPENDICES	XVIII
CHAPTER 1 INTRODUCTION.....	1
1.1 Background on Nonlinear Ground Response Analysis.....	1
1.2 Objectives and Methodology	4
1.3 Organization of the Memoire	5
CHAPTER 2 LITERATURE REVIEW.....	7
2.1 Seismic Amplification Factors (AF) in Ground Response Analyses.....	7
2.2 Site-Specific Ground Response Analysis.....	7
2.2.1 Maximum Shear Modulus (G_{max}) and Shear Wave Velocity (V_s).....	8
2.2.2 Modulus Reduction and Damping (MRD) Curves	9
2.2.3 Linear Analysis	12
2.2.4 Equivalent Linear (EL) Analysis.....	14
2.2.5 Nonlinear (NL) Analysis	15
CHAPTER 3 METHODOLOGY OF SOIL PROFILING BASED ON FIELD DATA.....	25
3.1 Objectives of the Code	25
3.2 Logic Flow	25

3.3	Step-by-Step Description of the Code.....	27
3.3.1	Importation and Processing of Available Field Data	27
3.3.2	Definition of Soil Layers Basic Properties.....	28
3.3.3	Calibration of Estimated Shear Wave Velocities (<i>V_s</i>).....	32
3.3.4	Calibration of Estimated Undrained Shear Strength (<i>S_u</i>)	33
3.3.5	Grouping of Soil Layers for Analysis in Deepsoil	35
3.3.6	Computation of Soil Dynamic Properties	36
3.3.7	Exporting the Soil Layers' Properties to Deepsoil.....	37
3.4	An Example of Using the MATLAB Code.....	37
CHAPTER 4	DEEPSOIL SIMULATION RESULTS.....	40
4.1	Procedures of Using Deepsoil for Nonlinear (NL) Analyses.....	40
4.1.1	Analysis Type Definition	40
4.1.2	Soil Profile Definition	41
4.1.3	Input Motion Selection.....	43
4.1.4	Viscous/Small-Strain Damping Definition	46
4.1.5	Analysis Control Definition	47
4.1.6	Exporting Results	47
4.2	Analysis of Results.....	47
4.2.1	Summary of the Three Cases of Comparisons Found.....	48
4.2.2	Amplification Factors (<i>AF</i>) for Scenario 1	51
4.2.3	Amplification Factors (<i>AF</i>) for Scenario 2	52
4.2.4	Amplification Factors (<i>AF</i>) for Scenario 3	54
4.2.5	Residuals Analysis for Scenario 1.....	55
4.2.6	Residuals Analysis for Scenario 2.....	57

4.2.7 Residuals Analysis for Scenario 3.....	59
4.2.8 Correlations Between Residuals and Various Parameters	60
CHAPTER 5 CONCLUSION AND FUTURE WORKS	74
REFERENCES.....	76

LIST OF TABLES

Table 1 – V_s30 based soil classification (Finn et Wightman, 2003)	2
Table 2 – Soil classifications used by the Eurocode 8 (Pitilakis et al., 2012).....	3
Table 3 – Raw CPT data from soil profile #29	38
Table 4 – Output of the MATLAB code for soil profile #29 (part 1)	39
Table 5 – Output of the MATLAB code for soil profile #29 (part 2 – strain points, g)	39
Table 6 – Output of the MATLAB code for soil profile #29 (part 3 – modulus reduction, MR)..	39
Table 7 – Output of the MATLAB code for soil profile #29 (part 4 – damping, D)	40
Table 8 – Additional information on each soil profile analysed	42
Table 9 – Description of the 3 Scenarios of Input Motions Used in this Study	43
Table 10 – Input Seismic Motions Used in Scenario 1	45
Table 11 - Input Seismic Motions Used in Scenario 2.....	46
Table 12 – Input Seismic Motions Used in Scenario 3	46

LIST OF FIGURES

Figure 1 – Example of a Backbone Curve (Stewart et al, 2008).....	8
Figure 2 – Example of a set of Modulus Reduction and Damping Curves.....	9
Figure 3 – Definition of Damping (Yniesta et al., 2017)	10
Figure 4 – Amplification Peaks for Uniform Undamped Soil on Rigid Rock (Kramer, 1996)	13
Figure 5 – Amplification Peaks for Uniform Damped Soil on Rigid Rock (Kramer, 1996)	13
Figure 6 – Local Coordinate System for Layered Damped Soil on Elastic Rock.....	14
Figure 7 – One dimensional simplification of soil layer system (Stewart et al, 2008)	16
Figure 8 – Overestimation of damping by the Masing rules (Phillips and Hashash, 2009).....	18
Figure 9 – Improvement of the GQ/H model over the MKZ model (Groholski et al., 2016).....	20
Figure 10 – The MRDF formulation versus the Masing rules (Groholski et al., 2016).....	22
Figure 11 – Comparison between one, two and four modes Rayleigh formulation (Phillips and Hashash, 2009)	24
Figure 12 – Logic flow summary of the MATLAB code	26
Figure 13 – Example of curve-smoothing of q_c measurements	27
Figure 14 – Example of curve-smoothing of V_s measurements	28
Figure 15 – Example of segmentation.....	30
Figure 16 – Example of calibration of V_s measurements	33
Figure 17 – Correction factors of Bjerrum (1973)	34
Figure 18 – Calibration of Nkt	34
Figure 19 – Calibration of Su	35
Figure 20 – Distribution of V_{s30} for the soil profiles analysed (Zhang et al., 2020).....	41
Figure 21 – Input motions of a) Scenario 1, b) Scenario 2 and c) Scenario 3 compared to the target seismic hazard (Huo-Kang and Yniesta, 2020).....	44

Figure 22 – Amplification Factors from NL Analysis Versus NBCC 2015 – Case 1 (Scenario 1, soil profile #11)	49
Figure 23 - Amplification Factors of NL Analysis Versus NBCC 2015 – Case 1 (Scenario 1, soil profile #4).....	49
Figure 24 - Amplification Factors of NL Analysis Versus NBCC 2015 – Case 2 (Scenario 1, soil profile #15).....	50
Figure 25 - Amplification Factors of NL Analysis Versus NBCC 2015 – Case 3 (Scenario 1, soil profile #1).....	51
Figure 26 – Comparison of <i>AF</i> differences between NL analysis and NBCC 2015 at the natural site period between a) Scenario 1 and b) Scenario 2 – Case 2 (soil profile #1)	53
Figure 27 – Comparison of <i>AF</i> differences between NL analysis and NBCC 2015 at the natural site period between a) Scenario 1 and b) Scenario 2 – Case 3 (soil profile #1)	54
Figure 28 – Residuals of Amplification Factors for All Motions and All Soil Profiles (Scenario 1)	56
Figure 29 – Residuals of Amplification Factors for All Motions and All Soil Profiles Versus Normalized Periods (Scenario 1)	57
Figure 30 – Residuals of Amplification Factors for All Motions and All Soil Profiles (Scenario 2)	58
Figure 31 – Residuals of Amplification Factors for All Motions and All Soil Profiles Versus Normalized Periods (Scenario 2)	58
Figure 32 – Residuals of Amplification Factors for All Motions and All Soil Profiles (Scenario 3)	59
Figure 33 – Residuals of Amplification Factors for All Motions and All Soil Profiles Versus Normalized Periods (Scenario 3)	60
Figure 34 – Residuals versus V_{s30} at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	61
Figure 35 – Residuals versus T_0 at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	62

Figure 36 – Residuals versus PGA at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	64
Figure 37 – Residuals versus $Sa(0.1s)$ at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	65
Figure 38 – Residuals versus root-mean-square of acceleration ($arms$) at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	66
Figure 39 – Residuals versus sustained max acceleration (SMA) at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	67
Figure 40 – Residuals versus $Sa(1s)$ at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	68
Figure 41 – Residuals versus $Sa(10s)$ at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	69
Figure 42 – Residuals versus Arias intensity (AI) at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	70
Figure 43 – Residuals versus cumulative absolute velocity (CAV) at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	71
Figure 44 – Residuals versus $A95$ at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	72
Figure 45 – Residuals versus $PSA[T0]$ at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$	73

LIST OF SYMBOLS AND ABBREVIATIONS

1D	One dimension
AF	Amplification factors
AI	Arias intensity
ARCS	Axis rotation and cubic spline
a_{rms}	Root-mean-square of acceleration
$a(t)$	Acceleration time history
CAV	Cumulative absolute velocity
CPT	Cone penetration test
D	Damping ratio
ξ	Damping ratio
D_M	Damping according to Masing rules
D_{min}	Minimum damping
EL	Equivalent linear
f	Frequency
f_s	Local friction from CPT
γ	Strain
γ_1	User-defined transitional shear strain
γ_b	Bulk unit weight
γ_{eff}	Effective shear strain
γ_{max}	Maximum shear strain
γ_w	Unit weight of water
g	Acceleration of gravity

G	Shear modulus
G_{max}	Maximum shear modulus or elastic shear modulus
I_c	Soil classification index
K_o	In-situ stress ratio
M	Magnitude of the earthquake
MR	Modulus reduction
MRD	Modulus reduction and damping
NBCC 2015	National Building Code of Canada 2015
NEHRP	National Earthquake Hazards Reduction Program
NL	Nonlinear
OCR	Overconsolidation ratio
ρ	Density
p_a	Atmospheric pressure
PI	Plasticity index
PGA	Peak ground acceleration
$PSA[T_0]$	Pseudo spectral acceleration at the natural site period
q_c	Tip resistance from CPT
q_t	Corrected cone resistance from CPT
σ'_o	Mean effective stress
S_a	Spectral response
S_{a_i}	Spectral response of the seismic input motion
S_{a_s}	Spectral response at the surface
S_g	Spectral acceleration

SMA	Sustained maximum acceleration
SPT	Standard penetration test
S_T	Target seismic hazard
S_u	Undrained shear strength
τ	Shear stress
τ_{max}	Maximum shear stress
T	Period
T_0	Natural site period
UHS	Uniform hazard design spectrum
ϕ	Friction angle
V_s	Shear wave velocity
V_{s30}	Average shear wave velocity of the first 30 meters below surface

LIST OF APPENDICES

Appendix A	Amplification Factors for Scenario 1	80
Appendix B	Amplification Factors for Scenario 2.....	86
Appendix C	Amplification Factors for Scenario 3.....	92
Appendix D	Correlations between residuals and <i>Vs30</i>	98
Appendix E	Correlations between residuals and <i>T0</i>	101
Appendix F	Correlations between residuals and <i>PGA</i>	104
Appendix G	Correlations between residuals and <i>Sa(0.1)</i>	107
Appendix H	Correlations between residuals and <i>Sa(1)</i>	110
Appendix I	Correlations between residuals and <i>Sa(10)</i>	113
Appendix J	Correlations between residuals and Root-Mean-Square of Acceleration (<i>arms</i>).....	116
Appendix K	Correlations between residuals and Sustained Maximum Acceleration (<i>SMA</i>) ...	119
Appendix L	Correlations between residuals and Arias Intensity (<i>AI</i>).....	122
Appendix M	Correlations between residuals and Cumulative Absolute Velocity (<i>CAV</i>)	125
Appendix N	Correlations between residuals and <i>A95</i>	128
Appendix O	Correlations between residuals and Pseudo Spectral Acceleration at T_0 (<i>PSA[T0]</i>)	131

CHAPTER 1 INTRODUCTION

1.1 Background on Nonlinear Ground Response Analysis

New editions of the National Building Code of Canada have made seismic hazard an increasingly important design factor to consider in geotechnical engineering in Eastern Canada. The seismic hazard at the bedrock of any coordinates in the form of spectral acceleration is typically provided for the reference rock class C, as shown in Table 1. In the event of an earthquake, seismic waves emitted from the bedrock are usually altered when propagated through the overlaying soil. This is due to the impedance contrast between soft soil deposits and the underlying bedrock, and the nonlinearities that can develop in soft soil layers. The alteration can be an attenuation of the seismic waves by the soil medium, but more problematically, it can also be an amplification by the soil (Buech, 2010). The effect which soils have on the propagation of seismic waves depends on a combination of parameters from the soil and the seismic motion, making an accurate prediction of the soil amplification very difficult. The National Building Code of Canada 2015 (NBCC 2015) provides a simple approach based on Borcherdt (1994) to predict the seismic wave amplification of soil by finding amplification factors (AF) based the average shear wave velocity of the first 30 meters below the surface or the foundation level (V_{s30}) as shown in Table 1. Spectral accelerations at the surface are then obtained by applying these AF to the spectral acceleration at the bedrock. This method can yield very inaccurate results as it does not depend on key dynamic properties such as the modulus reduction and damping (MRD) curves, nor effects such as the frequency content of the motion or the duration of the input seismic motion (Finn and Wightman 2003). In fact, the NBCC 2015 AF were derived from a limited database with few recorded earthquakes on soft soils. The NBCC 2005 and 2010 AF were obtained from the Loma Prieta earthquake and associated ground response simulations (Finn and Wightman, 2003), while the NBCC 2015 AF were derived from a ground motion model of Boore and Atkinson (2008) with few data from soft sites, thus not recommended for soils with $V_{s30} < 180$ m/s. The NBCC 2015 AF can therefore be particularly inaccurate for regions such as the basin of the Saint Lawrence River where large deposits of sensitive clay are present. Design codes in other regions of the world such as the Eurocode 8 used in Europe and the ASCE-7 used in the USA adopted a similar approach also based on the works of Borcherdt (1994), namely consisting of utilizing similar soil type classifications to determine soil amplification coefficients which were derived from the Loma Prieta earthquake and others

(Pitilakis et al., 2012, Huang et al., 2010). Hence, the same limitations apply to those approaches. Table 2 shows the soil classifications used by the Eurocode 8. Due to these drawbacks, the superior approach when determining the amplification effect of a soft soil deposit is typically through a site-specific ground response analysis, where the MRD curves specific to the soil deposit and other parameters such as the frequency content and the duration of the input seismic motion are considered.

Table 1 – V_{s30} based soil classification (Finn et Wightman, 2003)

Site class	Site class name and generic description	Site class definition
A	Hard rock	$V_{s30} > 1500$ m/s
B	Rock	$760 < V_{s30} \leq 1500$ m/s
C	Very dense soil and soft rock	$360 < V_{s30} \leq 760$ m/s, $\bar{N} > 50$, or $\bar{S}_u > 100$ kPa
D	Stiff soil	$180 < V_{s30} \leq 360$ m/s, $15 \leq \bar{N} \leq 50$, or $50 \leq \bar{S}_u \leq 100$ kPa
E	Soil profile with soft clay	$V_{s30} < 180$ m/s; plasticity index $PI > 20$, water content $w > 40\%$, and $\bar{S}_u \leq 25$ kPa
F	Site-specific geotechnical investigations and dynamic site response analyses: (i) soils vulnerable to potential failure or collapse under seismic loading (liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils, etc.); (ii) peats and (or) highly organic clays ($H > 3$ m of peat and (or) highly organic clay, where H is thickness of soil); (iii) very high plasticity clays ($H > 8$ m with $PI > 75$); (iv) very thick “soft – medium-stiff clays” ($H > 36$ m)	

Table 2 – Soil classifications used by the Eurocode 8 (Pitilakis et al., 2012)

Ground type	Description of stratigraphic profile	Parameters		
		$V_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	C_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	>800	-	-
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70-250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with V_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $V_s > 800$ m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index ($PI > 40$) and high water content	<100 (indicative)	-	10-20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1 .			

Incidentally, the dynamic properties of soils, such as the shear wave velocity (V_s), the initial shear stiffness (G_{max}) and the modulus reduction and damping (MRD) curves, are essential factors to consider when performing site-specific ground response analysis. The simplest and most widely used method which considers all these factors is the equivalent linear (EL) method, which solves the 1-D equation of wave propagation in the frequency domain (Seed et Idriss, 1969, Schnabel et al., 1972). However, the EL method does not consider the variations of the dynamic properties during shaking. A more reliable and robust method is the nonlinear (NL) analysis, as it considers the changes to the MRD curves during seismic shaking based on the shear strain amplitude (Kaklamanos et al., 2015, Stewart et al., 2008).

Unfortunately, there are challenges to the use of NL analysis. Due to its complexity and a lack of expertise, the NL method is seldomly used within the Eastern Canadian industry practices, despite

the availability of computer software which can reliably perform NL ground response analyses. One such software is Deepsoil (Hashash et al., 2018) which is already well established in the research field. However, a challenge of using Deepsoil is the time-consuming “Soil Layer Definition” step, in which the user needs to separate the soil profile into numerous layers with small thicknesses. Then, several basic soil properties and the dynamic properties of each layer need to be defined one at a time. The “Soil Layer Definition” step is particularly time-consuming if there is a large number of layers to be defined. It is also imperative to accurately define the dynamic properties, such as the MRD curves, as the accuracy of the NL analysis results largely depend on the accuracy of these properties. Choosing the best constitutive models to use to formulate the MRD curves is not always straightforward, as different models could be best suited for different soil types. Typically, the soil basic and dynamic properties are both interpreted based on field data such as CPT data or, for major projects, using an extensive laboratory testing program.

1.2 Objectives and Methodology

As previously mentioned, the commonly used NBCC 2015 approach for determining the amplification of seismic waves by soft soil deposits can be inaccurate particularly for regions with sensitive clays, while the equivalent linear (EL) approach can also be inaccurate as it does not consider the variations of the soil dynamic properties during shaking. On the other hand, the nonlinear (NL) analysis is a more reliable approach that is can be suited for any type of soil if given reliable input soil properties. However, performing the NL analysis is typically tedious and time-consuming. Therefore, the primary objective of this project was to develop a computer code which can facilitate the use of NL analysis by reducing the time-consuming nature of defining the soil layers properties, enabling the user to perform a large number of NL analysis within a reasonable amount of time. As part of code development, the best suited constitutive models needed to be selected to adequately formulate the MRD curves of the sample soil profiles involved in this project to be analysed by Deepsoil. Next, the secondary but equally pertinent objective of this project is to perform a set of NL analysis on the sample soil profiles using the computer code, and highlight any differences in the results between the NBCC 2015 approach and the NL analysis.

Firstly, 33 anonymous soil profiles from the basin of the Saint Lawrence River provided by WSP Inc., the industrial partner of this project, were compiled and examined for common field data

sources which can offer interpretations of soil properties. A MATLAB code was then written to automatically import and interpret the relevant field data in order to produce an exportable table of subdivided layers with corresponding basic and dynamic properties. This code was also able to discern whether a layer is classified as clay or sand to accurately attribute basic soil properties. The set of empirical relationships used by the code to attribute MRD curves to soil layers is from Darendeli (2001), a widely accepted set of relationships applicable for both sand and clay.

Next, the table of soil layers with corresponding properties produced by the MATLAB code was imported into Deepsoil at the “Soil Layer Definition” step, significantly reducing the time required to complete this step. The ARCS model from Yniesta and al. (2017), a mathematical model which can accurately curve-fit MRD curves independent of the soil type, is used such that the MRD curves can be processed by the Deepsoil software. A set of 48 input seismic motions, filtered and scaled to be consistent with the seismic hazard in Montreal, were selected. NL analysis were then performed for all 33 soil profiles, and the results were collected.

Lastly, soil AF were derived from the results of the NL analysis, which included spectral accelerations of the input motions at the bedrock and the acceleration time histories at the surface. These AF were then compared to AF obtained using the NBCC 2015 approach to examine the differences between the two methods. A particular interest was to investigate whether there would be amplifications at certain periods of certain soil profiles which would be underestimated by the NBCC 2015, posing potential design risks; or whether the NBCC 2015 could be overly conservative at certain periods, leading to needlessly costly design. The residuals between the AF of the two methods were also calculated and plotted against strain to further highlight the differences. The comparisons were discussed, and any correlations between the residuals and various parameters specific to a soil profile or to a motion were investigated.

1.3 Organization of the Memoire

The remainder of this memoire is organized as follows:

(i) LITERATURE REVIEW presents the concepts relevant to this project and research findings studied prior to the start of the design of the MATLAB code. These research findings include

subjects such as the seismic amplification factors (AF), the different types of site-specific analysis and various constitutive models for formulating MRD curves.

(ii) METHODOLOGY OF SOIL PROFILING describes the procedure for creating the MATLAB code which can import and interpret field data, and output a desired table of soil layers and corresponding properties.

(iii) DEEPSOIL SIMULATION RESULTS presents and discusses the AF from the NL analysis and the residuals between the latter and the AF obtained from the NBCC 2015 approach. Correlations between the residuals and various site-specific and motion-specific parameters are also presented.

(iv) CONCLUSION AND FUTURE WORK summarizes the work presented in this memoir and recommends future steps pertinent for future research.

CHAPTER 2 LITERATURE REVIEW

2.1 Seismic Amplification Factors (AF) in Ground Response Analyses

Seismic hazard analysis, such as the ones provided by the Canadian Geological Survey, generally provide the seismic hazard at a site as a response spectrum for rock conditions. In order to quantify the potential of soil deposits to amplify seismic waves propagating from the bedrock to the overlaying soil surface, amplification factors (AF) are typically used. These are used as multipliers applied to the spectral acceleration at the bedrock to estimate the spectral acceleration at the surface. The NBCC defines AF based on NEHRP site classification, which is associated with significant uncertainty, as it does not consider important dynamic properties such as the modulus reduction and damping (MRD) curves, nor effects such as the frequency content of the motion or the duration of the input seismic motion (Finn and Wightman 2003). In the NBCC 2005 and 2010 editions, the AF were derived based on recordings of the Loma Prieta earthquake and ground responses simulations (Finn and Wightman 2003). In the NBCC 2015 edition, factors were obtained from the ground motion model of Boore and Atkinson (2008), itself derived from a ground motion database with only a few recordings at soft sites, with $V_{s30} < 180$ m/s. Due to these limitations, it is recommended that site-specific ground response analysis be performed for sites with $V_{s30} < 180$ m/s. In fact, site-specific analysis is required by the NBCC 2015 for soils of class F, such as liquefiable soils and sensitive clay.

2.2 Site-Specific Ground Response Analysis

Site response analysis is typically preferred whenever possible and even required for soils of class F, as described in Table 1. In current industry practices, site response analyses are commonly performed in one dimension (1-D), by simulating the seismic response of a column of soil, discretized in elements. Such analyses are referred to as ground response analysis, which determines the alteration of seismic waves propagated from the bedrock to the surface of the soft soil deposit due to the impedance contrast between the rock and the soil, and by considering parameters that are specific to the site or the seismic motions involved in the analysis.

Two and three-dimensional site response analysis are seldomly performed in practice, but can assess basin and topographic effects, which can also affect the ground motion propagation. However, they are not considered in this project.

2.2.1 Maximum Shear Modulus (G_{max}) and Shear Wave Velocity (V_s)

The maximum shear modulus (G_{max}) and the shear wave velocity (V_s) are two important parameters required by ground response analysis.

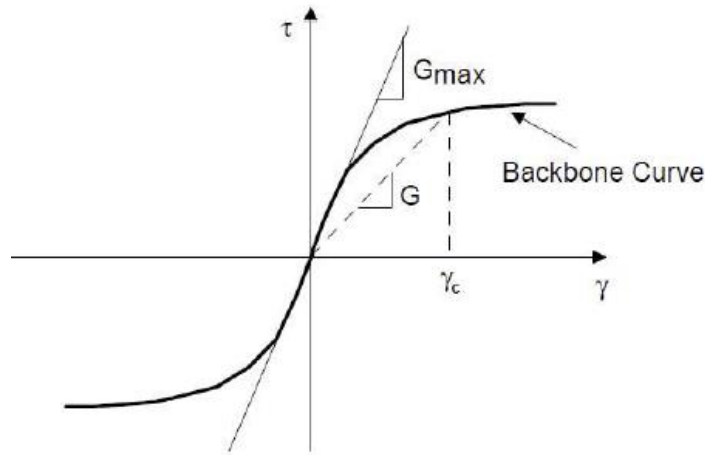


Figure 1 – Example of a Backbone Curve (Stewart et al, 2008)

As shown on Figure 1, the stress-strain curve, commonly referred to as the backbone curve, describes the behaviour of the soil upon monotonic loading, (also called initial loading, i.e. before unloading and reloading) and shows the variations of secant shear modulus with strain. The secant shear modulus (G), as a function of strain (γ), is defined as the slope of a straight line from the origin to the corresponding stress value on the backbone curve. The maximum shear modulus (G_{max}) is the slope of the backbone curve when $\gamma = 0$, and can be defined as follows:

$$G_{max} = \rho * V_s^2 \quad \text{Equation 1}$$

where ρ is the density and V_s is the shear wave velocity. G_{max} is the elastic shear modulus of the soil. The G_{max} can also be approximated as the G at a strain smaller than the linear cyclic threshold shear strain, γ_{tl} , below which the soil behaves as a linearly elastic material (Vucetic, 1994).

The shear wave velocity (V_s) of a soil is a mechanical property that can be determined either through laboratory testing or in-situ measurements. Although associated with more error, they can also be derived from empirical correlations such as those based on SPT data (e.g. Rollins et al., 1998, Brandenburg et al., 2010) or CPT data (e.g. Karray and Hussien, 2017).

2.2.2 Modulus Reduction and Damping (MRD) Curves

The dynamic properties required by ground response analysis include the modulus reduction and the damping (MRD) curves associated with each layer of soil.

The modulus reduction curve is the variation of normalized shear modulus with respect to shear strain amplitude. The normalized modulus reduction curve is defined as the ratio between the secant shear modulus (G) and the maximum shear modulus (G_{max}). Figure 2 shows a normalized modulus reduction curve, where $G(\gamma)/G_{max}$ is typically plotted against the logarithm of γ . Note that since $G(0) = G_{max}$, $G(0)/G_{max}$ is always equal to 1.

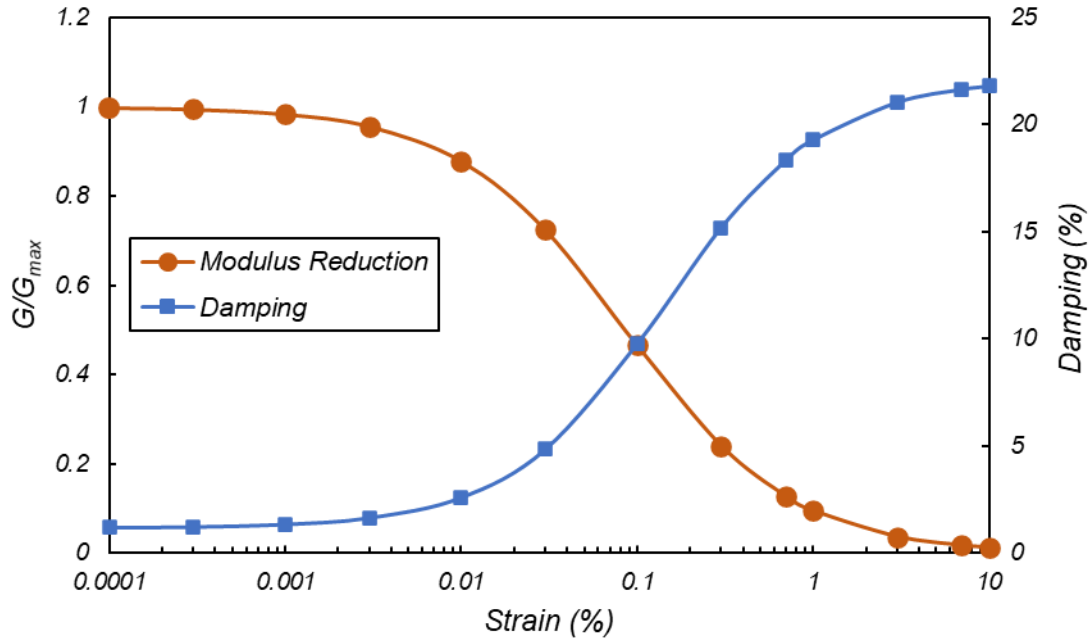


Figure 2 – Example of a set of Modulus Reduction and Damping Curves

Following the initial loading, the subsequent unloading/reloading cycles can define the damping curve, which portrays the capacity of a layer to dissipate energy upon cyclic loading. As shown in

Figure 3, the value of the hysteretic damping (D) at any given γ is computed as the ratio between areas A and B for any given loading loop. A series of tests performed at numerous γ values can be performed to obtain a damping curve. Figure 2 also shows an example of a damping curve. Typically in practice, data at smaller strains can be obtained via resonant column tests while data at larger strains can be obtained via triaxial tests.

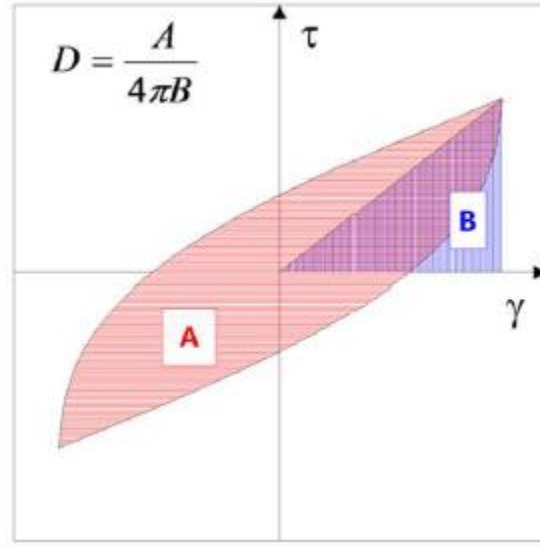


Figure 3 – Definition of Damping (Yniesta et al., 2017)

2.2.2.1 Darendeli's (2001) Empirical Relationships

Although the MRD curves of a soil element can be determined through extensive laboratory experiments, they are more commonly obtained through established empirical relationships. For instance, Darendeli (2001) proposed empirical relationships applicable for clays, silts and sands to determine MRD curves as a function of the overconsolidation ratio (OCR), the mean effective stress (σ'_o) and the plasticity index (PI).

First, the modulus reduction curve of Darendeli (2001) for each layer with respect to the deformation is calculated as follows:

$$\frac{G(\gamma)}{G_{max}} = \frac{1}{1 + \left(\frac{\gamma}{\gamma_r}\right)^\alpha} \quad \text{Equation 2}$$

where:

$$\alpha = 0.92,$$

$$\gamma_r(\%) = (\varphi_1 + \varphi_2 * PI * OCR^{\varphi_3}) * \left(\frac{\sigma'_o}{p_a}\right)^{\varphi_4}, \quad \text{Equation 3}$$

with:

$$\sigma'_o = \text{mean effective stress} = \sigma'_v * \frac{1+2*K_o}{3}, \quad \text{Equation 4}$$

and $p_a = \text{atmospheric pressure}$.

In Darendeli's empirical equations, the coefficients $\varphi_1, \varphi_2, \varphi_3, \varphi_4, \varphi_6, \varphi_7, \varphi_8, \varphi_9, \varphi_{10}, \varphi_{11}$ and φ_{12} have different average values for different types of soils.

Next, the hysteretic damping curves of Darendeli (2001) with respect to the deformation is calculated as follows:

$$D(\gamma) = D_{min} + b * D_M(\gamma) * \left(\frac{G(\gamma)}{G_{max}}\right)^{0.1} \quad \text{Equation 5}$$

where:

$$D_{min} = (\varphi_6 + \varphi_7 * PI * OCR^{\varphi_8}) * \left(\frac{\sigma'_o}{p_a}\right)^{\varphi_9} * (1 + \varphi_{10} * \ln(freq)) \quad \text{Equation 6}$$

with:

$$freq = 1 \text{ Hz},$$

$$b = \varphi_{11} + \varphi_{12} * \ln(N), \quad \text{Equation 7}$$

$$N = \text{number of cycle, } 10.$$

$D_M(\gamma)$ is the hysteretic damping according to the Masing rules as further explained in section 2.2.5.2.1, and can be empirically obtained as follows:

$$D_M(\gamma) = c_1 * D_{M,\alpha=1} + c_2 * D_{M,\alpha=1}^2 + c_3 * D_{M,\alpha=1}^3, \quad \text{Equation 8}$$

$$D_{M,\alpha=1}(\gamma)[\%] = \frac{100}{\pi} \left[4 * \frac{\gamma - \gamma_r * \ln\left(\frac{\gamma - \gamma_r}{\gamma_r}\right)}{\frac{\gamma^2}{\gamma + \gamma_r}} - 2 \right], \quad \text{Equation 9}$$

$$c_1 = 0.2523 + 1.8618 * \alpha - 1.1143 * \alpha^2, \quad \text{Equation 10}$$

$$c_2 = -0.0095 - 0.0710 * \alpha + 0.0805 * \alpha^2, \quad \text{Equation 11}$$

$$et\ c_3 = 0.0003 + 0.0002 * \alpha - 0.0005 * \alpha^2. \quad \text{Equation 12}$$

Darendeli's empirical curves are consistent with the findings from Vucetic and Dobry (1991) and from Stokoe (1999) which found that soil behaves more linearly as the plasticity index and confining pressure increase. However, Darendeli's equations are limited to strains lower than 0.3% as the database from which the equations are derived did not have sufficient data at larger strains. Other established relationships include those derived by Menq (2003) which are applicable for gravels and sands, Zhang et al (2005) which are applicable for Central and Eastern US soils, Kishida et al (2009) which are applicable for organic soils, and more.

2.2.3 Linear Analysis

The linear analysis is the easiest site-specific analysis to perform. It consists of solving the propagation of waves through a column of soils using the wave equation for the 1-D elastic system. However, the dynamic properties are assumed to be constant for the linear analysis, and therefore the method should not be used, because it is not an accurate representation of the soil behaviour. For the case of uniform undamped soil on rigid rock, the wave equation is represented by the following differential equation:

$$\frac{\partial^2 u}{\partial t^2} = V_s^2 \frac{\partial^2 u}{\partial x^2} \quad \text{Equation 13}$$

where t is time, x is the position along the one dimension, and u is the particle movement in the transverse direction at the location of x . In this scenario, the amplification is at its peak when $kH = \frac{\pi}{2}$ or when $\omega = \frac{\pi V_s}{2H}$ as shown in Figure 4. k and ω are the wave number and the circular frequency respectively. This in turn means that in this scenario, the amplification peaks occur when the frequency is equal to $V_s/4H$, the site's natural frequency.

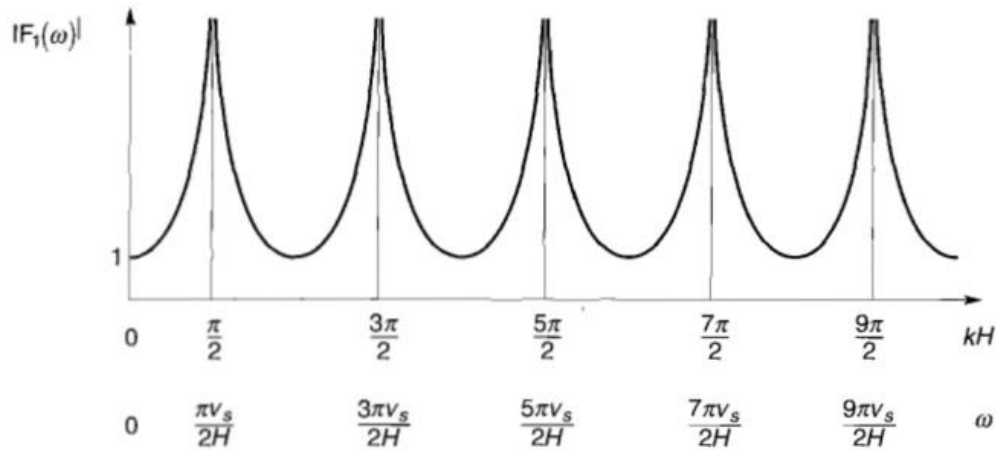


Figure 4 – Amplification Peaks for Uniform Undamped Soil on Rigid Rock (Kramer, 1996)

However, as shown in Figure 4, at every harmonic frequency, the amplification goes to infinity, which is unrealistic. Therefore, damping should be introduced. For the case of uniform damped soil on rigid rock, the wave equation is

$$\frac{\partial^2 u}{\partial t^2} = V_s^2 \left(\frac{\partial^2 u}{\partial x^2} + \frac{2D}{\omega} * \frac{\partial^2 u}{\partial z^2 \partial t} \right). \quad \text{Equation 14}$$

Since linear solutions cannot address hysteretic damping, the equivalent viscous damping ratio, D , is used. As shown on Figure 5, the amplification no longer peaks at infinity, but has a maximum at $f = V_s/4H$, the natural frequency of the site.

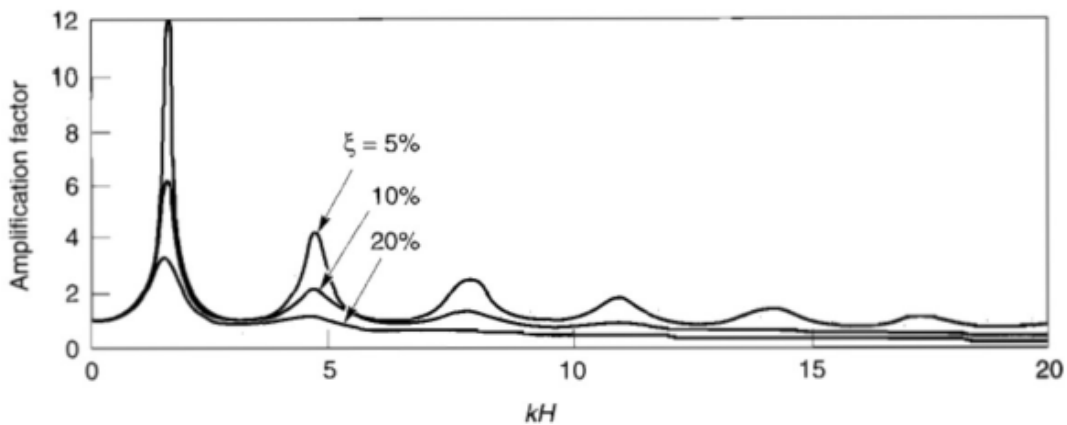


Figure 5 – Amplification Peaks for Uniform Damped Soil on Rigid Rock (Kramer, 1996)

The most realistic case of linear analysis is the scenario of layered damped soil on elastic rock. Figure 6 shows the coordinate of the layered system. In this scenario, the wave equation solutions

$$\gamma_{eff} = \frac{M-1}{10} * \gamma_{max} \quad \text{Equation 15}$$

where M is the magnitude of the earthquake. Then, updated guesses of G and D for each layer can be found from the MRD curves using the γ_{eff} of each layer. With the new G and D values, another linear site response analysis is carried out to find a new γ_{eff} from the γ_{max} . This iterative process is to be repeated until the updated guesses for G and D of a new iteration converge with those of a previous iteration. Finally, the linear analysis results from the last iteration of this procedure are the results of the EL analysis.

The EL analysis is still commonly used due to its low computational requirements. However, the iterative procedure to determine G and D may not always converge, or simply produces results lacking in accuracy when compared to results from nonlinear analysis, particularly for soils with highly nonlinear behaviours or when large strains ($> 0.05\%$) or plastic response are expected (Kaklamanos et al. 2013, Stewart et al. 2008).

2.2.5 Nonlinear (NL) Analysis

The nonlinear (NL) analysis (Kaklamanos et al., 2015) is the most rigorous and accurate method of ground response analysis while considering the nonlinearity of the dynamic behaviour of soils. The NL analysis solves the equation of motion in the time domain at every time step:

$$[M] * \{\ddot{u}\} + [C] * \{\dot{u}\} + [K] * \{u\} = -[M] * \{I\} * \ddot{u}_g \quad \text{Equation 16}$$

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

2.2.5.1 Hypothesis

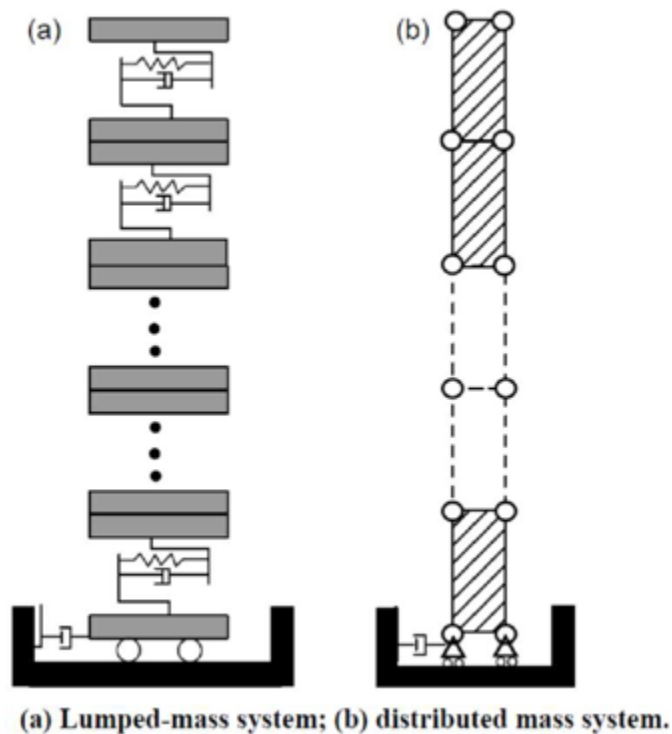


Figure 7 – One dimensional simplification of soil layer system (Stewart et al, 2008)

As seen in Figure 7, the soil column layers can be modelled either in a lumped-mass or a distributed mass system. The lumped-mass system is a multi-degree-of-freedom system where layers are each assigned a mass lumped together by discrete springs and dashpots. This system is used by the software Deepsoil, the NL modelling software used for this project. The distributed mass system is a continuum system where every layer is split into smaller elements assigned with a distributed mass. The distributed mass system is adapted for other software such as OpenSees, SUMDES and TESS, which are not covered in this project.

2.2.5.2 Nonlinear (NL) Soil Models/Formulations

Since NL analysis is performed in the time domain, it requires a time-stepping integration algorithm to solve the equation of motion. The time-stepping scheme commonly used in NL analysis is the Newmark method (Newmark, 1959). At every time step, the stiffness matrix is updated according to a chosen constitutive model representing the soil's cyclic behaviour. In 1D ground response analysis, the latter is typically described by (i) a backbone curve representing the

initial loading behaviour and (ii) a series of unloading and reloading curves representing the soil's hysteretic behaviour. The MRD curves initially obtained from empirical relationships such as those from Darendeli (2001) cannot be directly used as input parameters for the NL models in software such as Deepsoil. Thus, a set of chosen constitutive NL soil models/formulations are calibrated to match the empirically obtained input MRD curves. In the following sub-sections, a few constitutive models will be briefly presented.

2.2.5.2.1 Masing Rules

The first set of unloading/reloading rules was developed by Masing (1926). The extended Masing rules are as follow:

- (i) During initial loading, the stress as a function of strain precisely follows the backbone curve:

$$\tau = F_{bb}(\gamma) \quad \text{Equation 17}$$

where $F_{bb}(\gamma)$ is the backbone function.

- (ii) When a stress-strain reversal occurs at a point $(\gamma_{rev}, \tau_{rev})$, the stress as a function of strain follows a shape that is twice the size of the backbone curve:

$$\frac{\tau - \tau_{rev}}{2} = F_{bb}\left(\frac{\gamma - \gamma_{rev}}{2}\right) \quad \text{Equation 18}$$

- (iii) When the unloading/reloading curve exceeds the point at which the unloading/reloading started, the stress-strain curve continues to follow the backbone curve until another stress-strain reversal occurs.

- (iv) When an unloading/reloading curve intersects an unloading/reloading curve of a previous cycle, it follows the curve of the previous cycle.

At small strains, the Masing rules produce damping that is almost zero, which is unrealistic. Viscous damping at small strains, described in section 2.2.5.3, are therefore needed. At large strains, the Masing rules can lead to significant overestimations of damping, as shown in Figure 8.

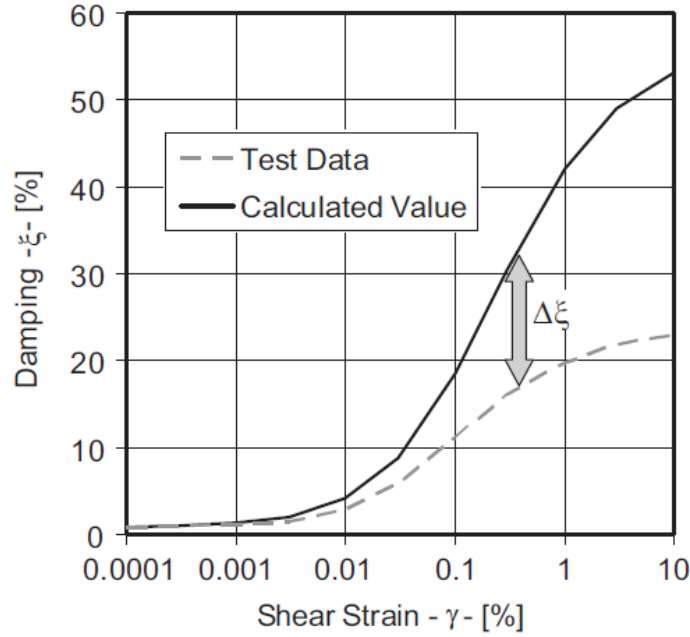


Figure 8 – Overestimation of damping by the Masing rules (Phillips and Hashash, 2009)

2.2.5.2.2 MKZ model

The MKZ model by Matasovic (1993), modified from the earlier KZ model by Kondner and Zelasko (1963), describes the initial loading curve of the soil's cyclical behaviour. The MKZ model improved upon its predecessor by increasing its ability to predict small-strain behaviour, as typical soil seismic responses are solicited at strains lower than 1-3%. The relationship between shear stress and shear strain in the MKZ model can be described by the hyperbolic equation as follows:

$$\frac{G(\gamma)}{G_{max}} = \frac{1}{1 + \beta \left(\frac{\gamma}{\gamma_{r,MKZ}} \right)^\alpha} \quad \text{Equation 19}$$

where

$$\gamma_{r,MKZ} = \tau_{mo} / G_{mo}. \quad \text{Equation 20}$$

β is a curve-fitting parameter that adjusts the position of the hyperbolic curve. α is a curve-fitting parameter which adjusts the curvature. τ_{mo} is the “ τ ordinate corresponding approximately to the upper boundary of this dominant shear strain range” (Matasovic, 1993) of 1-3% shear strain. G_{mo} is the initial tangent shear modulus. The MKZ model is equivalent to the KZ model when $\beta = \alpha = 1$. A limitation of the MKZ model is its inability to match a target shear strength at large shear

strains, including the implied shear strength of the soil which can be defined as the shear stress of the backbone curve when $\gamma = 10\%$, as demonstrated in Figure 9 in section 2.2.5.2.4.

2.2.5.2.3 Model of Yee et al.

Yee et al. (2013) developed a model which conserves the small-strain behaviour of the MKZ model while adjusting the large-strain behaviour to the actual failure strength of the soil by altering shear stresses at strains larger than the user-defined transitional shear strain, γ_1 . At strains smaller than γ_1 , the original hyperbolic modulus reduction curve is used; however, at strains larger than γ_1 , the modulus reduction curve follows a hybrid of two hyperbolic curves:

$$\frac{G}{G_{max}} = \frac{\frac{\gamma_1}{1+(\gamma_1/\gamma_r)^\alpha} + \frac{(G_{\gamma_1}/G_{max})/\gamma'}{1+(\gamma'/\gamma'_{ref})}}{\gamma} \quad \text{Equation 21}$$

where:

$$\gamma' = \gamma - \gamma_1, \quad \text{Equation 22}$$

$$\gamma'_{ref} = \frac{S_u - \tau_1}{G_{\gamma_1}}, \quad \text{Equation 23}$$

$$\tau_1 = \frac{G_{max} * \gamma_1}{1 + (\frac{\gamma_1}{\gamma_r})^\alpha}, \quad \text{Equation 24}$$

$$\frac{G_{\gamma_1}}{G_{max}} = \frac{1 + (1 - \alpha) * (\frac{\gamma_1}{\gamma_r})^\alpha}{(1 + (\frac{\gamma_1}{\gamma_r})^\alpha)^2}. \quad \text{Equation 25}$$

To maintain the continuity in the slopes of the stress-strain curve, the tangent shear modulus of the original hyperbola at $\gamma = \gamma_1$ (G_{γ_1}) is equal to the initial modulus of the hybrid hyperbola.

2.2.5.2.4 GQ/H model

The GQ/H model by Groholski et al. (2016) was developed to address the issue of shear strength overestimation at large strains as well. The general equation for the GQ/H hyperbolic model is as follows:

$$A\gamma^2 + B\gamma\tau + C\tau^2 + D\gamma + E\tau + F = 0 \quad \text{Equation 26}$$

where A, B, C, D, E and F are constant fitting parameters. Under the conditions of setting a τ_{max} which should be reached but not exceeded, the general hyperbolic model can be represented by

$$\theta_\tau = \frac{\left(\frac{G}{G_{max}}\right) + \left(\frac{G}{G_{max}}\right) * \left(\frac{\gamma}{\gamma_r}\right) - 1}{\left(\frac{G}{G_{max}}\right)^2 * \left(\frac{\gamma}{\gamma_r}\right)} \quad \text{Equation 27}$$

where

$$\theta_\tau = \theta_1 + \theta_2 \left(\frac{\theta_4 * \left(\frac{\gamma}{\gamma_r}\right)^{\theta_5}}{\theta_3^{\theta_5} + \theta_4 * \left(\frac{\gamma}{\gamma_r}\right)^{\theta_5}} \right) \leq 1. \quad \text{Equation 28}$$

$\theta_1, \theta_2, \theta_3, \theta_4$ and θ_5 are curve-fitting parameters to provide the best fit to the modulus reduction curve. The user can also assign a minimum percentage of match between the implied and the maximum shear strength of the layer. Typically, the GQ/H model can ensure a reasonable match between the implied shear strength and the maximum shear strength. Figure 9 showcases that while both the modulus reduction curves of the MKZ model and of the GQ/H model match fairly well to the reference curve from Darendeli (2001), the GQ/H model can provide a significantly better prediction of the maximum shear strength (τ_{max}).

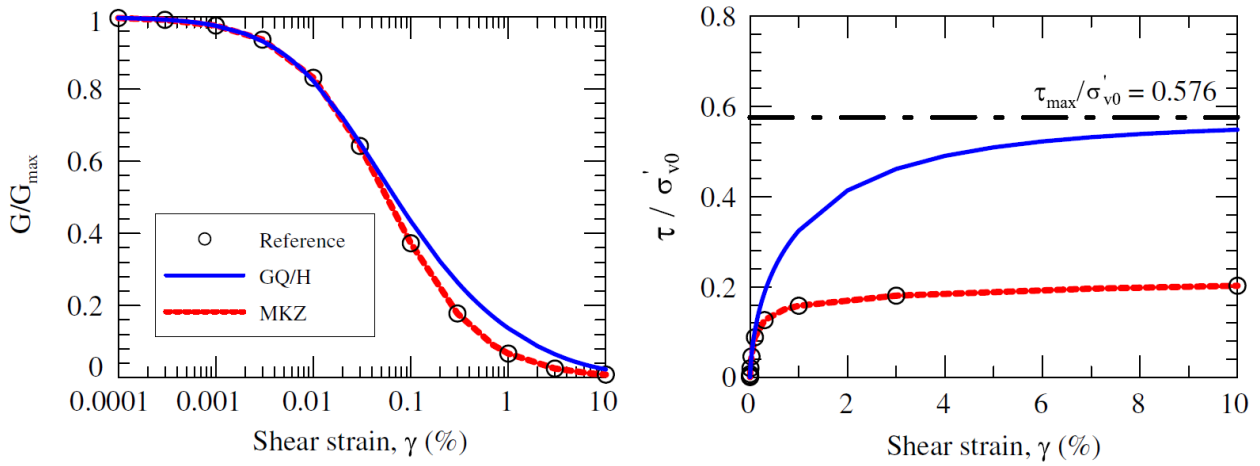


Figure 9 – Improvement of the GQ/H model over the MKZ model (Groholski et al., 2016)

2.2.5.2.5 MRDF formulation

Phillips and Hashash (2009) developed a set of unloading/reloading rules using a reduction factor (MRDF) that improved on the Masing rules, particularly at large strains. The MRDF procedure consists of first computing a backbone curve obtained from the chosen curve-fit of the input modulus reduction curve. Then, a corresponding damping curve is to be calculated from the backbone curve using the Masing rules. Lastly, three reduction factor parameters, p_1 , p_2 and p_3 , need to be optimized to provide the best curve-fit to the damping curve. The three parameters are obtained by the following equation:

$$F(\gamma_m) = p_1 - p_2 \left(1 - \frac{G_{\gamma_m}}{G_{max}}\right)^{p_3}. \quad \text{Equation 29}$$

$F(\gamma_m)$ is defined as the adjustment factor to the Masing damping curve and is calculated as follows:

$$F(\gamma_m) = \frac{\text{input damping curve} - \text{input minimum damping}}{\text{damping curve using Masing rules}}. \quad \text{Equation 30}$$

G_{γ_m} is defined as the secant modulus associated with the maximum shear strain, γ_m , of each unloading/reloading cycle.

The MRDF formulation can provide fairly accurate curve-fittings for the damping curves at all strains, including strains larger than 1%. Figure 10 shows an example of a comparison between the MRDF formulation and the Masing rules while using the same backbone curve from the GQ/H model, presented in section 2.2.5.2.4. The reference curve in Figure 10 is obtained using the empirical relationships of Darendeli (2001).

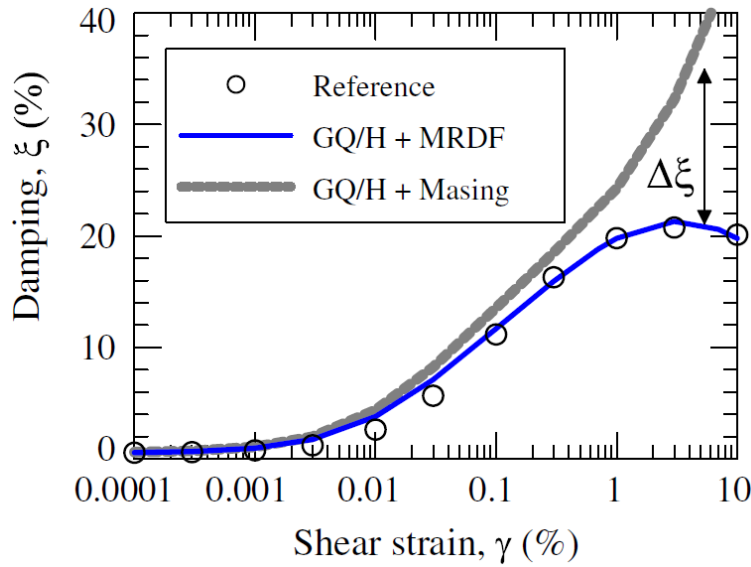


Figure 10 – The MRDF formulation versus the Masing rules (Groholski et al., 2016)

2.2.5.2.6 ARCS model

The model ARCS, developed by Yniesta et al. (2017), addresses the issues associated with the models previously presented. The Axis Rotation and Cubic Spline (ARCS) formulation is used to accurately mathematically curve-fit the input MRD curves.

For the modulus reduction curves, ARCS curve-fits through every single input empirical data point with the cubic spline interpolation, creating a precise and continuous fit. The ARCS formulation also offers a curve-fitting of the input empirical damping curve which is able to perfectly match damping at all strains, unlike the Masing rules which overpredict damping at high strains or the newer MRDF formulation which can still predict inaccurate damping values at certain strain values (Yniesta et al., 2017). The unloading/reloading damping behaviour of the ARCS formulation performs coordinate transformations to compute the shear stress, and abides by the following rules:

- “(i) The secant modulus of the stress-strain loops matches a user-defined modulus-reduction curve,
- (ii) When subject to uniform cyclic strain amplitude input, the stress-strain loops close and repeat, exhibiting no cyclic degradation or stiffening,

- (iii) The area inside the stress-strain loops matches a user-defined damping curve, even at small strains where the modulus-reduction value is one,
- (iv) The stress-strain loops are concave about the secant modulus line.” (Yniesta et al., 2017)

More details about the ARCS formulation are provided in Yniesta et al. (2017).

2.2.5.3 Viscous Damping

2.2.5.3.1 Frequency dependent damping formulation

Typically, NL models do not introduce small-strain hysteretic damping since the behaviour of such at small strains is essentially elastic. Viscous damping formulation is thus needed to model small-strain damping so that there is realistic damping even at small strains. Traditionally, the Rayleigh damping (Rayleigh and Lindsay. 1945) is used to introduce small-strain damping by representing the damping matrix, $[C]$, as a combination of the mass and stiffness matrices:

$$[C] = a_0[M] + a_1[K] \quad \text{Equation 31}$$

where $[M]$ and $[K]$ are the mass and stiffness matrices respectively, and a_0 and a_1 are scalar values computed as follows:

$$\begin{bmatrix} \xi_i \\ \xi_j \end{bmatrix} = \frac{1}{4\pi} \begin{bmatrix} \frac{1}{f_i} & f_i \\ \frac{1}{f_j} & f_j \end{bmatrix} \begin{Bmatrix} a_0 \\ a_1 \end{Bmatrix} \quad \text{Equation 32}$$

where ξ_i is the damping ratio for frequency f_i and ξ_j is the damping ratio for frequency f_j . Among various recommendations in the literature, f_i is commonly chosen to be the natural frequency of the soil and f_j is chosen to be the predominant frequency of the input seismic motion. The solution calculated from the Rayleigh and Lindsay (1945) approach is frequency dependent, which is typically contradicted by laboratory results showing that solutions are rather frequency independent at small strains. In addition, Park and Hashash (2008) implemented an extended Rayleigh damping involving four frequencies in the Deepsoil software. Figure 11 shows the accuracy of one, two and four modes (extended) solutions by comparing the ratio between the predicted damping (ξ_{Model}) and the target damping (ξ_{Target}) values. As seen in Figure 11, the four modes solution is closer to

the target than the two modes solution at frequencies shorter than 10Hz, particularly between 0.7 Hz and 6 Hz, while the two modes solution is closer to the target at frequencies larger than 10Hz.

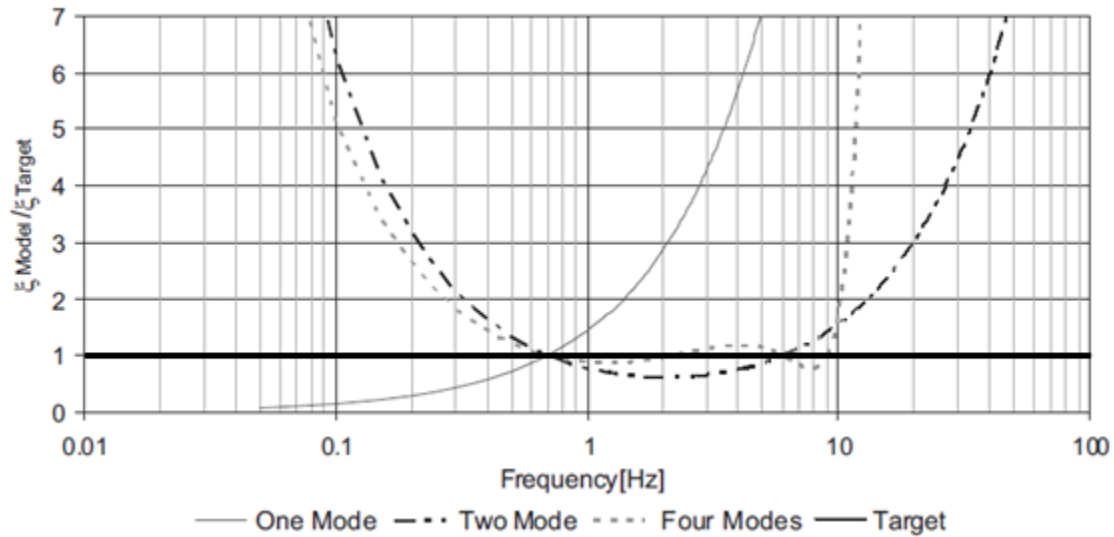


Figure 11 – Comparison between one, two and four modes Rayleigh formulation (Phillips and Hashash, 2009)

2.2.5.3.2 Frequency independent damping formulation

Based on the works of Liu and Gorman (1995), Phillips and Hashash (2009) developed a frequency independent Rayleigh damping formulation, which is more consistent with experimental results at small strains. This method involves calculating eigenvalues and eigenvectors of the matrix $[M]^{-1}[K]$ to determine the system's real modal matrix and natural frequencies, which causes this method to be computationally demanding. More details on the frequency independent damping formulation can be found in Phillips and Hashash (2009).

CHAPTER 3 METHODOLOGY OF SOIL PROFILING BASED ON FIELD DATA

In this section, the step-by-step use of the MATLAB code to perform soil profiling based on available field data is described. The MATLAB programming language was selected due to the speed and flexibility of implementing the code in the chosen language. However, it is also possible to implement the code in other programming languages, such as Python, in potential future works.

3.1 Objectives of the Code

In this project, NL analysis were performed for 33 soil profiles to showcase the importance of NL analysis over the use of amplification factors (AF) from the NBCC 2015. However, to efficiently analyse all 33 soil profiles, the creation of a MATLAB code was required to automatically produce tables of basic and dynamic soil properties necessary for defining soil layers on the NL analysis software used in this project, Deepsoil. The purpose of this MATLAB code was to reliably enable a large number of NL analysis to be done in a reasonable amount of time.

3.2 Logic Flow

Before diving into the details of the MATLAB code, presented in section 3.3, a summary of its logic flow is presented here. First, the basic CPT data (q_c , f_s and q_t), the ground water level and any available V_s and S_u field measurement of each soil profile is imported into MATLAB as inputs for the code. After smoothing the raw CPT data using a locally weighted linear regression approach, soil classification indices (I_c), along with bulk unit weights (γ), are calculated. The I_c are then used to label each layer as either clay ($I_c \geq 2.6$) or sand ($I_c < 2.6$), which allows for V_s , shear strengths, PI and other properties to be estimated from the CPT data according to each layer's soil type. If for a soil profile, field measurements of V_s and undrained shear strength (S_u) are available, the previously CPT-based V_s and S_u estimations are then calibrated to match field measurements. Next, due to the time-consuming runtime of the Deepsoil NL analysis, CPT data points are grouped together in layers to significantly reduce the total amount of soil layers for analysis, and in turn, significantly reduce Deepsoil's runtime. For each grouped layer, the dynamic properties of the soil are then computed based on the empirical relationships from Darendeli

(2001). Lastly, tables of finalized layers and their corresponding properties required as input into the Deepsoil software are exported. The summary of the logic flow is shown in Figure 12.

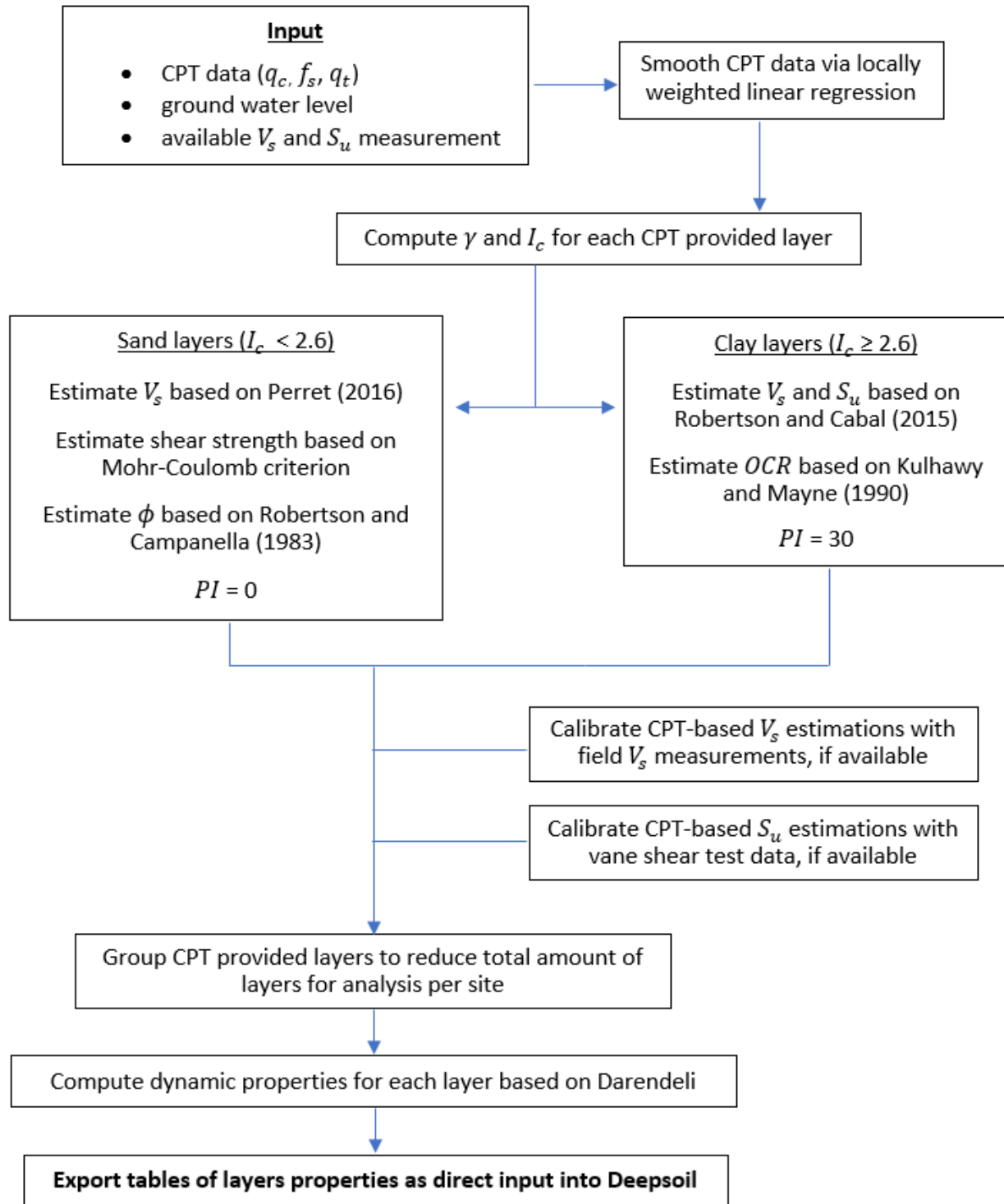


Figure 12 – Logic flow summary of the MATLAB code

3.3 Step-by-Step Description of the Code

3.3.1 Importation and Processing of Available Field Data

First, for each soil profile, the MATLAB code imports the available CPT data of cone resistance (q_c), sleeve friction resistance (f_s) and corrected cone resistance (q_t) located in specified dedicated columns in EXCEL Worksheets. For most soil profiles, these CPT resistance factors were measured consistently at every 1 cm in depth, but for some of these profiles, they were measured at every 2 cm or 2.5 cm. Regardless, the intervals of depth at which CPT measurements were made are initially taken as layer thicknesses by the MATLAB code. To improve the interpretation of the field data, the extracted CPT data are curve-smoothed by the method of local regression using weighted linear least squares over the entire depth without loss of resolution, as shown in Figure 13.

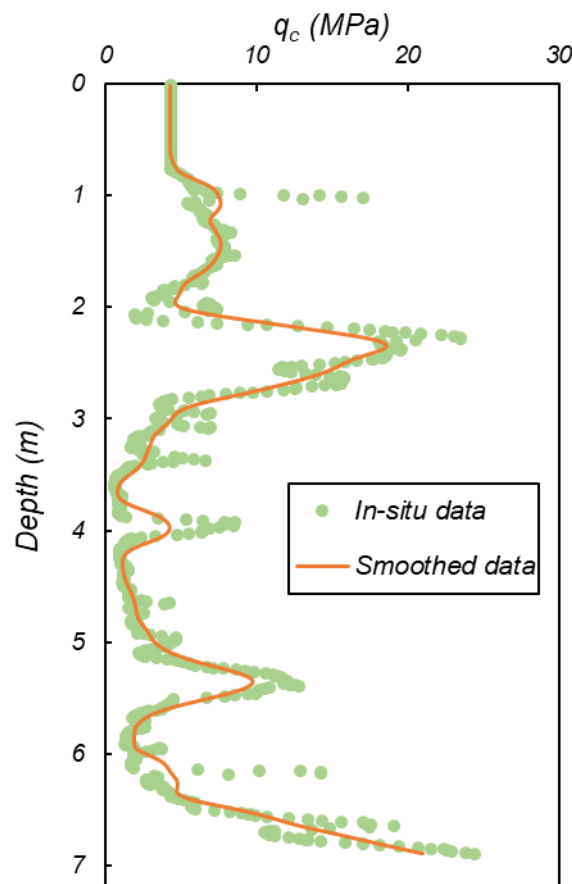


Figure 13 – Example of curve-smoothing of q_c measurements

In addition, from the EXCEL Worksheets containing the SCPT tests data, the shear wave velocity (V_s) measurements are extracted and smoothed, as shown in Figure 14, without loss of resolution, by the Gaussian weighted moving average method, a curve-smoothing technique which also eliminates outliers. However, for the majority of soil profiles involved in this study, V_s measurements were not available; in such case, V_s values firstly determined through the interpolation of CPT data would not be calibrated by any field V_s measurements.

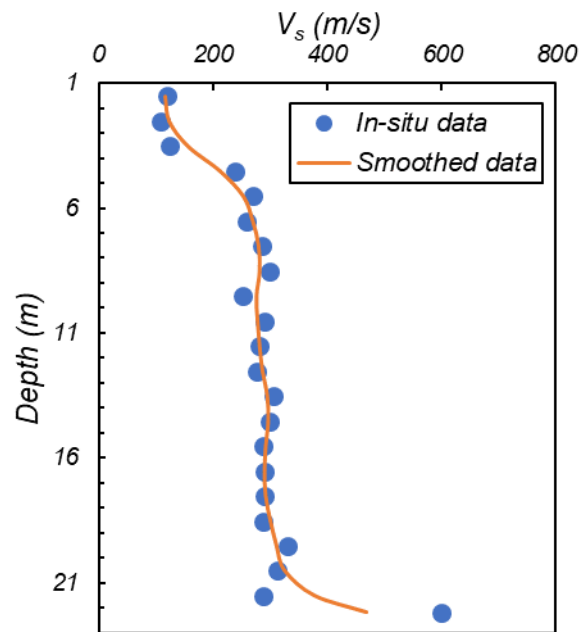


Figure 14 – Example of curve-smoothing of V_s measurements

Subsequently, the measurements of shear strength from vane shear tests and of the water table level are imported manually because this information are obtained from PDF files provided by WSP Inc.

3.3.2 Definition of Soil Layers Basic Properties

In order to adequately define the soil layers' properties, the stratigraphy must be defined first. Since sand and clay soils have different physical properties, each soil layer is classified as either sand or clay depending on its classification index, I_c , calculated from the imported basic CPT data (Robertson, 1990) as follows:

$$I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5} \quad \text{Equation 33}$$

with:

$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}}, \quad \text{Equation 34}$$

$$\text{and } F_r = \left(\frac{f_s}{q_t - \sigma_{vo}} \right) * 100\%. \quad \text{Equation 35}$$

Typically, a soil layer with an I_c value greater than or equal to 2.6 is considered to be a clayey soil, while a soil layer with an I_c value less than 2.6 is considered to be a sandy soil. However, for this project, modifications to this criterion by Robertson (1990) needed to be made due to unreasonable fluctuations of I_c caused by the inherent uncertainty and variability of CPT tests. The possibility that these fluctuations could be caused by varved clays containing interlayers of sands is disregarded due to the unknown seismic behaviour of the latter. In this project, if the I_c limit value of 2.6 is strictly kept over the entire depth of most of the soil profiles, an unrealistically large number of segments, when compared to the boring logs, could be created at certain intervals of depth at certain soil profiles. In this project, a segment is defined as a set of continuous sublayers. Initially when stratifying a soil profile, a new soil segment is identified when the I_c value goes from a value lower than 2.6 to a higher value, or vice versa. However, if over a certain interval of depth, the total number of soil layer changes decreases by changing the I_c threshold criterion from 2.6 to another value between 2.6 and 2.4, the I_c threshold criterion will be adjusted iteratively for the interval in question. This will minimize the total number of soil type changes and create a more realistic representation of the profile's soil composition. An example is shown in Figure 15 in which the I_c threshold criterion is changed from 2.6 to 2.43 between the depths of 5.2m and 8.4m, which has become a single segment of clay instead of several segments of sand and clay if the criterion was to stay at 2.6.

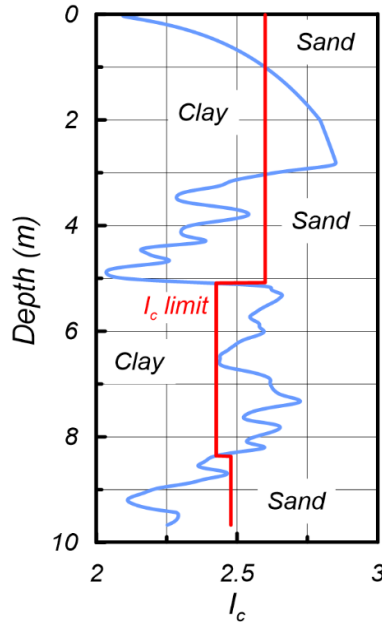


Figure 15 – Example of segmentation

3.3.2.1 Bulk Unit Weight (γ)

The bulk unit weight (γ) of each soil layer is calculated from the CPT data according to the empirical relationships of Robertson (2010):

$$\gamma = [0.27(\log R_f) + 0.36(\log \frac{q_t}{p_a}) + 1.236] * \gamma_w \quad \text{Equation 36}$$

where:

$$R_f = \left(\frac{f_s}{q_t} \right) * 100\%, \quad \text{Equation 37}$$

γ_w = unit weight of water,

and p_a = atmospheric pressure.

3.3.2.2 Shear Strength

The undrained shear strength (S_u) of each clay layer is calculated by Robertson and Cabal (2015):

$$S_u = \frac{q_t - \sigma_v}{N_{kt}} \quad \text{Equation 38}$$

where N_{kt} is a dimensionless parameter with an average value of 14, but calibrated by the available field measurements of undrained shear strength such that estimated values of S_u are consistent with field measurements. More details on this calibration are found in section 3.3.4.

The shear strength (τ_{max}) of each sand layer is calculated with the Mohr-Coulomb criterion without the cohesion factor:

$$\tau_{max} = \sigma'_{vo} * \tan(\phi) \quad \text{Equation 39}$$

where σ'_{vo} is the initial effective stress and ϕ is the friction angle calculated by Robertson and Campanella (1983):

$$\tan(\phi') = \frac{1}{2.68} * [\log \frac{q_c}{\sigma'_{vo}} + 0.29]. \quad \text{Equation 40}$$

3.3.2.3 Plasticity Index (PI)

Due to the impossibility of accurately predicting the plasticity indices (PI) using the available data in this project, PI of sand layers are assumed to be 0, while PI of clay layers are assumed to be 30, a conservative value for typical clay found in Quebec. Each layer's PI is therefore automatically assigned according to its soil type.

3.3.2.4 Estimating Over-Consolidation Ratios (OCR)

The over-consolidation ratios (OCR) are calculated with the empirical relations from Kulhawy and Mayne (1990):

$$OCR = k * \frac{(q_t - \sigma_{vo})}{\sigma'_{vo}}, \quad \text{Equation 41}$$

where an average value of the constant $k = 0.33$ is taken (Robertson, 2015).

3.3.2.5 Coefficient of Earth Pressure at Rest (K_o)

The in-situ stress ratios (K_o) are also estimated with empirical relations from Kulhawy and Mayne (1990):

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

3.3.2.6 Shear Wave Velocity (V_s)

Since shear wave velocities (V_s) measurements are only available for a minority of soil profiles involved in this project, V_s of all layers for all profiles are initially estimated from CPT data. V_s of clayey layers are calculated by Robertson et Cabal (2015), while V_s of sandy layers are calculated by Perret (2016):

$$V_s = \left[10^{(0.55I_c + 1.68)} * \frac{q_t - \sigma_v}{p_a} \right]^{0.5} / \left(\frac{p_a}{\sigma'_{vo}} \right)^{0.25} \quad (\text{clayey layers}), \quad \text{Equation 43}$$

$$V_s = 28.27 * q_t^{0.137} * f_s^{0.013} * \sigma'_{vo}{}^{0.17} \quad (\text{sandy layers}). \quad \text{Equation 44}$$

3.3.3 Calibration of Estimated Shear Wave Velocities (V_s)

After estimating the shear wave velocities (V_s) from the CPT data via Equations 43 and 44, the MATLAB code calibrates the estimated V_s if field measurements for a soil profile are available. First, every V_s measurement is categorized into the appropriate segment of soil according to the depth of the measurement. Then, within each segment having one field V_s measurement, the MATLAB code calibrates all previously estimated V_s via a scaling factor such that they are consistent with field measurements. For a segment with at least two field V_s data points, the scaling factor varies linearly with respect to depth. This linearly varying scaling factor is obtained via a linear curve fit of the ratios between the in-situ V_s data within the segment and the corresponding CPT-estimated V_s at the same depths. For a segment without any field V_s data point, scaling factors are linearly interpolated from the two closest factors of adjacent segments. However, at the edge of the profile, segments without field V_s data are assigned a constant scaling factor equal to the closest scaling factor found in closest segment with field V_s . Additionally, near the surface, all V_s values must be at least 50 m/s to ensure realistic V_s values. Figure 16 shows an example of a calibration of V_s for a soil profile. Note that Equations 43 and 44 are not always able to reliably compute V_s estimations which accurately match field V_s values. In part, this is because V_s measurements are taken in the field by causing small strains, within the elastic range, to the soil, whereas CPT data are obtained by causing large strains and plastic failure to the soil. Thus, the availability of field V_s measurements is important to properly calibrate any CPT-based V_s estimations.

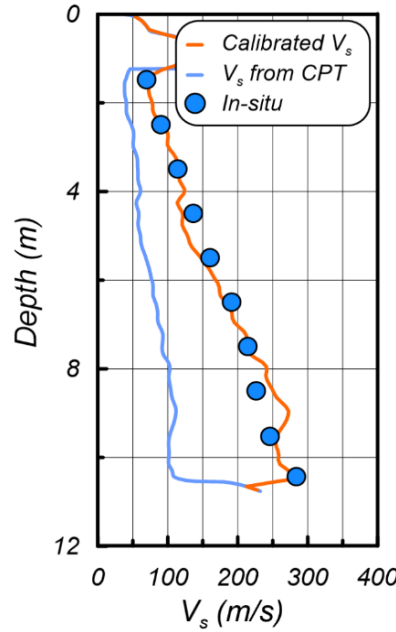


Figure 16 – Example of calibration of V_s measurements

3.3.4 Calibration of Estimated Undrained Shear Strength (S_u)

After estimating the undrained shear strength (S_u) of clay layers from CPT data via Equations 38, they are then calibrated to match field vane shear measurements if the latter are available. First, the vane shear test measurements are corrected according to Bjerrum (1973) to adjust for the testing method's tendency to overestimate the S_u of clays with increasing plasticity. As shown in Figure 17, the higher the PI of the clay, the more reduction to the vane shear measurement is required. Then, the MATLAB code identifies in which segment each vane shear measurement was made. For each clay segment with at least one measurement, the ideal N_{kt} value is calculated for each measurement, and the averaged ideal N_{kt} is used to re-calculate the adjusted S_u of all layers in said clay segment. In clay segments without any field S_u measurements, an interpreted N_{kt} value according to each layer's I_c value, as shown in Figure 18, is used to re-calculate each layer's S_u value. Figure 18 is obtained by compiling all the calibration factors for all vane shear measurements with respect to their I_c value found in clay segments available among all soil profiles. If a layer has an I_c within the limits of $2.52 \leq I_c \leq 2.75$, the following equation is used to estimate the layer's N_{kt} :

$$N_{kt} = -522.67I_c^3 + 4399.6I_c^2 - 12339I_c + 11544. \quad \text{Equation 45}$$

However, if $I_c < 2.52$ or if $I_c > 2.75$,

$$N_{kt}(I_c < 2.52) = N_{kt}(I_c = 2.52) = 24.65$$

$$N_{kt}(I_c > 2.75) = N_{kt}(I_c = 2.75) = 13.82.$$

Figure 19 shows an example of an adjustment of S_u for a soil profile.

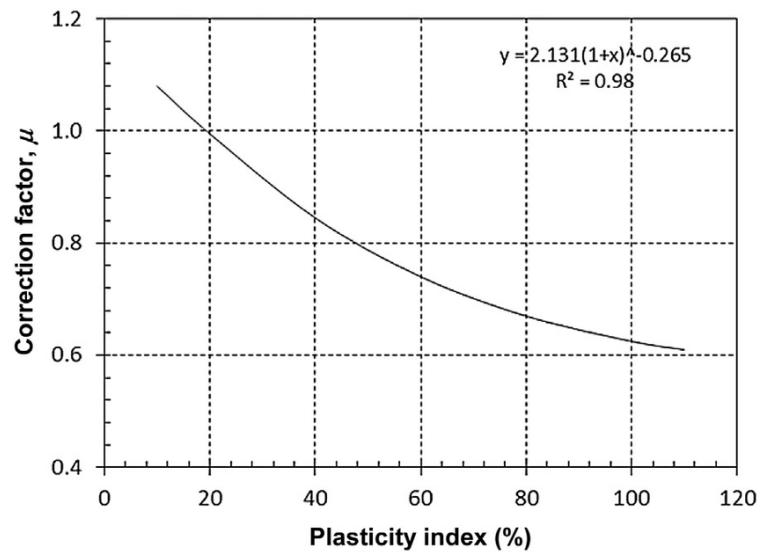


Figure 17 – Correction factors of Bjerrum (1973)

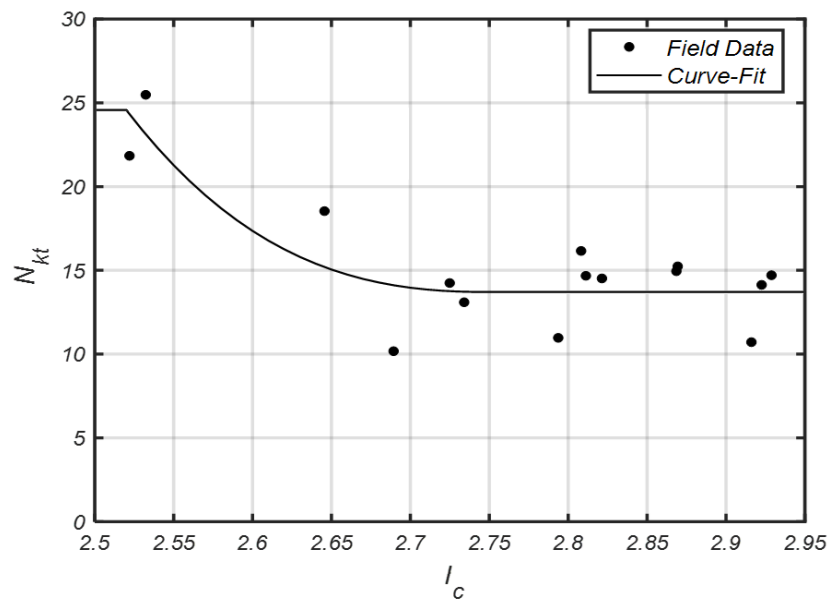


Figure 18 – Calibration of N_{kt}

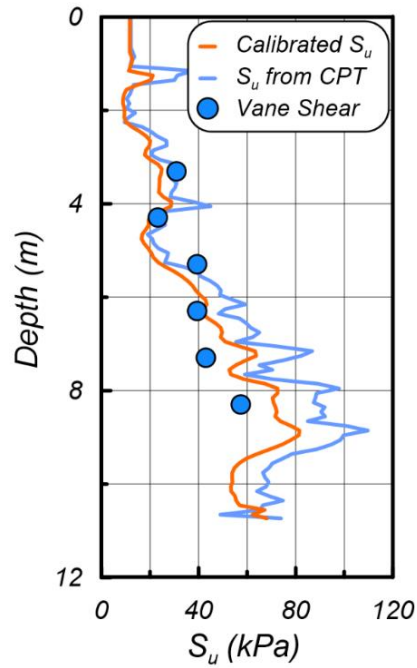


Figure 19 – Calibration of S_u

3.3.5 Grouping of Soil Layers for Analysis in Deepsoil

When Deepsoil performs an analysis for a soil profile, the more layers the profile has, the more time-consuming is the software's runtime due to the size of the stiffness and mass matrices that need to be computed and inverted. For the soil profiles in this project, initial layer thicknesses are either 1 cm, 2 cm or 2.5 cm depending on the interval of depth at which CPT measurements were taken. Regardless, in order to significantly decrease Deepsoil's runtime, the layers initially obtained based on the CPT data sheets are grouped together to reduce the total amount of layers for each profile. However, this must be done while respecting the Lysmer criterion (Kuhlemeyer and Lysmer, 1973) to prevent artificial damping, as well as respecting the maximum allowable thickness for each layer such that the input seismic motion can be adequately propagated in the soil:

$$\Delta H < \frac{V_{s,minimum}/f}{10}, \quad \text{Equation 46}$$

where a maximum propagable frequency of $f = 50 \text{ Hz}$ is used. This ensures that the maximum thickness of a layer would be less than one tenth of the smallest wavelength ($V_{s,minimum}/f$) that would be propagated in the soil. Note that due to the rock conditions in Eastern Canada, the

earthquakes recorded in Quebec are typically of higher frequencies than those recorded in Western Canada. If this study were done in Western Canada, a lower maximum propagable frequency could have been used, allowing for larger ΔH . For this reason, the MATLAB code which automates the creation of soil layers is particularly useful for soil profiles in Eastern Canada, as opposed to Western Canada, as the maximum allowable thicknesses required for soil layers are smaller, and therefore a larger amount of soil layers is needed for each soil profile.

3.3.6 Computation of Soil Dynamic Properties

For each finalized layer, dynamic properties are then computed according to the empirical relationships of Darendeli (2001). First, the modulus reduction curve of Darendeli for each layer with respect to the deformation is calculated as a function of PI , OCR and K_o following Equation 2 from section 2.2.2.1 using:

$$\varphi_1 = 0.0352, \quad \varphi_2 = 0.0010, \quad \varphi_3 = 0.3246, \quad et \quad \varphi_4 = 0.3483,$$

which are the average constant values from both sandy and clayey soils (Darendeli, 2001) that are also used by Deepsoil.

The input Darendeli modulus reduction curves are then strength adjusted according to Yee and al. (2013), as the Darendeli curves do not always produce implied shear strengths that match the maximum shear strengths of the layers. To do so, a user-defined transition strain (γ_1), as presented in section 2.2.5.2.3, is chosen such that the modulus reduction curve no longer follows the Darendeli curve for $\gamma > \gamma_1$ but rather the hybrid hyperbolic curve described by Equation 21, which is dependent of the soil shear strengths. To choose the γ_1 , the initial guess for γ_1 is each time set to be the value at which it would produce a τ_1 , as described by Equation 24, equal to 30% of the maximum shear strength of the layer. The initial guess of γ_1 must also not be more than 0.3%, which is approximately the maximum strain value usable in the Darendeli models. However, if γ_1 does not lead to a backbone curve implying a shear strength equal to or greater than 90% of the maximum shear strength of the soil layer, then γ_1 would be decreased iteratively until the implied shear strength is at least 90% of the maximum shear strength of the soil layer.

Next, the hysteretic damping curves with respect to the deformation ($D(\gamma)$) and the minimum damping (D_{min}) of Darendeli (2001) are calculated as a function of PI , OCR and K_o following Equation 5 and 6 respectively from section 2.2.2.1 using

$$\begin{aligned}\varphi_6 &= 0.8005, & \varphi_7 &= 0.0129, & \varphi_8 &= -0.1069, & \varphi_9 &= -0.2889, \\ \varphi_{10} &= 0.2919, & \varphi_{11} &= 0.6329, & \varphi_{12} &= -0.0057,\end{aligned}$$

which are also the average constant values from both sandy and clayey soils (Darendeli, 2001) and used by Deepsoil. The hysteretic damping according to the Masing rules ($D_M(\gamma)$), required by Equation 5, is calculated following Equation 8 from section 2.2.2.1.

Then, the resulting empirically obtained input Darendeli curves will need to be curve-fitted mathematically to be interpretable by NL models in Deepsoil v.7.0. The constitutive model chosen in this study is the ARCS model (Yniesta et al., 2017), which accurately match the input modulus reduction curve with cubic splines and the input unloading/reloading damping curve via a coordinate transformations technique as described in section 2.2.5.2.6. The ARCS model is implemented in Deepsoil as a user-defined model, and imported as a dynamic link library (*dll*). The model automatically curve-fits the input of the Darendeli curves into Deepsoil as described in section 3.3.7.

3.3.7 Exporting the Soil Layers' Properties to Deepsoil

Finally, when the properties of all layers of the soil profile to be analysed are completed, the MATLAB code exports the columns of thicknesses, unit weights, shear wave velocities, minimum damping values, strength adjusted modulus reduction curves and damping curves into an EXCEL Worksheet. From there, the user can copy-paste the columns of information into Deepsoil's "Soil Profile Definition" step, thereby significantly decreasing the time required to perform a NL analysis on Deepsoil.

3.4 An Example of Using the MATLAB Code

To demonstrate the simplicity in the usage of the MATLAB code, an example is shown here for soil profile #29. The input into the MATLAB code include the raw CPT data available for this

profile, of which an excerpt is shown in Table 3, and the water table level identified at a depth of 1.05 m. Field V_s and S_u measurements were not available for soil profile #29.

Table 3 – Raw CPT data from soil profile #29

Depth (m)	Tip resistance (MPa)	Local friction (MPa)
...	...	...
0.51	0.306	0.011
0.52	0.426	0.005
0.53	0.431	0.004
0.54	0.421	0.004
0.55	0.373	0.005
0.56	0.341	0.008
0.57	0.331	0.008
0.58	0.314	0.011
0.59	0.324	0.012
0.6	0.341	0.013
0.61	0.35	0.014
0.62	0.357	0.015
0.63	0.357	0.015
0.64	0.364	0.016
0.65	0.358	0.02
0.66	0.357	0.024
0.67	0.365	0.023
0.68	0.369	0.021
0.69	0.383	0.02
0.7	0.393	0.018
0.71	0.409	0.017
0.72	0.416	0.016
0.73	0.442	0.017
0.74	0.463	0.018
0.75	0.501	0.018
0.76	0.555	0.017
0.77	0.616	0.016
0.78	0.745	0.017
0.79	0.81	0.018
0.8	0.844	0.022
0.81	0.865	0.023
0.82	0.893	0.022
0.83	0.91	0.021
0.84	0.918	0.021
0.85	0.907	0.022
0.86	0.876	0.023
0.87	0.855	0.024
0.88	0.841	0.027
0.89	0.811	0.032
0.9	0.771	0.036
...	...	...

Table 7 – Output of the MATLAB code for soil profile #29 (part 4 – damping, D)

D1 (%)	D2 (%)	D3 (%)	D4 (%)	D5 (%)	D6 (%)	D7 (%)	D8 (%)	D9 (%)	D10 (%)	D11 (%)	D12 (%)	D13 (%)	D14 (%)	D15 (%)
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
0.031	0.070	0.159	0.359	0.796	1.71	3.49	6.44	10.35	14.35	17.53	19.48	20.22	20.05	19.31
0.028	0.064	0.146	0.329	0.731	1.58	3.24	6.06	9.90	13.95	17.25	19.33	20.19	20.10	19.41
0.026	0.059	0.134	0.302	0.672	1.46	3.02	5.71	9.47	13.54	16.95	19.17	20.15	20.15	19.50
0.025	0.056	0.127	0.287	0.640	1.39	2.89	5.50	9.22	13.30	16.77	19.07	20.12	20.17	19.56
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

CHAPTER 4 DEEPSOIL SIMULATION RESULTS

4.1 Procedures of Using Deepsoil for Nonlinear (NL) Analyses

In this project, the software Deepsoil V7.0 (Hashash et al. 2017) was used to carry out the nonlinear (NL) ground response analysis. This software is well established in the research field, but also frequently used in industry practices on the West coast of North America, and increasingly used elsewhere. In addition to NL analysis, Deepsoil also has the ability to carry out linear and equivalent linear (EL) analysis. The following subsections present the procedure to perform a NL analysis in Deepsoil.

4.1.1 Analysis Type Definition

First, in the “Analysis Type Definition step”, the total stress analysis is chosen over the effective stress analysis. This is because the pore water generation occurring typically in liquefaction is not considered in this project, but also the accuracy of the predictions of effective stress models for ground response analysis is not as well validated. Furthermore, the ARCS model (Yniesta et al. 2017) is chosen to curve-fit the input modulus reduction and damping (MRD) curves computed using Darendeli’s equations (Darendeli 2001), and modified using the Yee et al. (2013) procedure. Since the ARCS model is not a built-in model available in Deepsoil, it is imported into Deepsoil as a user-defined model in the form of a dynamic link library (dll). The ARCS model was chosen as it can accurately curve-fit all the input MRD curves.

4.1.2 Soil Profile Definition

The second step is the definition of the soil profile, where all the soil layers and their corresponding properties, produced by the MATLAB code, are imported to define all the layers of the soil profile being evaluated. The position of the water table level and the properties of the bedrock are also defined at this step.

4.1.2.1 Supplemental Information of the Soil Profiles Analysed

In this project, 33 anonymous soil profiles were provided by WSP Inc. 16 profiles are of class E type soil and 17 are of class D type soil, with an average V_{s30} across all soil profiles of 164 m/s. It is important to note that the total depth for most of the profiles analysed are lower than 30 m, which means that the V_{s30} for most of the soil profiles are equivalent to their average V_s without consideration for the bedrock. Figure 20 shows the distribution of V_{s30} among the profiles analysed in this project. Table 8 shows each profile's total depth, average V_s , soil class and natural site period (T_0).

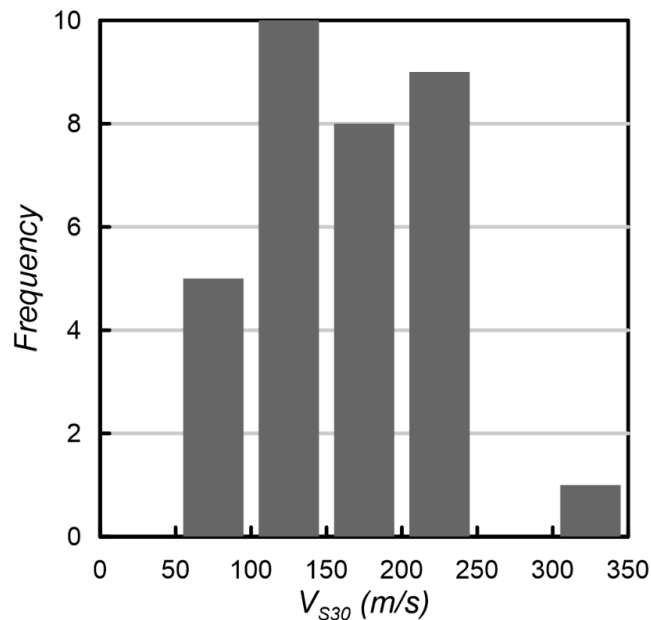


Figure 20 – Distribution of V_{s30} for the soil profiles analysed (Zhang et al., 2020)

Table 8 – Additional information on each soil profile analysed

Profile #	Total Depth (m)	Average V_s (m/s)	Soil Class	Natural Site Period, T_0 (s)
1	41.74	174	E	0.9602
2	6.90	247	D	0.1117
3	10.76	139	E	0.3088
4	9.67	342	D	0.1130
5	4.66	145	E	0.1283
6	8.63	134	E	0.2573
7	7.52	185	D	0.1627
8	20.51	227	D	0.3615
9	22.22	198	D	0.4497
10	22.63	234	D	0.3872
11	22.91	222	D	0.4135
12	22.60	228	D	0.3973
13	16.35	116	E	0.5632
14	17.24	108	E	0.6361
15	16.36	106	E	0.6190
16	16.89	107	E	0.6329
17	16.44	114	E	0.5749
18	10.44	184	D	0.2266
19	10.68	183	D	0.2338
20	13.05	184	D	0.2835
21	20.65	211	D	0.3908
22	19.77	217	D	0.3650
23	22.62	225	D	0.4026
24	18.14	217	D	0.3339
25	15.93	198	D	0.3217
26	13.88	194	D	0.2856
27	4.68	129	E	0.1455
28	3.17	100	E	0.1274
29	10.17	81	E	0.5027
30	15.65	96	E	0.6490
31	3.42	113	E	0.1208
32	2.6	100	E	0.1037
33	5.76	100	E	0.2315

4.1.2.2 Bedrock Properties Used

To represent a bedrock potentially found in Quebec, the bedrock under each soil profile in this study is assigned a shear wave velocity (V_s) of 1600m/s (Nastev et al., 2016), a unit weight (γ) of 25kN/m³ (Motazedian et al., 2010) and a damping ratio of 2% (Hashash et al. 2017). Note that the

shear wave velocity of the bedrock in a ground response simulation performed using Deepsoil has a limited influence.

4.1.3 Input Motion Selection

The third step is the selection of the input motions. A total of 48 input seismic motions, consistent with the seismic hazard in Montréal, are selected from the master's thesis of Huo-Kang (2021). These input motions are selected from the NGA-West2 database (Ancheta et al., 2014) due to lack of available recordings in the Eastern Canada, a stable continental region. They are also selected based on a deaggregation analysis of the seismic hazard, a process which divides the total seismic hazard into different source contributions based on periods, distance and magnitude. Six scenarios, as listed in Huo-Kang and Yniesta (2020), can be combined to represent the total seismic hazard in Montréal. These scenarios were separated into either the short period interval (0.01s – 0.2s), medium period interval (0.2s – 1.0s) or long period interval (1.0s – 10s). However, due to time constraints on this project, only the most influential scenario within each of the three period intervals is selected in this project, as listed in Table 9.

Table 9 – Description of the 3 Scenarios of Input Motions Used in this Study

Criteria	Scenario 1	Scenario 2	Scenario 3
Magnitude of input motion within each scenario	5 – 6	6 – 7	6.5 – 7.5
Distance between location of recording and the source (km)	0 – 20	20 – 40	20 – 40
Period interval of interest (secs)	0.01 – 0.2	0.2 – 1.0	1.0 – 10

Each seismic input motion selected for each scenario needs to abide by the following criteria:

- i. has a magnitude falling within the range of magnitude specified for the scenario,
- i. recorded on a bedrock within a certain distance, specified for the scenario, from the source of the seismic event,

- ii. any scaling factor used to scale the original recording to the target spectrum over the period interval of interest, specified for the scenario, must be greater than 0.5 but lower than 2,
- iii. the average input motion must be greater than 90% of the target spectrum within the period interval of interest specified by the scenario.

Figure 21 summarizes the spectral accelerations (S_g) of all input motions used in each scenario and the target seismic hazard in Montréal ($S_T(T)$) at the period interval of interest corresponding to each scenario. Table 10, 11 and 12 provides the names, magnitude and recording station of the 22, 14 and 12 input motions used in Scenario 1, 2 and 3 respectively, obtained from the NGA-West2 database (Ancheta et al. 2014).

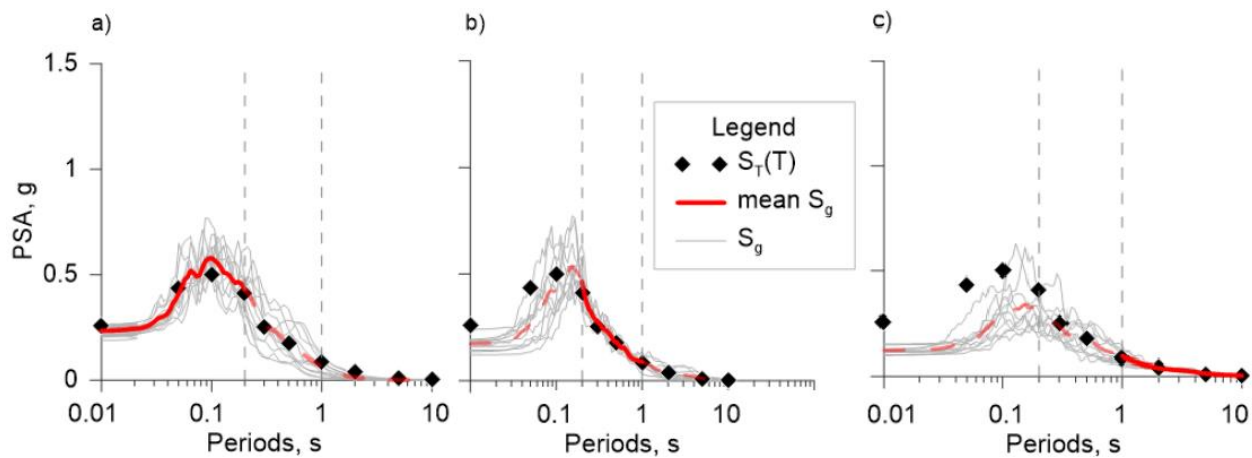


Figure 21 – Input motions of a) Scenario 1, b) Scenario 2 and c) Scenario 3 compared to the target seismic hazard (Huo-Kang and Yniesta, 2020)

Table 10 – Input Seismic Motions Used in Scenario 1

Record Sequence Number	Name of Seismic Event	Magnitude	Recording Station
98 Horizontal 1	Hollister-03, USA	5.14	Gilroy Array #1
98 Horizontal 2	Hollister-03, USA	5.14	Gilroy Array #1
156 Horizontal 1	Norcia, Italy	5.90	Cascia
156 Horizontal 2	Norcia, Italy	5.90	Cascia
381 Horizontal 1	Coalinga-02, USA	5.09	Oil Fields Fire Station
381 Horizontal 2	Coalinga-02, USA	5.09	Oil Fields Fire Station
1642 Horizontal 1	Sierra Madre, USA	5.61	Cogswell Dam – Right Abutment
1642 Horizontal 2	Sierra Madre, USA	5.61	Cogswell Dam – Right Abutment
2019 Horizontal 1	Gilroy, USA	4.90	Gilroy – Gavilan Coll.
2019 Horizontal 2	Gilroy, USA	4.90	Gilroy – Gavilan Coll.
3553 Horizontal 1	Fruili-03, Italy	5.50	Tarcento
3553 Horizontal 2	Fruili-03, Italy	5.50	Tarcento
3768 Horizontal 1	Northridge-06, USA	5.28	Tarzana – Club House
3768 Horizontal 2	Northridge-06, USA	5.28	Tarzana – Club House
4369 Horizontal 1	Umbria Marche (aftershock 1), Italy	5.5	Nocera Umbra
4369 Horizontal 2	Umbria Marche (aftershock 1), Italy	5.5	Nocera Umbra
4377 Horizontal 1	Umbria Marche (aftershock 8), Italy	5.2	Borgo-Cerreto Torre
4377 Horizontal 2	Umbria Marche (aftershock 8), Italy	5.2	Borgo-Cerreto Torre
8169 Horizontal 1	San Juan Bautista, USA	5.17	San Andreas Geophysical Obs.
8169 Horizontal 2	San Juan Bautista, USA	5.17	San Andreas Geophysical Obs.
8571 Horizontal 1	El Mayor-Cucapah, Mexico	7.20	Padua
8571 Horizontal 2	El Mayor-Cucapah, Mexico	7.20	Padua

Table 11 - Input Seismic Motions Used in Scenario 2

Record Sequence Number	Name of Seismic Event	Magnitude	Recording Station
72 Horizontal 1	San Fernando, USA	6.61	Lake Hughes #4
72 Horizontal 2	San Fernando, USA	6.61	Lake Hughes #4
81 Horizontal 1	San Fernando, USA	6.61	Pearblossom Pump
81 Horizontal 2	San Fernando, USA	6.61	Pearblossom Pump
288 Horizontal 1	Irpinia-01, Italy	6.90	Brienza
288 Horizontal 2	Irpinia-01, Italy	6.90	Brienza
511 Horizontal 1	N. Palm Springs, USA	6.06	Anza - Red Mountain
511 Horizontal 2	N. Palm Springs, USA	6.06	Anza - Red Mountain
598 Horizontal 1	Whittier Narrows-01, USA	5.99	Big Tujunga, Angeles Nat F
598 Horizontal 2	Whittier Narrows-01, USA	5.99	Big Tujunga, Angeles Nat F
782 Horizontal 1	Loma Prieta, USA	6.93	Monterey City Hall
782 Horizontal 2	Loma Prieta, USA	6.93	Monterey City Hall
954 Horizontal 1	Northridge-01, USA	6.69	Big Tujunga, Angeles Nat F
954 Horizontal 2	Northridge-01, USA	6.69	Big Tujunga, Angeles Nat F

Table 12 – Input Seismic Motions Used in Scenario 3

Record Sequence Number	Name of Seismic Event	Magnitude	Recording Station
782 Horizontal 1	Loma Prieta, USA	6.93	Monterey City Hall
782 Horizontal 2	Loma Prieta, USA	6.93	Monterey City Hall
1021 Horizontal 1	Northridge-01, USA	6.69	Lake Hughes #4 - Camp Mend
1021 Horizontal 2	Northridge-01, USA	6.69	Lake Hughes #4 - Camp Mend
4205 Horizontal 1	Niigata, Japan	6.63	NIG015
4205 Horizontal 2	Niigata, Japan	6.63	NIG015
4854 Horizontal 1	Chuetsu-oki, Japan	6.80	Nadachiku Joetsu City
4854 Horizontal 2	Chuetsu-oki, Japan	6.80	Nadachiku Joetsu City
4869 Horizontal 1	Chuetsu-oki, Japan	6.80	Kawaguchi
4869 Horizontal 2	Chuetsu-oki, Japan	6.80	Kawaguchi
6948 Horizontal 1	Darfield, New Zealand	7.00	OXZ
6948 Horizontal 2	Darfield, New Zealand	7.00	OXZ

4.1.4 Viscous/Small-Strain Damping Definition

In the fourth step, the small strain damping scheme is defined. In this study, the frequency independent damping formulation (Phillips and Hashash 2009), as presented in section 2.2.5.3.2, is used to introduce small strain damping. The use of the frequency independent damping

formulation used in conjunction with the ARCS model helps smooth out numerical noise that might arise during the simulations. The frequency-independent formulation is chosen over the Rayleigh method (Park and Hashash, 2008), as presented in section 2.2.5.3.1, as it eliminates the disadvantages of the latter by eliminating the higher damping at low and high frequencies and the lower damping between the intermediate frequencies. In addition, the damping matrix is updated at each time step as a function of the stiffness matrix, producing more realistic analyses but increasing the computational time.

4.1.5 Analysis Control Definition

The last step is to define the different options available to solve the equation of motion. A flexible strain increment option is chosen, where a maximum strain increment of 0.005% is specified, according to the default recommendation by the Deepsoil software. In addition, the implicit Newmark Beta method is chosen for the time integration method, and the linear time domain interpolation is chosen for the time-history interpolation method.

4.1.6 Exporting Results

After running the analysis, results can be exported into EXCEL files, or TEXT files, from which data of interest, such as spectral acceleration and acceleration time history at different depths, can be extracted and further analysed.

4.2 Analysis of Results

In this research project, seismic NL ground analyses were performed on 33 soil profiles located in Quebec. For each of the 48 input motions for every soil profile, the seismic amplification factors (AF) with respect to periods (T) from the NL analyses are obtained as follows:

$$AF(T) = \frac{S_{a_s}(T)}{S_{a_i}(T)} \quad \text{Equation 47}$$

The spectral response of the seismic input motion ($S_{a_i}(T)$) is obtained from the EXCEL file exported directly from Deepsoil after analysis. However, the spectral response at the surface ($S_{a_s}(T)$) is derived from the resulting acceleration time history at the surface, exported from Deepsoil, filtered with the same filters used for each corresponding input motion. For each motion,

AF are calculated for 113 spectral periods, distributed from the period of 0.01s to 10s. Then, AF from all motions belonging to the same scenario at every spectral period are averaged for each soil profile, with plus or minus one standard deviation (σ), and compared to the corresponding AF obtained from the NBCC 2015. Typically in industry practices, only one set of AF for the uniform hazard design spectrum (UHS) at the site location is computed when using the NBCC 2015 approach. However, in this project, AF are computed for each input motion in this study by both the NL analysis and the NBCC 2015 approach such that a fair comparison of AF for each motion, with different PGA, can be made between the two methods.

4.2.1 Summary of the Three Cases of Comparisons Found

The comparison of the results of ground response analysis and the amplification factors provided by the NBCC show that three cases emerge as presented in this section.

4.2.1.1 Case 1

The most frequent situation (Case 1), having an average occurrence of 21 of the 33 soil profiles among all three scenarios, is where the NL analysis yielded a peak in AF located around the natural site period which is significantly higher than the AF from the NBCC 2015. Note that the natural site period is the first fundamental mode of the system, calculated from the modal decomposition of the site's V_s profile. Figure 22 and Figure 23 show two examples of soil profiles belonging to Case 1. Among results belonging to Case 1, as shown in Figure 23, it was observed that the NL analysis can sometimes yield AF higher than those from the NBCC 2015 at periods shorter than the natural site period as well. As for the results at large periods, the NBCC 2015 typically yielded AF higher than those from the NL analysis. However, it is worth noting that the NL analysis results at large periods may not be reliable due to the impact of multidimensional long period effects, such as basin effects, which are not considered in 1D ground response analysis. Nevertheless, for the majority of the soil profiles across all three scenarios (Case 1), using the NBCC 2015 to design against seismic hazard would be potentially unsafe, especially around the natural site period.

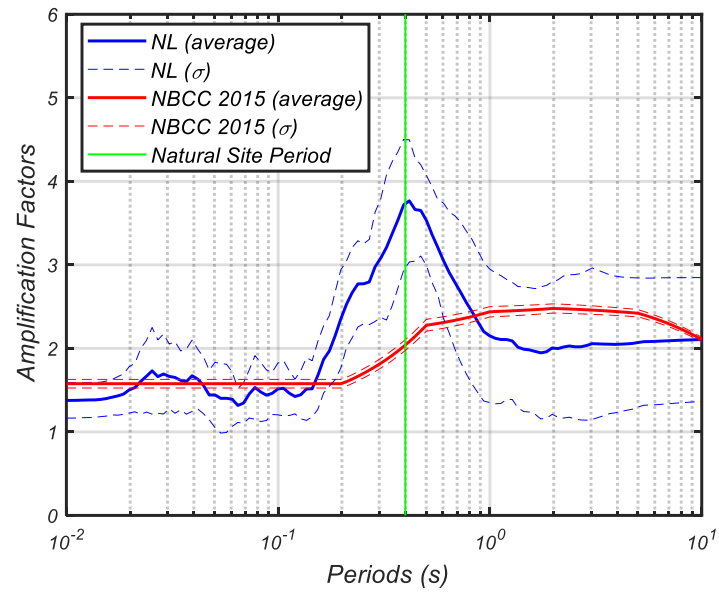


Figure 22 – Amplification Factors from NL Analysis Versus NBCC 2015 – Case 1 (Scenario 1, soil profile #11)

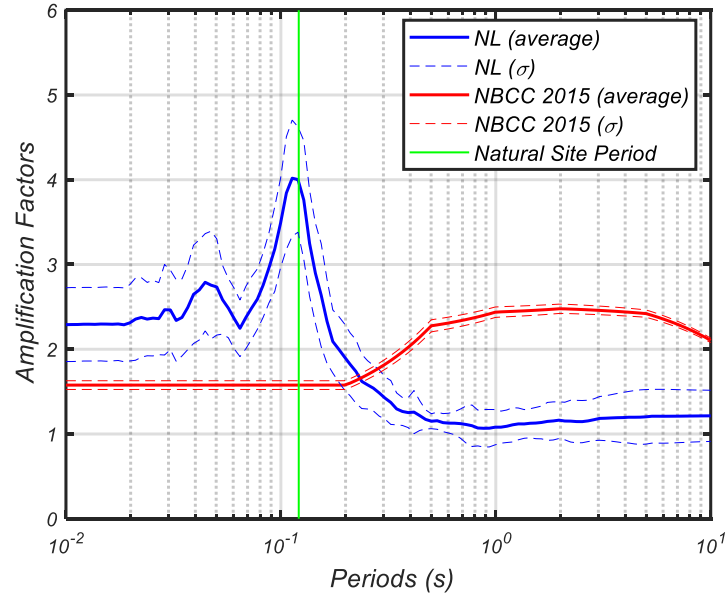


Figure 23 - Amplification Factors of NL Analysis Versus NBCC 2015 – Case 1 (Scenario 1, soil profile #4)

4.2.1.2 Case 2

In contrast to Case 1, the least frequently occurring situation is Case 2, an average occurrence of four of the 33 soil profiles among all three scenarios, where the NBCC 2015 predicts AF significantly higher than those from the NL analysis at all periods, as shown by an example in Figure 24. Thus, for certain soil profiles (Case 2), the NBCC 2015 can occasionally be overly conservative in estimating the amplification of soils during seismic events. It is observable that at times, as seen in Figure 24, the peak in the result of the NL analysis can be at a period slightly larger than the natural site period, which is due to the loss of stiffness of the system under cyclic loading, causing a corresponding gradual increase in the natural site period, which is often referred to as period lengthening. Note therefore that the natural site period used in this study is an elastic parameter, calculated based on the V_s profile.

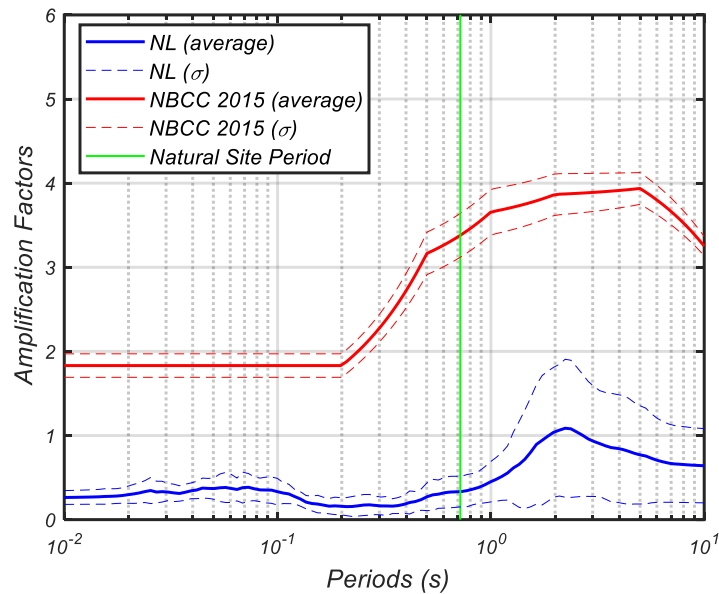


Figure 24 - Amplification Factors of NL Analysis Versus NBCC 2015 – Case 2 (Scenario 1, soil profile #15)

4.2.1.3 Case 3

Case 3 represents on average eight out of the 33 soil profiles across all three scenarios, for which the NBCC 2015 predicts only slightly higher AF than those from the NL analysis. Therefore, the NL analysis is sometimes fairly consistent with the NBCC 2015. However, as shown in an example

of a soil profile belonging to Case 3 in Figure 25, certain individual earthquake motions can still generate AF higher than those from the NBCC 2015 as demonstrated by the standard deviations of the NL analysis. It is also worth noting that in Figure 25, the peak amplification according to the NL analysis is also found around the natural site period.

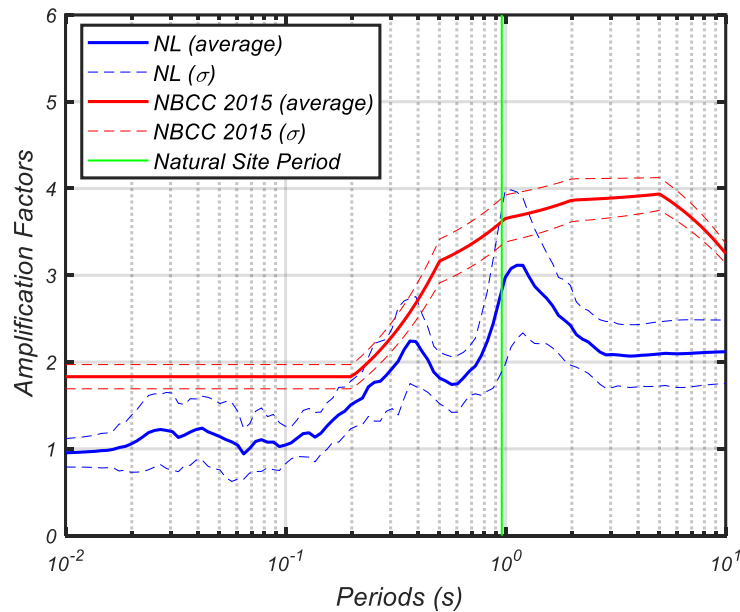


Figure 25 - Amplification Factors of NL Analysis Versus NBCC 2015 – Case 3 (Scenario 1, soil profile #1)

4.2.2 Amplification Factors (AF) for Scenario 1

All AF versus periods plots for all the soil profiles in Scenario 1 can be found in Appendix A. In Scenario 1, in which the input motions used are specified in Table 10, 19 out of the 33 soil profiles resulted in Case 1. The peak in difference between the NL analysis and the NBCC 2015 approach was always most noticeable around the natural site period, where the NL analysis can suggest AF up to more than twice as high as the factors suggested by NBCC 2015, as seen by an example from Scenario 1 in Figure 23. For nine of the soil profiles of Scenario 1 belonging to Case 1, the NL analysis predicted significantly higher AF than those from the NBCC 2015 at periods lower than the natural site period as well. As demonstrated by an example of Case 1 from Scenario 1 in Figure 22, although the NBCC 2015 can at times produce AF similar to those from the NL analysis at most of the periods, the amplification at the natural site period can still potentially be

underestimated by the NBCC 2015 approach. However, at large periods for all soil profiles belonging to Case 1, the NL analysis typically suggest AF significantly lower than those suggested by the NBCC 2015. The type of soils belonging to Case 1 in Scenario 1 include 15 soil profiles as class D type soils and 4 profiles as class E type soils. This shows that it is potentially more likely for stiffer soils (class D) to cause amplifications that are underestimated by the NBCC 2015, particularly around the natural site period.

In Scenario 1, five out of the 33 soil profiles account for Case 2. In fact, in the example belonging to Scenario 1 in Figure 24, the NL analysis yielded AF about 4 times lower than the factors predicted by the NBCC 2015. Thus, for certain profiles, the NBCC 2015 can be overly conservative in estimating the amplification of soils during seismic events. All 5 soil profiles in Case 2 of Scenario 1 are of class E type soils. However, this does not mean that all softer soils (class E) would fall under Case 2, since some class E soils fell under Case 1 and Case 3, which shows the complexity of the ground response analysis.

In Scenario 1, nine out of the 33 soil profiles account for Case 3, showing that in some cases, the NBCC 2015 is fairly consistent with the NL analysis. However, as seen by the standard deviation (σ) in the example belonging to Scenario 1 in Figure 25, specific motions might exceed the amplification provided in the NBCC 2015. Among the 9 soil profiles belonging to Case 3 in Scenario 1, 2 soil profiles are of class D type soils while 7 soil profiles are of class E type soils. This suggests that it may be more likely for softer soils (class E) to cause amplifications that are fairly well estimated by the NBCC 2015.

4.2.3 Amplification Factors (AF) for Scenario 2

All AF versus periods plots for all soil profiles in Scenario 2 can be found in the Appendix B. In Scenario 2, 14 different input motions, as specified in Table 11, were used on the 33 soil profiles of this project. Among the results in Scenario 2, 22 out of the 33 profiles belonged to Case 1, in which the NL analysis yielded AF higher than those from the NBCC 2015 particularly around the natural site period. The occurrence that AF from the NL analysis were larger than AF from the NBCC 2015 at periods shorter than the natural site period is present for 19 of these soil profiles in Scenario 2. Similarly to Scenario 1, the NBCC 2015 seems to be conservative when compared to

the NL analysis at large periods in Scenario 2. Among these 22 soil profiles, 17 are of class D type soils, while only five are of class E type soils, also showing that stiffer soils' (class D) amplifications may more likely be underestimated by the NBCC 2015, particularly around the natural site period. The higher number of soil profiles belonging to Case 1 in Scenario 2 in comparison with Case 1 soil profiles in Scenario 1 could be because Scenario 2 has input seismic motions with stronger magnitude on average. The average magnitude of input motions for Scenario 1 and Scenario 2 are 5.5 and 6.5 respectively. However, the reasons for this potential correlation are unclear and further research on this could be beneficial.

Consequently, there are only three soil profiles belonging to Case 2 in Scenario 2, and all three are of class E type soils. Just like in Scenario 1, the NL analysis results in Scenario 2 in Case 2 are also sometimes up to four times smaller than the results from the NBCC 2015; however, it is noticeable that in Scenario 2, there are slightly less differences between the NL analysis and the NBCC 2015, particularly at the periods where the peak AF of the NL analysis are situated as shown in Figure 26.

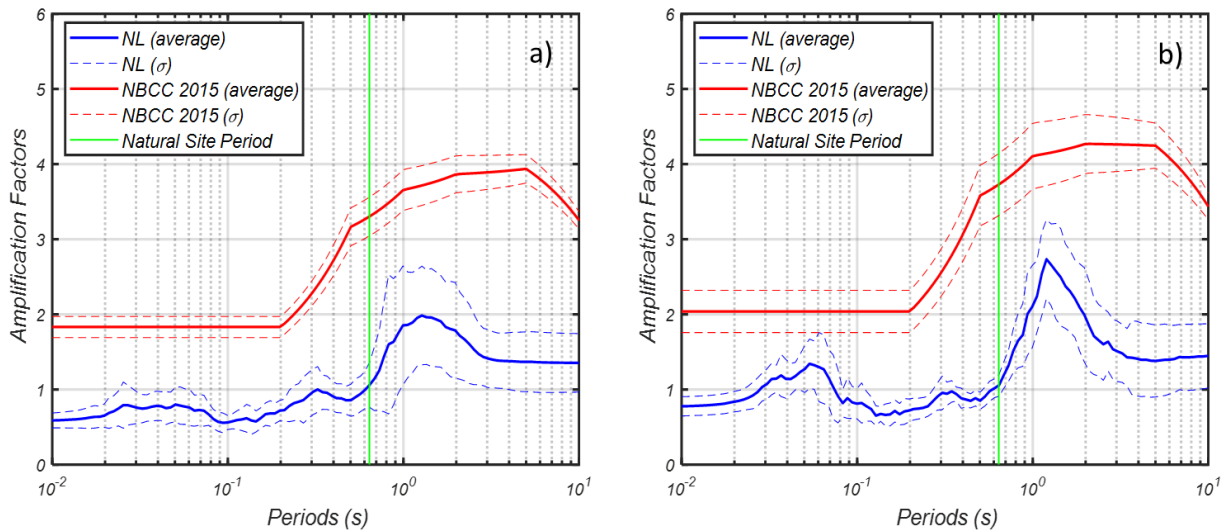


Figure 26 – Comparison of AF differences between NL analysis and NBCC 2015 at the natural site period between a) Scenario 1 and b) Scenario 2 – Case 2 (soil profile #1)

Likewise, since the stronger input motions of Scenario 2 have potentially caused more soil profiles to yield AF underpredicted by the NBCC 2015, only eight soil profiles belong to Case 3. Typically in Scenario 2, NL analysis in Case 3 yield AF slightly closer to AF from the NBCC 2015 around

the natural site period when compared to Case 3 of Scenario 1. A comparison example is shown in Figure 27 for soil profile #1. All eight profiles are of class E type soils.

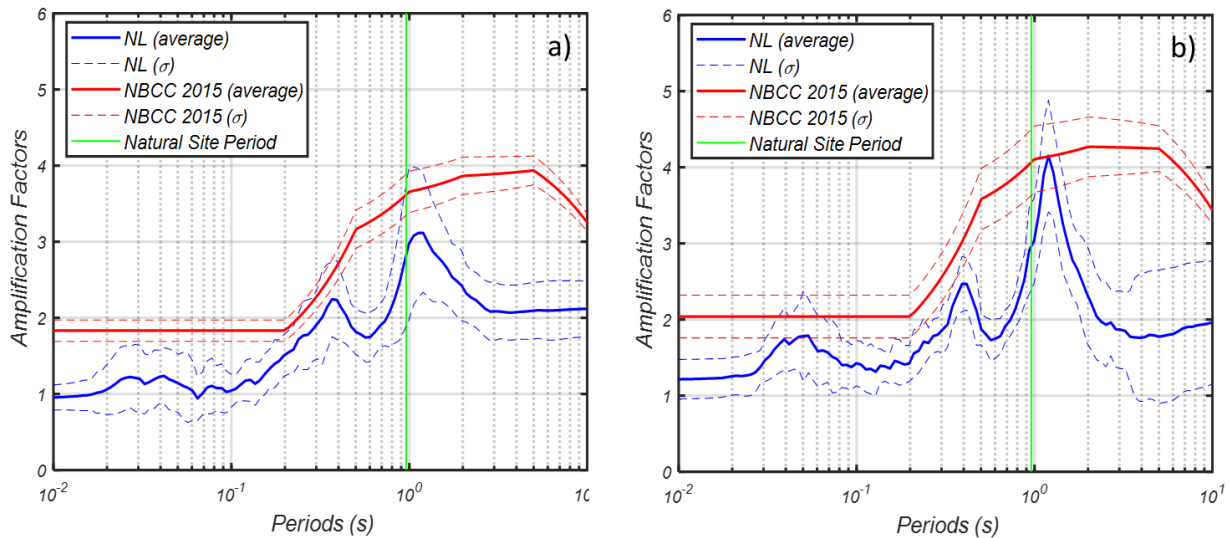


Figure 27 – Comparison of AF differences between NL analysis and NBCC 2015 at the natural site period between a) Scenario 1 and b) Scenario 2 – Case 3 (soil profile #1)

In short, the main differences observed between Scenario 2 and Scenario 1 are that the NL analysis of Scenario 2 yielded AF slightly higher than those observed in Scenario 1 particularly around the natural site period. Similarly, the AF from the NBCC 2015 for Scenario 2 were also slightly higher than those of Scenario 1 due to the PGA of input motions from Scenario 1 being higher than those from Scenario 2.

4.2.4 Amplification Factors (AF) for Scenario 3

All AF versus periods plots for all soil profiles in Scenario 3 can be found in the Appendix C. For Scenario 3, 22 out of 33 belonged to Case 1. These 22 profiles were the same profiles in Case 1 of Scenario 2. Just like in the other two Scenarios, in Scenario 3, the AF from the NL analysis can also be significantly higher than factors from the NBCC 2015 at periods shorter than the natural site period. Similarly, the NBCC 2015 is typically conservative at large periods. Out of the 22 profiles of Case 1, 17 are of class D type soils and five are of class E type soils, just like in Scenario 2.

In Scenario 3, three soil profiles belonged to Case 2, which are the same three profiles in Case 2 of Scenario 2. In this last Scenario, the NBCC 2015 can also predict, at certain periods, AF which are about four times higher than the AF obtained from the NL analysis. All three profiles in Case 2 of Scenario 3 are of class E type soils as well. Likewise, Case 3 of Scenario 3 also has the same eight soil profiles as were in Scenario 2. All eight profiles in Case 3 of Scenario 3 are also of class E type soils.

Overall, the NL analysis results from Scenario 3 are more similar to those from Scenario 2 than those from Scenario 1, potentially due to the similar average magnitude of input motions between Scenario 2 and 3. The average magnitude of input motions for Scenario 2 and Scenario 3 are 6.5 and 6.8 respectively. The differences observed in the AF between the NL analysis and the NBCC 2015 in Scenario 3 are also similar to those differences observed in Scenario 2.

4.2.5 Residuals Analysis for Scenario 1

Another way to examine the differences between the NL analysis and the NBCC 2015 approach is to compute residuals. Residuals are defined as the following:

$$\text{Residual} = \ln \left(\frac{AF \text{ from nonlinear analysis}}{AF \text{ from the NBCC 2015}} \right) \quad \text{Equation 48}$$

A residual value that is positive would indicate that the NL analysis yielded an AF larger than the factor from the NBCC 2015. Similarly, a negative residual value would indicate that the NBCC 2015 yielded an AF more conservative than the AF from the NL analysis. Figure 28 displays the residuals of all the soil profiles from all the motions in Scenario 1. It can be observed that on average for most of the periods, the NBCC 2015 is seen to be conservative when compared to the results from the NL analysis, which is especially true at periods larger than 1 second, the period after which the one standard deviation of the residuals are also negative values. This was expected since the NBCC 2015 factors are defined to account partially for some site responses that are not considered by 1D ground response analysis, such as basin effects. However, Figure 28 shows that for more than 25% of all the motions of all the soil profiles, NL analysis yield AF larger than the factors from the NBCC 2015 at periods shorter than 1s. Notice from Table 8 that the natural site period (T_0) of all the soil profiles are smaller than 1s, ranging from 0.1070s to 0.9599s, which may explain how the AF from NL analysis are higher at the shorter periods.

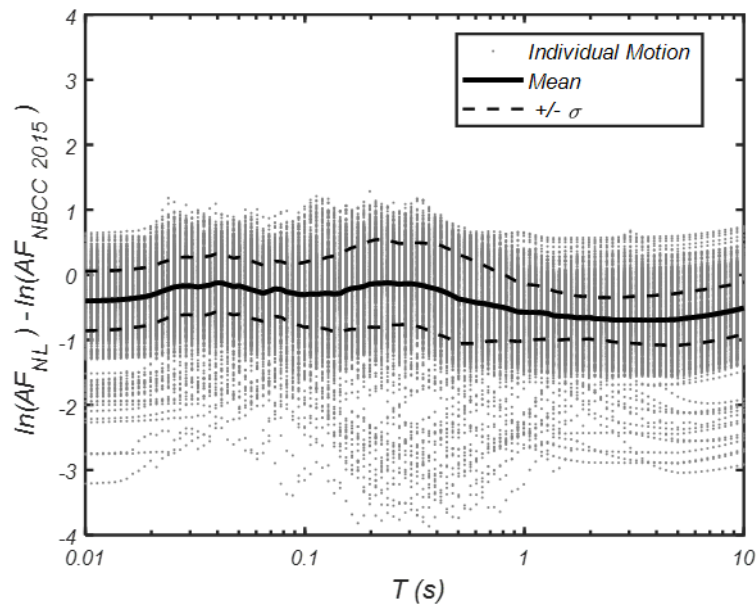


Figure 28 – Residuals of Amplification Factors for All Motions and All Soil Profiles (Scenario 1)

Figure 29 also displays the residuals from all the motions in Scenario 1 for all the soil profiles but plotted against the period normalized by the natural site period (T/T_0). From Figure 29, it is clearly observable that the peak of the residuals is found around $T/T_0 = 1$, the natural site period (T_0), where the NBCC 2015 underpredicts the soil seismic amplification. To be more specific, the peak is found at a period slightly longer than T_0 , which is potentially due to the soil softening effect during seismic loading.

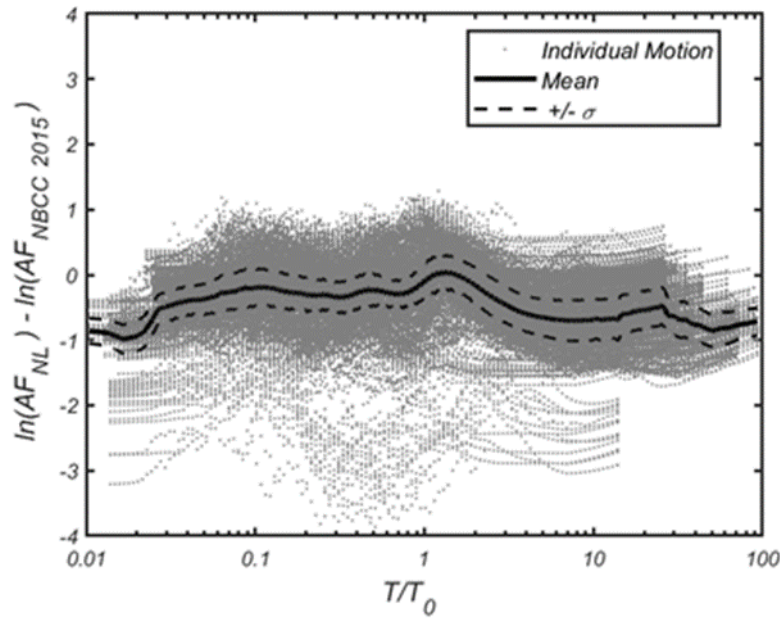


Figure 29 – Residuals of Amplification Factors for All Motions and All Soil Profiles Versus Normalized Periods (Scenario 1)

Note that the residuals found in Scenario 1 are relatively large for most of the data points in either Figure 28 or Figure 29. In a study by Kaklamanos et al. (2015), residuals computed between NL analysis and measured ground responses in earthquakes were comparatively lower. This suggests that the NL analysis method, in comparison to the NBCC 2015 approach, would be better in predicting more accurate soil AF during seismic events.

4.2.6 Residuals Analysis for Scenario 2

In Scenario 2, a similar behaviour is observed as was in Scenario 1. Most of the data points show a conservatism of the NBCC 2015 approach in predicting AF when compared to the NL method. However, in Scenario 2, between the periods of 0.02s and 0.08s as seen in Figure 30, the residuals show on average consistent predictions from both methods. Furthermore, at large periods, the residuals decrease even further than the decrease at large periods in Scenario 1, reaching on average residuals of 1 between the periods of 4s and 5s. The decreasing trend is also seen to have started from a shorter period of 0.7s instead of 1s in Scenario 1.

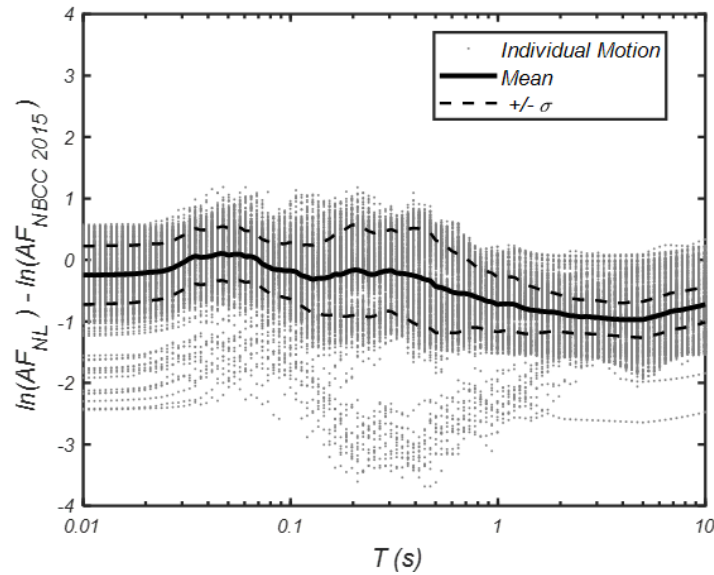


Figure 30 – Residuals of Amplification Factors for All Motions and All Soil Profiles (Scenario 2)

As seen in Figure 31, the residuals plotted against the normalized periods show a peak of residuals which occurs around T_0 as well. There is also a second peak, slightly lower than the peak at T_0 , at shorter normalized periods, potentially reflecting the group of higher residuals at shorter periods observed in Figure 30.

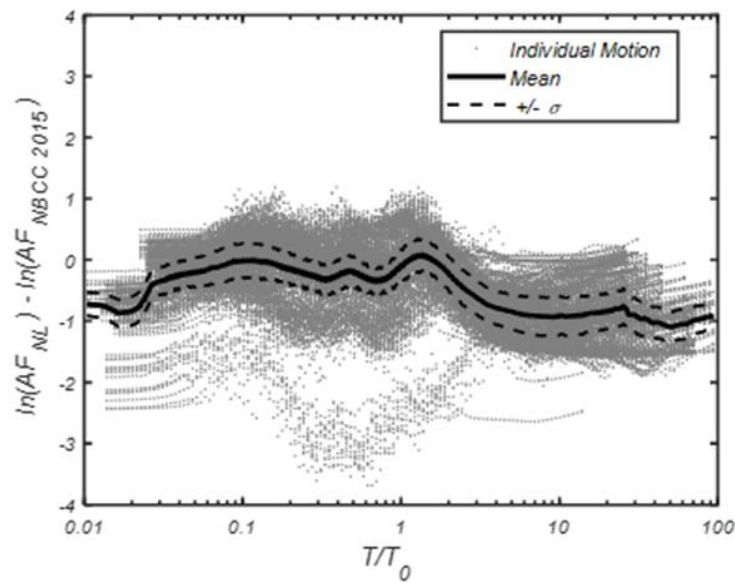


Figure 31 – Residuals of Amplification Factors for All Motions and All Soil Profiles Versus Normalized Periods (Scenario 2)

4.2.7 Residuals Analysis for Scenario 3

In Scenario 3, the residuals show an overall conservatism as well. Similar to Scenario 2, in Scenario 3, there is an observed overall decrease starting at 0.7s, with an average residual of slightly below -1 past the period of 2s, as seen in Figure 32. There is also a peak of consistent predictions between the AF from the NBCC 2015 and the NL method at short periods between 0.05s and 0.07s.

In Figure 33, the residuals plotted against normalized periods still show the highest peak around T_0 . However, at shorter normalized periods of around 0.1, there is another peak almost as high as the peak at T_0 . This secondary peak was somewhat visible in Scenario 2 as well, although less so. This could be due to the higher average magnitude of input seismic motions used in Scenario 2 and even more so in Scenario 3 when compared to Scenario 1. The average magnitude of input motions used in Scenario 1, 2 and 3 are 5.5, 6.5 and 6.8 respectively. The reason for this potential correlation is unclear and further research on this could be beneficial.

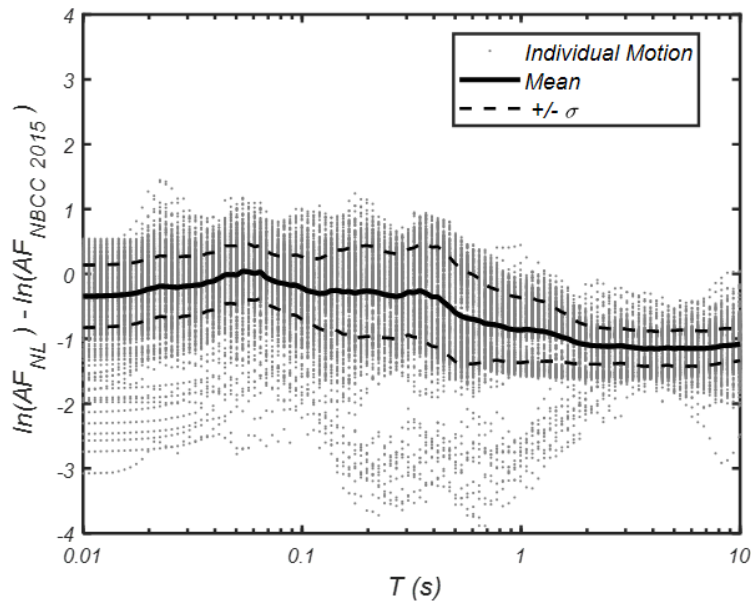


Figure 32 – Residuals of Amplification Factors for All Motions and All Soil Profiles (Scenario 3)

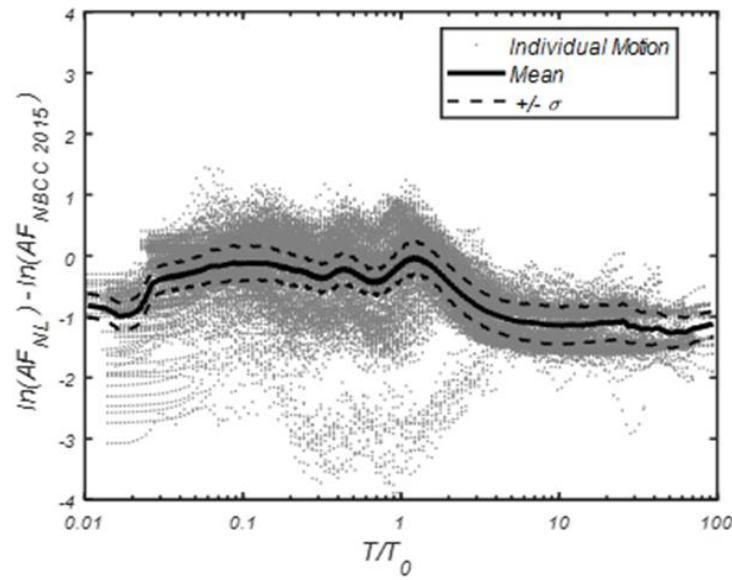


Figure 33 – Residuals of Amplification Factors for All Motions and All Soil Profiles Versus Normalized Periods (Scenario 3)

4.2.8 Correlations Between Residuals and Various Parameters

In this section, for all the soil profiles and scenarios analysed, correlations between the residuals at four distinct spectral periods ($T = 0.01s$, $T = 0.1s$, $T = 1s$ and $T = 10s$) and various parameters are investigated to analyze whether the NBCC2015 are biased, and how they can be improved. It is important to recall that results at $T = 10s$ may be unreliable due to multidimensional long period effects, such as basin effects, which are not considered in 1D site response analysis. The correlations plotted for all three Scenarios combined are found in section 4.2.8.1, whereas the correlations plotted for each individual Scenario are found in Appendix D through Appendix P.

4.2.8.1 Summary of Correlations between Residuals and Various Parameters

Among all the residuals plots with all tested parameters, one stark difference between soil profiles of class D and of class E can be made. Typically, as seen in the examples of Figure 34, for soils of class D, average residuals are closer to 0 in comparison to soils of class E, which means that the AF obtained from the NBCC 2015 are on average more consistent with factors derived from the nonlinear (NL) analyses for soils of class D. This also suggests that if a soil is of class D, it is more likely that a NL analysis will showcase an AF higher than the factor from the NBCC 2015, since

significantly more class D data points have positive residual values compared to class E data points. As for soils of class E, the average residuals are typically below 0, meaning that the NBCC 2015 is typically more conservative than NL analyses on average. This observation is most noticeable in the correlation of residuals with respect to V_{s30} (Figure 34). As a reminder, if a soil profile is shallower than 30 m in this project, its V_{s30} would be equivalent to its average V_s without consideration for the bedrock. As seen in Figure 34 involving V_{s30} , for all cases and periods, residuals steadily increase from negative values at low V_{s30} towards 0 as the V_{s30} of a soil profile increase. Since V_{s30} is an indicator of a soil's stiffness, as shown in Table 1, this further suggests that the stiffer the soil deposit, the more likely the NBCC 2015 would underpredict AF , and that the softer the soil, the more conservative would likely be the predictions of NBCC 2015. It is important to note that due to the large variability in the results, both types of soils would benefit from NL analyses.

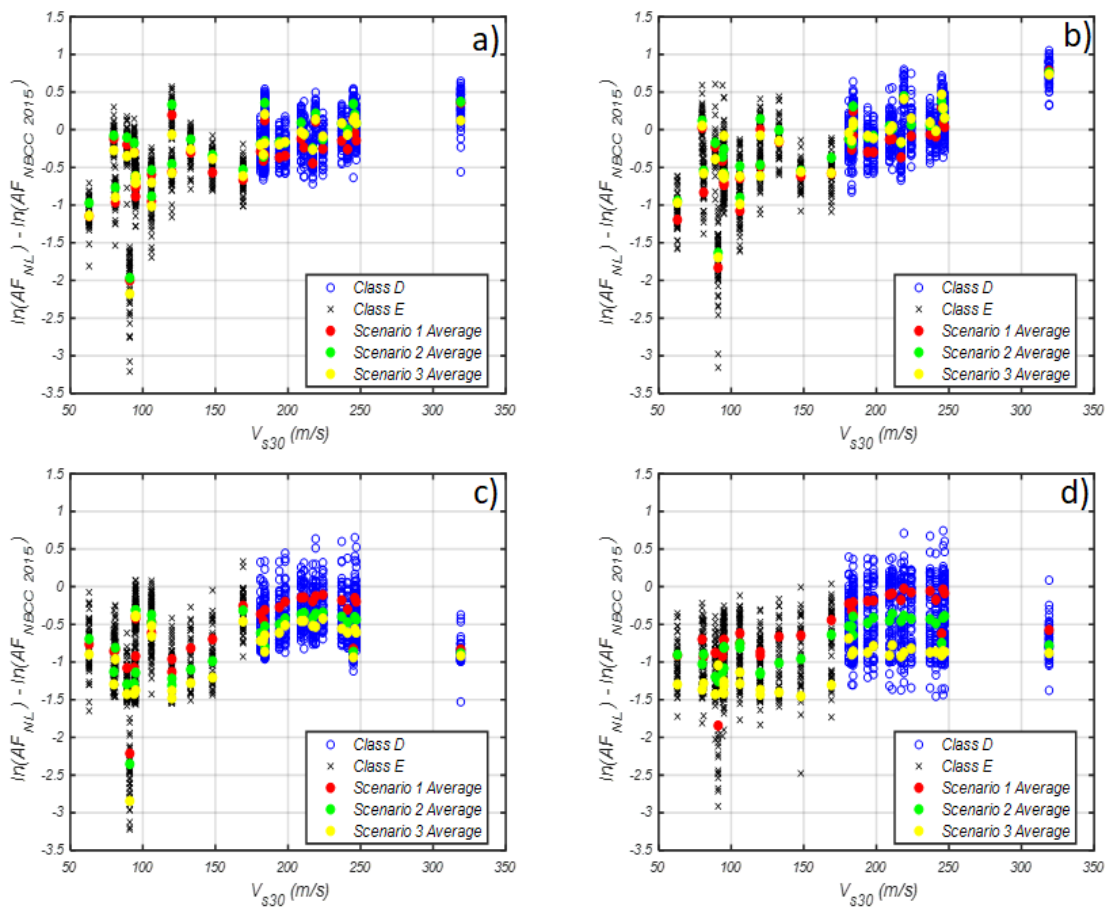


Figure 34 – Residuals versus V_{s30} at a) T=0.01s, b) T=0.1s, c) T=1s and d) T=10s

For the other parameters, their influences on the residuals are less obvious. As for the natural site period (T_0) shown in Figure 35, at lower periods ($T = 0.01s$ and $T = 0.1s$), the residuals decrease on average from 0 to negative values as T_0 increases. More specifically at low periods, soil profiles with low T_0 ($<0.5s$) yield AF consistent between NL analyses and NBCC 2015, while for higher T_0 ($>0.5s$), it appears that NBCC 2015 is more conservative than NL analyses. At higher periods ($T = 1s$ and $T = 10s$), the data on average shows that the NBCC 2015 yield AF more conservative than those from NL analyses. At the higher periods, there is a slight increase in residuals from negative values towards 0 as T_0 increase, which is contrary to the trend at lower periods.

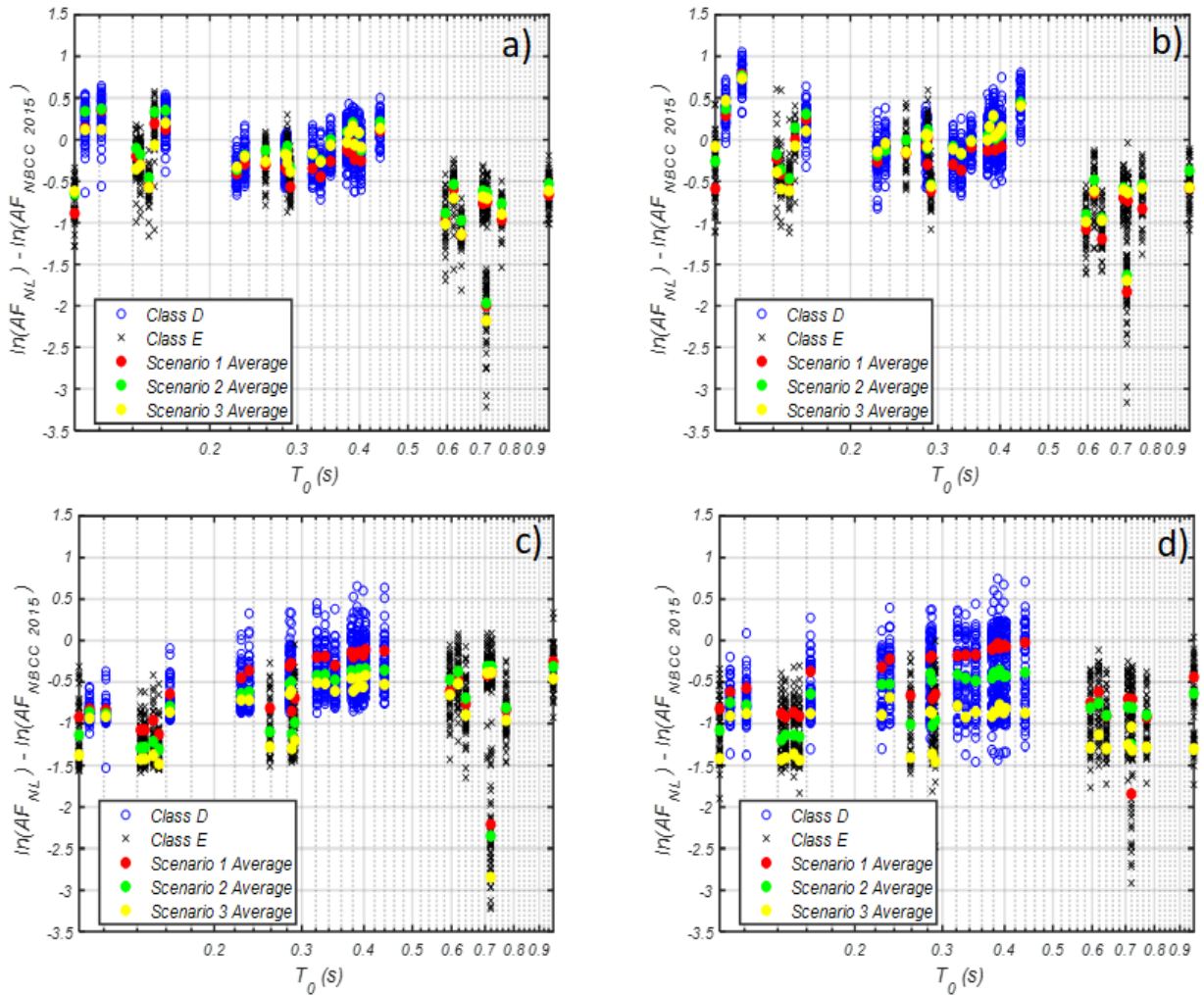


Figure 35 – Residuals versus T_0 at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$

The PGA , the $Sa(0.1s)$, the root-mean-square of acceleration (a_{rms}) and the sustained max acceleration (SMA) of input motions, as shown in Figure 36, Figure 37, Figure 38 and Figure 39 respectively, have similar effects on the residuals. a_{rms} (McCaan and Shah, 1979) is defined as:

$$a_{rms} = \left(\frac{1}{t_{total}} \int_0^{t_{total}} a(t)^2 dt \right)^{1/2}, \quad \text{Equation 47}$$

where t_{total} is the total time of the acceleration time history ($a(t)$), and SMA (Nuttli, 1979) is defined as the third highest absolute value in an acceleration time history. At low periods, both these parameters hover slightly below 0 with seemingly no variation in residuals as the parameters increase in value. However, at higher periods, low PGA , $Sa(0.1s)$, a_{rms} and SMA are correlated with clearly below 0 residuals, suggesting on average conservative estimations from the NBCC 2015, whereas high PGA , $Sa(0.1s)$, a_{rms} and SMA are seen with an increase in residual values but still below 0 on average. The NBCC 2015 is therefore on average most conservative at higher periods when the PGA , $Sa(0.1s)$, a_{rms} and SMA are low. Otherwise, the NBCC 2015 provides fair yet not overly conservative AF .

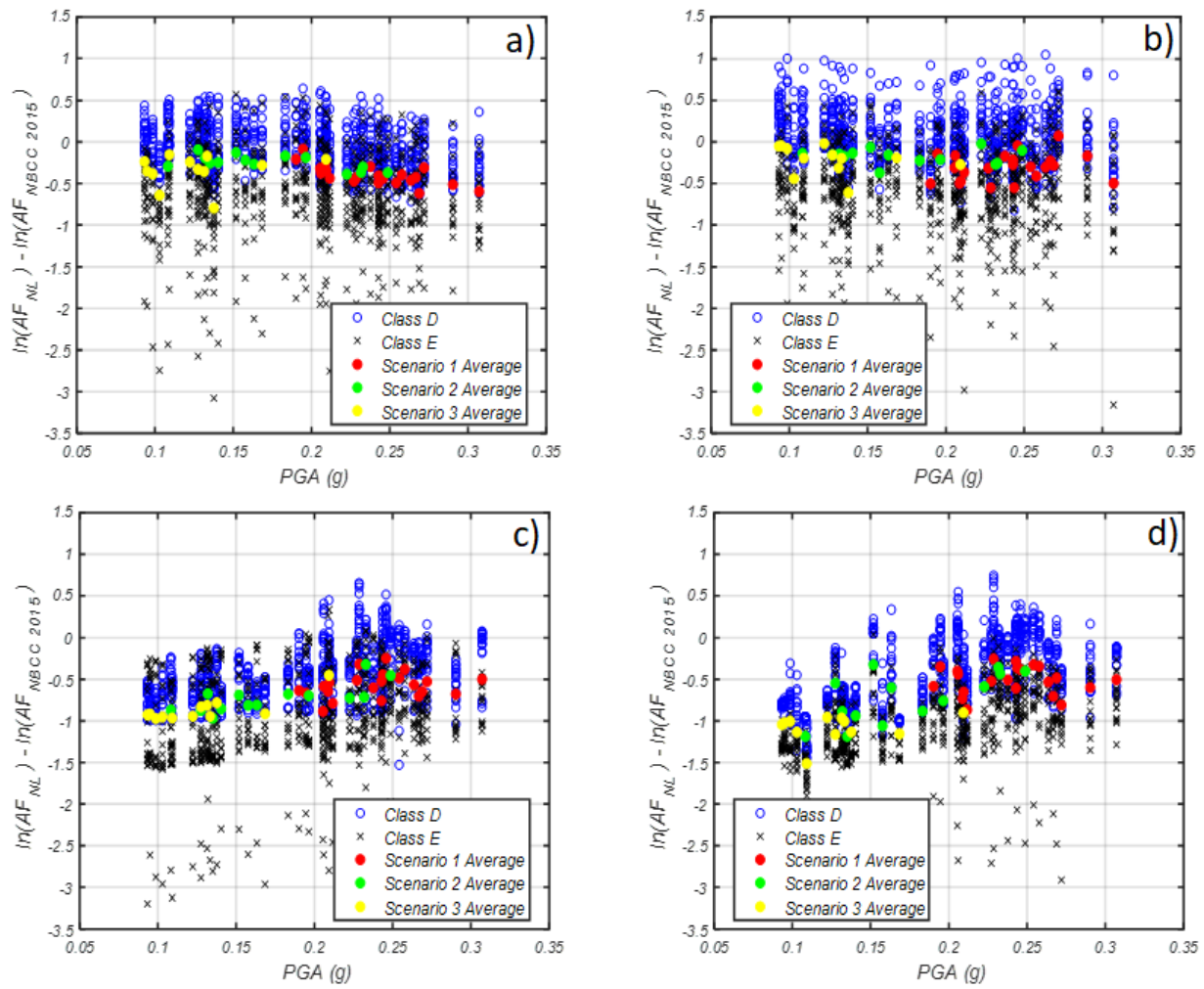


Figure 36 – Residuals versus PGA at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$

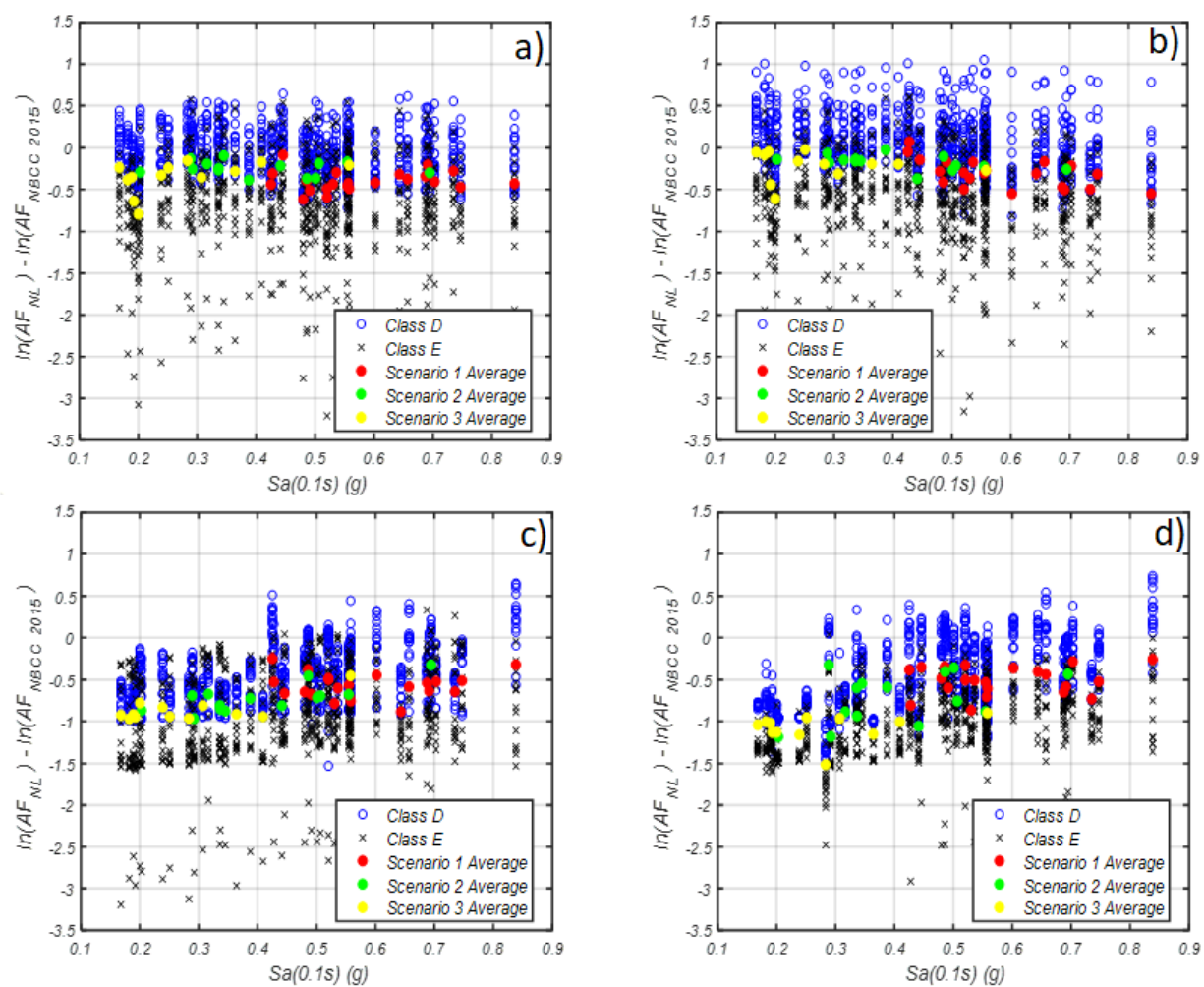


Figure 37 – Residuals versus $Sa(0.1s)$ at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$

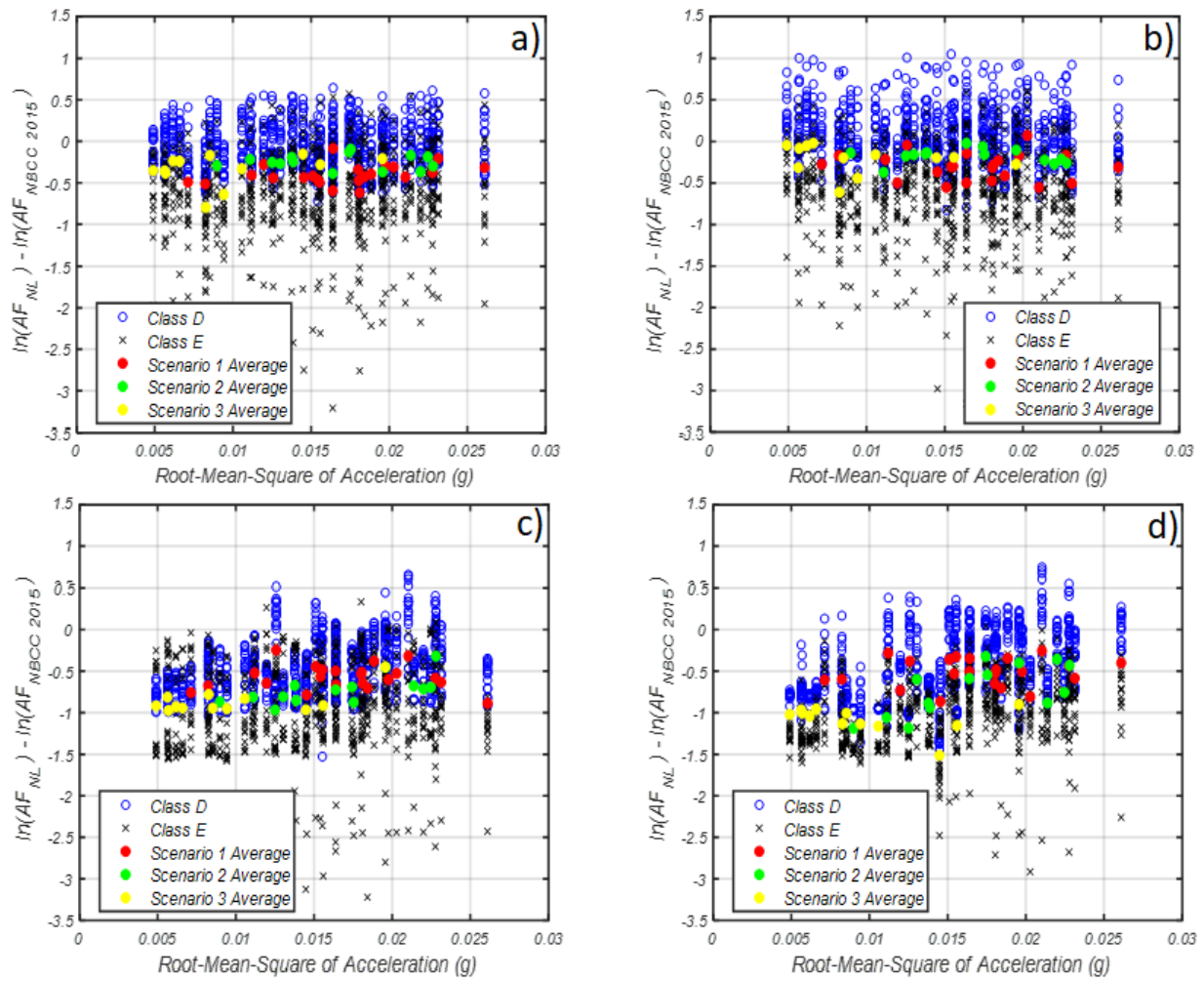


Figure 38 – Residuals versus root-mean-square of acceleration (a_{rms}) at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$

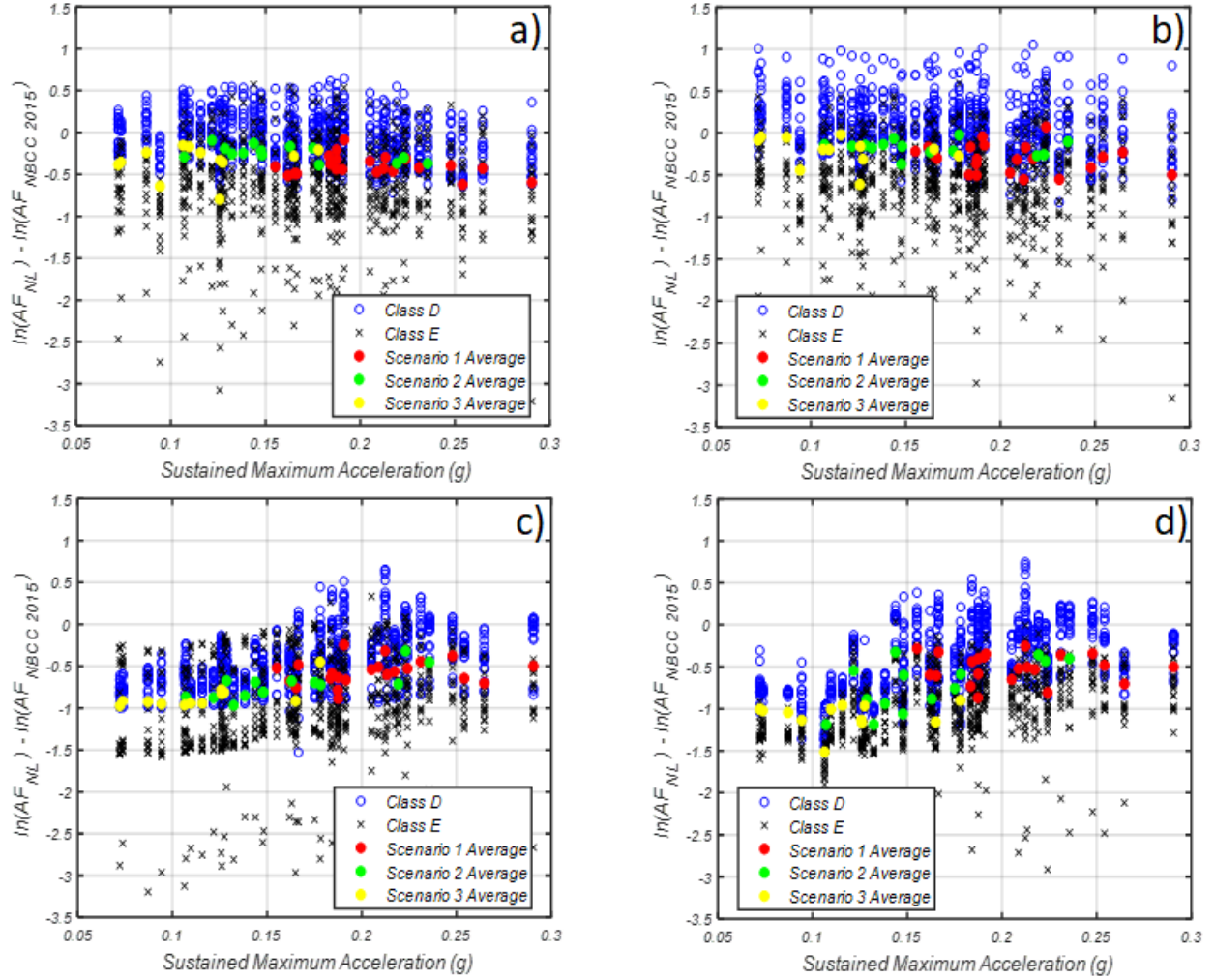


Figure 39 – Residuals versus sustained max acceleration (*SMA*) at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$

Additionally, residuals as a function of $Sa(1s)$, $Sa(10s)$, Arias intensity (*AI*) and cumulative absolute velocity (*CAV*) of input motions, as shown in Figure 40, Figure 41, Figure 42 and Figure 43 respectively, all show a similar trend. *AI* (Arias, 1970) is defined as:

$$AI = \frac{\pi}{2g} \int_0^{t_{total}} a(t)^2 dt, \quad \text{Equation 48}$$

where g is the acceleration of gravity, and *CAV* (EPRI-NP-5930, 1988) is defined as:

$$CAV = \int_0^{t_{total}} |a(t)| dt. \quad \text{Equation 49}$$

At low periods, these parameters are also hovering slightly below 0, indicating fair yet not overly conservative AF estimations from the NBCC 2015. At high periods, on average, the lower these four parameters are, the more consistent are NBCC 2015 and NL analyses predictions, and the higher the four parameters are, the more conservative the NBCC 2015 is.

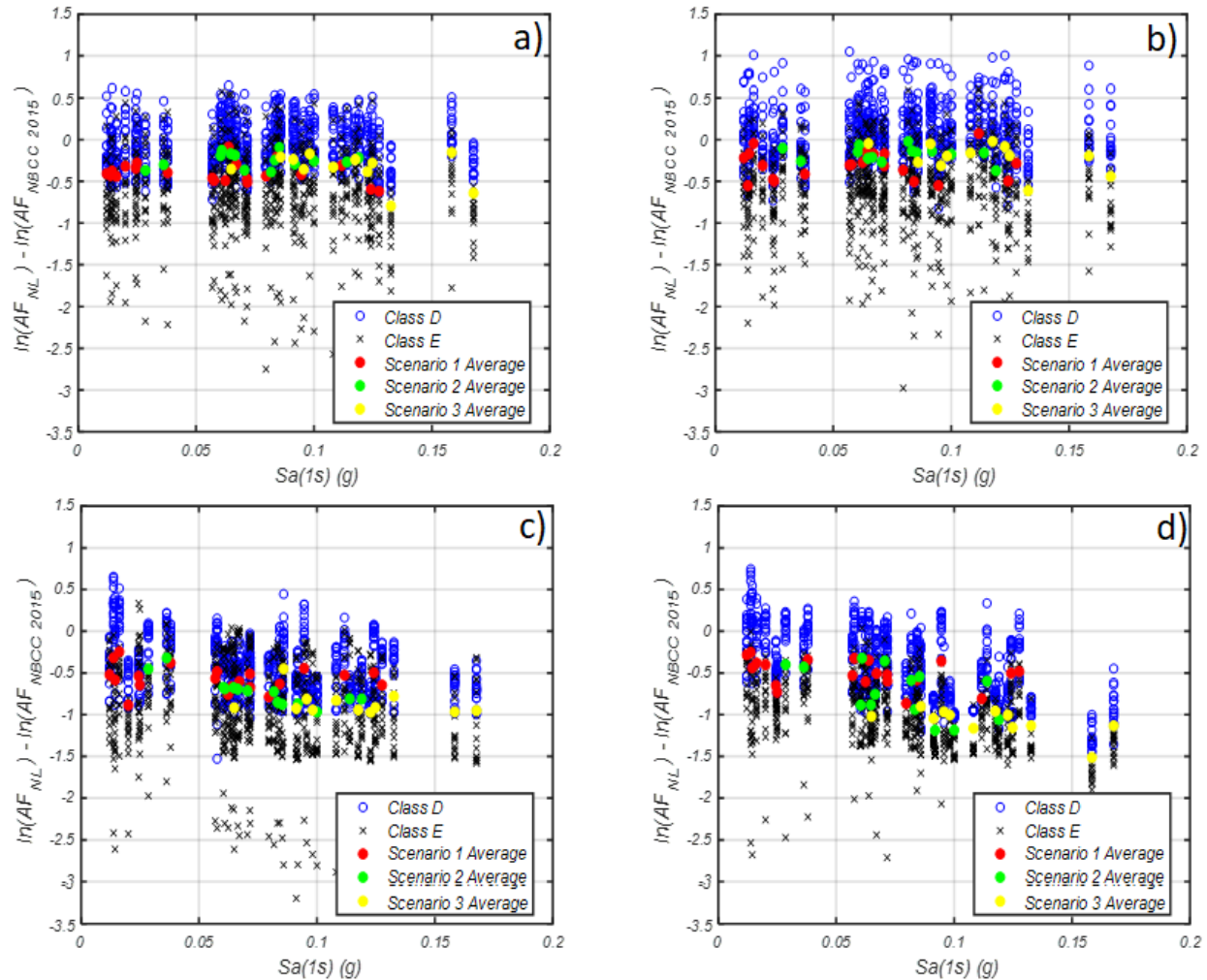


Figure 40 – Residuals versus $Sa(1s)$ at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$

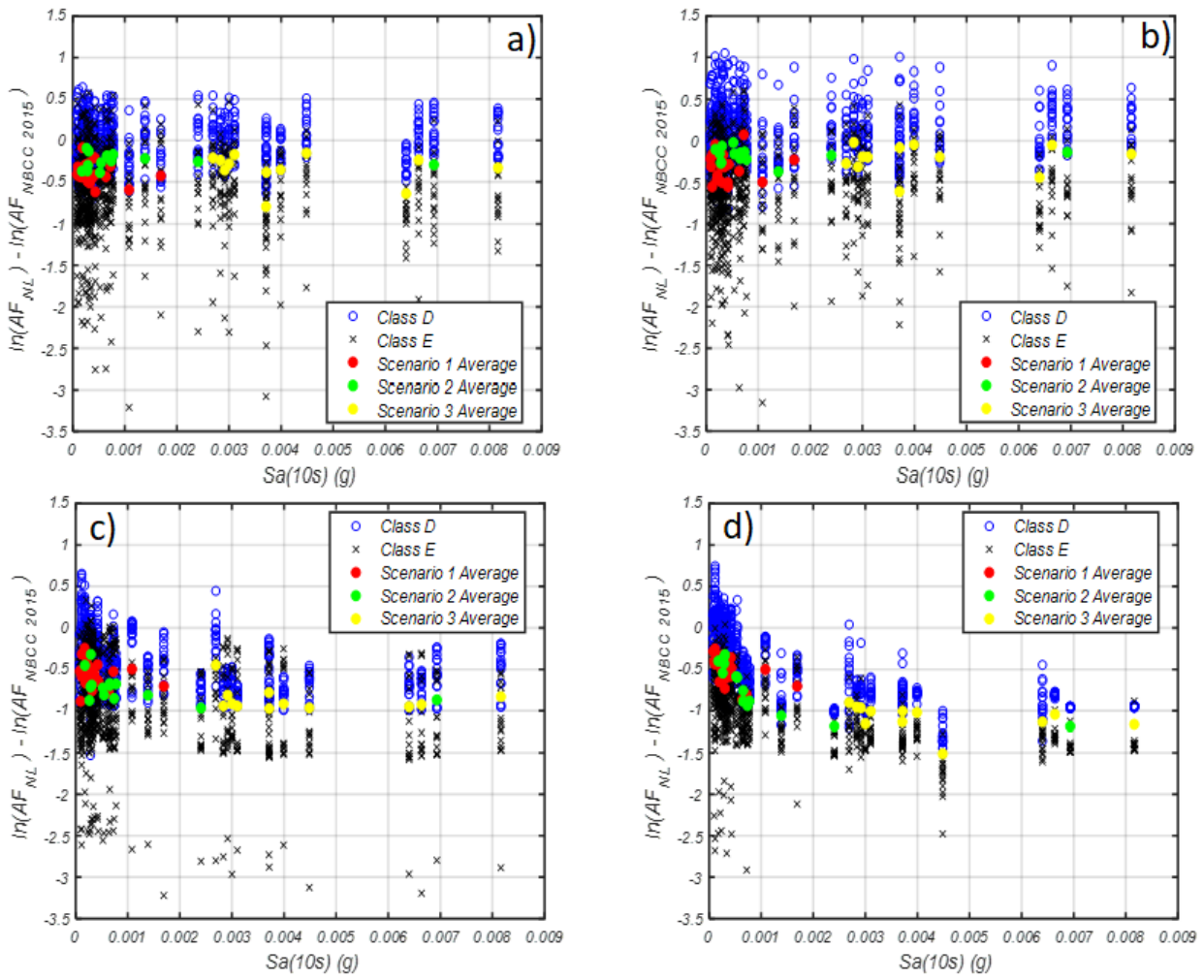


Figure 41 – Residuals versus $Sa(10s)$ at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$

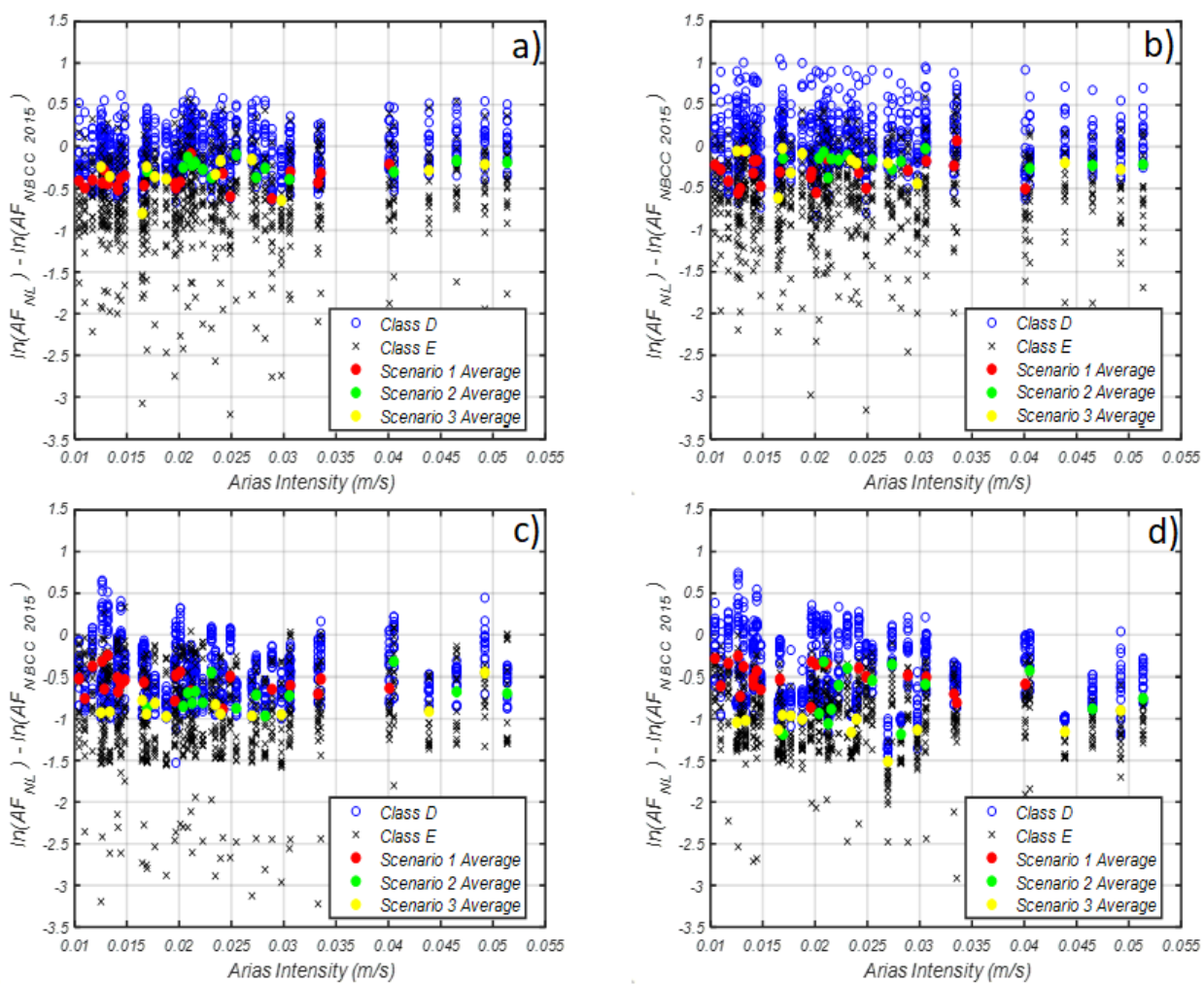


Figure 42 – Residuals versus Arias intensity (AI) at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$

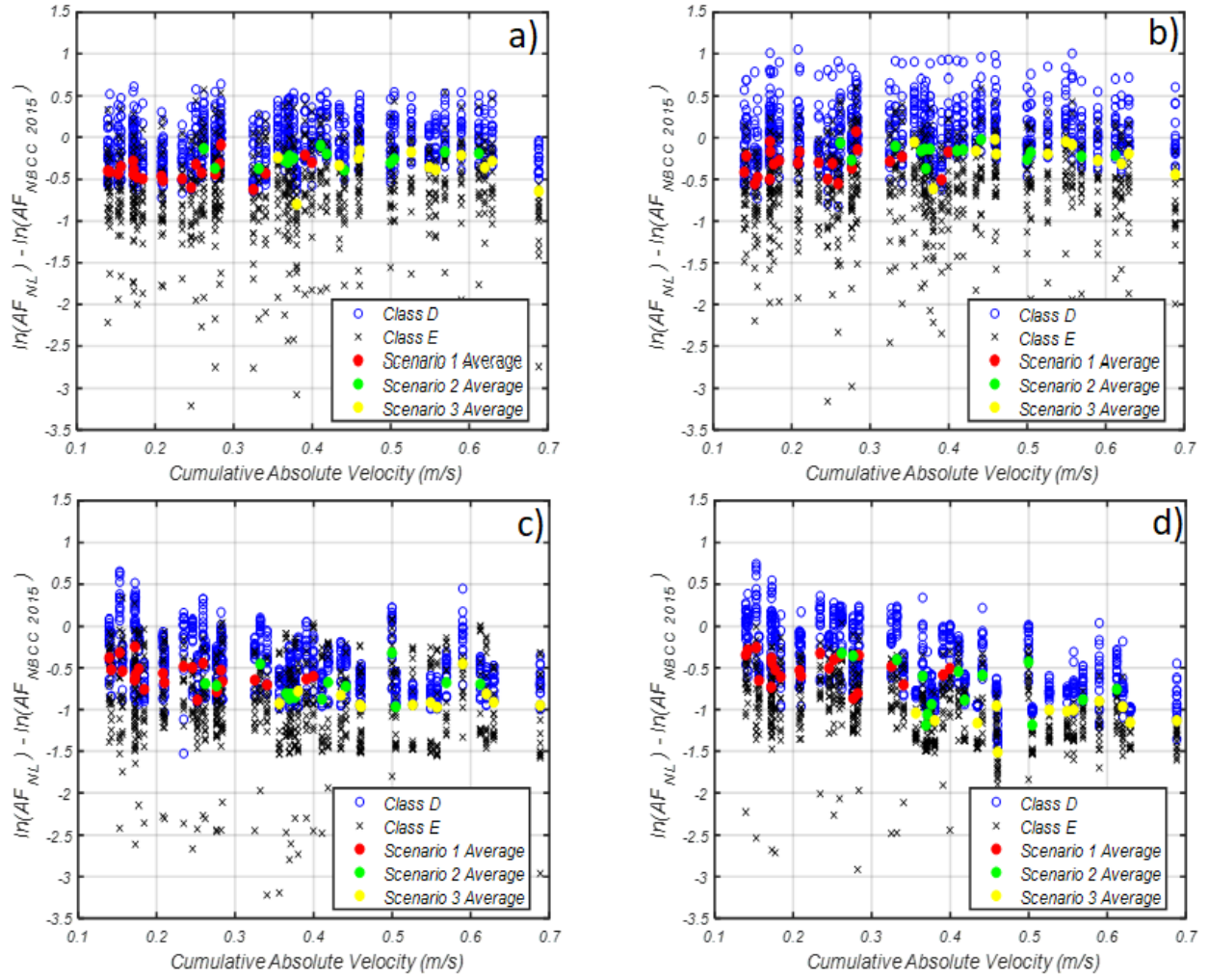


Figure 43 – Residuals versus cumulative absolute velocity (CAV) at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$

A_{95} is a parameter against which residuals seem to be fairly independent as shown in Figure 44.

A_{95} (Sarma and Yang, 1987) is defined as:

$$a(t_{95}) \text{ where } \left(\frac{\pi}{2g} \int_0^{t_{95}} a(t)^2 dt \right) / AI = 0.95. \quad \text{Equation 50}$$

On average, at low periods, A_{95} correspond to slightly below 0 residuals, while at high periods, A_{95} correspond to even lower residuals, indicating again that the NBCC 2015 is more conservative at higher periods.

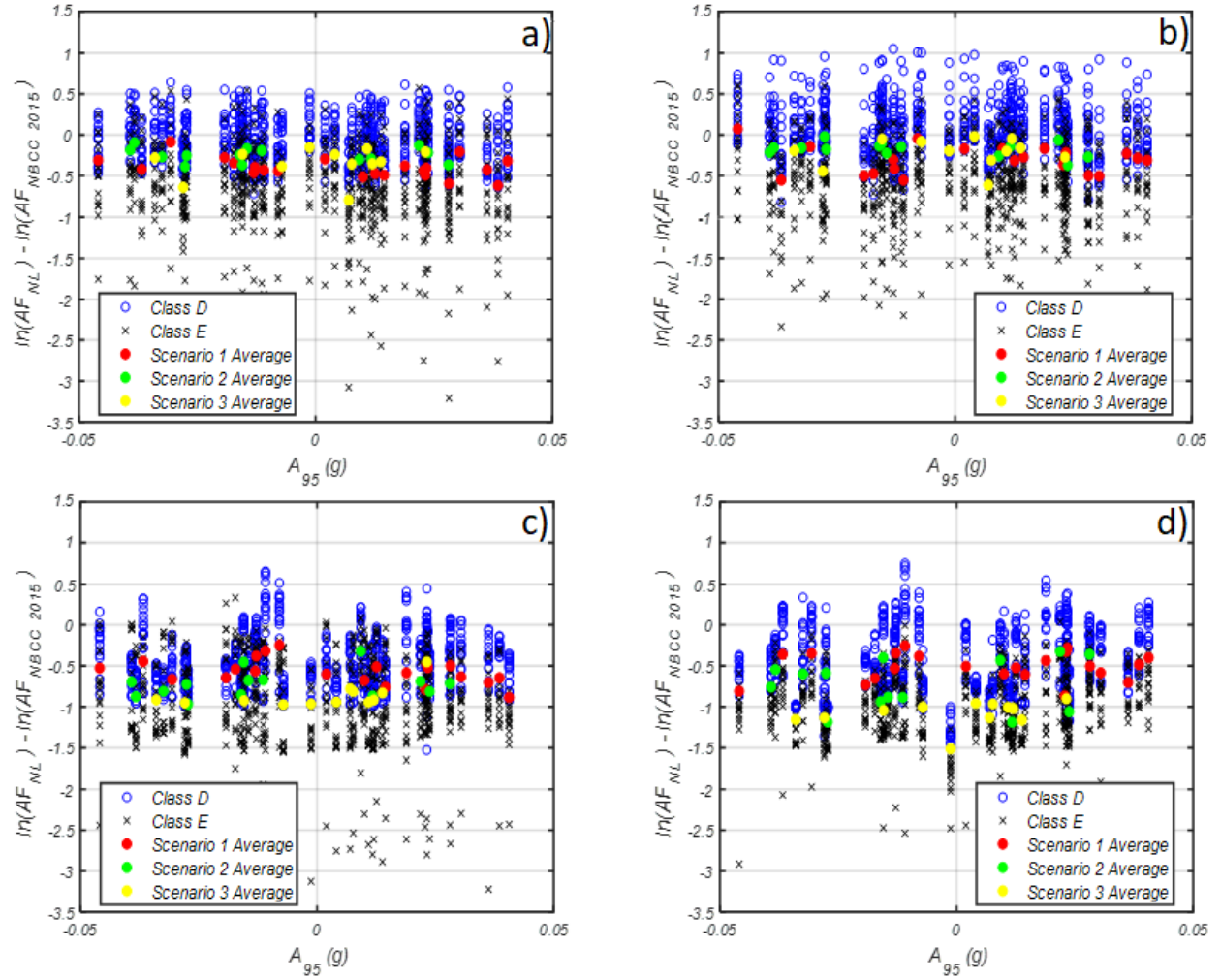


Figure 44 – Residuals versus A_{95} at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$

The pseudo spectral acceleration at the natural site period ($PSA[T_0]$), defined as the spectral acceleration at the fundamental period of the site (T_0) (Dashti and Karimi, 2017), seems to be a parameter that is also independent of residuals as shown in Figure 45. However, it can be subtly observed that at low periods, the NBCC 2015 on average yield more consistent results with NL analyses as the residuals are closer to 0, while at high periods, the NBCC 2015 is more conservative on average as the residuals are further away from 0.

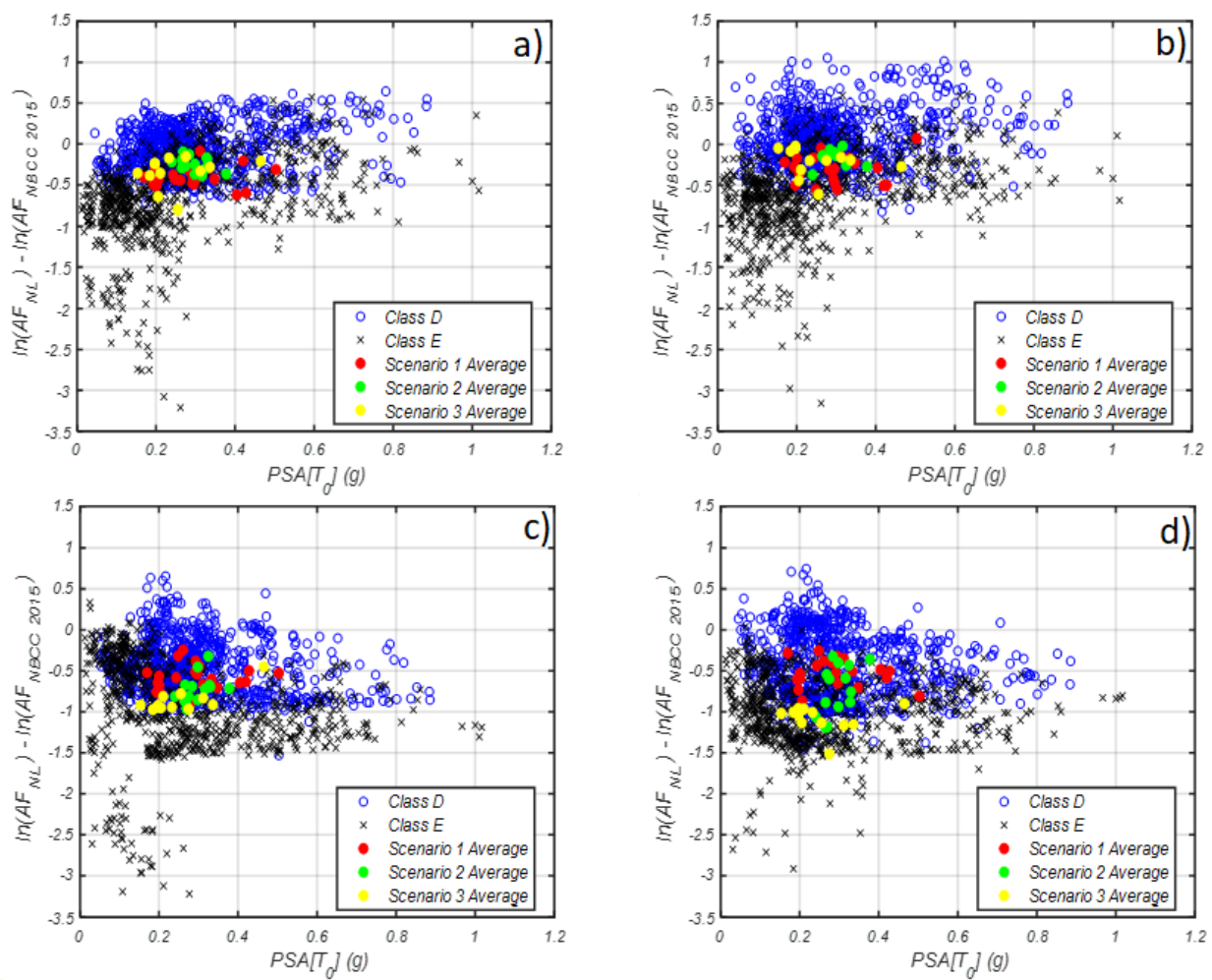


Figure 45 – Residuals versus $PSA[T_0]$ at a) $T=0.01s$, b) $T=0.1s$, c) $T=1s$ and d) $T=10s$

CHAPTER 5 CONCLUSION AND FUTURE WORKS

In current Eastern Canadian industry practices, amplification factors (AF) directly taken from the NBCC 2015 is the most commonly used method for predicting how much a soft soil deposit will amplify seismic waves propagated from the underlying bedrock. On the other hand, site-specific NL analysis is a significantly more accurate method of analysis, but requires a higher level of understanding of the soil behaviour and is much more time-consuming to perform. Therefore, as part of this project, a computer code was created in order to efficiently perform the NL analysis of 33 anonymous soil profiles, located in Quebec in the basin of the Saint-Lawrence River, subjected to 48 input seismic motions per soil profile. The AF resulting from the NL analysis were then compared to AF from the NBCC 2015 to highlight any differences between the two methods.

Through this project, a MATLAB code was successfully written to automatically interpret and output a series of subdivided soil layers with corresponding basic and dynamic properties given any soil profile with CPT data. If field V_s and S_u data were available for a soil profile, they were used to calibrate the basic properties estimated from the CPT data. From those basic properties, the MRD curves were also estimated through constitutive models integrated into the MATLAB code. The code effectively reduced the time it takes to conduct NL analysis on the Deepsoil software. This allowed for a large number of NL analysis to be completed within a reasonable timeframe, and their results to be compared with those from the NBCC 2015 approach.

From the results of this project, NL analysis at times yielded AF significantly lower than factors found in the NBCC 2015, suggesting that the NBCC 2015 can at times overestimate the seismic design requirements, causing unnecessarily costly designs. Other times, NL analysis predicted AF approximately similar to those found in the NBCC 2015, suggesting that the NBCC 2015 can at times predict accurately the seismic design requirements. However, for the majority of the soil profiles, NL analysis yielded AF significantly higher than the factors in NBCC 2015 around the natural site period. This is problematic, as it can lead to unsafe engineering designs against seismic hazard. Furthermore, from the average residual analysis, the NBCC 2015 yielded conservative AF at most of the periods but generally underestimated the AF around the natural site periods. Additionally, from the analysis of the correlations between the residuals and various site-specific or motion-specific parameters, it seems to be more likely that the underestimation of AF by the NBCC 2015 is even more significant on average for soils of class D than for soils of class E, but

that there might be a significant overestimation of AF by the NBCC 2015 on the softer class E type soil. Therefore, it is equally important to perform NL analysis for both soft and stiff soils.

Since the soil profiles analysed in this project were all located in the Saint-Lawrence River basin, where significant formations of soft sensitive clay are found, it may be of interest for future research activities to compare AF from NL analysis and from the NBCC 2015 for different geographical regions of Eastern Canada as well, as this could determine whether the observations made in this memoire are applicable to other regions with different soil compositions.

In addition, since it is evident that the natural site period can hugely influence the amplification of seismic waves, it could be worthwhile for future research efforts to investigate methods to incorporate the effect of the natural site periods as an influencing factor when determining the seismic wave amplifications in soft soil deposits. For example, one idea could be to incorporate into the NBCC 2015 an interval of periods surrounding the natural site period within which the user must multiply the AF from the NBCC 2015 by another scaling factor. This additional multiplying factor's aim would be to ensure that the amplification surrounding the natural site periods predicted by the NBCC 2015 would represent fairly well the peak of AF observed from the NL analysis. However, since this is outside the scope of this research project, detailed ideas on potential implementations of the natural site periods in the determination of soil seismic amplification factors are not further discussed here.

As a disclaimer, a minor error was found in the MATLAB code following the memoire's presentation. The V_s values used as input for Deepsoil were in fact V_{s1} values according to Robertson and Cabal (2015). However, re-computations of the NL analysis with the correct V_s values have already begun, and the results so far indicate only minimally observable differences between the AF obtained from the use of V_s and those obtained from the use of V_{s1} . Therefore, the conclusions of this memoire should remain the same. Once completed, the re-computed results in this project, including all AF and residual plots, will be uploaded online at <https://samuelyniesta.com/> prior to the publication date of this memoire.

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APPENDIX A AMPLIFICATION FACTORS FOR SCENARIO 1

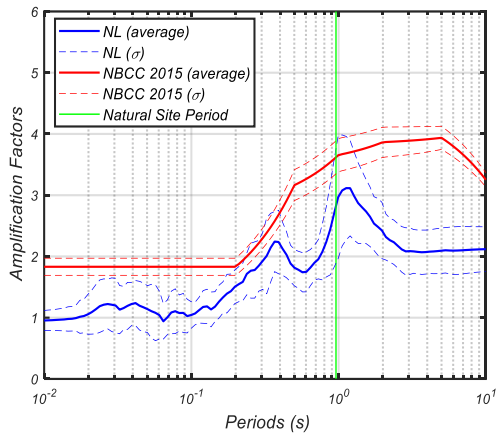


Figure A-1 – Soil Profile #1 Amplification, Case 3 (Scenario 1)

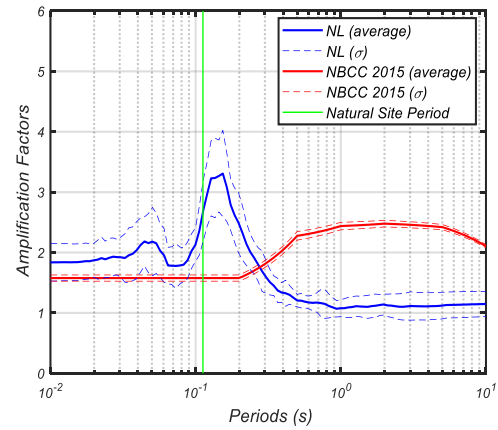


Figure A-2 – Soil Profile #2 Amplification, Case 1 (Scenario 1)

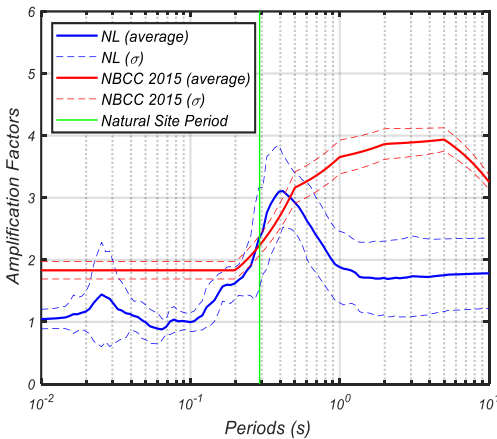


Figure A-3 – Soil Profile #3 Amplification, Case 3 (Scenario 1)

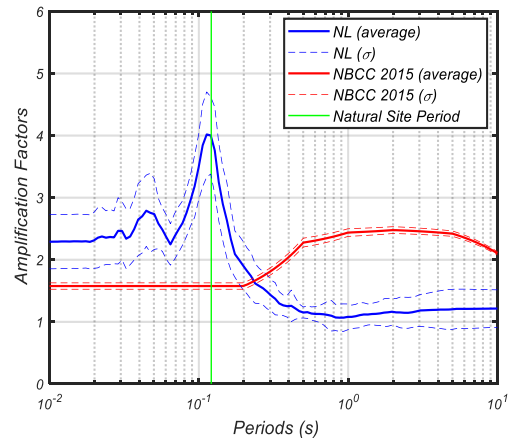


Figure A-4 – Soil Profile #4 Amplification, Case 1 (Scenario 1)

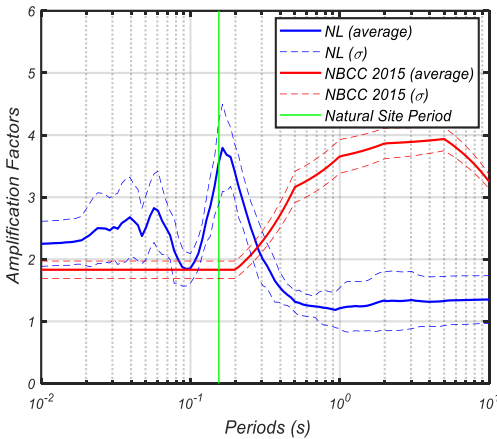


Figure A-5 – Soil Profile #5 Amplification, Case 1 (Scenario 1)

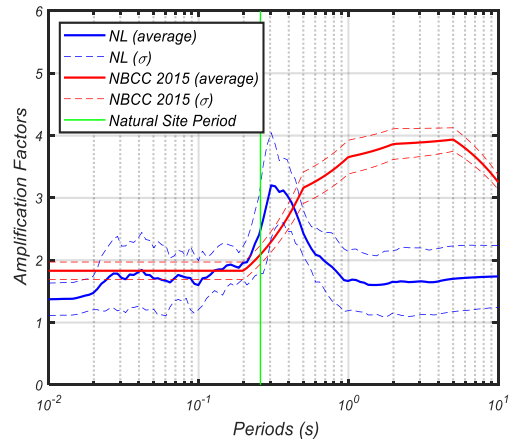


Figure A-6 – Soil Profile #6 Amplification, Case 1 (Scenario 1)

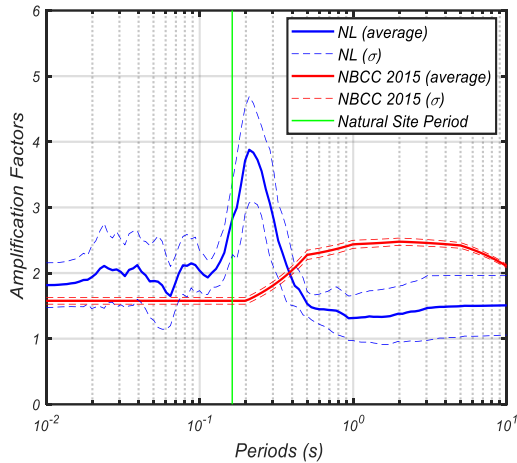


Figure A-7 – Soil Profile #7 Amplification, Case 1 (Scenario 1)

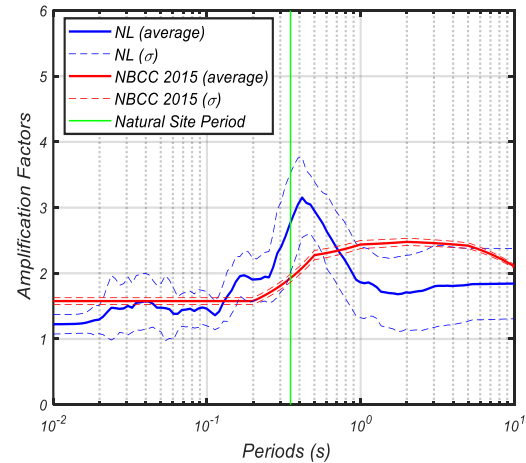


Figure A-8 – Soil Profile #8 Amplification, Case 1 (Scenario 1)

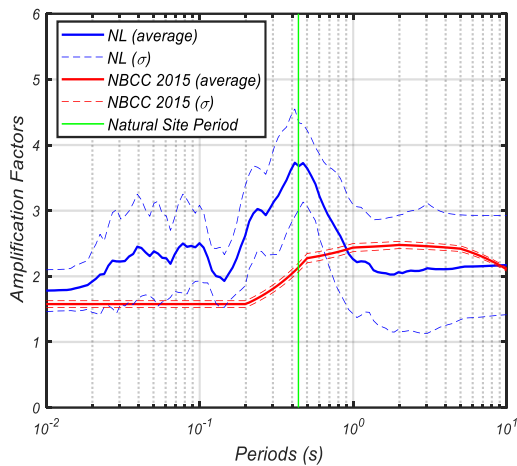


Figure A-9 – Soil Profile #9 Amplification, Case 1 (Scenario 1)

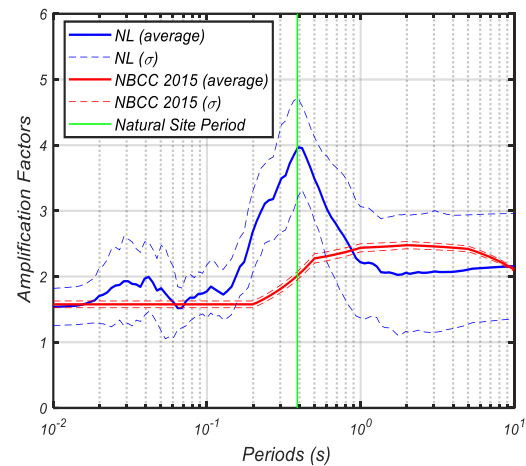


Figure A-10 – Soil Profile #10 Amplification, Case 1 (Scenario 1)

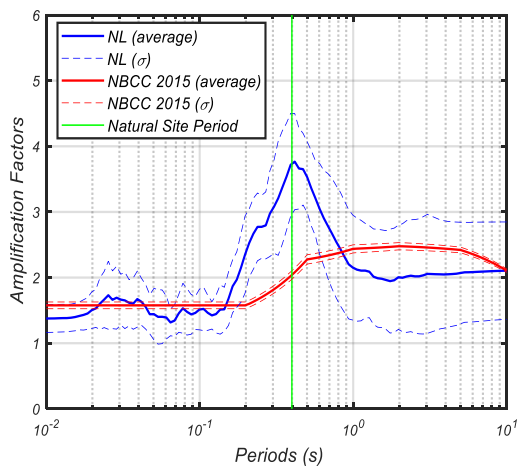


Figure A-11 – Soil Profile #11 Amplification, Case 1 (Scenario 1)

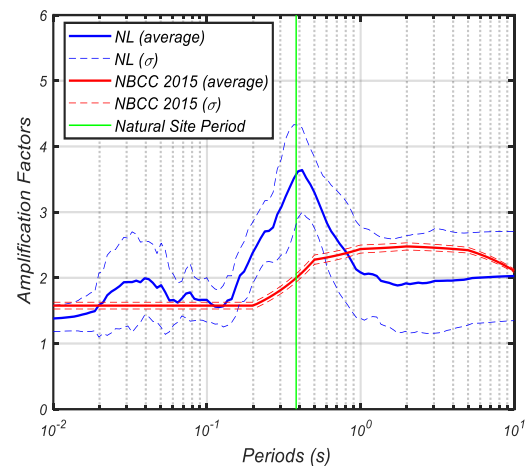


Figure A-12 – Soil Profile #12 Amplification, Case 1 (Scenario 1)

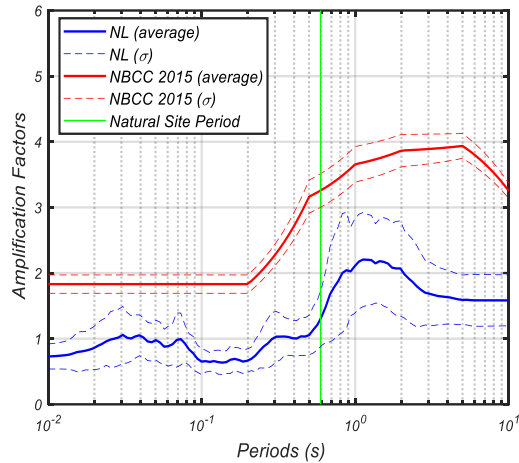


Figure A-13 – Soil Profile #13 Amplification, Case 2 (Scenario 1)

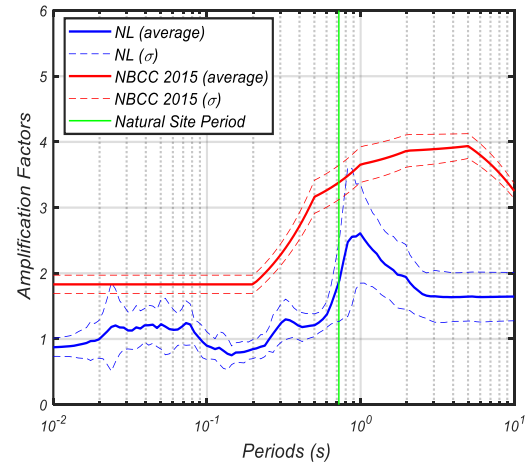


Figure A-14 – Soil Profile #14 Amplification, Case 3 (Scenario 1)

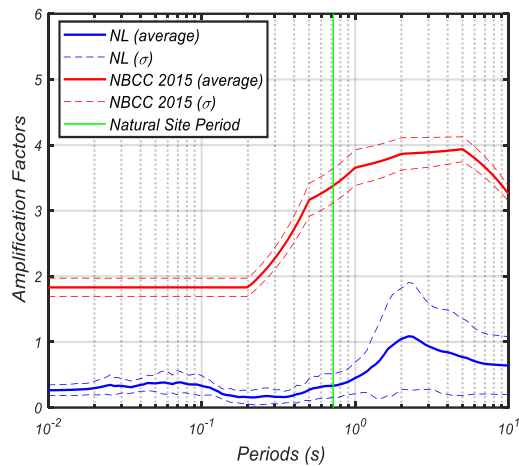


Figure A-15 – Soil Profile #15 Amplification, Case 2 (Scenario 1)

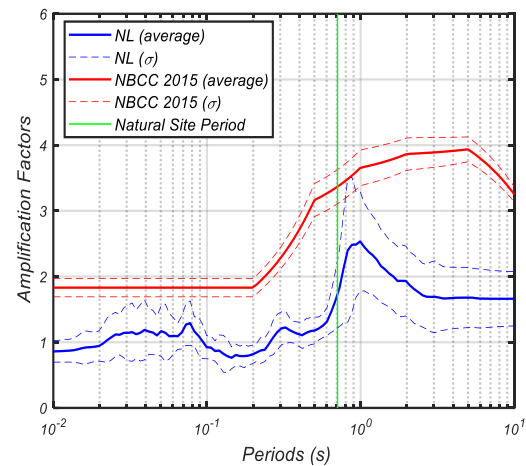


Figure A-16 – Soil Profile #16 Amplification, Case 2 (Scenario 1)

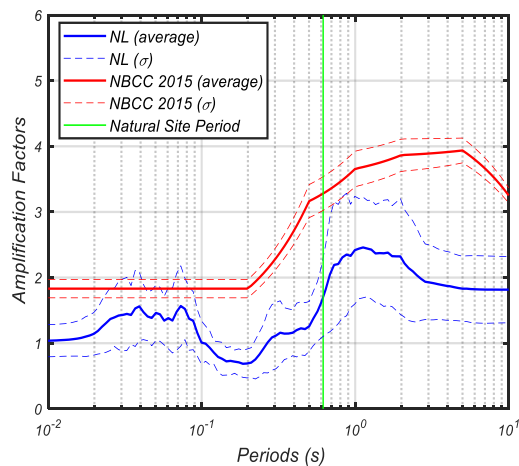


Figure A-17 – Soil Profile #17 Amplification, Case 3 (Scenario 1)

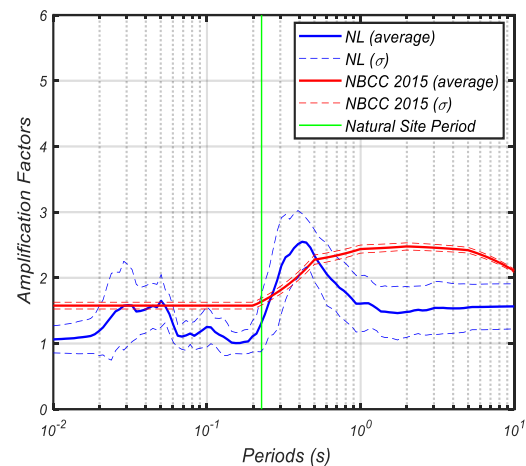


Figure A-18 – Soil Profile #18 Amplification, Case 3 (Scenario 1)

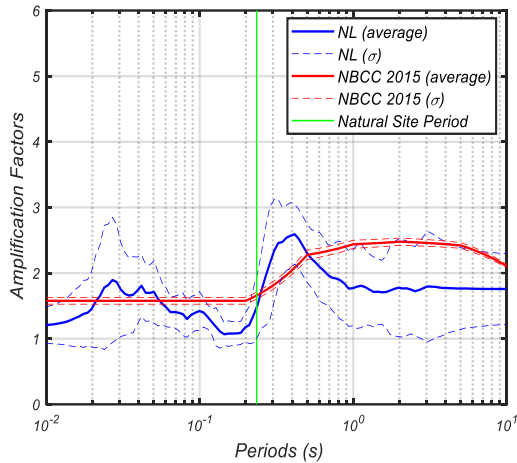


Figure A-19 – Soil Profile #19 Amplification, Case 3 (Scenario 1)

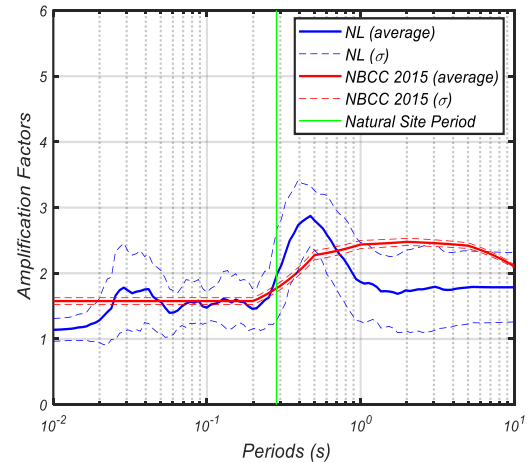


Figure A-20 – Soil Profile #20 Amplification, Case 1 (Scenario 1)

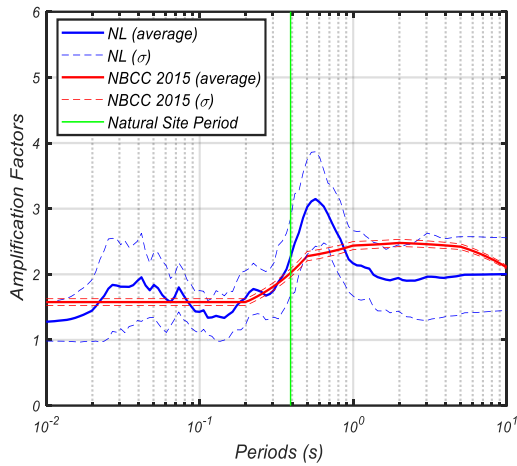


Figure A-21 – Soil Profile #21 Amplification, Case 1 (Scenario 1)

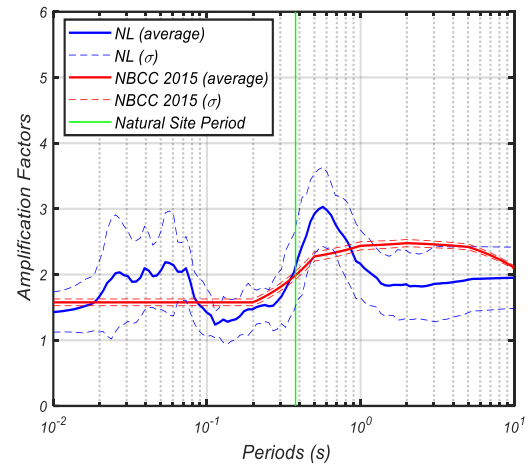


Figure A-22 – Soil Profile #22 Amplification, Case 1 (Scenario 1)

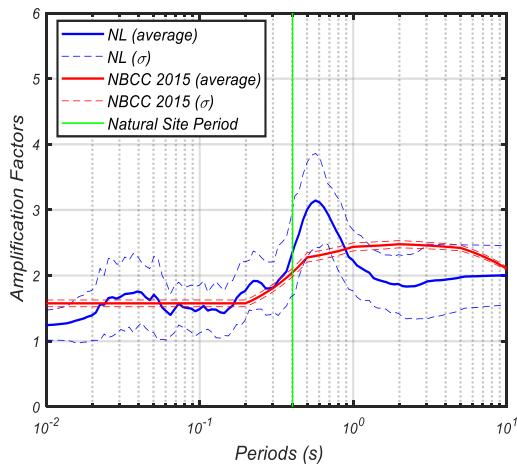


Figure A-23 – Soil Profile #23 Amplification, Case 1 (Scenario 1)

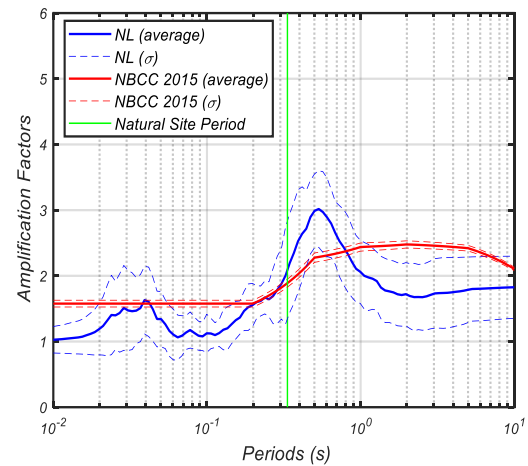


Figure A-24 – Soil Profile #24 Amplification, Case 1 (Scenario 1)

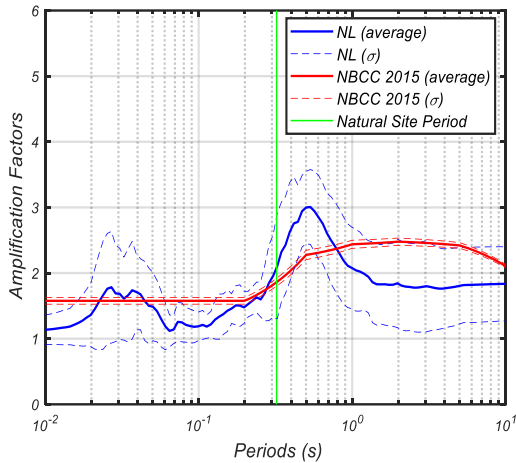


Figure A-25 – Soil Profile #25 Amplification, Case 1 (Scenario 1)

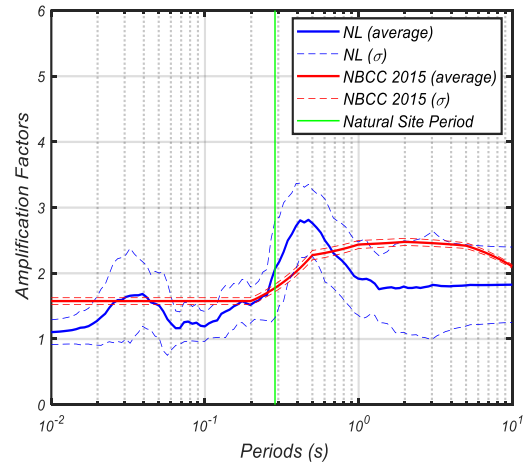


Figure A-26 – Soil Profile #26 Amplification, Case 1 (Scenario 1)

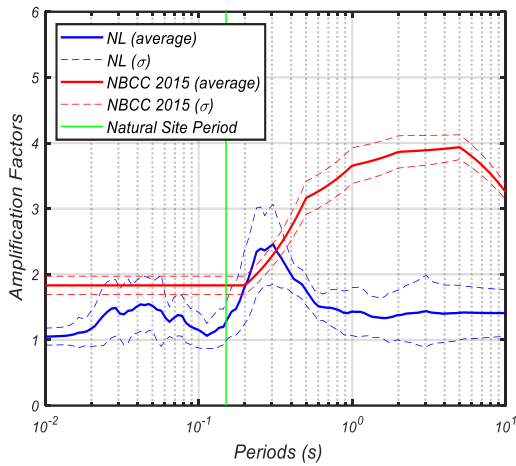


Figure A-27 – Soil Profile #27 Amplification, Case 3 (Scenario 1)

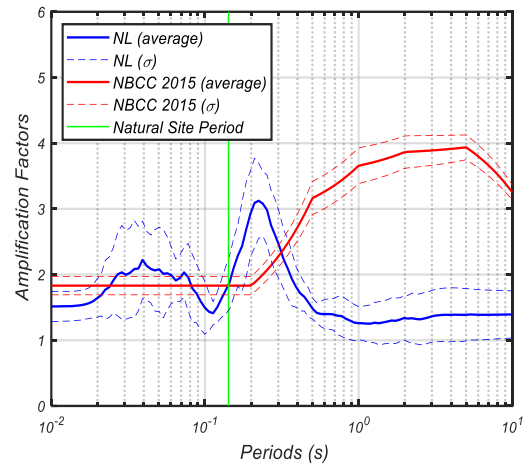


Figure A-28 – Soil Profile #28 Amplification, Case 1 (Scenario 1)

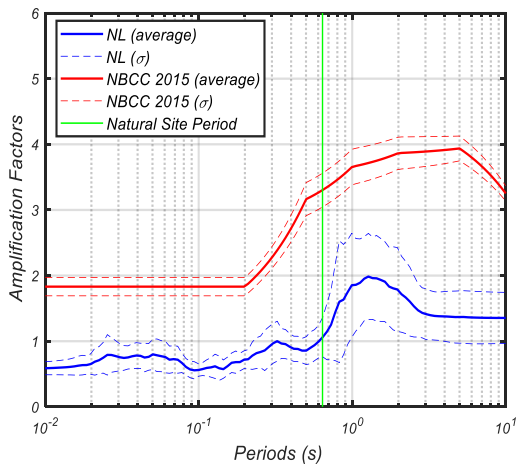


Figure A-29 – Soil Profile #29 Amplification, Case 2 (Scenario 1)

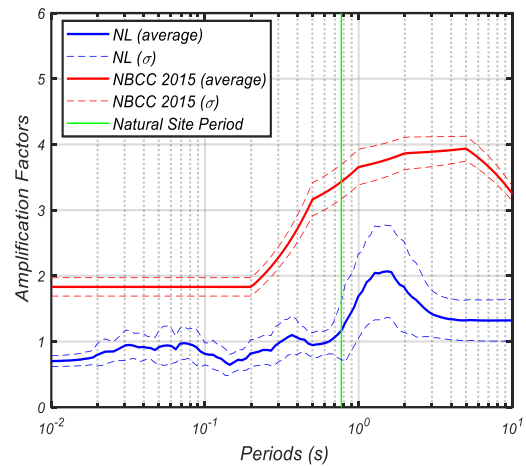


Figure A-30 – Soil Profile #30 Amplification, Case 2 (Scenario 1)

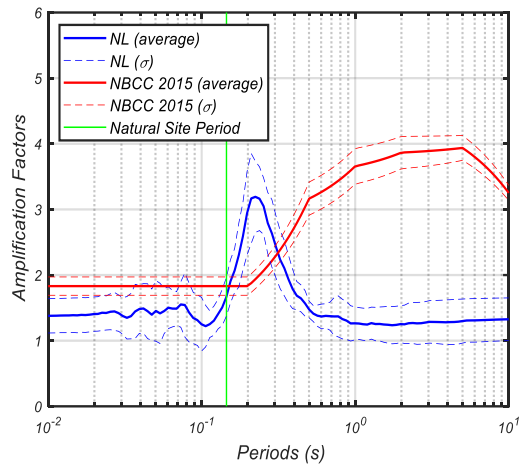


Figure A-31 – Soil Profile #31 Amplification, Case 1 (Scenario 1)

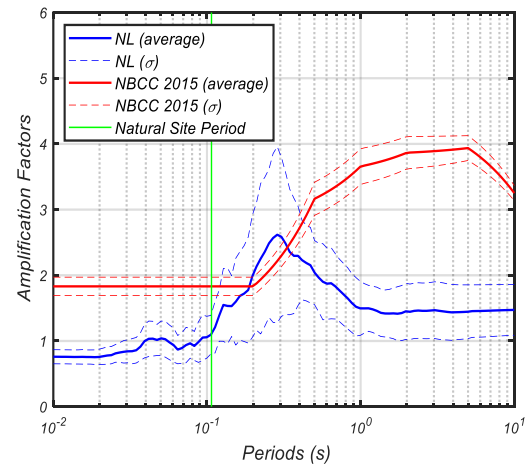


Figure A-32 – Soil Profile #32 Amplification, Case 3 (Scenario 1)

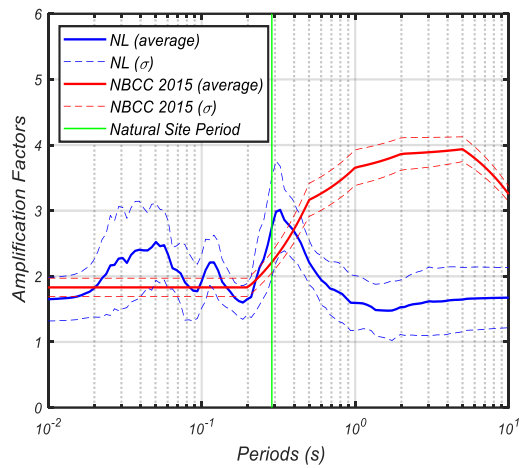


Figure A-33 – Soil Profile #33 Amplification, Case 3 (Scenario 1)

APPENDIX B AMPLIFICATION FACTORS FOR SCENARIO 2

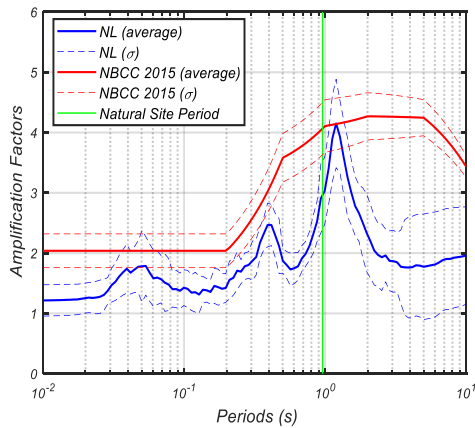


Figure B-1 – Soil Profile #1 Amplification, Case 3 (Scenario 2)

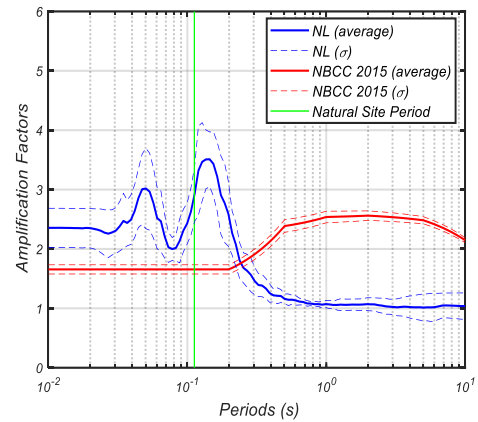


Figure B-2 – Soil Profile #2 Amplification, Case 1 (Scenario 2)

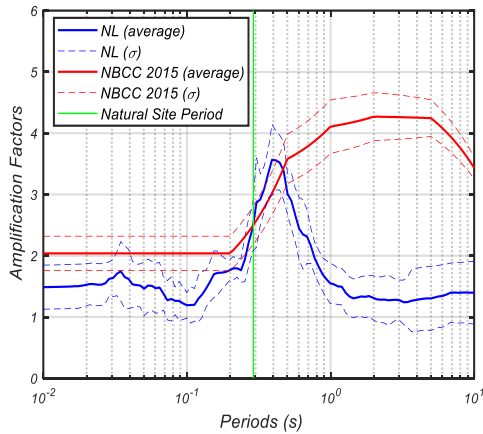


Figure B-3 – Soil Profile #3 Amplification, Case 3 (Scenario 2)

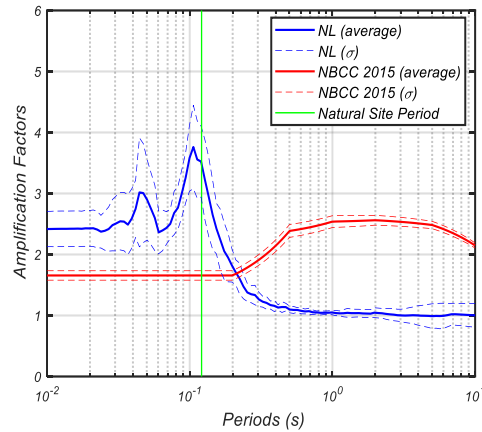


Figure B-4 – Soil Profile #4 Amplification, Case 1 (Scenario 2)

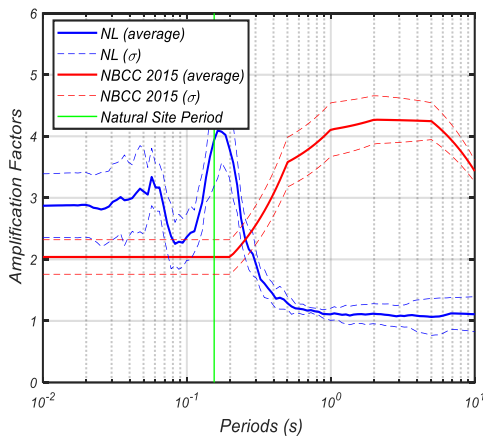


Figure B-5 – Soil Profile #5 Amplification, Case 1 (Scenario 2)

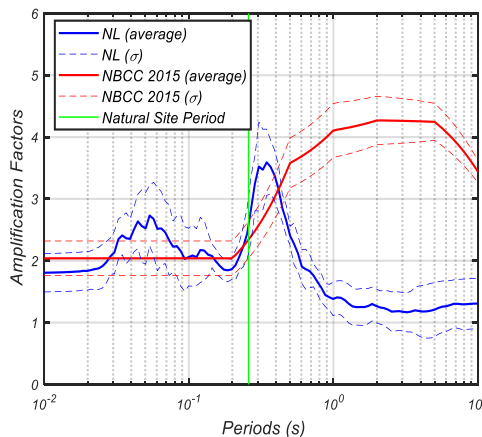


Figure B-6 – Soil Profile #6 Amplification, Case 1 (Scenario 2)

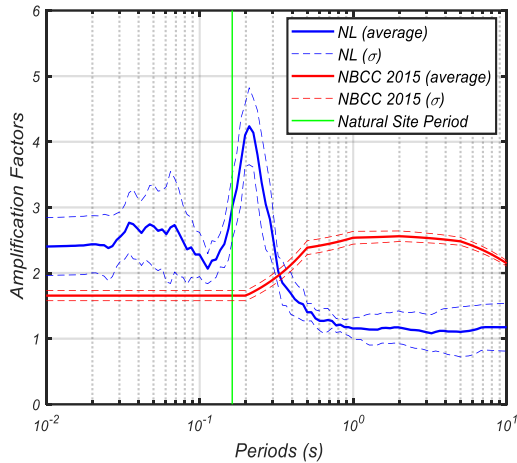


Figure B-7 – Soil Profile #7 Amplification, Case 1 (Scenario 2)

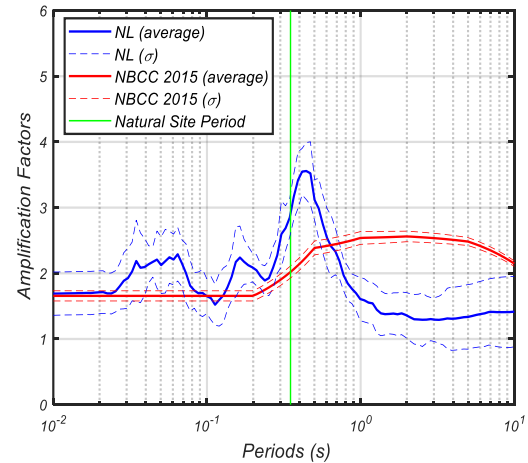


Figure B-8 – Soil Profile #8 Amplification, Case 1 (Scenario 2)

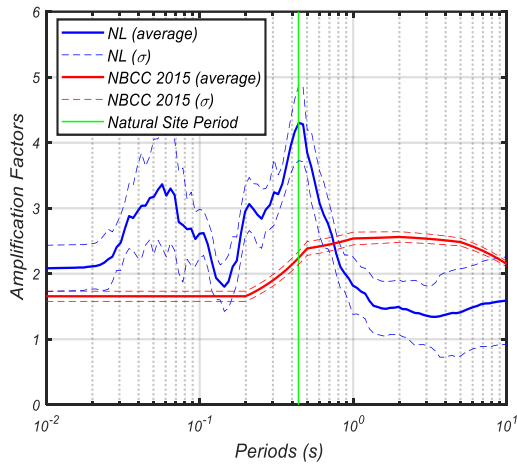


Figure B-9 – Soil Profile #9 Amplification, Case 1 (Scenario 2)

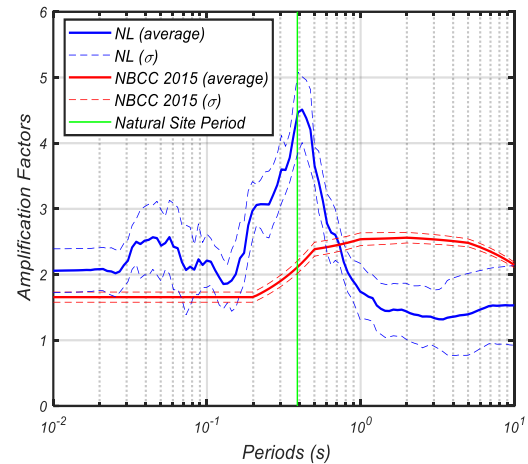


Figure B-10 – Soil Profile #10 Amplification, Case 1 (Scenario 2)

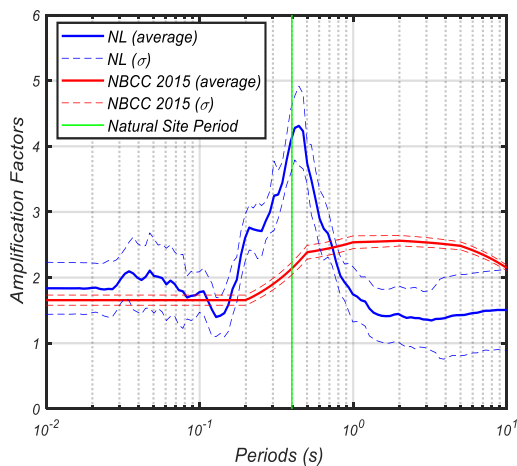


Figure B-11 – Soil Profile #11 Amplification, Case 1 (Scenario 2)

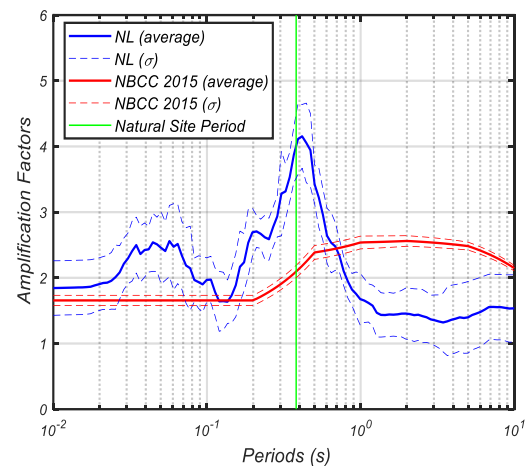


Figure B-12 – Soil Profile #12 Amplification, Case 1 (Scenario 2)

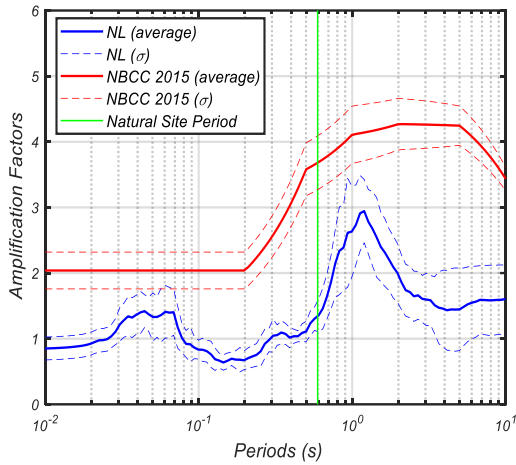


Figure B-13 – Soil Profile #13 Amplification, Case 2 (Scenario 2)

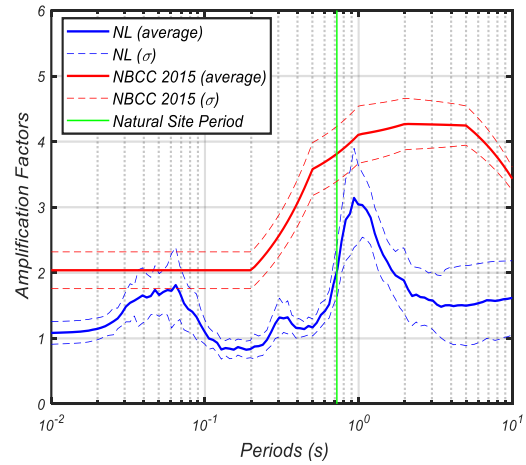


Figure B-14 – Soil Profile #14 Amplification, Case 3 (Scenario 2)

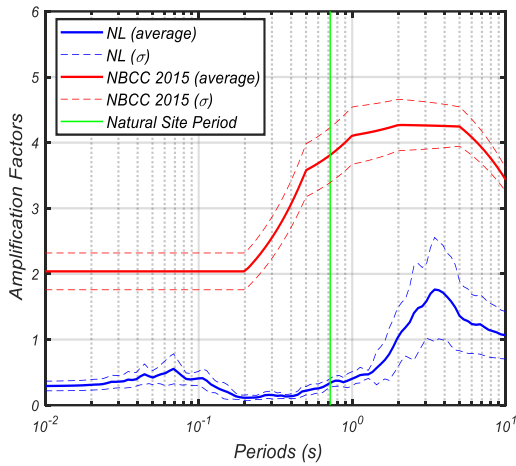


Figure B-15 – Soil Profile #15 Amplification, Case 2 (Scenario 2)

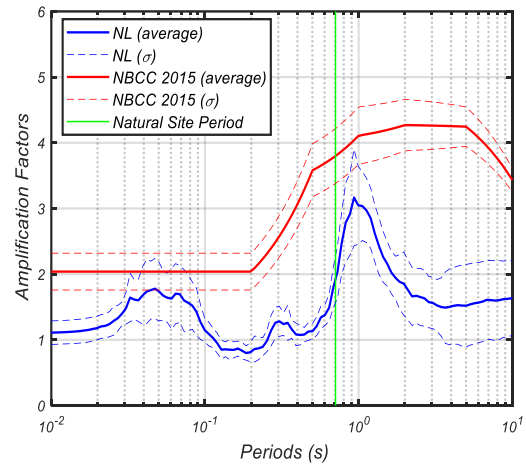


Figure B-16 – Soil Profile #16 Amplification, Case 3 (Scenario 2)

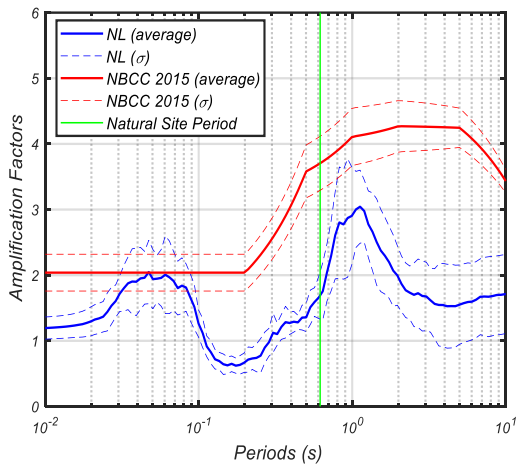


Figure B-17 – Soil Profile #17 Amplification, Case 3 (Scenario 2)

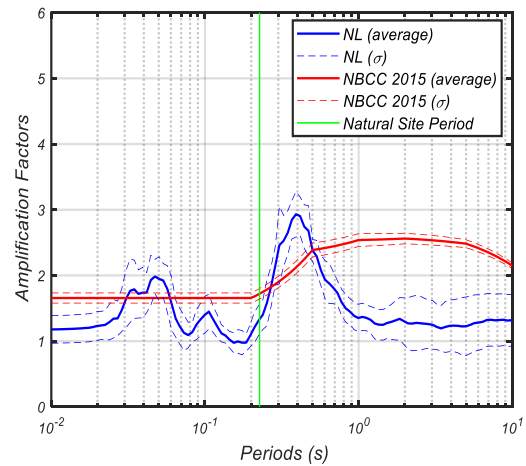


Figure B-18 – Soil Profile #18 Amplification, Case 1 (Scenario 2)

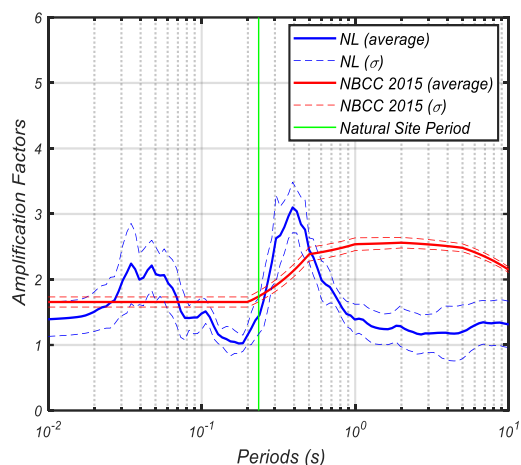


Figure B-19 – Soil Profile #19 Amplification, Case 1 (Scenario 2)

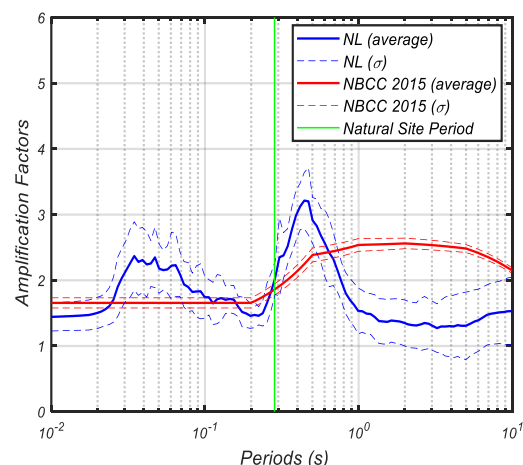


Figure B-20 – Soil Profile #20 Amplification, Case 1 (Scenario 2)

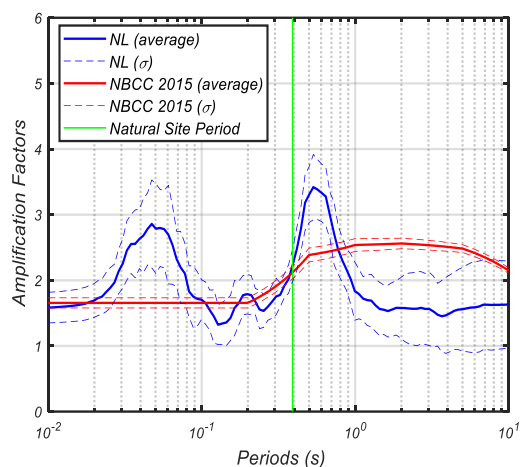


Figure B-21 – Soil Profile #21 Amplification, Case 1 (Scenario 2)

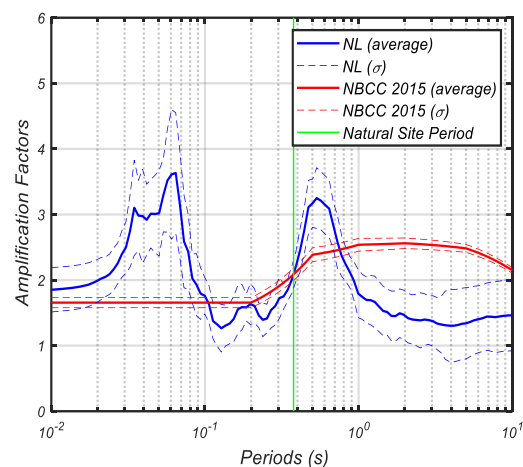


Figure B-22 – Soil Profile #22 Amplification, Case 1 (Scenario 2)

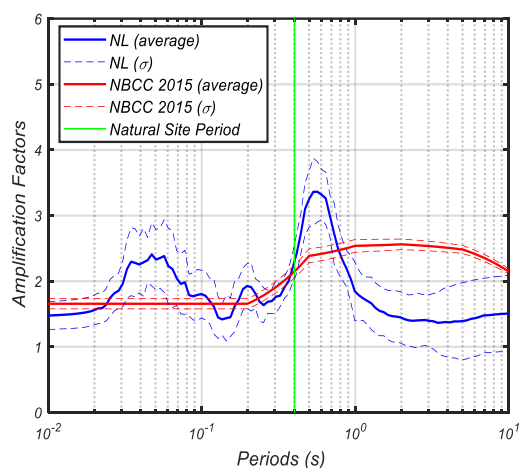


Figure B-23 – Soil Profile #23 Amplification, Case 1 (Scenario 2)

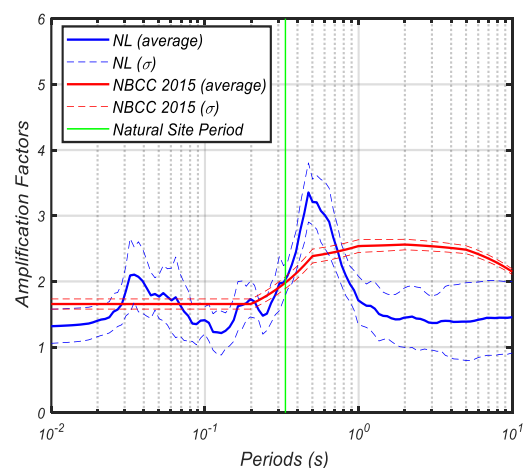


Figure B-24 – Soil Profile #24 Amplification, Case 1 (Scenario 2)

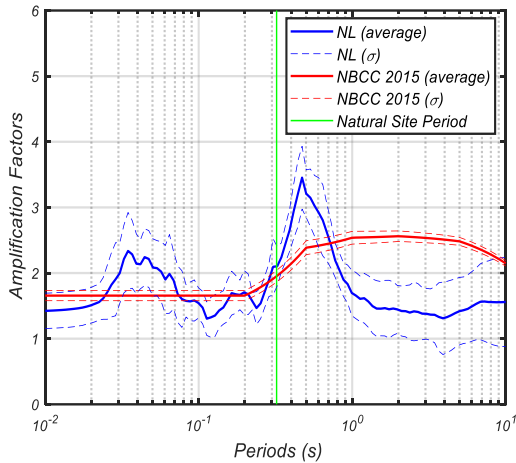


Figure B-25 – Soil Profile #25 Amplification, Case 1 (Scenario 2)

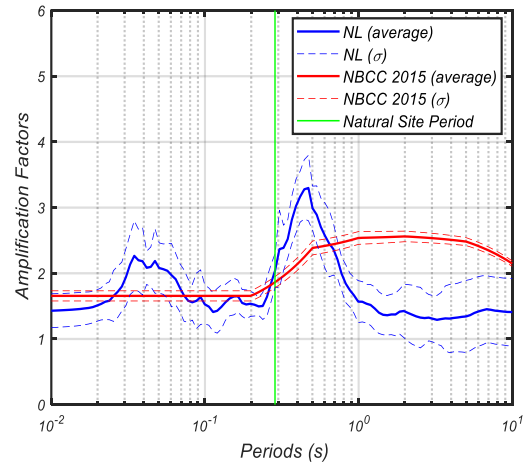


Figure B-26 – Soil Profile #26 Amplification, Case 1 (Scenario 2)

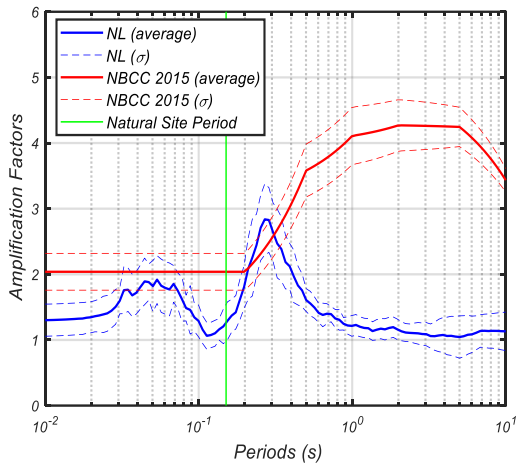


Figure B-27 – Soil Profile #27 Amplification, Case 3 (Scenario 2)

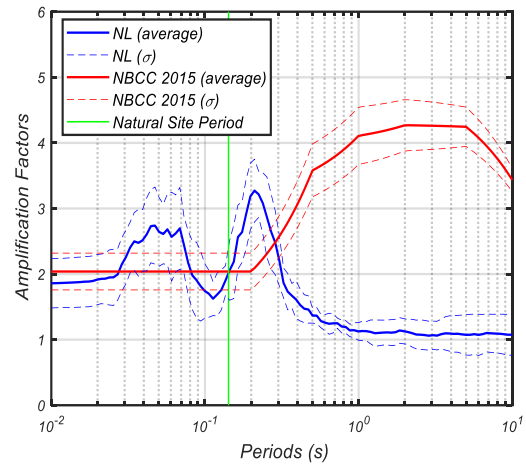


Figure B-28 – Soil Profile #28 Amplification, Case 1 (Scenario 2)

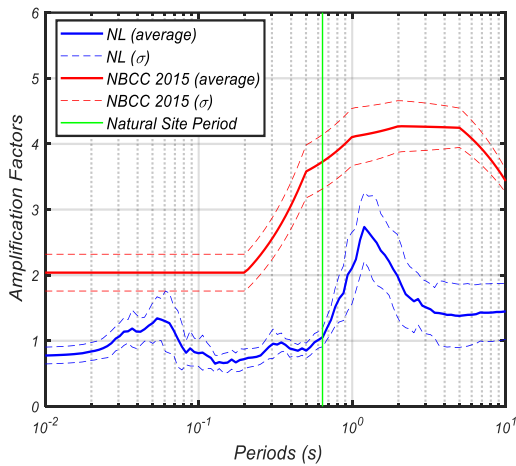


Figure B-29 – Soil Profile #29 Amplification, Case 2 (Scenario 2)

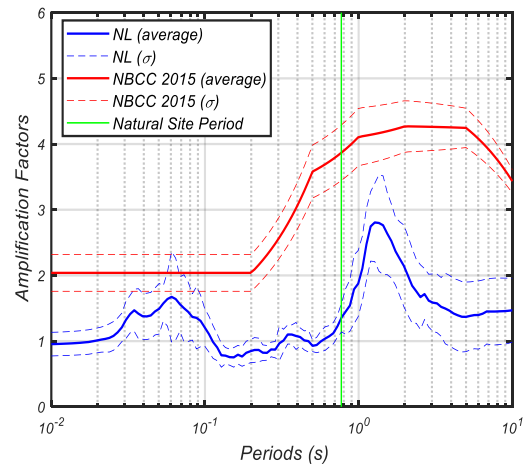


Figure B-30 – Soil Profile #30 Amplification, Case 3 (Scenario 2)

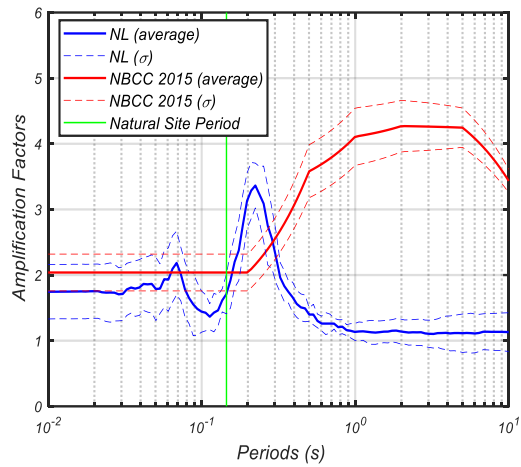


Figure B-31 – Soil Profile #31 Amplification, Case 1 (Scenario 2)

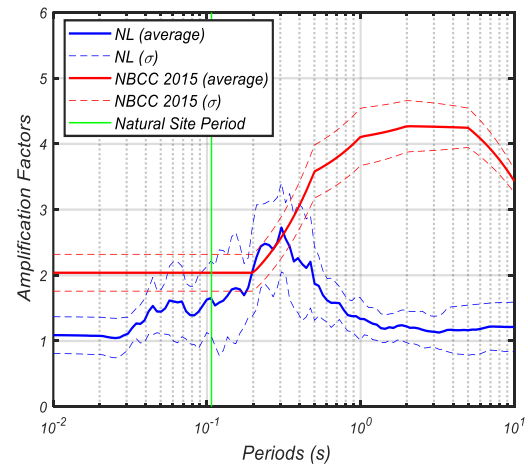


Figure B-32 – Soil Profile #32 Amplification, Case 3 (Scenario 2)

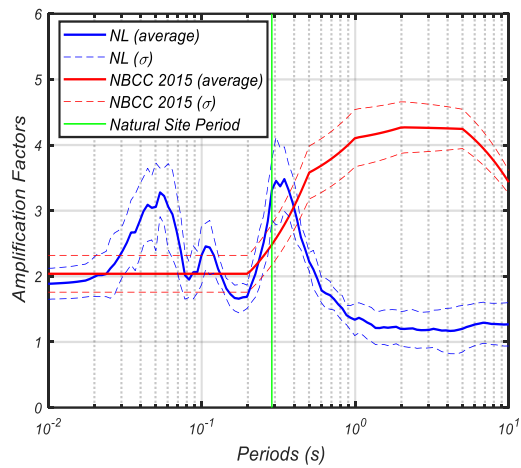


Figure B-33 – Soil Profile #33 Amplification, Case 1 (Scenario 2)

APPENDIX C AMPLIFICATION FACTORS FOR SCENARIO 3

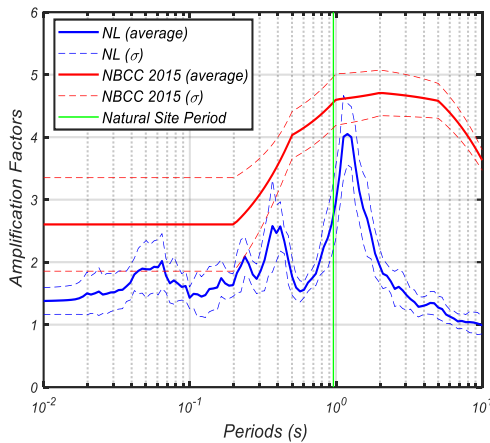


Figure C-1 – Soil Profile #1 Amplification, Case 3 (Scenario 3)

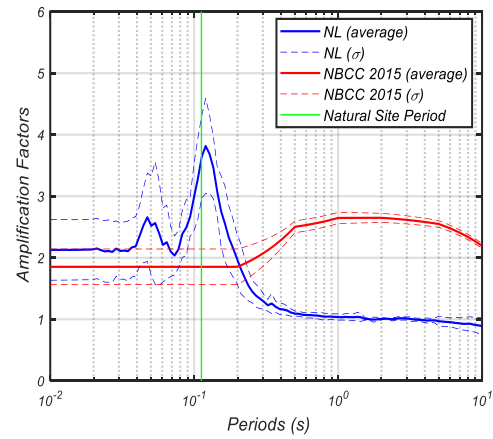


Figure C-2 – Soil Profile #2 Amplification, Case 1 (Scenario 3)

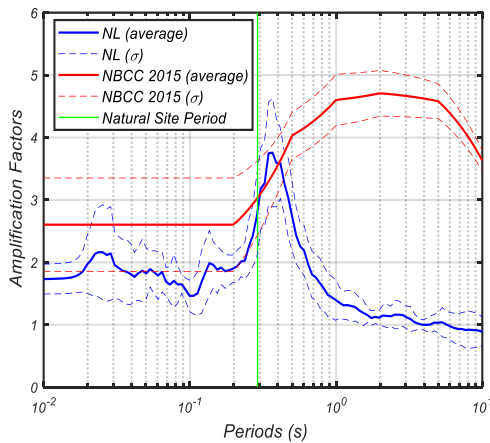


Figure C-3 – Soil Profile #3 Amplification, Case 3 (Scenario 3)

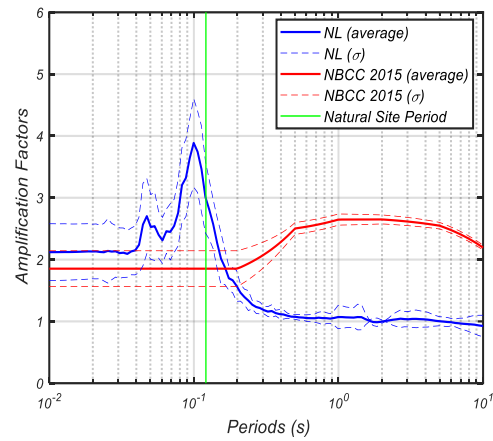


Figure C-4 – Soil Profile #4 Amplification, Case 1 (Scenario 3)

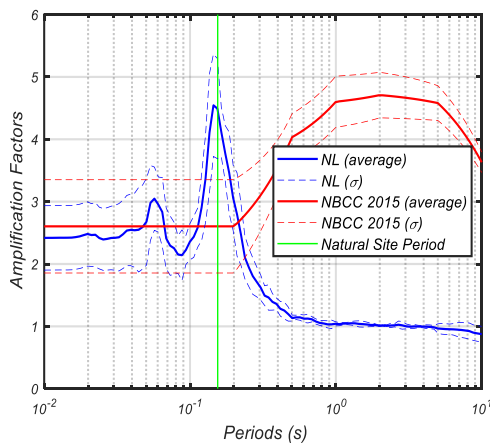


Figure C-5 – Soil Profile #5 Amplification, Case 1 (Scenario 3)

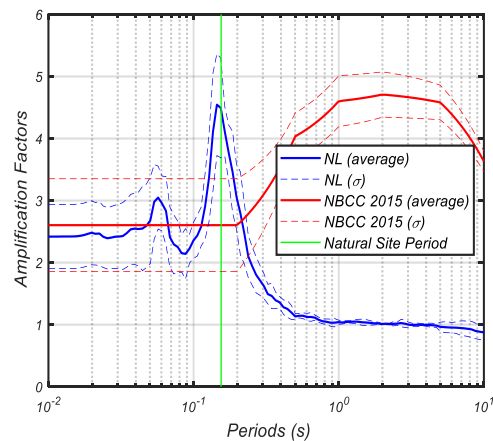


Figure C-6 – Soil Profile #6 Amplification, Case 1 (Scenario 3)

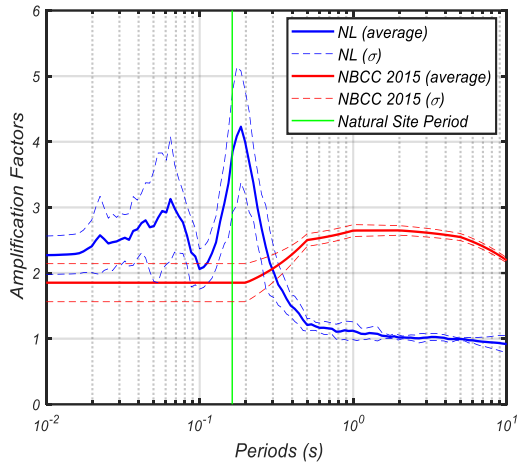


Figure C-7 – Soil Profile #7 Amplification, Case 1 (Scenario 3)

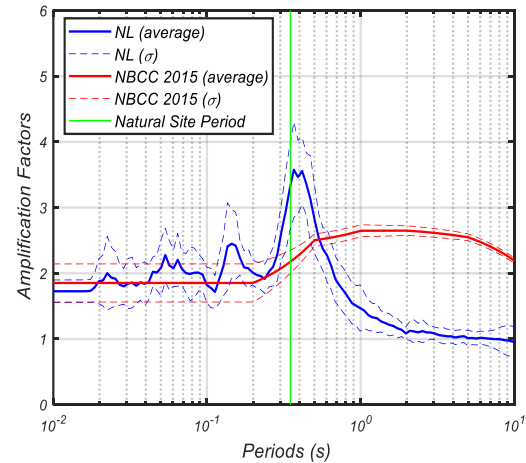


Figure C-8 – Soil Profile #8 Amplification, Case 1 (Scenario 3)

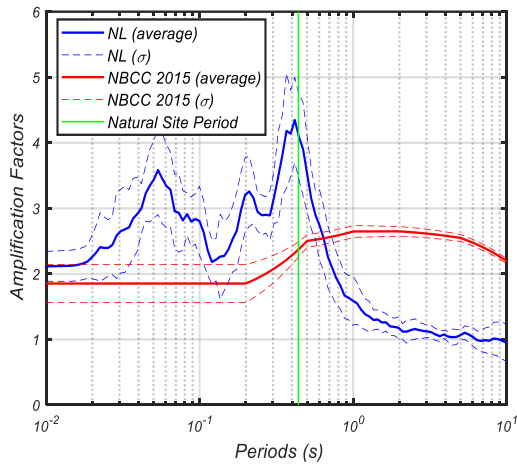


Figure C-9 – Soil Profile #9 Amplification, Case 1 (Scenario 3)

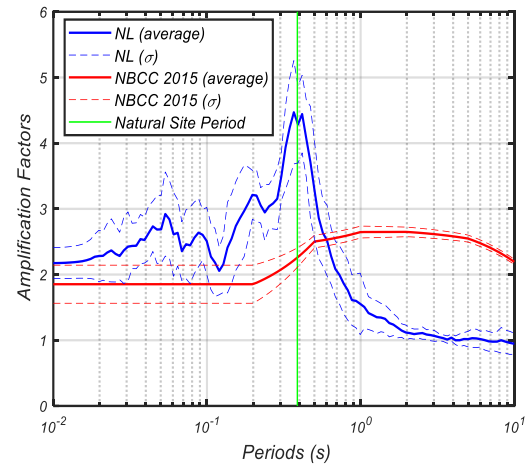


Figure C-10 – Soil Profile #10 Amplification, Case 1 (Scenario 3)

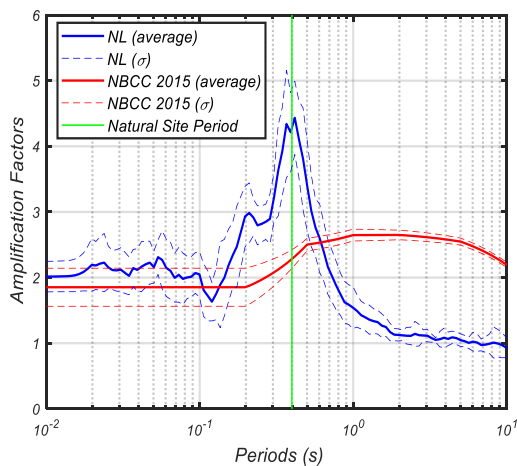


Figure C-11 – Soil Profile #11 Amplification, Case 1 (Scenario 3)

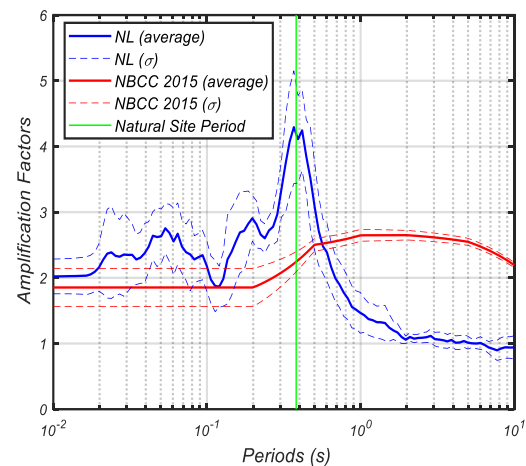


Figure C-12 – Soil Profile #12 Amplification, Case 1 (Scenario 3)

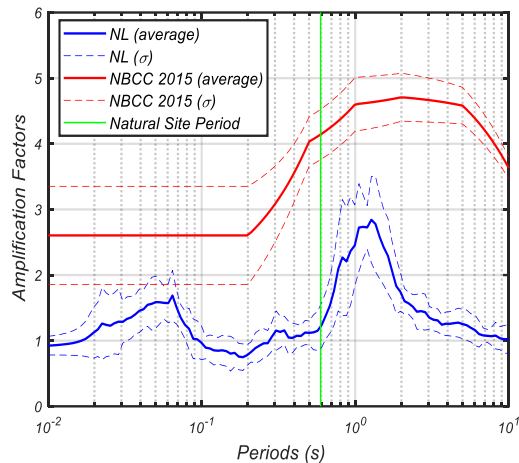


Figure C-13 – Soil Profile #13 Amplification, Case 2 (Scenario 3)

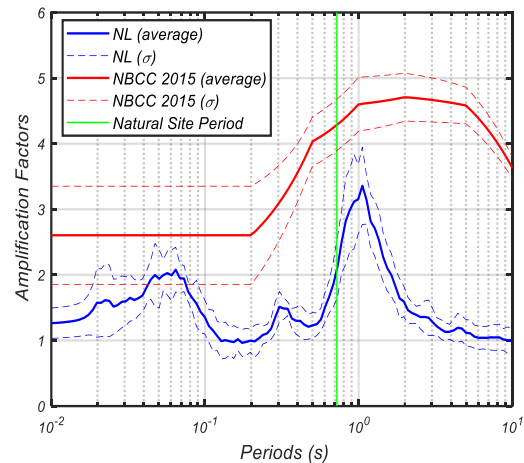


Figure C-14 – Soil Profile #14 Amplification, Case 3 (Scenario 3)

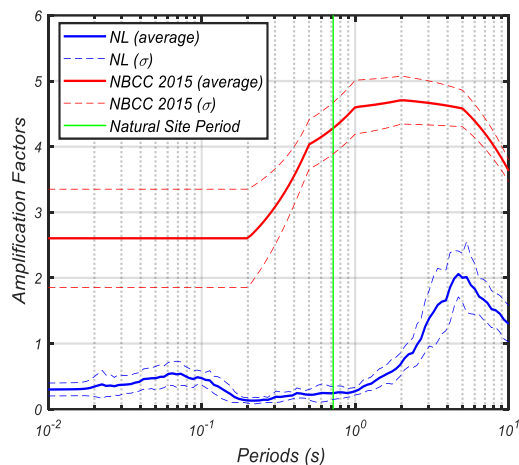


Figure C-15 – Soil Profile #15 Amplification, Case 2 (Scenario 3)

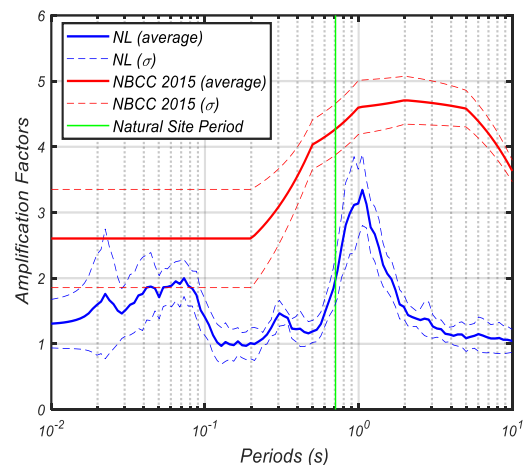


Figure C-16 – Soil Profile #16 Amplification, Case 3 (Scenario 3)

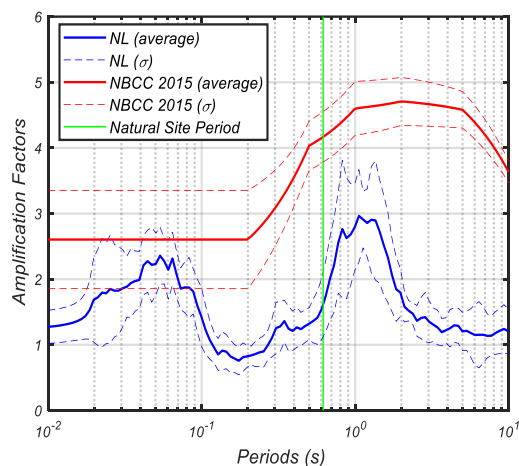


Figure C-17 – Soil Profile #17 Amplification, Case 3 (Scenario 3)

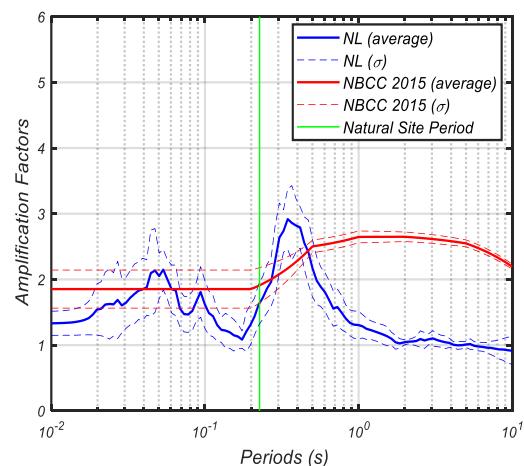


Figure C-18 – Soil Profile #18 Amplification, Case 1 (Scenario 3)

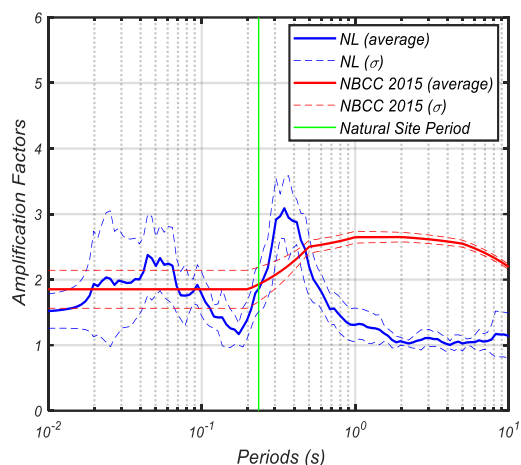


Figure C-19 – Soil Profile #19 Amplification, Case 1 (Scenario 3)

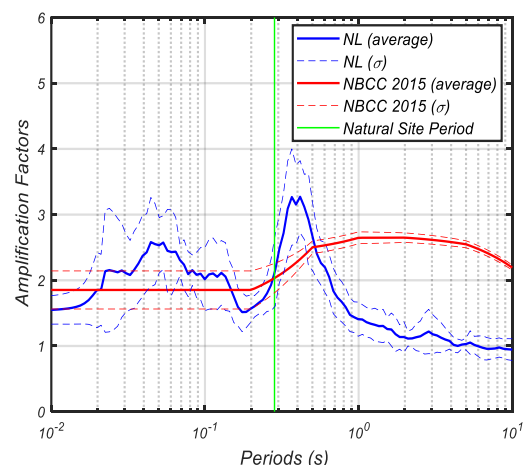


Figure C-20 – Soil Profile #20 Amplification, Case 1 (Scenario 3)

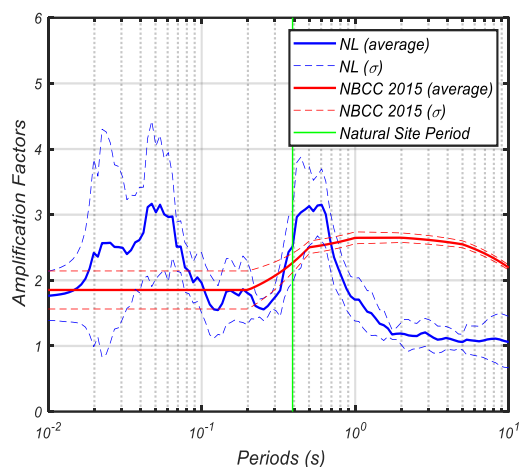


Figure C-21 – Soil Profile #21 Amplification, Case 1 (Scenario 3)

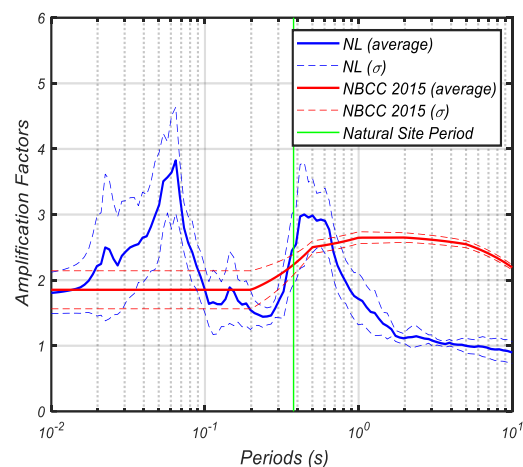


Figure C-22 – Soil Profile #22 Amplification, Case 1 (Scenario 3)

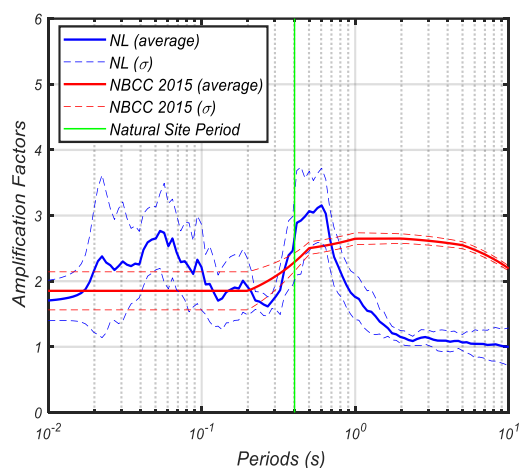


Figure C-23 – Soil Profile #23 Amplification, Case 1 (Scenario 3)

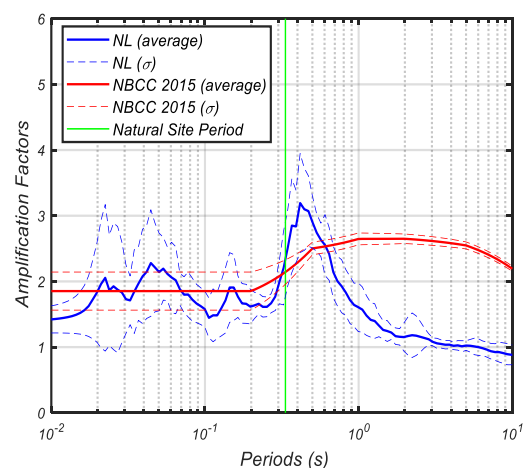


Figure C-24 – Soil Profile #24 Amplification, Case 1 (Scenario 3)

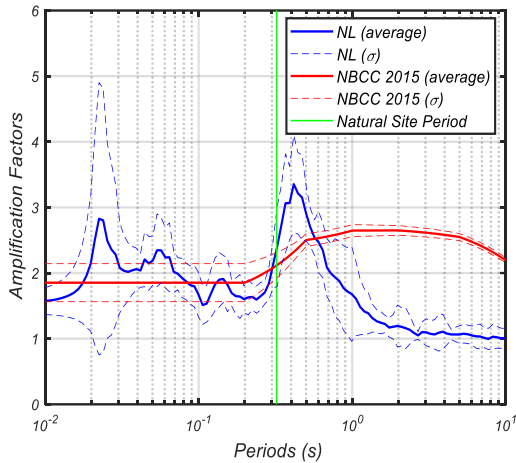


Figure C-25 – Soil Profile #25 Amplification, Case 1 (Scenario 3)

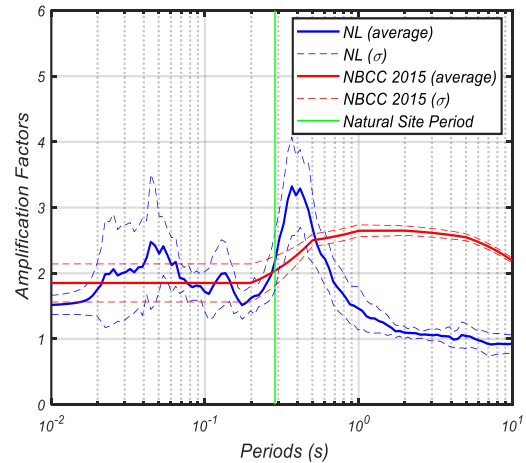


Figure C-26 – Soil Profile #26 Amplification, Case 1 (Scenario 3)

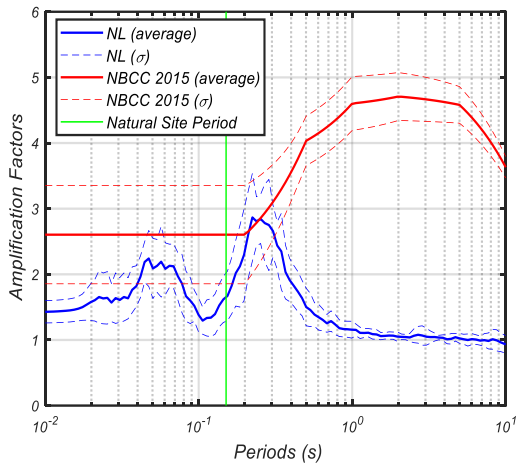


Figure C-27 – Soil Profile #27 Amplification, Case 3 (Scenario 3)

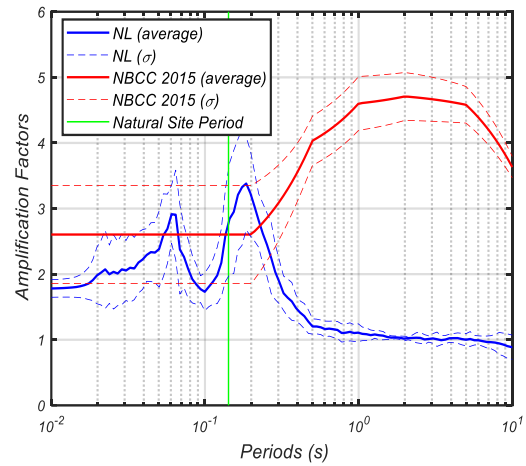


Figure C-28 – Soil Profile #28 Amplification, Case 1 (Scenario 3)

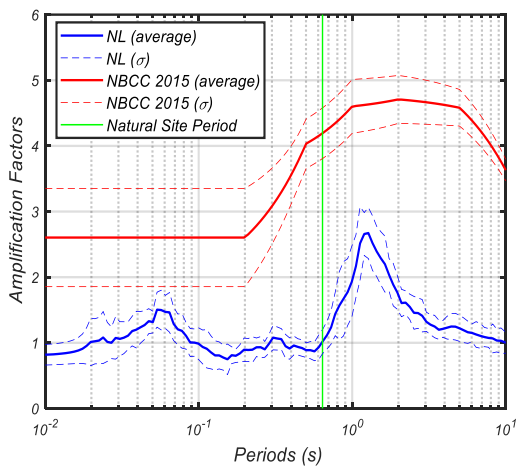


Figure C-29 – Soil Profile #29 Amplification, Case 2 (Scenario 3)

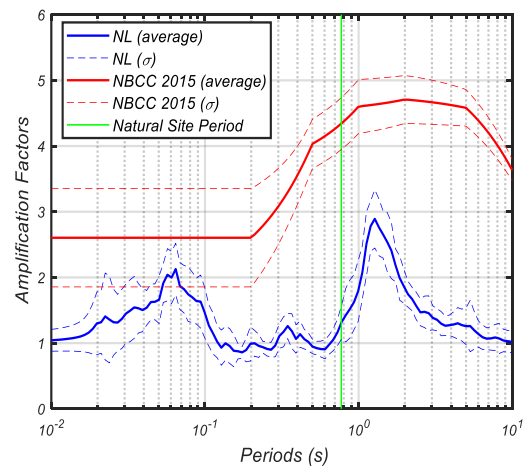


Figure C-30 – Soil Profile #30 Amplification, Case 3 (Scenario 3)

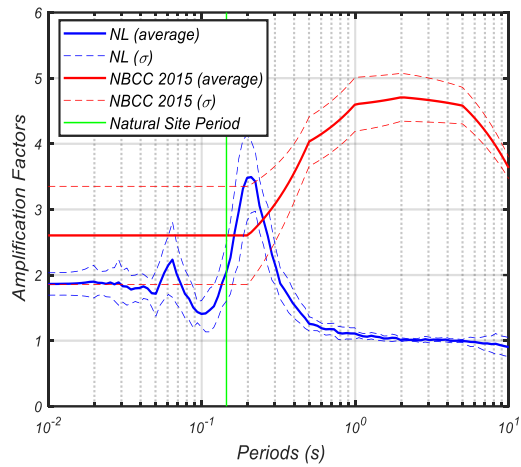


Figure C-31 – Soil Profile #31 Amplification, Case 1 (Scenario 3)

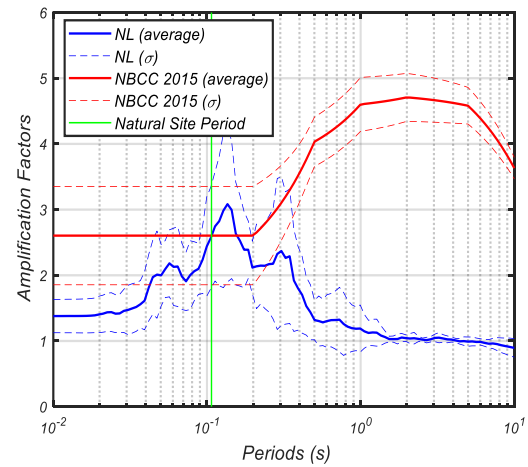


Figure C-32 – Soil Profile #32 Amplification, Case 3 (Scenario 3)

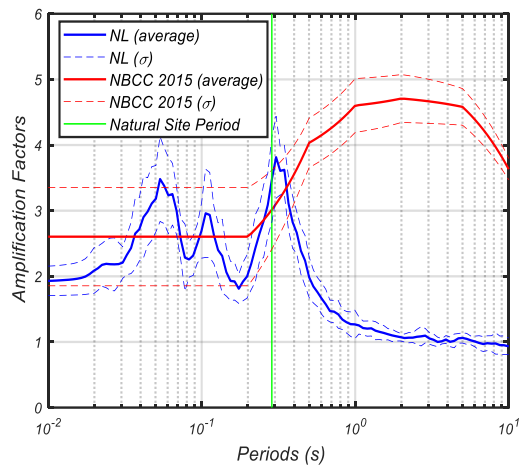


Figure C-33 – Soil Profile #33 Amplification, Case 1 (Scenario 3)

APPENDIX D CORRELATIONS BETWEEN RESIDUALS AND V_{s30}

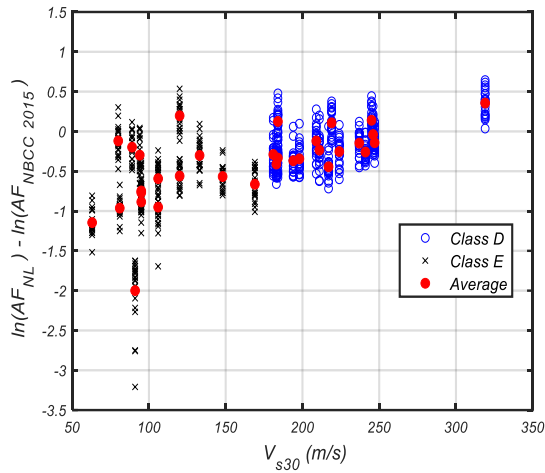


Figure D-1 – Residuals vs. V_{s30} at $T=0.01s$ (Scenario 1)

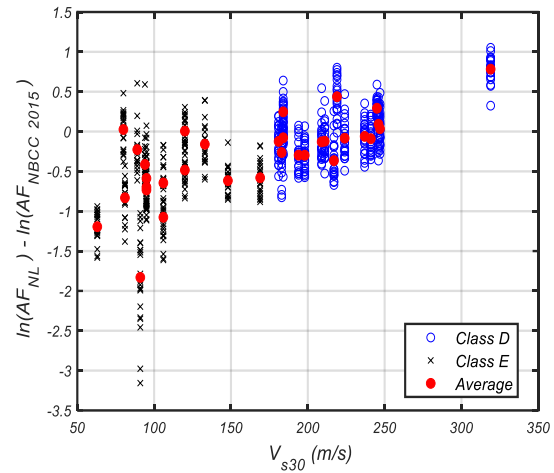


Figure D-2 – Residuals vs. V_{s30} at $T=0.1s$ (Scenario 1)

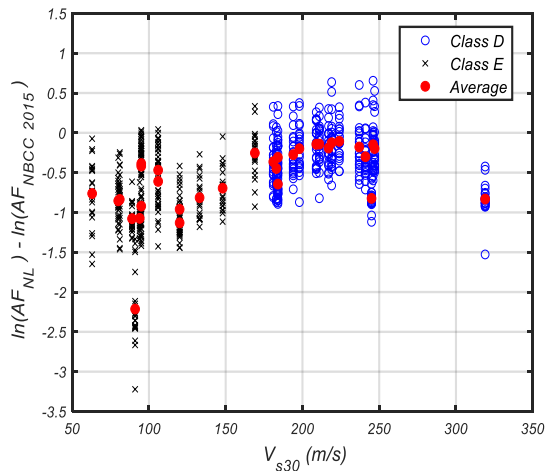


Figure D-3 – Residuals vs. V_{s30} at $T=1s$ (Scenario 1)

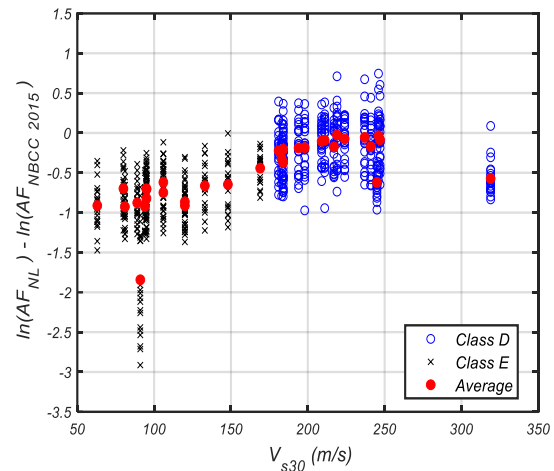


Figure D-4 – Residuals vs. V_{s30} at $T=10s$ (Scenario 1)

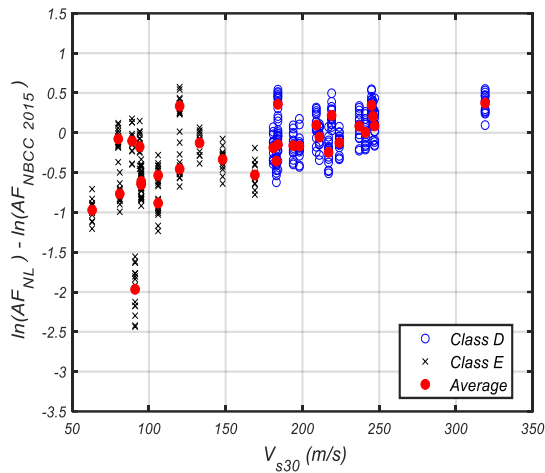


Figure D-5 – Residuals vs. V_{s30} at $T=0.01s$ (Scenario 2)

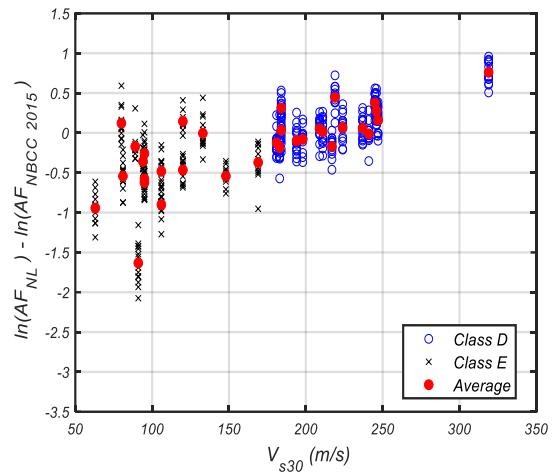


Figure D-6 – Residuals vs. V_{s30} at $T=0.1s$ (Scenario 2)

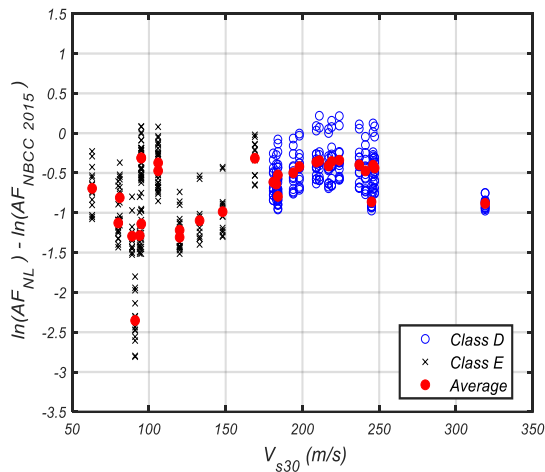


Figure D-7 – Residuals vs. V_{s30} at $T=1s$ (Scenario 2)

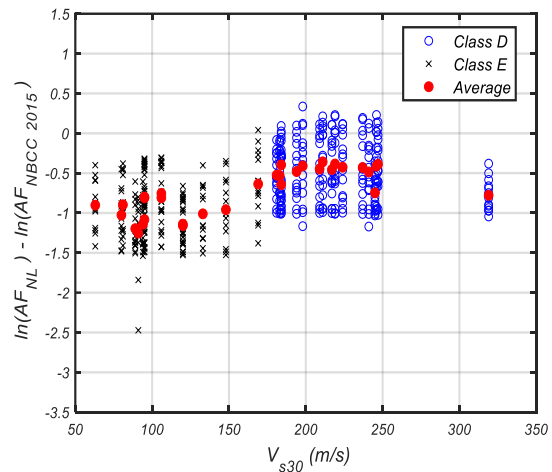


Figure D-8 – Residuals vs. V_{s30} at $T=10s$ (Scenario 2)

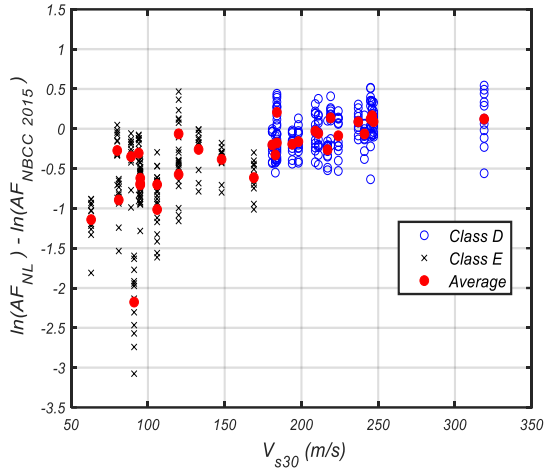


Figure D-9 – Residuals vs. V_{s30} at $T=0.01s$ (Scenario 3)

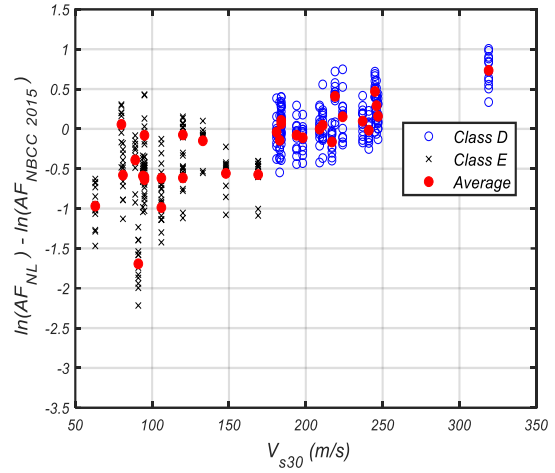


Figure D-10 – Residuals vs. V_{s30} at $T=0.1s$ (Scenario 3)

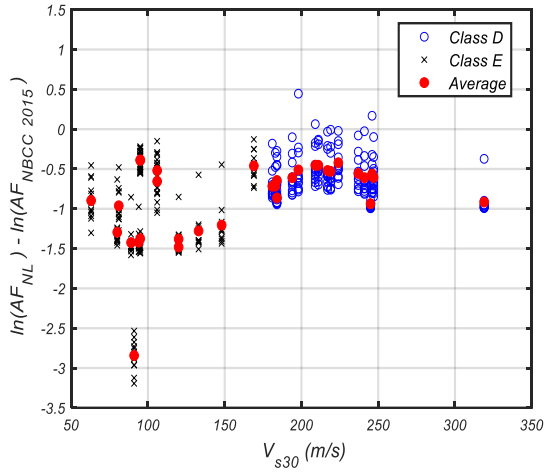


Figure D-11 – Residuals vs. V_{s30} at $T=1s$ (Scenario 3)

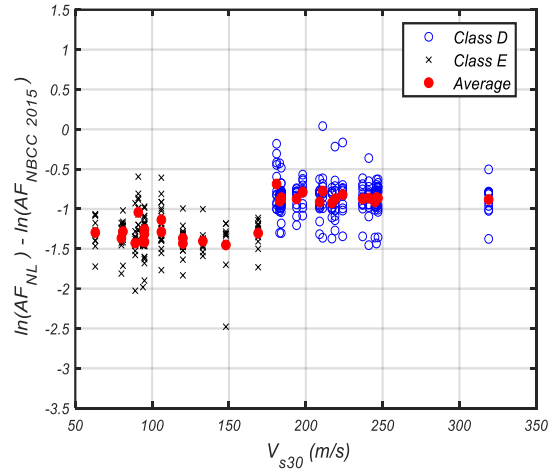


Figure D-12 – Residuals vs. V_{s30} at $T=10s$ (Scenario 3)

APPENDIX E CORRELATIONS BETWEEN RESIDUALS AND T_0

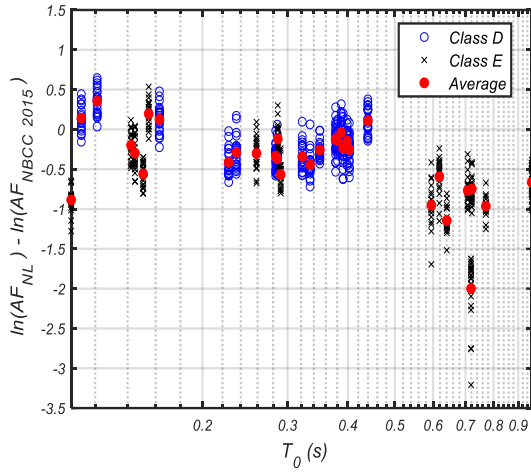


Figure E-1 – Residuals vs. T_0 at $T=0.01s$
(Scenario 1)

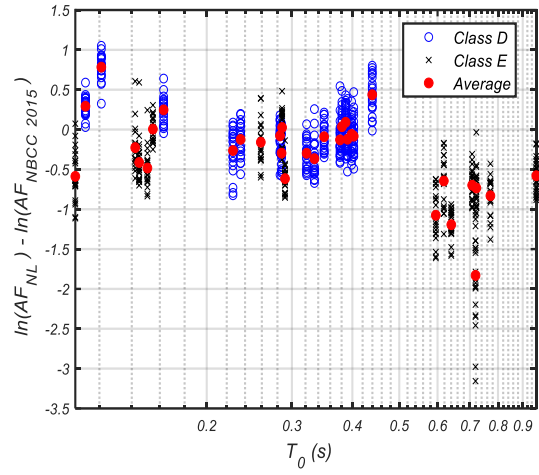


Figure E-2 – Residuals vs. T_0 at $T=0.1s$
(Scenario 1)

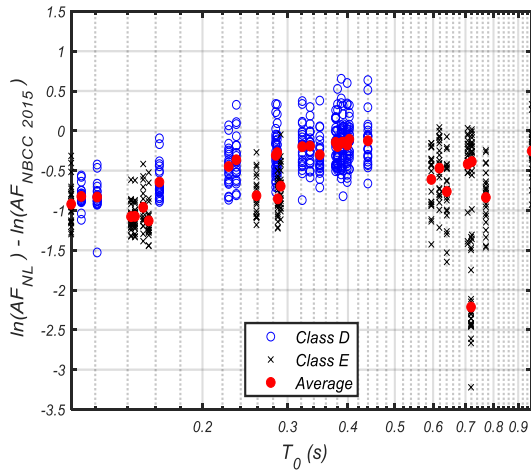


Figure E-3 – Residuals vs. T_0 at $T=1s$
(Scenario 1)

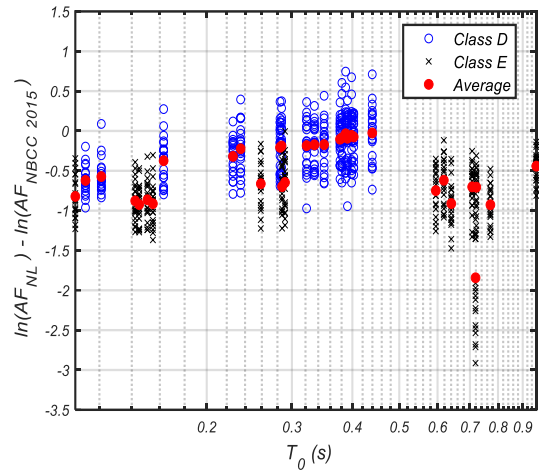


Figure E-4 – Residuals vs. T_0 at $T=10s$
(Scenario 1)

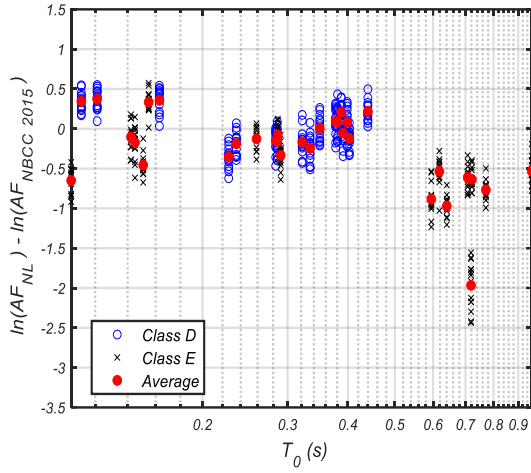


Figure E-5 – Residuals vs. T_0 at $T=0.01s$
(Scenario 2)

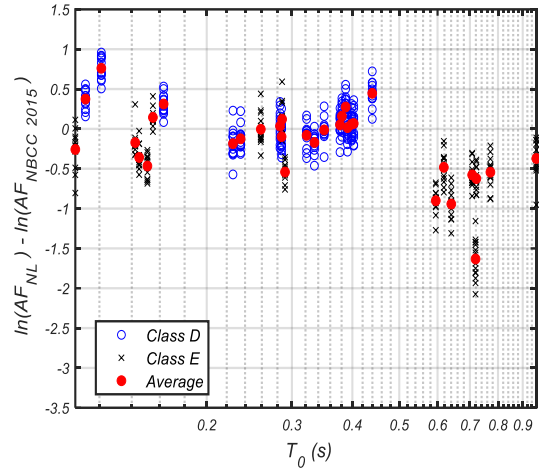


Figure E-6 – Residuals vs. T_0 at $T=0.1s$
(Scenario 2)

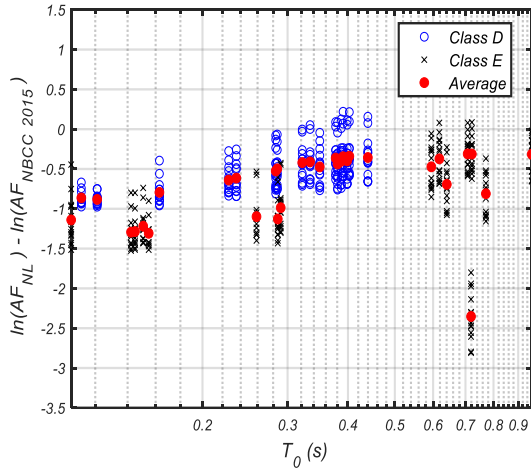


Figure E-7 – Residuals vs. T_0 at $T=1s$
(Scenario 2)

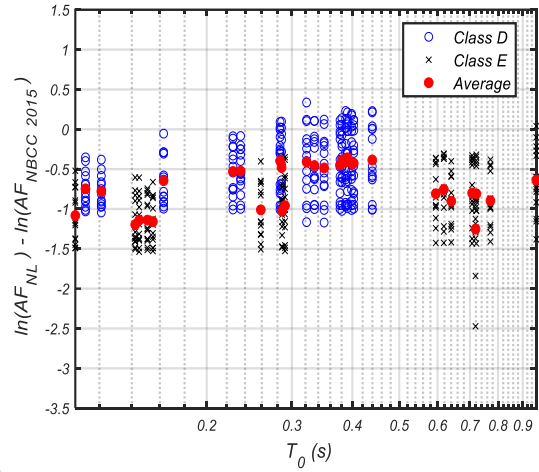


Figure E-8 – Residuals vs. T_0 at $T=10s$
(Scenario 2)

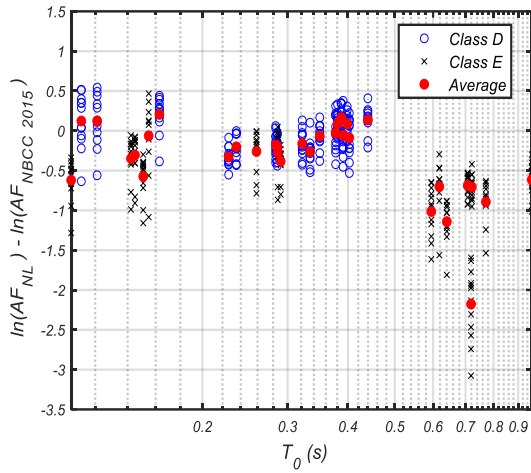


Figure E-9 – Residuals vs. T_0 at $T=0.01s$
(Scenario 3)

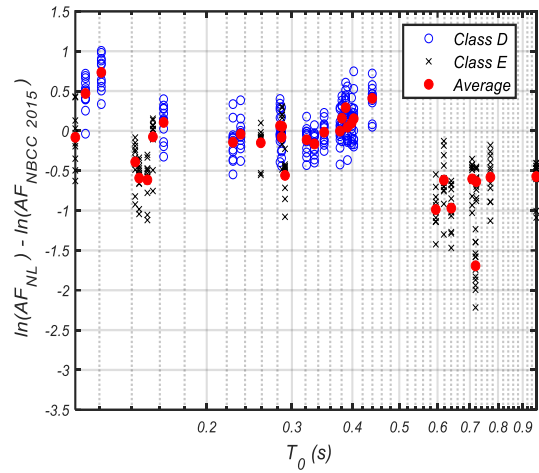


Figure E-10 – Residuals vs. T_0 at $T=0.1s$
(Scenario 3)

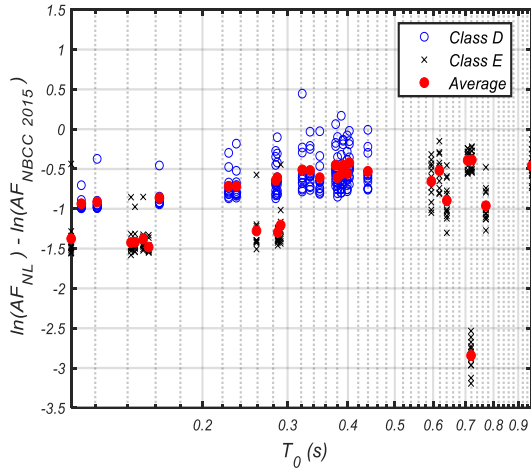


Figure E-11 – Residuals vs. T_0 at $T=1s$
(Scenario 3)

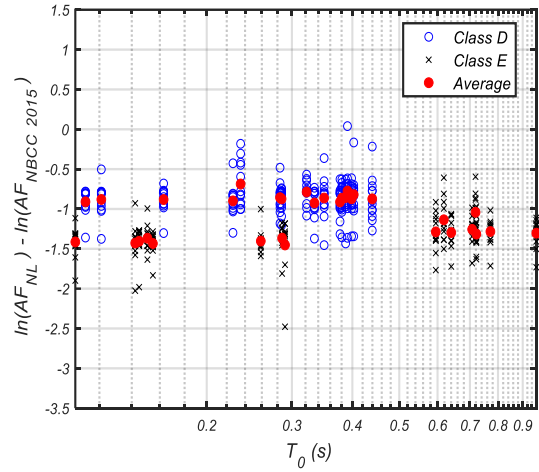


Figure E-12 – Residuals vs. T_0 at $T=10s$
(Scenario 3)

APPENDIX F CORRELATIONS BETWEEN RESIDUALS AND *PGA*

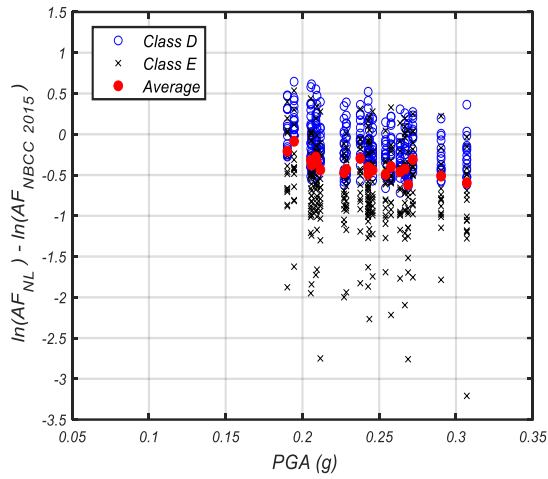


Figure F-1 – Residuals vs. PGA at $T=0.01s$ (Scenario 1)

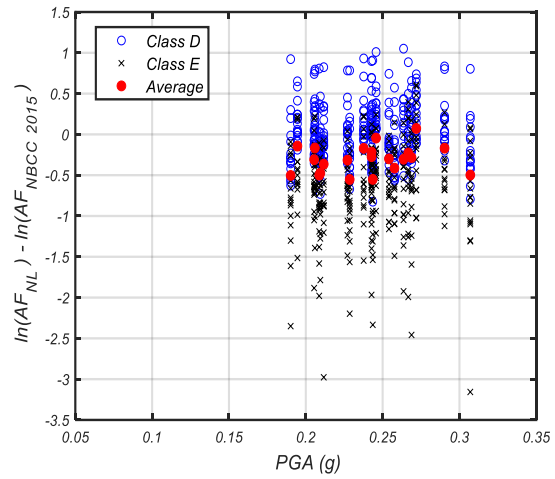


Figure F-2 – Residuals vs. PGA at $T=0.1s$ (Scenario 1)

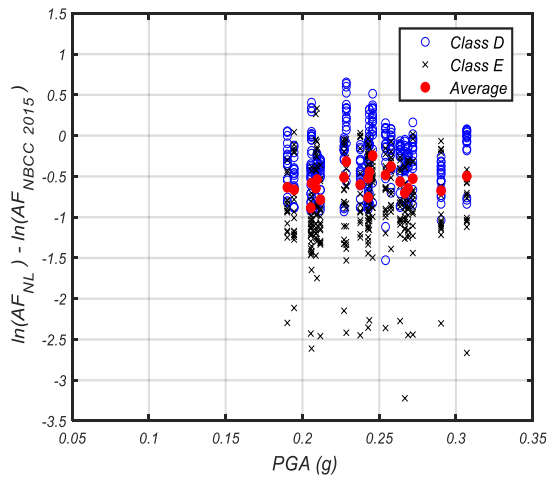


Figure F-3 – Residuals vs. PGA at $T=1s$ (Scenario 1)

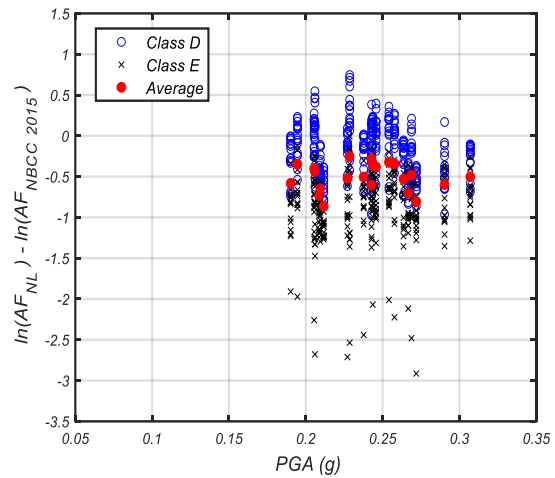


Figure F-4 – Residuals vs. PGA at $T=10s$ (Scenario 1)

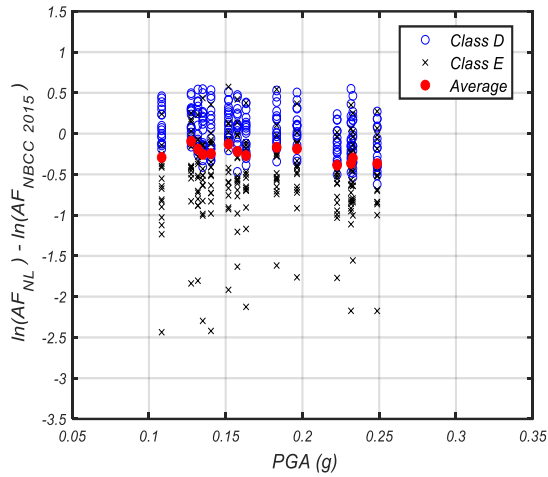


Figure F-5 – Residuals vs. PGA at T=0.01s
(Scenario 2)

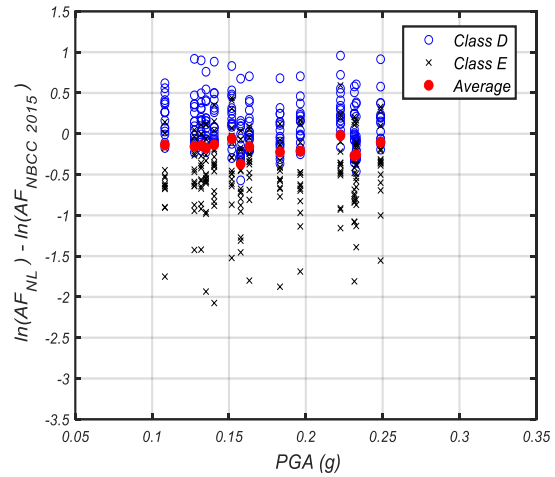


Figure F-6 – Residuals vs. PGA at T=0.1s
(Scenario 2)

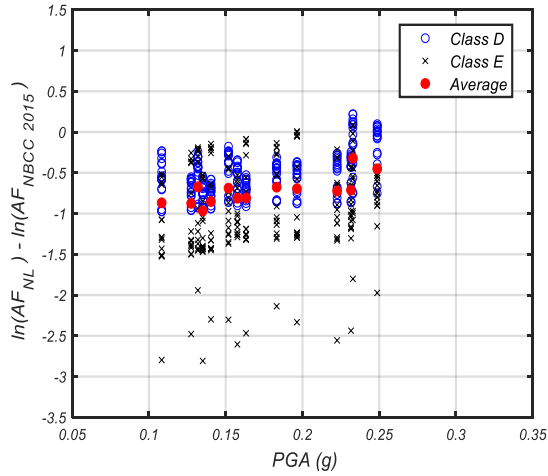


Figure F-7 – Residuals vs. PGA at T=1s
(Scenario 2)

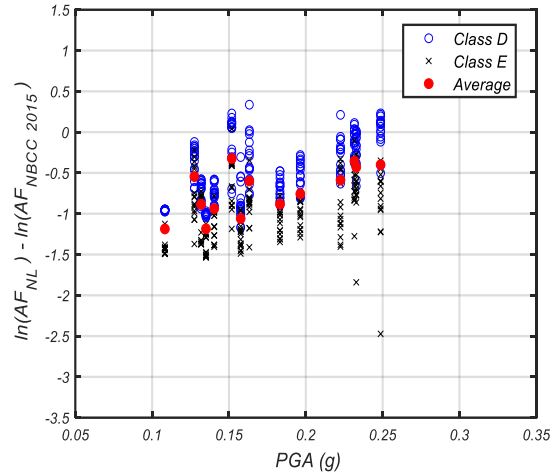


Figure F-8 – Residuals vs. PGA at T=10s
(Scenario 2)

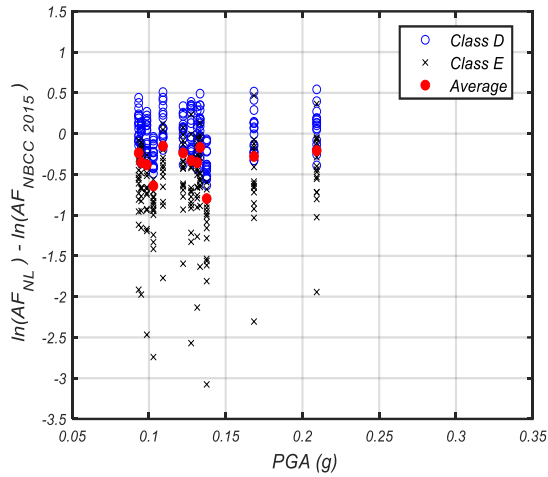


Figure F-9 – Residuals vs. PGA at T=0.01s
(Scenario 3)

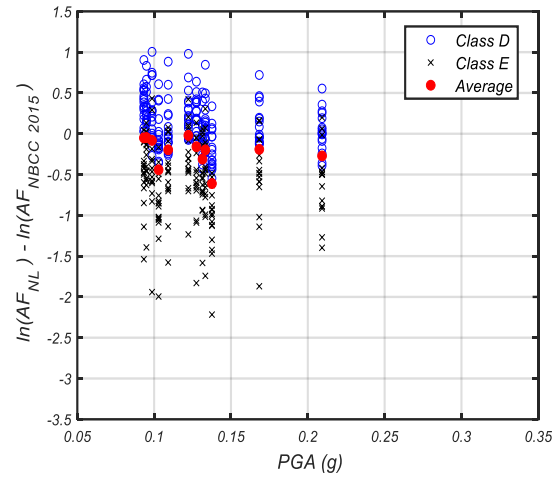


Figure F-10 – Residuals vs. PGA at T=0.1s
(Scenario 3)

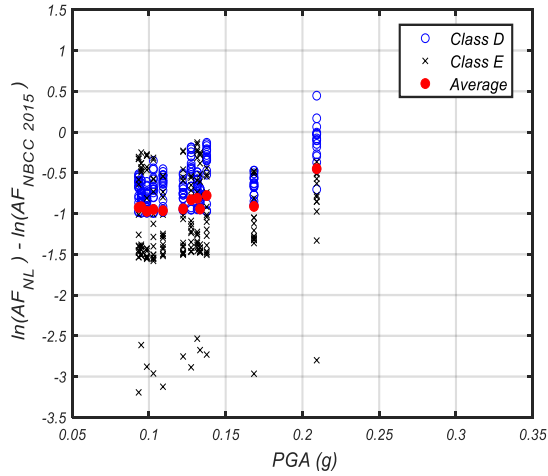


Figure F-11 – Residuals vs. PGA at T=1s
(Scenario 3)

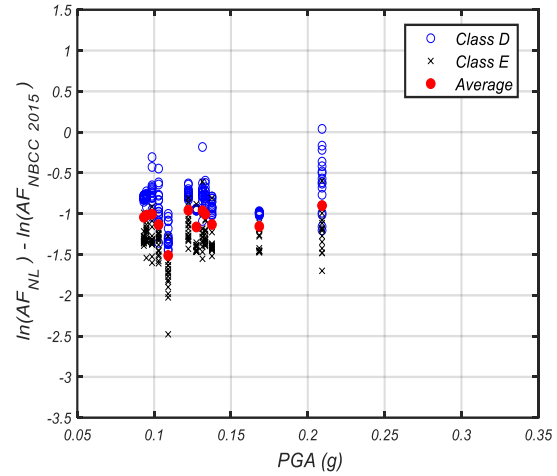


Figure F-12 – Residuals vs. PGA at T=10s
(Scenario 3)

APPENDIX G CORRELATIONS BETWEEN RESIDUALS AND $Sa(0.1)$

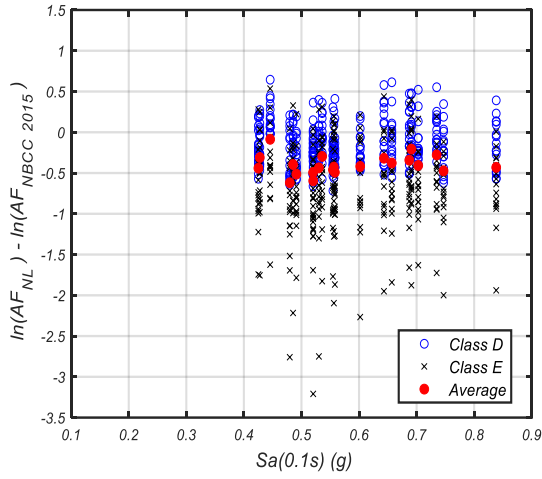


Figure G-1 – Residuals vs. $Sa(0.1s)$ at $T=0.01s$ (Scenario 1)

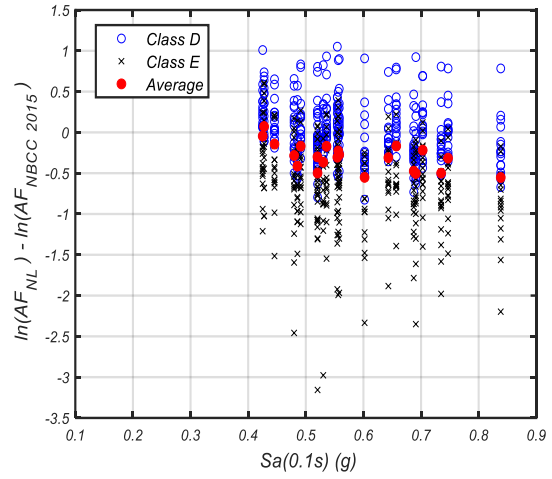


Figure G-2 – Residuals vs. $Sa(0.1s)$ at $T=0.1s$ (Scenario 1)

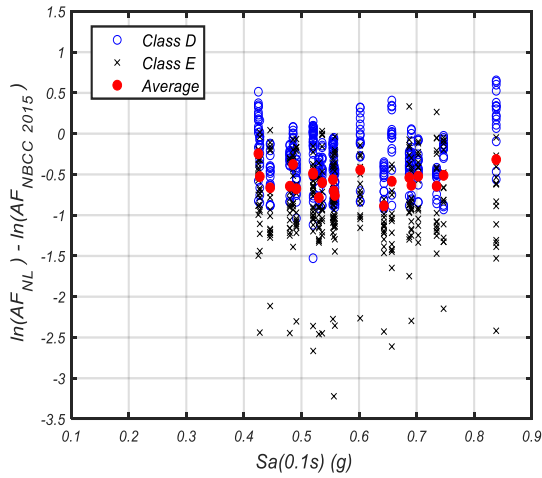


Figure G-3 – Residuals vs. $Sa(0.1s)$ at $T=1s$ (Scenario 1)

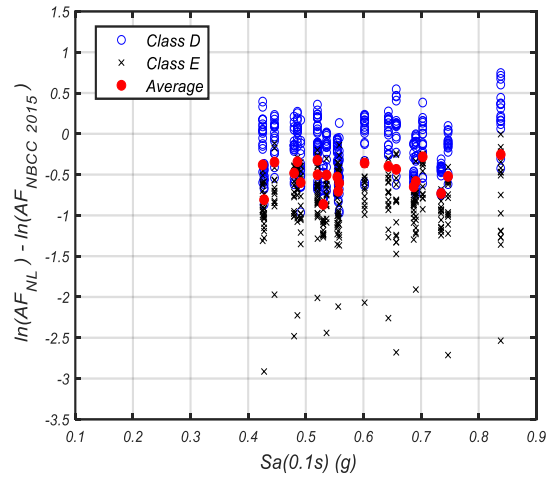


Figure G-4 – Residuals vs. $Sa(0.1s)$ at $T=10s$ (Scenario 1)

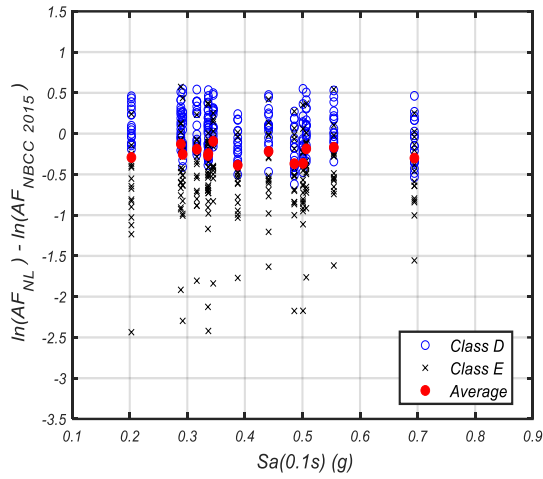


Figure G-5 – Residuals vs. $Sa(0.1s)$ at $T=0.01s$ (Scenario 2)

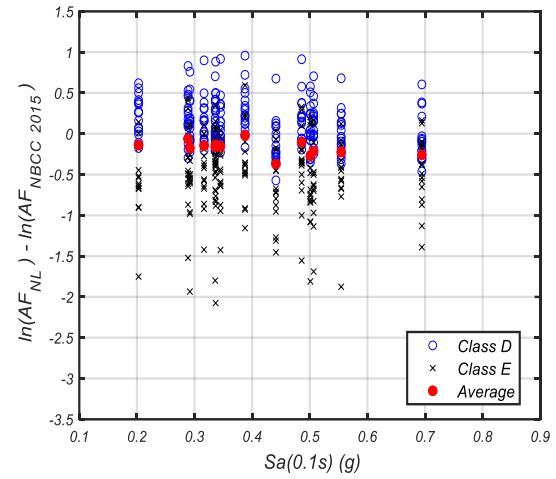


Figure G-6 – Residuals vs. $Sa(0.1s)$ at $T=0.1s$ (Scenario 2)

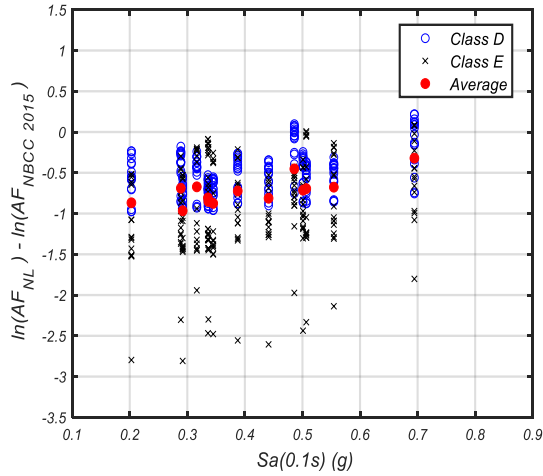


Figure G-7 – Residuals vs. $Sa(0.1s)$ at $T=1s$ (Scenario 2)

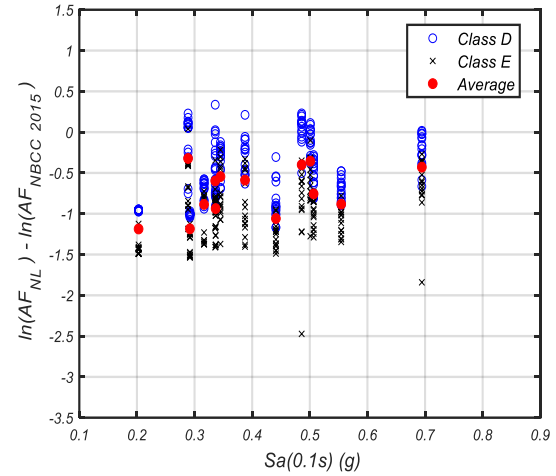


Figure G-8 – Residuals vs. $Sa(0.1s)$ at $T=10s$ (Scenario 2)

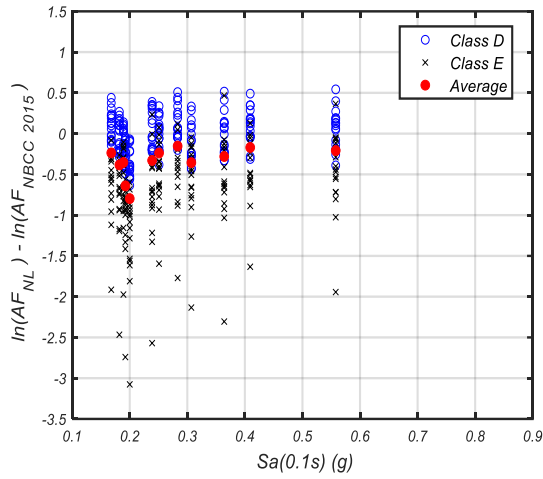


Figure G-9 – Residuals vs. $Sa(0.1s)$ at $T=0.01s$ (Scenario 3)

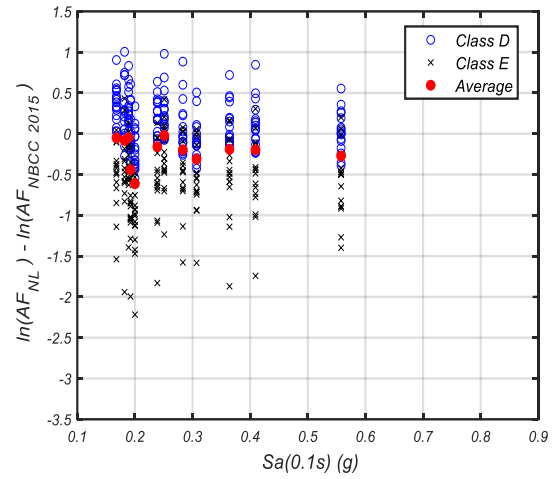


Figure G-10 – Residuals vs. $Sa(0.1s)$ at $T=0.1s$ (Scenario 3)

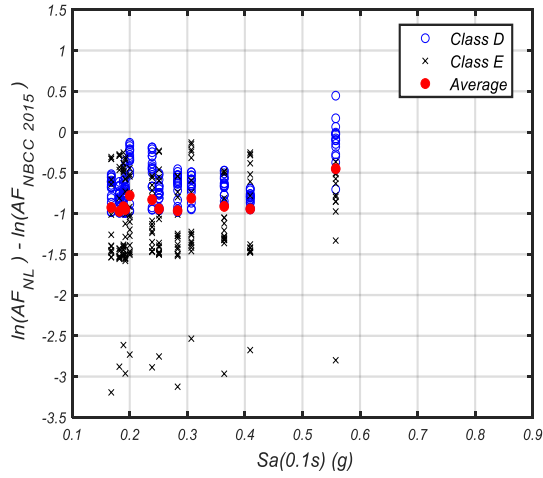


Figure G-11 – Residuals vs. $Sa(0.1s)$ at $T=1s$ (Scenario 3)

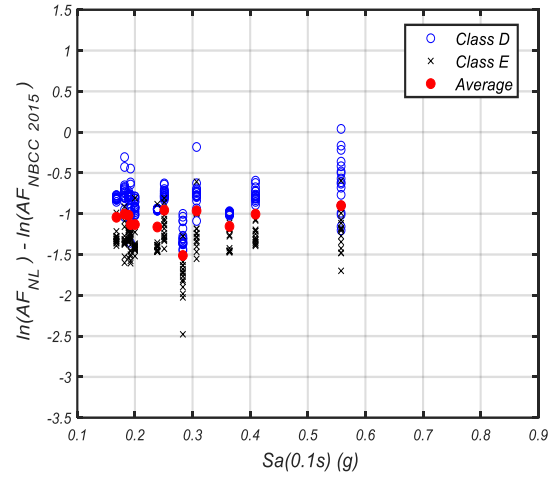


Figure G-12 – Residuals vs. $Sa(0.1s)$ at $T=10s$ (Scenario 3)

APPENDIX H CORRELATIONS BETWEEN RESIDUALS AND $Sa(1)$

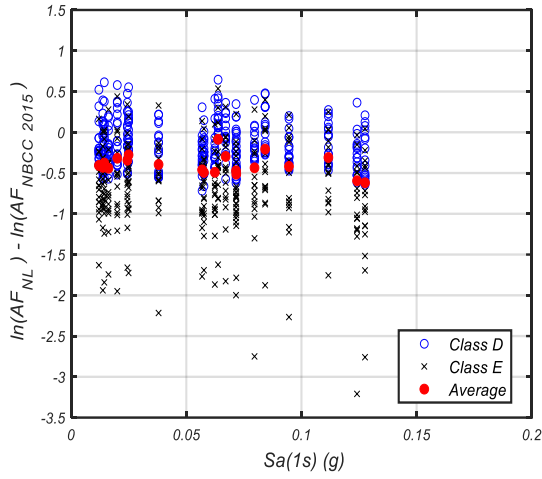


Figure H-1 – Residuals vs. $Sa(1s)$ at $T=0.01s$ (Scenario 1)

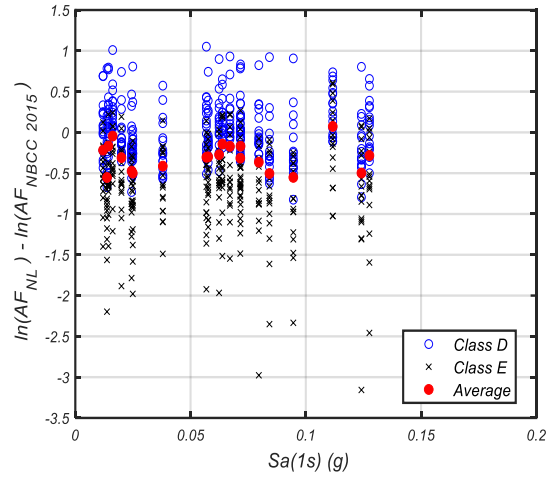


Figure H-2 – Residuals vs. $Sa(1s)$ at $T=0.1s$ (Scenario 1)

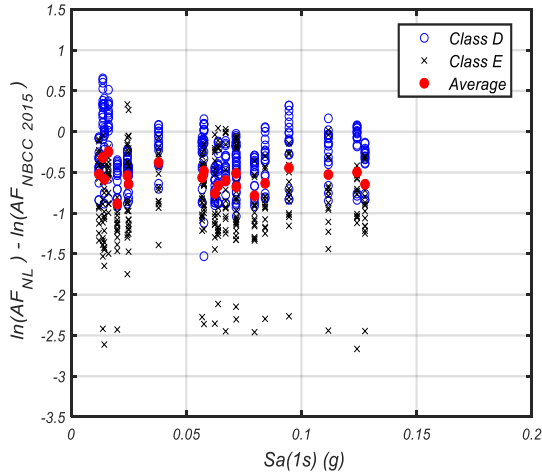


Figure H-3 – Residuals vs. $Sa(1s)$ at $T=1s$ (Scenario 1)

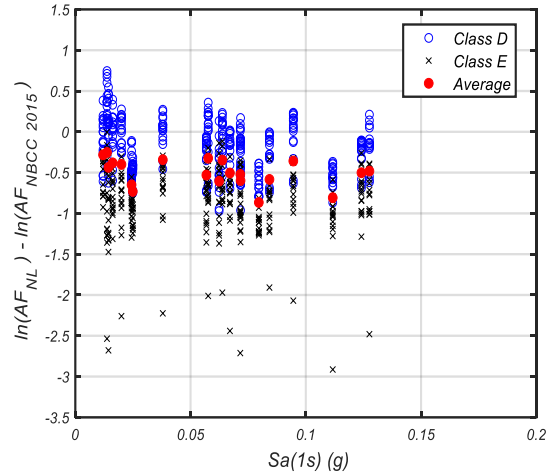


Figure H-4 – Residuals vs. $Sa(1s)$ at $T=10s$ (Scenario 1)

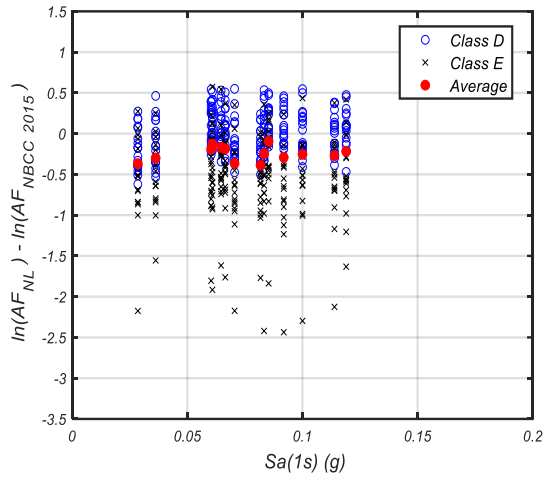


Figure H-5 – Residuals vs. $Sa(1s)$ at $T=0.01s$ (Scenario 2)

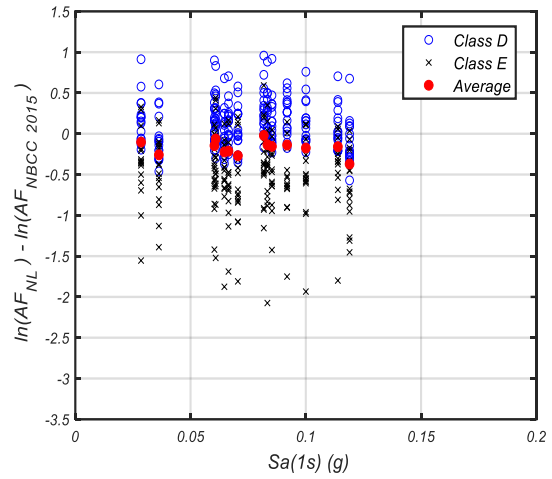


Figure H-6 – Residuals vs. $Sa(1s)$ at $T=0.1s$ (Scenario 2)

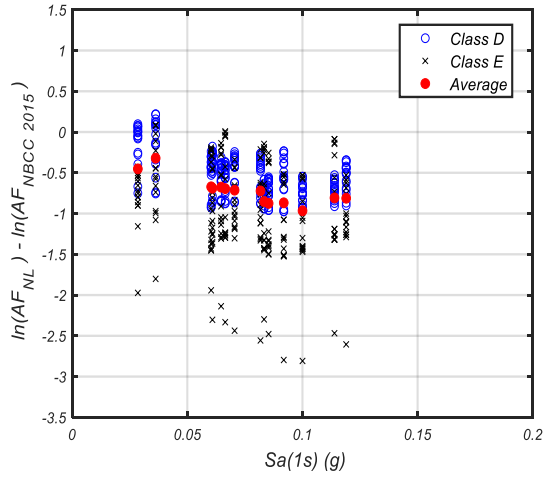


Figure H-7 – Residuals vs. $Sa(1s)$ at $T=1s$ (Scenario 2)

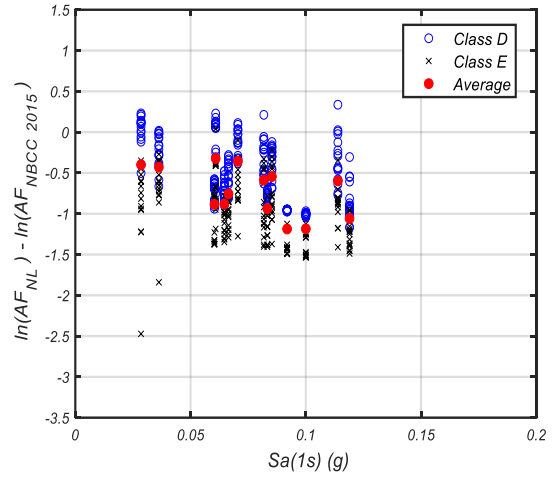


Figure H-8 – Residuals vs. $Sa(1s)$ at $T=10s$ (Scenario 2)

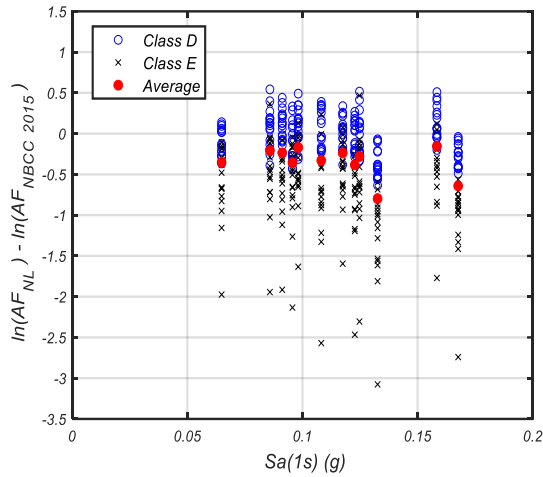


Figure H-9 – Residuals vs. $Sa(1s)$ at $T=0.01s$ (Scenario 3)

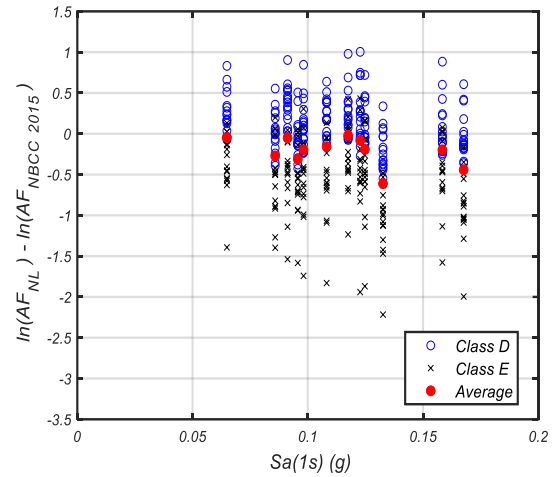


Figure H-10 – Residuals vs. $Sa(1s)$ at $T=0.1s$ (Scenario 3)

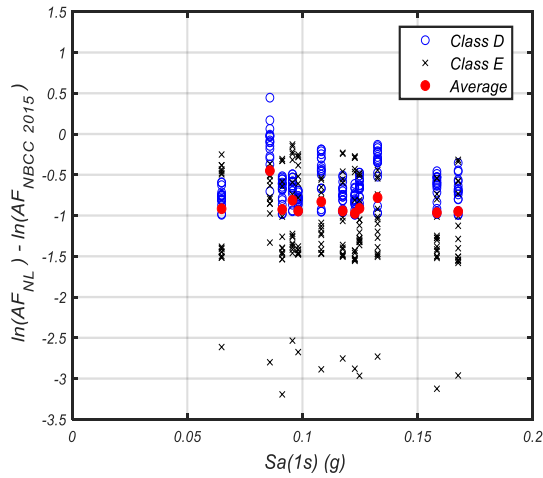


Figure H-11 – Residuals vs. $Sa(1s)$ at $T=1s$ (Scenario 3)

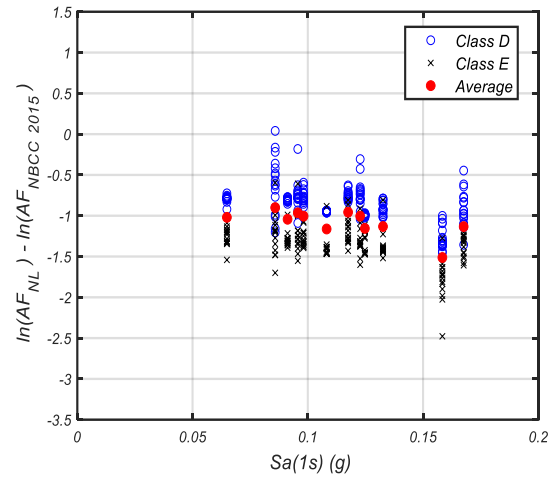


Figure H-12 – Residuals vs. $Sa(1s)$ at $T=10s$ (Scenario 3)

APPENDIX I CORRELATIONS BETWEEN RESIDUALS AND $Sa(10s)$

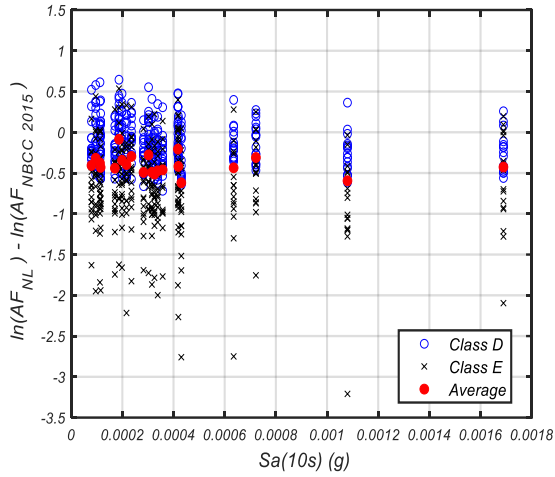


Figure I-1 – Residuals vs. $Sa(10s)$ at $T=0.01s$ (Scenario 1)

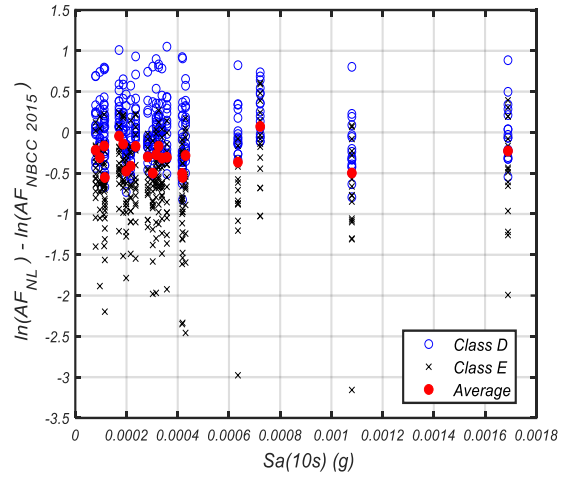


Figure I-2 – Residuals vs. $Sa(10s)$ at $T=0.1s$ (Scenario 1)

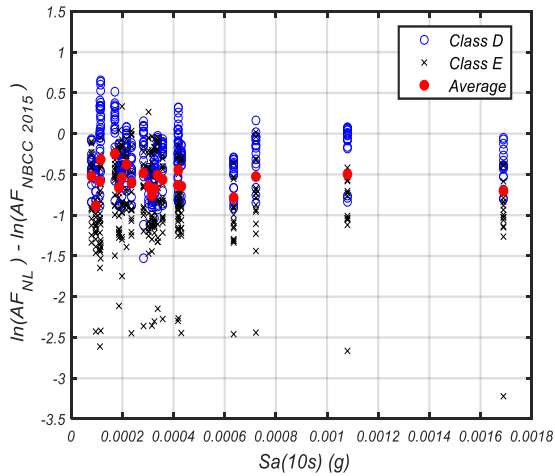


Figure I-3 – Residuals vs. $Sa(10s)$ at $T=1s$ (Scenario 1)

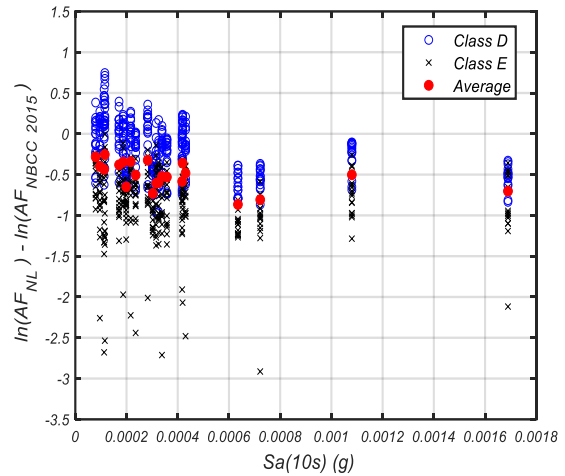


Figure I-4 – Residuals vs. $Sa(10s)$ at $T=10s$ (Scenario 1)

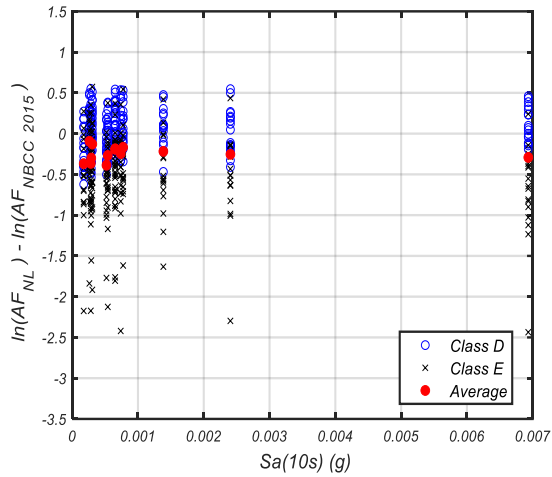


Figure I-5 – Residuals vs. $Sa(10s)$ at $T=0.01s$ (Scenario 2)

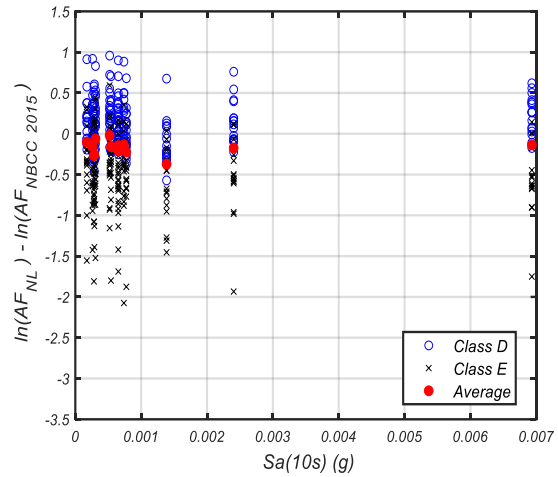


Figure I-6 – Residuals vs. $Sa(10s)$ at $T=0.1s$ (Scenario 2)

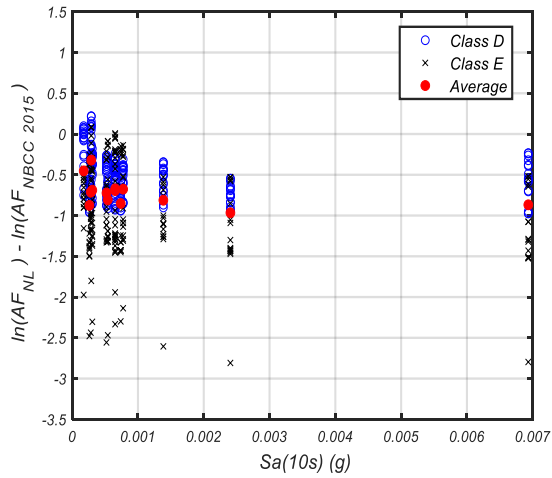


Figure I-7 – Residuals vs. $Sa(10s)$ at $T=1s$ (Scenario 2)

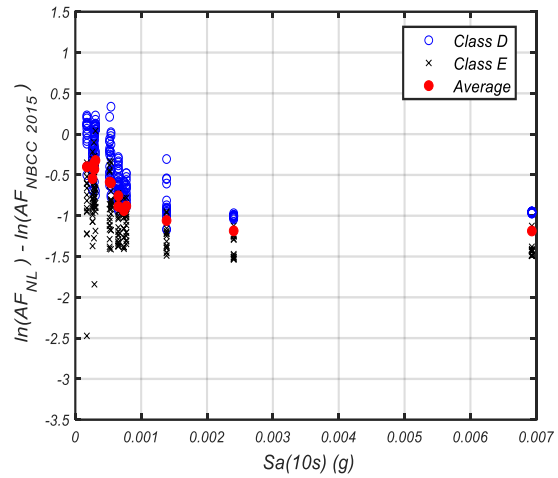


Figure I-8 – Residuals vs. $Sa(10s)$ at $T=10s$ (Scenario 2)

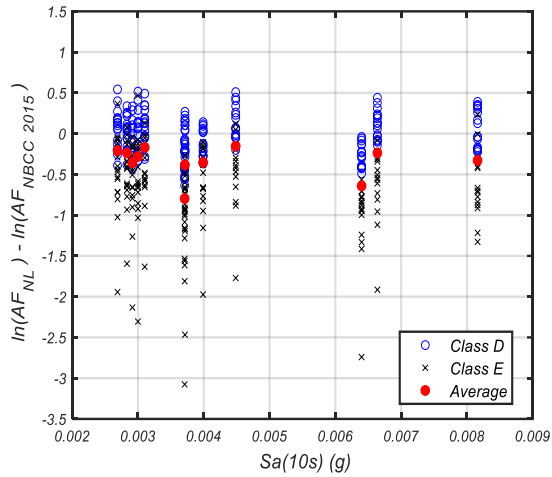


Figure I-9 – Residuals vs. $Sa(10s)$ at $T=0.01s$ (Scenario 3)

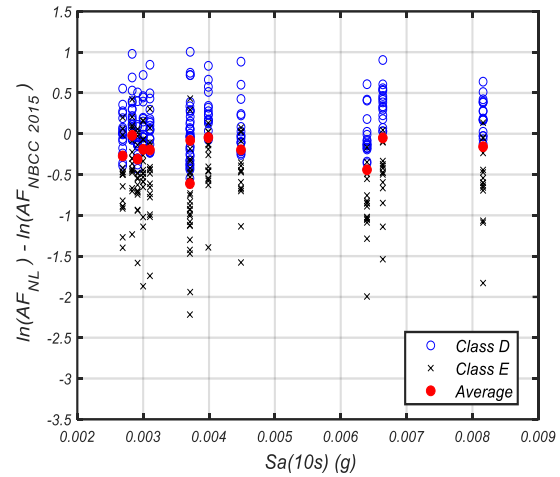


Figure I-10 – Residuals vs. $Sa(10s)$ at $T=0.1s$ (Scenario 3)

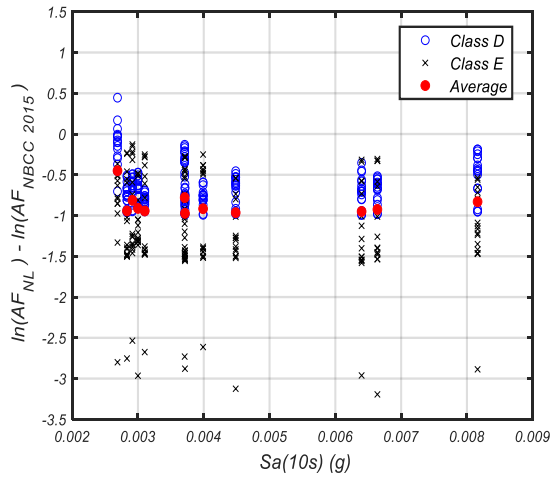


Figure I-11 – Residuals vs. $Sa(10s)$ at $T=1s$ (Scenario 3)

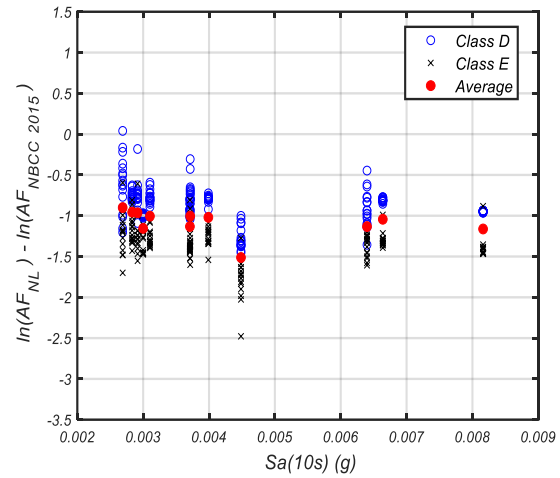


Figure I-12 – Residuals vs. $Sa(10s)$ at $T=10s$ (Scenario 3)

APPENDIX J CORRELATIONS BETWEEN RESIDUALS AND ROOT-MEAN-SQUARE OF ACCELERATION (a_{rms})

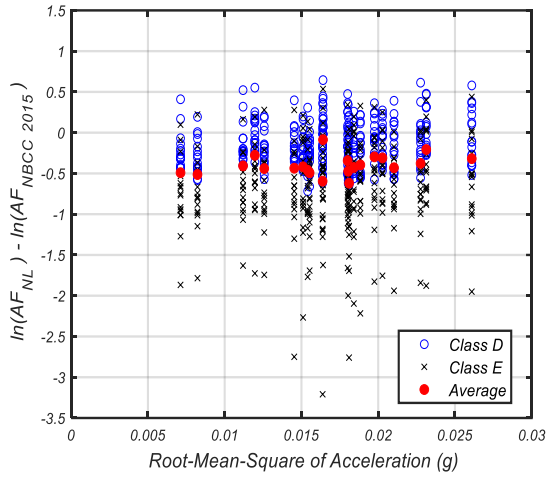


Figure J-1 – Residuals vs. a_{rms} at $T=0.01s$ (Scenario 1)

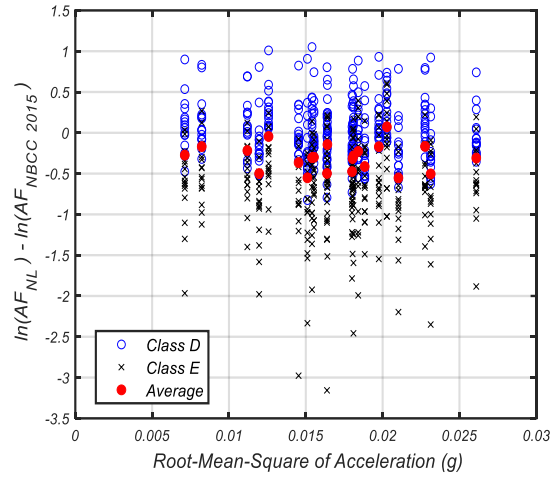


Figure J-2 – Residuals vs. a_{rms} at $T=0.1s$ (Scenario 1)

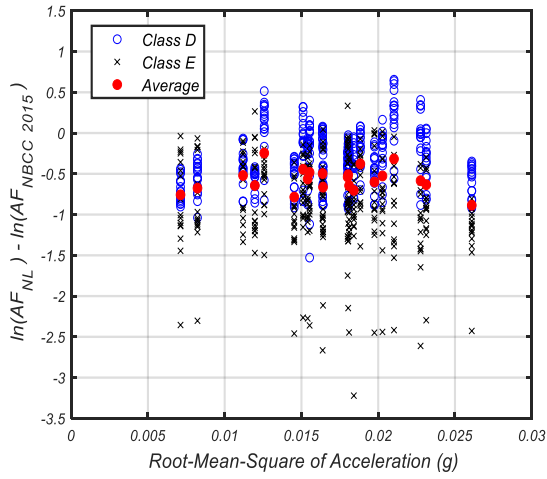


Figure J-3 – Residuals vs. a_{rms} at $T=1s$ (Scenario 1)

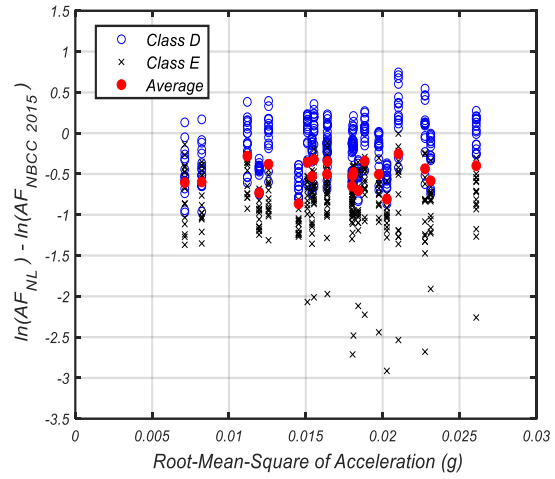


Figure J-4 – Residuals vs. a_{rms} at $T=10s$ (Scenario 1)

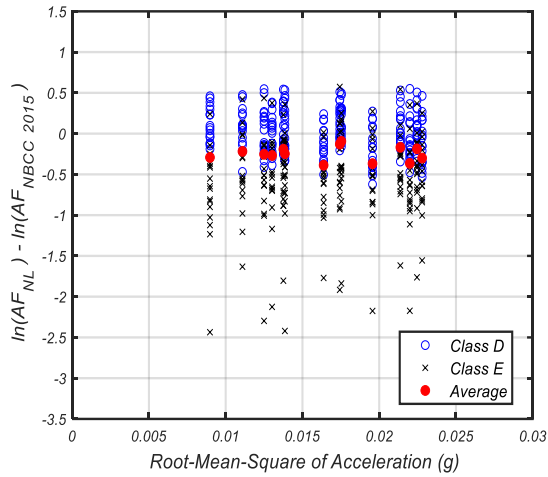


Figure J-5 – Residuals vs. a_{rms} at $T=0.01s$
(Scenario 2)

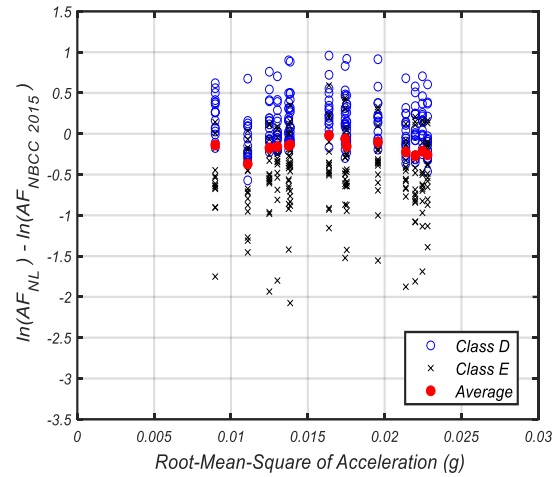


Figure J-6 – Residuals vs. a_{rms} at $T=0.1s$
(Scenario 2)

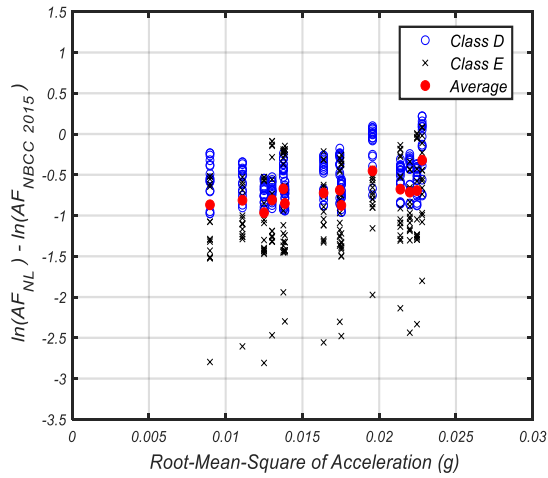


Figure J-7 – Residuals vs. a_{rms} at $T=1s$
(Scenario 2)

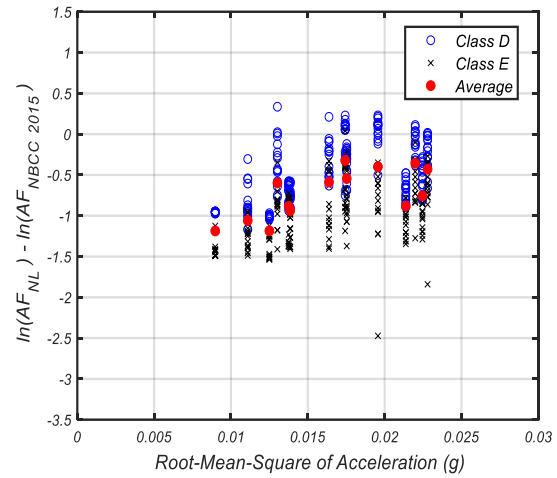


Figure J-8 – Residuals vs. a_{rms} at $T=10s$
(Scenario 2)

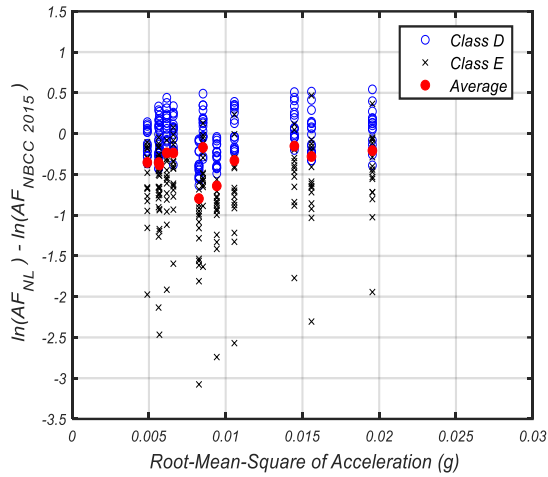


Figure J-9 – Residuals vs. a_{rms} at $T=0.01s$
(Scenario 3)

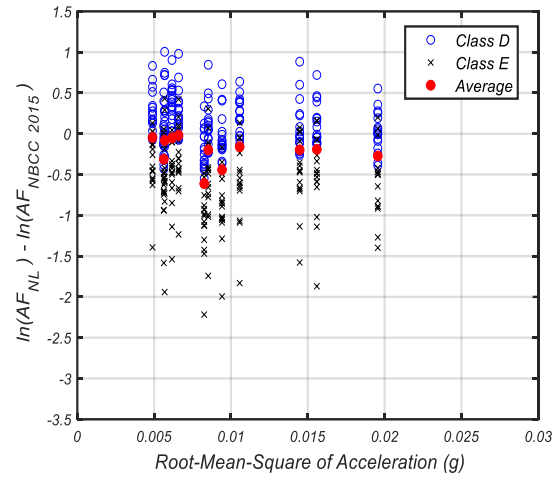


Figure J-10 – Residuals vs. a_{rms} at $T=0.1s$
(Scenario 3)

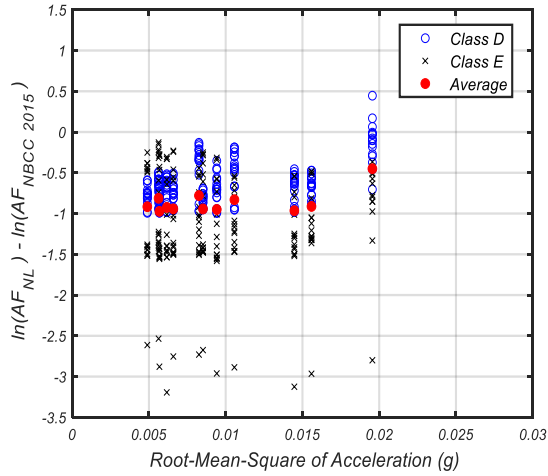


Figure J-11 – Residuals vs. a_{rms} at $T=1s$
(Scenario 3)

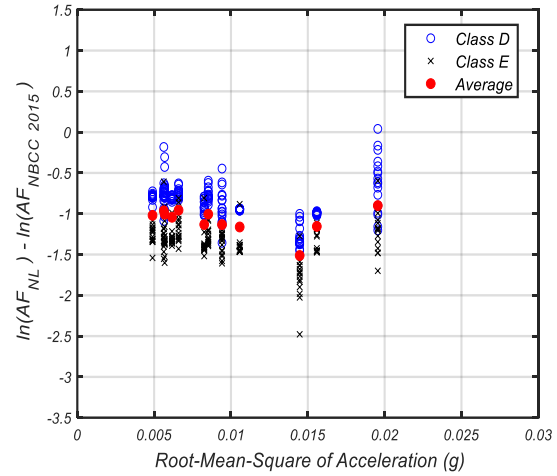


Figure J-12 – Residuals vs. a_{rms} at $T=10s$
(Scenario 3)

APPENDIX K CORRELATIONS BETWEEN RESIDUALS AND SUSTAINED MAXIMUM ACCELERATION (SMA)

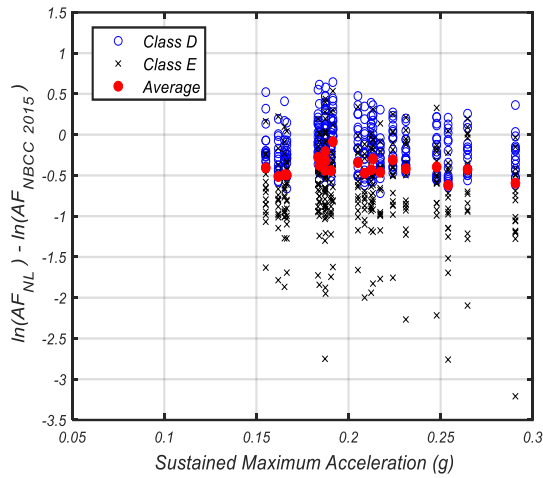


Figure K-1 – Residuals vs. SMA at T=0.01s
(Scenario 1)

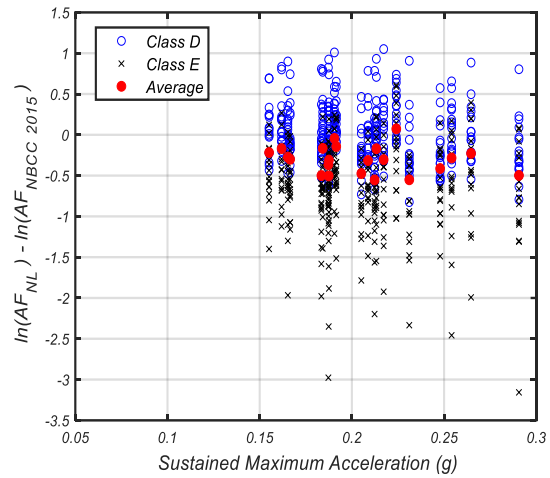


Figure K-2 – Residuals vs. SMA at T=0.1s
(Scenario 1)

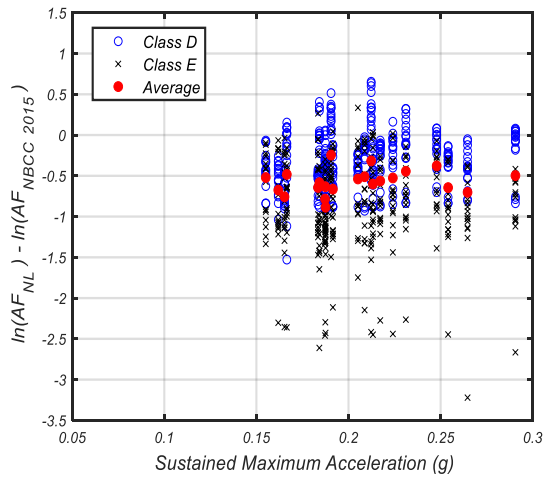


Figure K-3 – Residuals vs. SMA at T=1s
(Scenario 1)

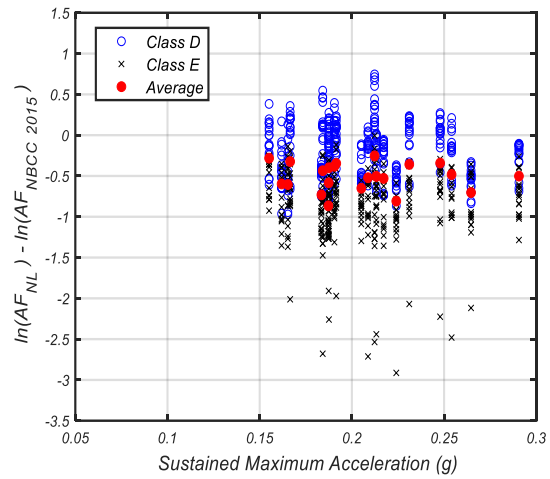


Figure K-4 – Residuals vs. SMA at T=10s
(Scenario 1)

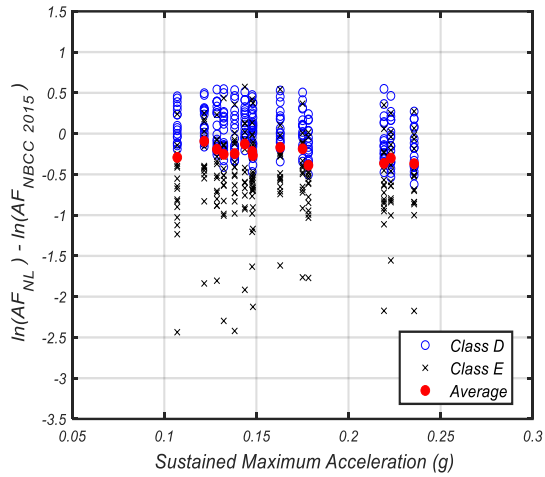


Figure K-5 – Residuals vs. SMA at T=0.01s
(Scenario 2)

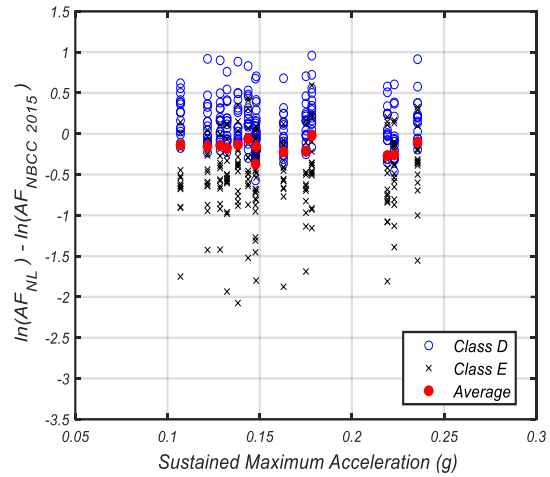


Figure K-6 – Residuals vs. SMA at T=0.1s
(Scenario 2)

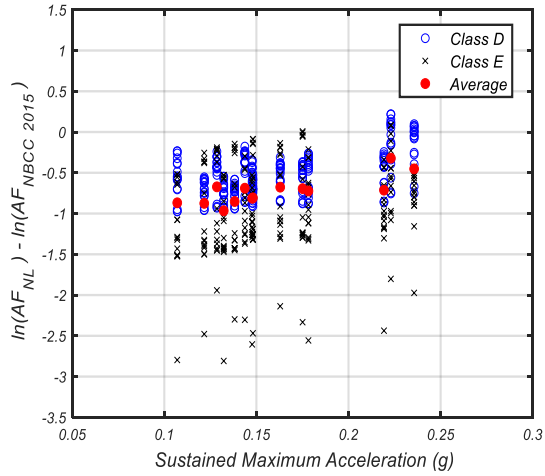


Figure K-7 – Residuals vs. SMA at T=1s
(Scenario 2)

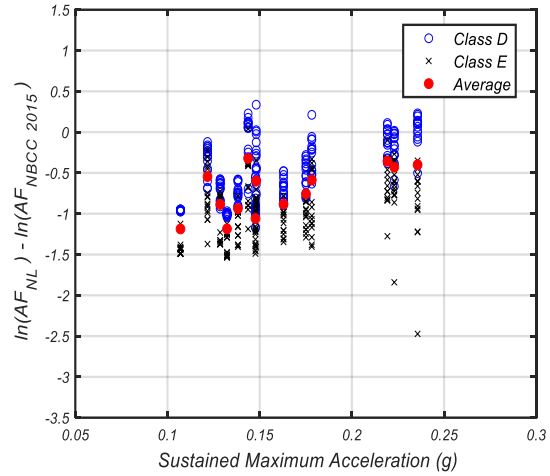


Figure K-8 – Residuals vs. SMA at T=10s
(Scenario 2)

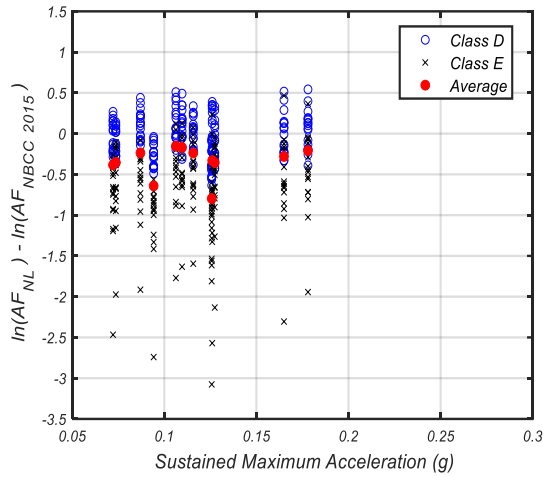


Figure K-9 – Residuals vs. SMA at T=0.01s
(Scenario 3)

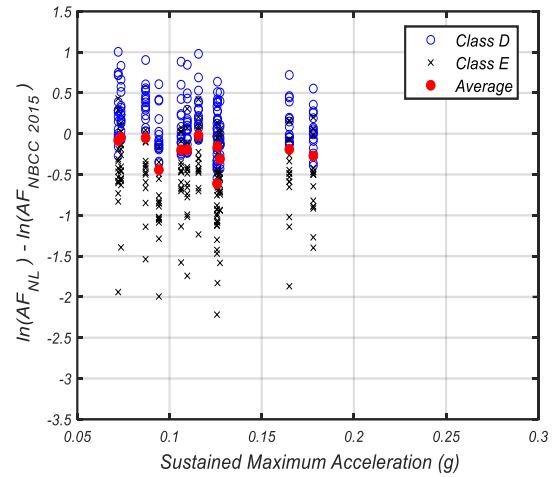


Figure K-10 – Residuals vs. SMA at T=0.1s
(Scenario 3)

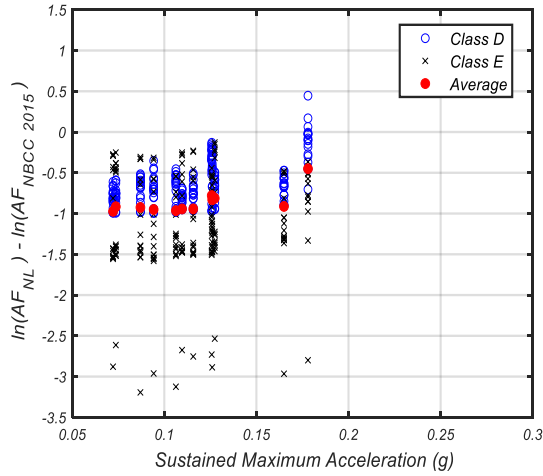


Figure K-11 – Residuals vs. SMA at T=1s
(Scenario 3)

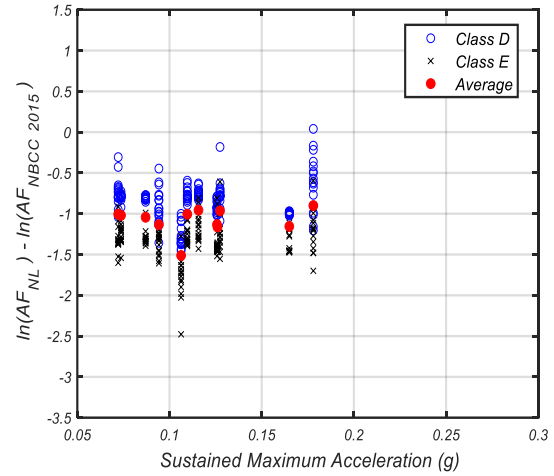


Figure K-12 – Residuals vs. SMA at T=10s
(Scenario 3)

APPENDIX L CORRELATIONS BETWEEN RESIDUALS AND ARIAS INTENSITY (AI)

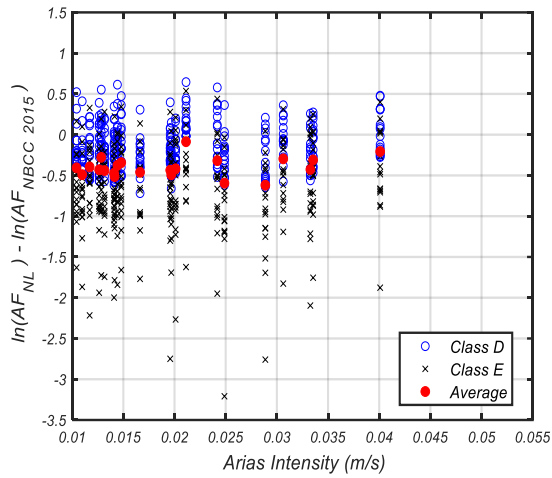


Figure L-1 – Residuals vs. AI at T=0.01s (Scenario 1)

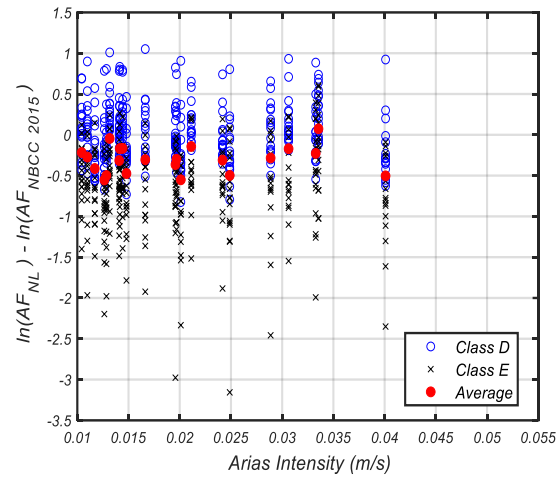


Figure L-2 – Residuals vs. AI at T=0.1s (Scenario 1)

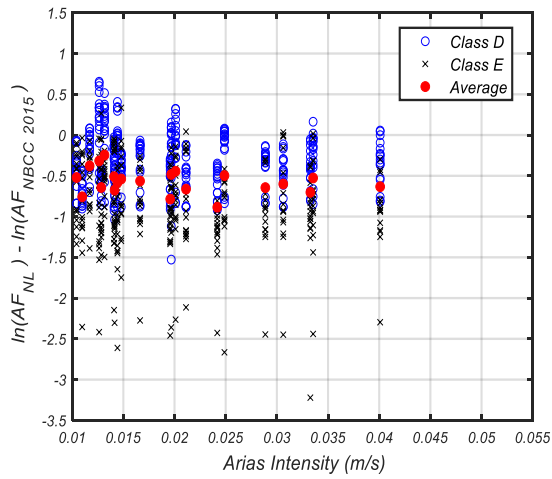


Figure L-3 – Residuals vs. AI at T=1s (Scenario 1)

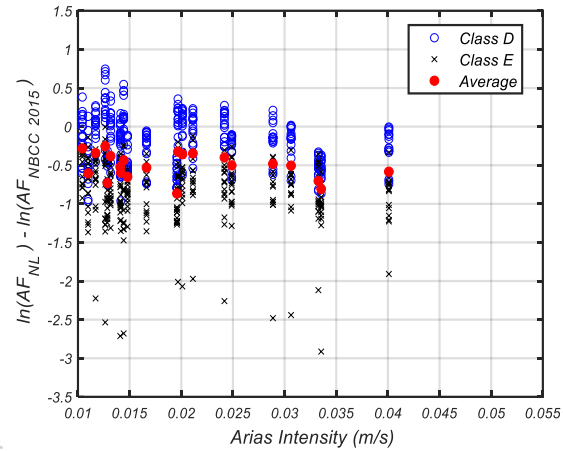


Figure L-4 – Residuals vs. AI at T=10s (Scenario 1)

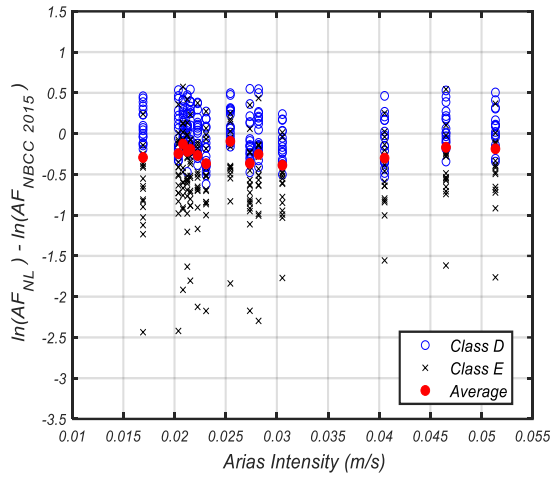


Figure L-5 – Residuals vs. AI at T=0.01s
(Scenario 2)

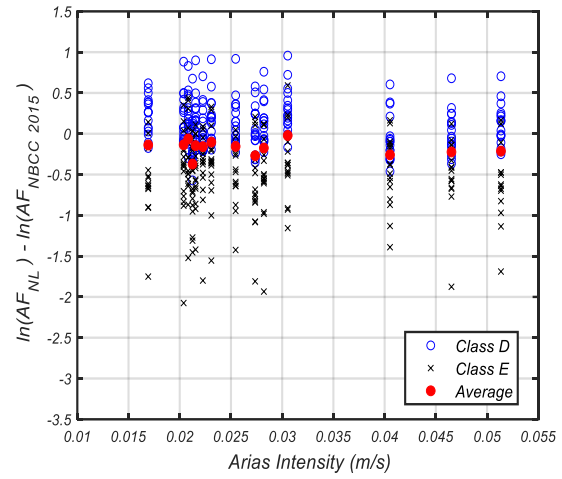


Figure L-6 – Residuals vs. AI at T=0.1s
(Scenario 2)

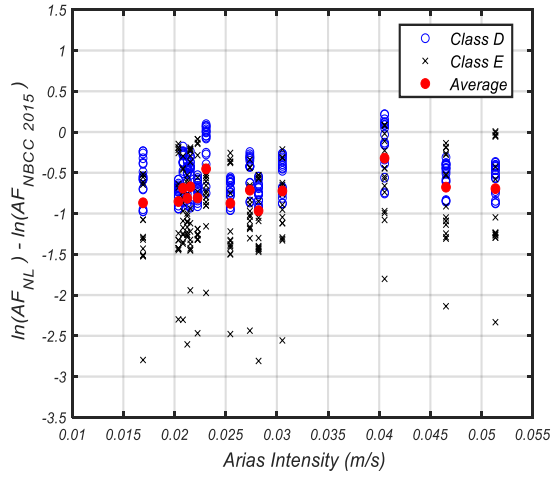


Figure L-7 – Residuals vs. AI at T=1s
(Scenario 2)

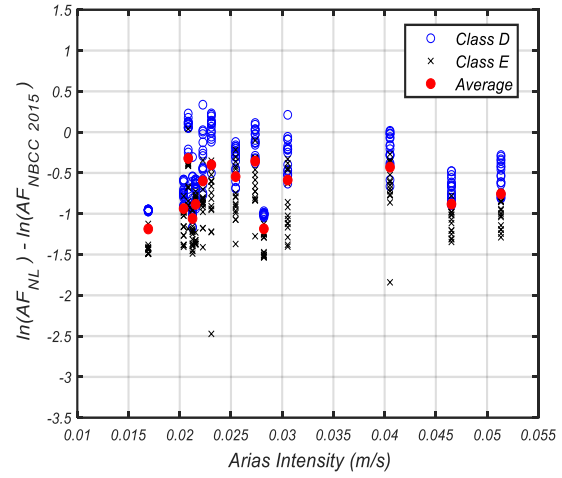


Figure L-8 – Residuals vs. AI at T=10s
(Scenario 2)

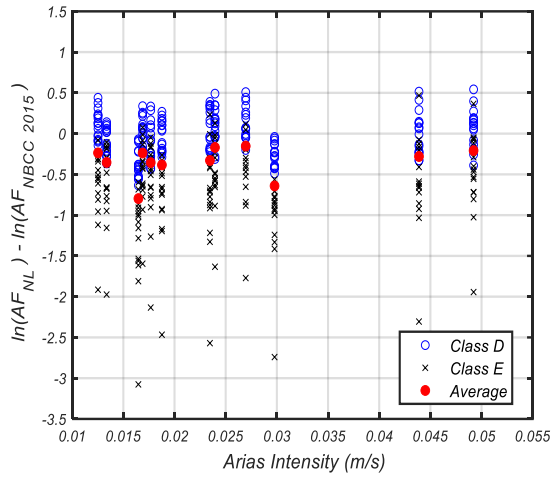


Figure L-9 – Residuals vs. AI at T=0.01s
(Scenario 3)

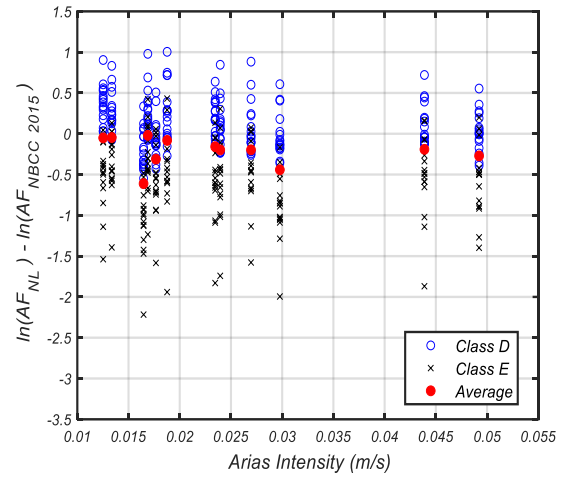


Figure L-10 – Residuals vs. AI at T=0.1s
(Scenario 3)

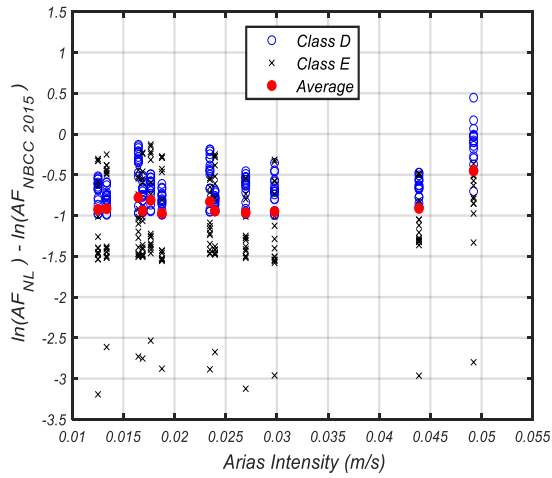


Figure L-11 – Residuals vs. AI at T=1s
(Scenario 3)

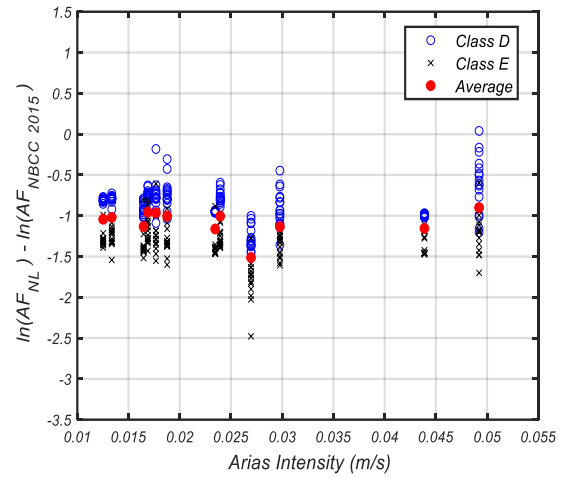


Figure L-12 – Residuals vs. AI at T=10s
(Scenario 3)

APPENDIX M CORRELATIONS BETWEEN RESIDUALS AND CUMULATIVE ABSOLUTE VELOCITY (CAV)

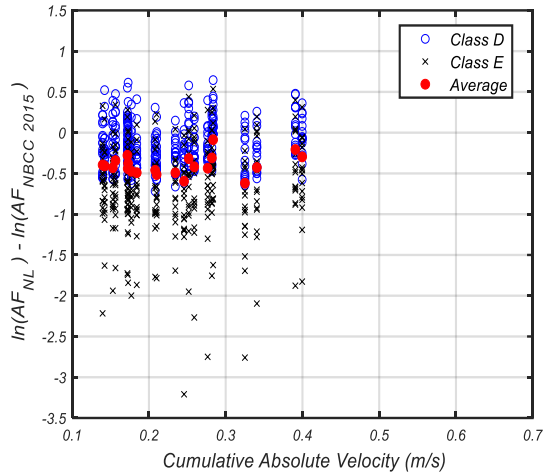


Figure M-1 – Residuals vs. CAV at T=0.01s
(Scenario 1)

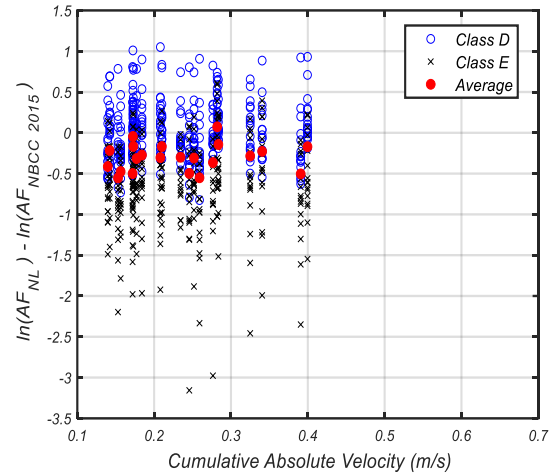


Figure M-2 – Residuals vs. CAV at T=0.1s
(Scenario 1)

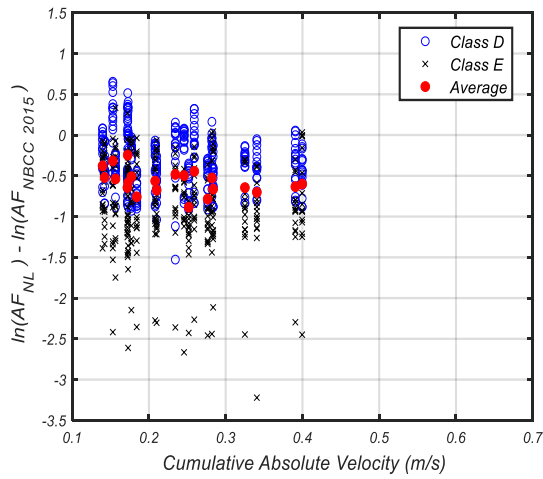


Figure M-3 – Residuals vs. CAV at T=1s
(Scenario 1)

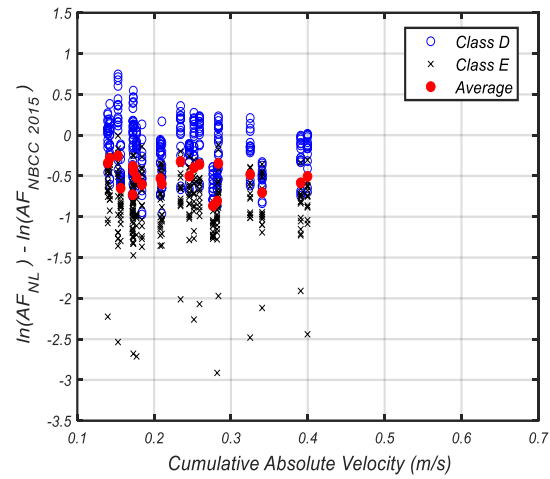


Figure M-4 – Residuals vs. CAV at T=10s
(Scenario 1)

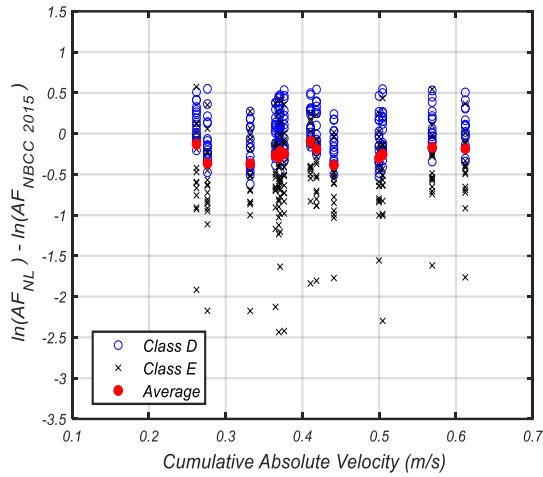


Figure M-5 – Residuals vs. CAV at T=0.01s
(Scenario 2)

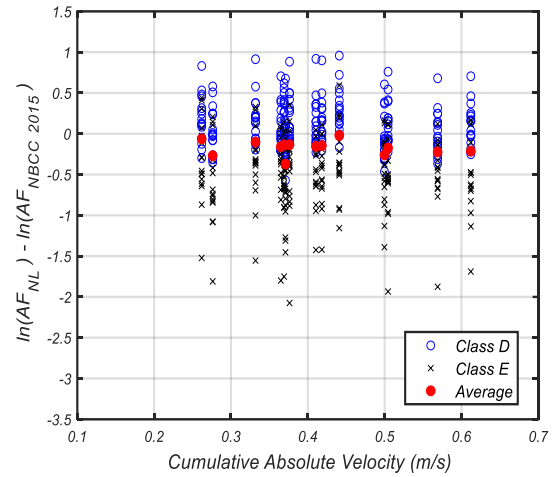


Figure M-6 – Residuals vs. CAV at T=0.1s
(Scenario 2)

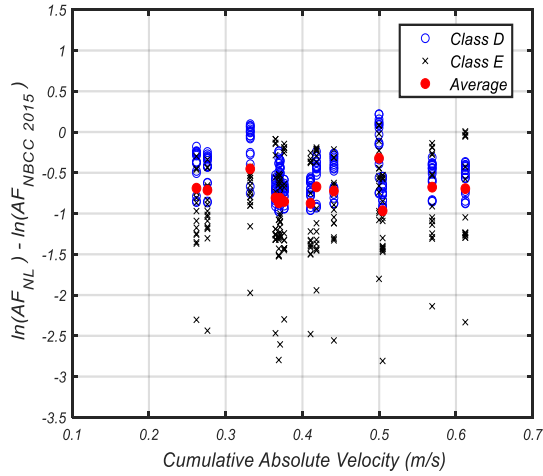


Figure M-7 – Residuals vs. CAV at T=1s
(Scenario 2)

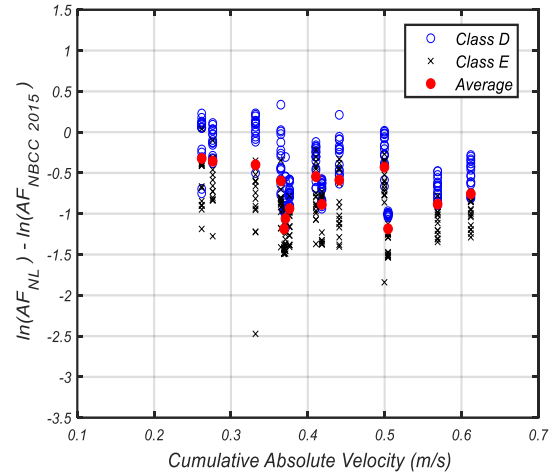


Figure M-8 – Residuals vs. CAV at T=10s
(Scenario 2)

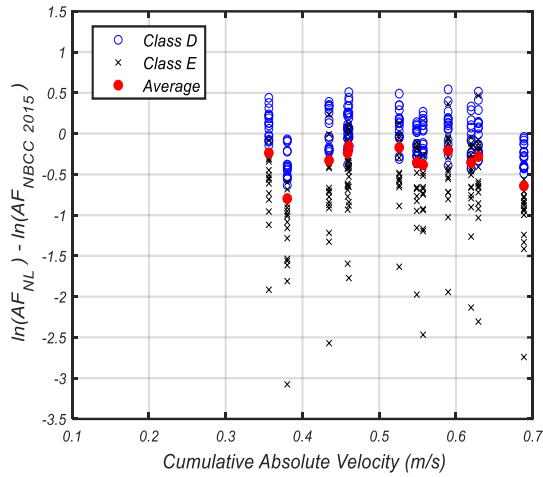


Figure M-9 – Residuals vs. CAV at T=0.01s
(Scenario 3)

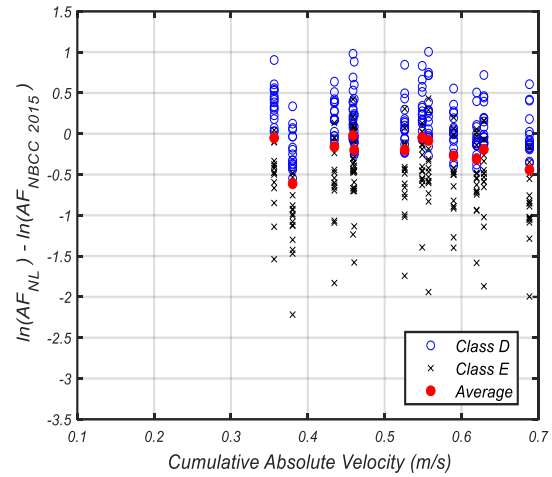


Figure M-10 – Residuals vs. CAV at T=0.1s
(Scenario 3)

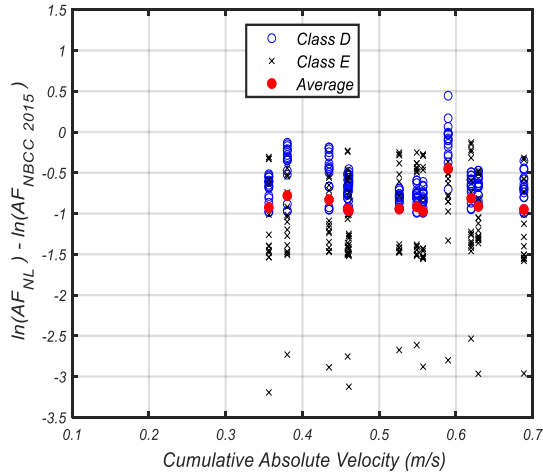


Figure M-11 – Residuals vs. CAV at T=1s
(Scenario 3)

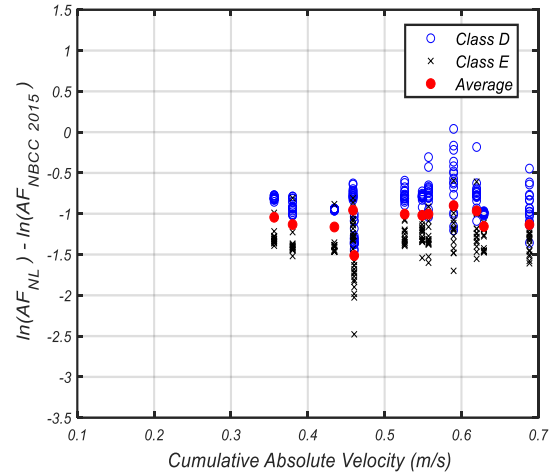


Figure M-12 – Residuals vs. CAV at T=10s
(Scenario 3)

APPENDIX N CORRELATIONS BETWEEN RESIDUALS AND A_{95}

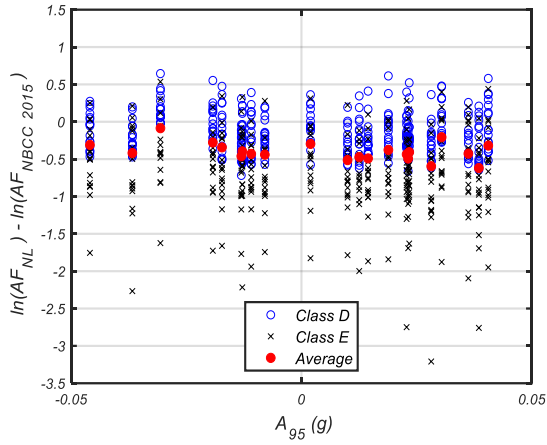


Figure N-1 – Residuals vs. A_{95} at $T=0.01s$ (Scenario 1)

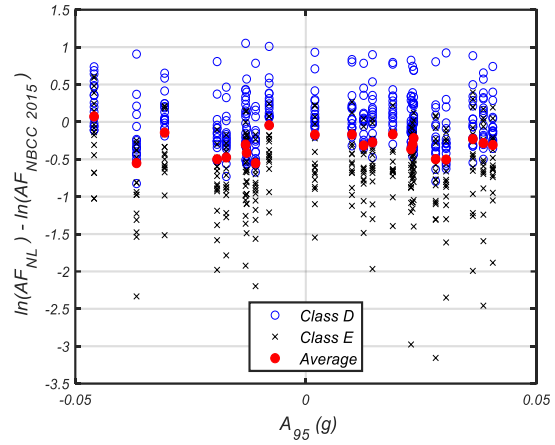


Figure N-2 – Residuals vs. A_{95} at $T=0.1s$ (Scenario 1)

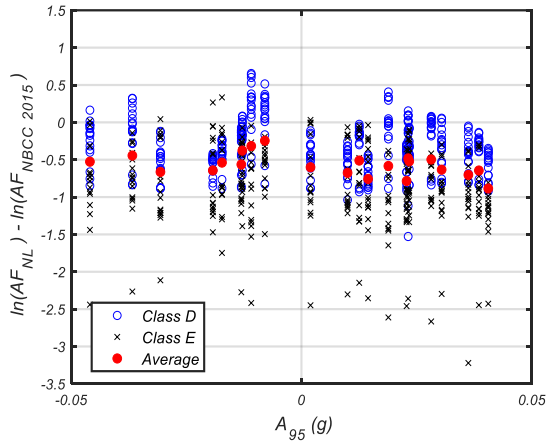


Figure N-3 – Residuals vs. A_{95} at $T=1s$ (Scenario 1)

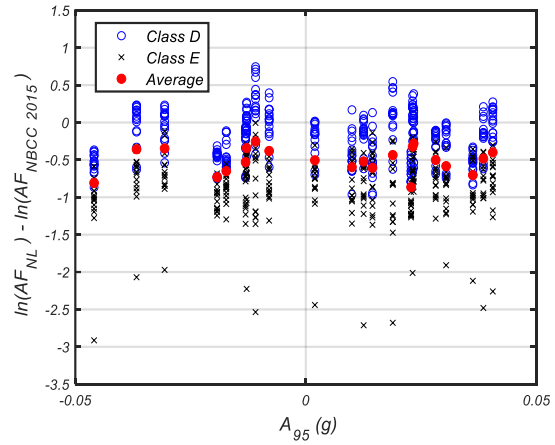


Figure N-4 – Residuals vs. A_{95} at $T=10s$ (Scenario 1)

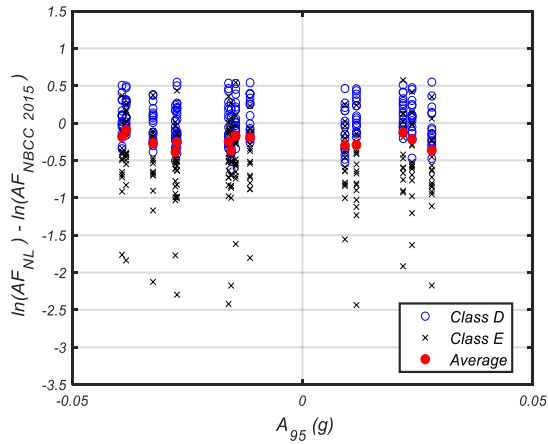


Figure N-5 – Residuals vs. A_{95} at $T=0.01s$
(Scenario 2)

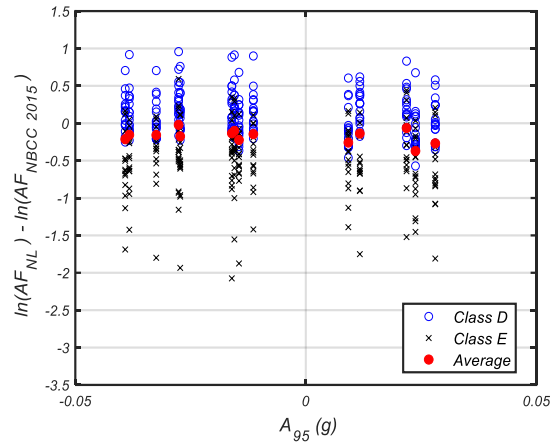


Figure N-6 – Residuals vs. A_{95} at $T=0.1s$
(Scenario 2)

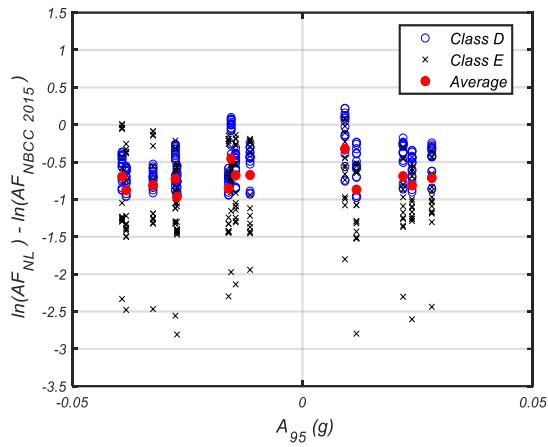


Figure N-7 – Residuals vs. A_{95} at $T=1s$
(Scenario 2)

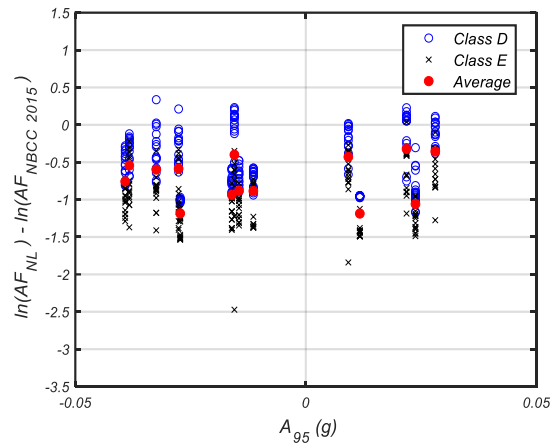


Figure N-8 – Residuals vs. A_{95} at $T=10s$
(Scenario 2)

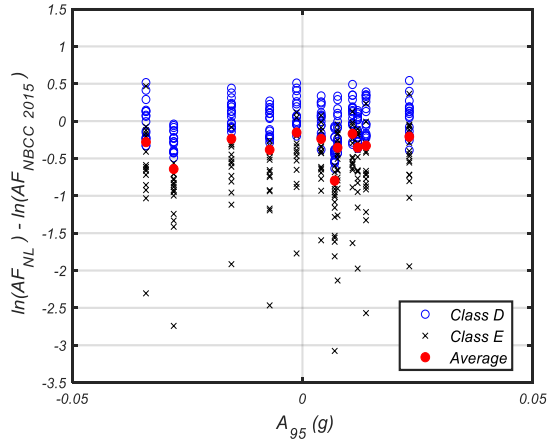


Figure N-9 – Residuals vs. A_{95} at $T=0.01s$ (Scenario 3)

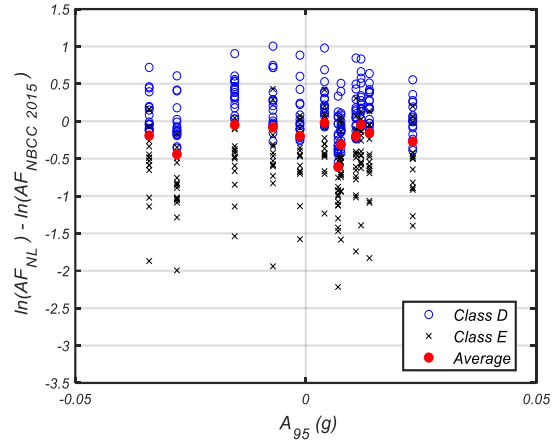


Figure N-10 – Residuals vs. A_{95} at $T=0.1s$ (Scenario 3)

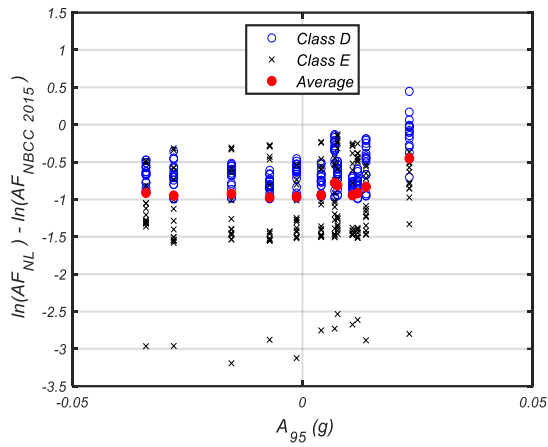


Figure N-11– Residuals vs. A_{95} at $T=1s$ (Scenario 3)

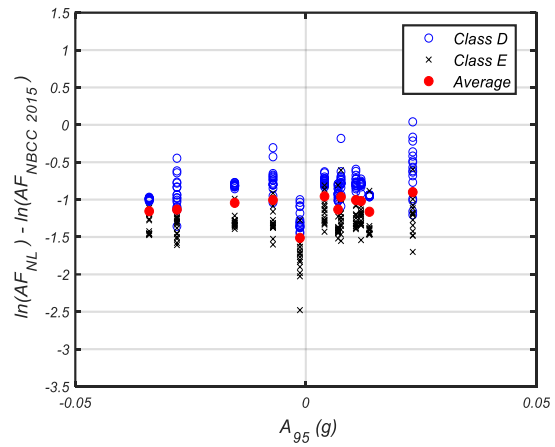


Figure N-12 – Residuals vs. A_{95} at $T=10s$ (Scenario 3)

APPENDIX O CORRELATIONS BETWEEN RESIDUALS AND PSEUDO SPECTRAL ACCELERATION AT T_0 ($PSA[T_0]$)

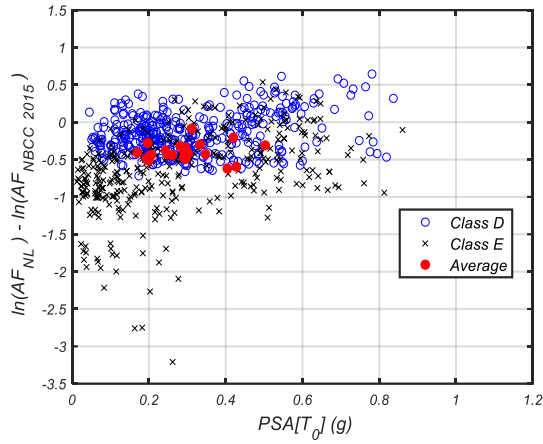


Figure O-1 – Residuals vs. $PSA[T_0]$ at $T=0.01s$ (Scenario 1)

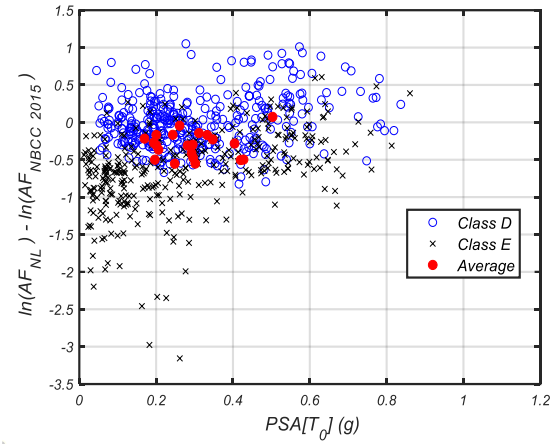


Figure O-2 – Residuals vs. $PSA[T_0]$ at $T=0.1s$ (Scenario 1)

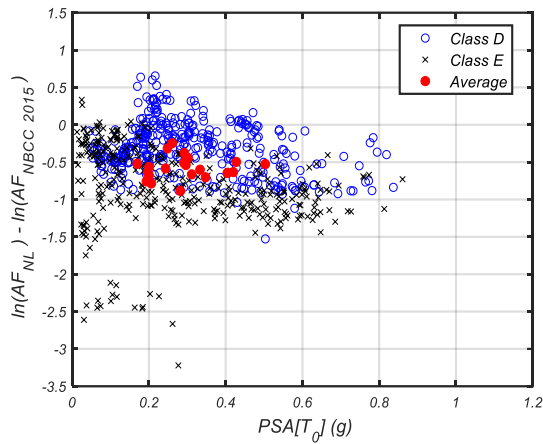


Figure O-3 – Residuals vs. $PSA[T_0]$ at $T=1s$ (Scenario 1)

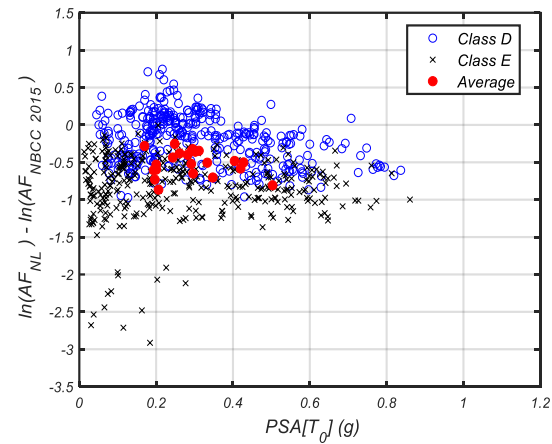


Figure O-4 – Residuals vs. $PSA[T_0]$ at $T=10s$ (Scenario 1)

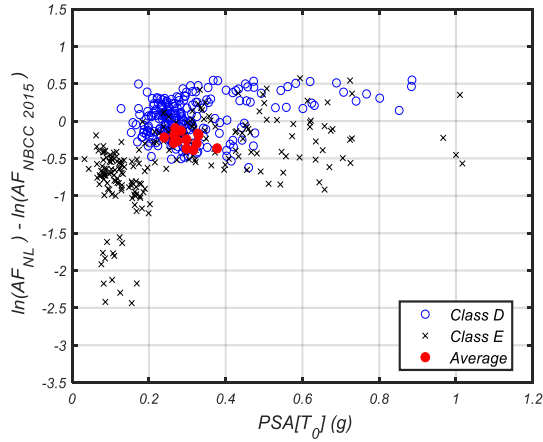


Figure O-5 – Residuals vs. $PSA[T_0]$ at $T=0.01s$ (Scenario 2)

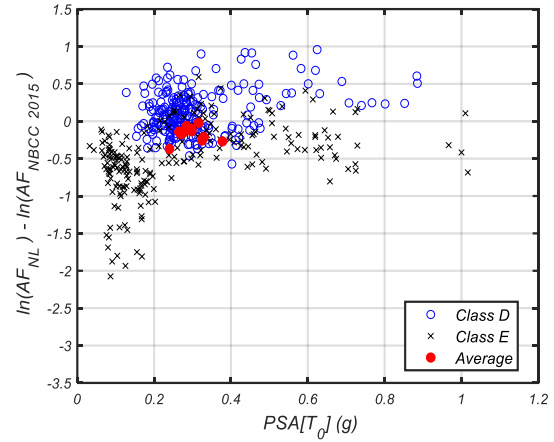


Figure O-6 – Residuals vs. $PSA[T_0]$ at $T=0.1s$ (Scenario 2)

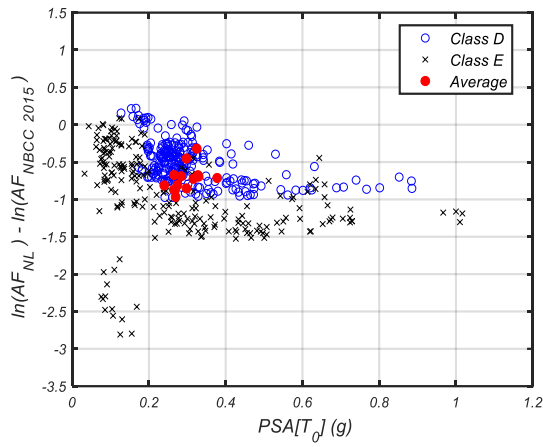


Figure O-7 – Residuals vs. $PSA[T_0]$ at $T=1s$ (Scenario 2)

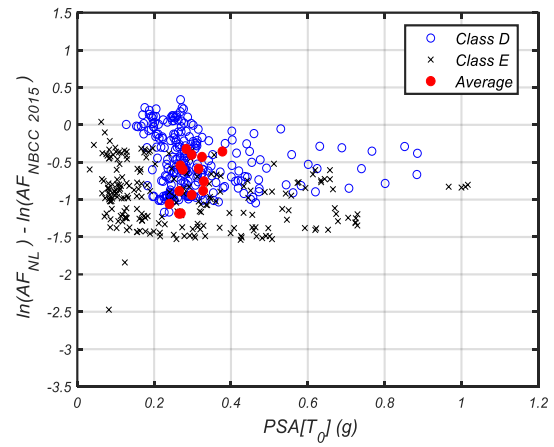


Figure O-8 – Residuals vs. $PSA[T_0]$ at $T=10s$ (Scenario 2)

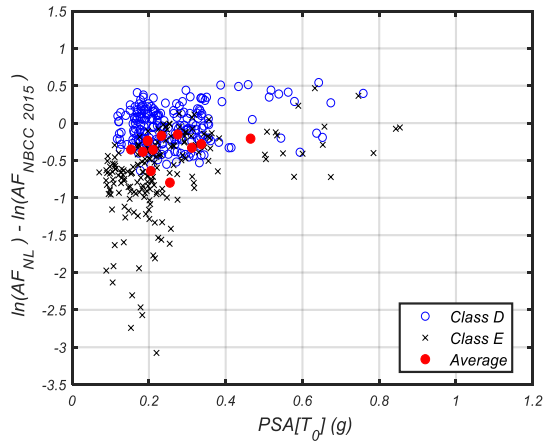


Figure O-9 – Residuals vs. $PSA[T_0]$ at $T=0.01$ s (Scenario 3)

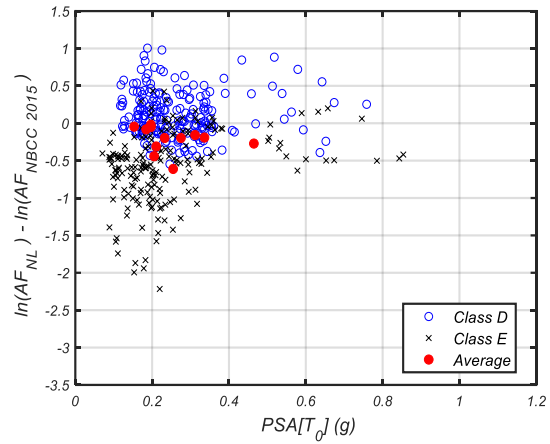


Figure O-10 – Residuals vs. $PSA[T_0]$ at $T=0.1$ s (Scenario 3)

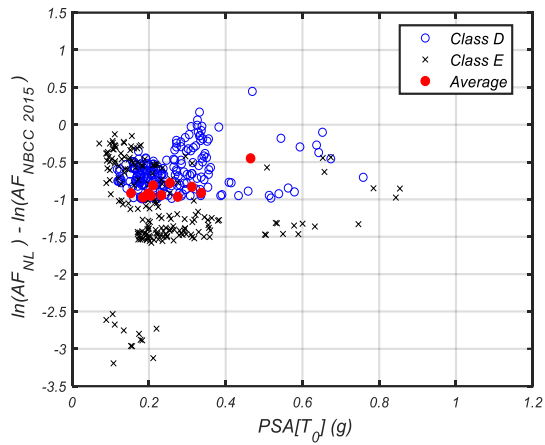


Figure O-11 – Residuals vs. $PSA[T_0]$ at $T=1$ s (Scenario 3)

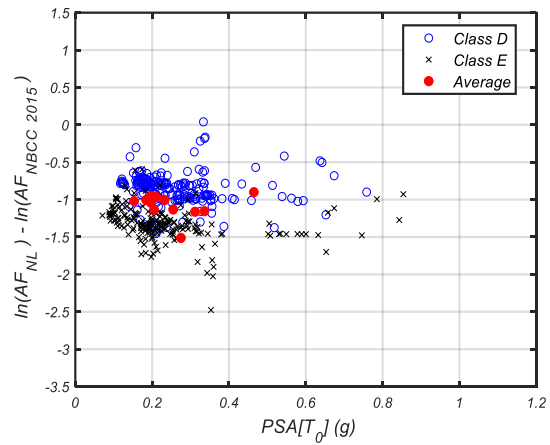


Figure O-12 – Residuals vs. $PSA[T_0]$ at $T=10$ s (Scenario 3)