# Site-Specific Response Analyses
# New I-26 Volvo Interchange
# Berkeley County, South Carolina
# S&ME Project No. 1413-15-114

Prepared for:

**Thomas & Hutton**
**1501 Main Street**
**Columbia, South Carolina 29201**


Prepared by:

**S&ME, Inc.**
**620 Wando Park Boulevard**
**Mt Pleasant, SC 29464**


**January 13, 2015**

January 13, 2016

Thomas & Hutton
1501 Main Street
Columbia, South Carolina 29201

Attention: Mr. Doyle Kelley

Reference: **Site-Specific Seismic Response Analyses**
**New I-26 Volvo Interchange**
Berkeley County, South Carolina
S&ME Project No. 1413-15-114

Dear Mr. Kelley:

We have completed site-specific response analyses (SSRA) for the proposed I-26 Volvo interchange in Berkeley County, South Carolina. This report presents our methodology, results, and recommendations. Our services were performed pursuant to S&ME Proposal No. 14-1500509 dated May 15, 2015, and the Master Services Agreement between Thomas & Hutton Engineering Co. and S&ME, Inc. for Volvo-Project Soter Interchange dated August 25, 2015.

We appreciate the opportunity to be of service on this project. Please let us know if you have any questions.

Sincerely,

**S&ME, Inc.**

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER No. 19871 MICHAEL S. ULMER

SOUTH CAROLINA S & ME, INC. No. C00473 CERTIFICATE OF AUTHORIZATION

Ariful Haque

Md Ariful H. Bhuiyan, Ph.D., E.I.T.
Staff Professional

Michael S. Ulmer, P.E.
Principal Engineer/Project Manager

Michael Ulmer, P.E.

Digitally signed by Michael Ulmer, P.E.
DN: cn=Michael Ulmer, P.E., o=S&ME, Inc., ou, email=mulmer@smeinc.com, c=US
Date: 2016.01.13 17:46:27 -05'00'

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## Appendices

# 1.0 Project Information and Background

We understand a new interchange is proposed along I-26 approximately two miles south of SC Highway 27 (Exit 187) to support the Volvo plant that is currently under construction in Berkeley County. The project includes constructing a fully directional interchange including associated ramp embankments and three bridges. The structures will likely be supported on drilled shafts or driven piles. Ground improvement beneath embankments and around structures may be required to meet the SCDOT performance limits under static and seismic loading.

The South Carolina Department of Transportation (SCDOT) plans to construct the new interchange using a design-build contract. S&ME, Inc. has been contracted with Thomas & Hutton Engineering Company to explore the site, prepare a Geotechnical Base Line Report, and develop the acceleration design response spectra (ADRS) that will be provided to the design-build teams of this project. The Geotechnical Base Line Report[1] (GBLR) is submitted under separate cover. The specific tasks and requirements for the SSRA are summarized as follows:

- Seven synthetic acceleration time histories, generated by the Scenario_PC software, were provided by the SCDOT for each of the functional evaluation earthquake (FEE) and the safety evaluation earthquake (SEE) base "rock" hazards.
- Subsurface exploration including a total of 39 soil test borings (STB), cone penetration test (CPT) soundings, and dilatometer (DMT) soundings; three Multi-Channel Analysis of Surface Waves (MASW) tests; and laboratory testing.
- The analyses were performed using equivalent-linear and nonlinear procedures using DEEPSOIL with the Andrus et al. 2003 modulus and damping curves.
- The variability in stratigraphy, soil properties, and depth to rock were considered with appropriate sensitivity analyses.
- The analyses were peer reviewed. The standard 3-point ADRS were compared with the site-specific results, and with consent of the Peer Reviewer, the 70% limit on the maximum reduction between the final design spectra and the 3-point spectrum was not applied.
- Dr. Sanjoy Chakraborty, P.E. with CDM Smith was contracted by S&ME to provide the required peer review. Dr. Chakraborty's comments and concurrence are included in Appendix IV.

# 2.0 Methodology Overview

## 2.1 Total Stress Analyses

Our equivalent-linear (EL) and nonlinear (NL) analyses were performed with the one-dimensional wave propagation program DEEPSOIL v6.1 (Hashash et al. 2015). The program can perform the EL and NL analyses simultaneously. In all of the analyses, the curves were adjusted to match the corresponding measured or estimated soil shear strength values (Hashash et al. 2010, Yee et al. 2013) to avoid

[1] Geotechnical Base Line Report – New I-26 Volvo Interchange, December 4, 2015; S&ME, Inc.

unrealistically high or low implied shear strengths computed based on the standard normalized modulus reduction curves within the SSRA tool.

The required inputs for these analyses can be summarized as follows:

- Representative acceleration time histories for the design earthquakes. A total of 14 time histories were provided by the SCDOT and are discussed in Section 3.0
- Stratigraphy. The soil profile information (i.e. layering), soil classification, water table location, et cetera were developed based on our subsurface exploration and are discussed in Section 4.0.
- Shear wave velocity profiles. The MASW and MAM shear wave data were used to develop representative velocity profile for the SSRA analysis. The shear wave velocity ($V_s$) profiles are discussed in more detail in Section 4.0.
- Shear modulus and damping reduction curves. As required by the scope of services, the dynamic properties of all soils were modeled using the Andrus et al. (2003) curves. Necessary inputs for generating these curves are plasticity index (PI), $K_0$, unit weight, and effective overburden stress. Additionally, estimates of effective stress friction angle, undrained shear strength, and overconsolidation ratio (OCR) are required to adjust the standard dynamic curves for soil shear strength (SS) values. The development of the modulus and damping reduction curves and the shear strength correction are discussed in more detail in Section 5.

## 2.2 Effective Stress Analyses

Effective stress analyses were performed to evaluate the consequences of pore pressure generation on the computed spectral response. While the total stress nonlinear analyses may provide a better estimate of the site response for large strain conditions (as compared to equivalent-linear analyses), they do not consider the porewater pressure (PWP) generation and dissipation that may occur. More specifically, liquefaction-induced softening is not considered in total stress analyses which is likely to occur under the design events for the project site.

The Dobry-Matasovic model, as implemented in DEEPSOIL, was used for PWP generation and dissipation. Both sand and clay models were used. For each cycle, the model considers the shear strain, a threshold shear strain, the coefficient of consolidation, and several curve fitting parameters. For sands, the curve fitting parameters are related to $V_S$ and fines content (FC). For clays, the curve fitting parameters are related to PI and OCR.

## 2.3 Variability of Soil Parameters

Sensitivity analyses were performed to account for the uncertainty or variability associated with some of the input parameters that have significant effect on computed responses. Specifically, the acceleration time histories were input at multiple depths, the $V_s$ profiles were varied (faster and slower), and the soil properties used to develop the modulus and damping curves (e.g., PI, SS) were varied. Sensitivity analyses for both total and effective stress conditions are described in Section 4.3.

## 2.4 ADRS Curve Generation

The analysis results were used to generate the ADRS curves at the ground surface (GS) and also at the top of the Cooper Marl (TOM). We expect TOM will correspond to the approximate depth-to-motion for the structures that will be designed by the design-build teams.

# 3.0 Ground Motions

The shear wave velocity data is relatively consistent at this site. The coefficient of variability (COV) varies with depth from approximately 0.01 to 0.10 with an average COV over the entire profile of 0.05. The site stiffness ($\bar{V}s$) between the three MASW profiles ranges from 1,276 ft/sec to 1,569 ft/sec, which would suggest a Site Class Seismic Category C. However, down-hole seismic CPT data from the nearby plant site exhibited somewhat lower $\bar{V}s$ values, which ranged from 782 to 859 ft/sec in the top 100 ft. As such, it is our opinion the interchange Site Class Seismic Category should be D.

A seismic hazard disaggregation was performed using the USGS interactive disaggregation tool (http://eqint.cr.usgs.gov/deaggint/2002/index.php) to determine the moment magnitude ($M_w$) and epicentral distance data pairs needed to generate the acceleration time histories using Scenario PC. Review of this data indicates the ground motions are dominated by the nearby Summerville fault, and the following $M_w$ and epicentral distances were determined. The subsurface conditions are relatively consistent across the relatively small interchange site, and it is our opinion additional ground motions are not necessary.

- Mw1 (SEE) = 7.36 with an Epicentral Distance of 17.3 km
- Mw2 (FEE) = 7.35 with an Epicentral Distance of 18.7 km

This information was submitted to the SCDOT on the Consultant Seismic Information Request form, and we were provided a total of 28 synthetic acceleration time histories that were generated by the SCDOT using Scenario_PC: 7 unscaled FEE motions, 7 scaled FEE motions, 7 unscaled SEE motions, and 7 scaled SEE motions. The FEE and SEE motions correspond to events having a 15% and 3% probability of exceedance in 75 years, respectively. All time histories correspond to an outcrop motion on rock with a shear wave velocity of 700 m/s (2,300 fps). The FEE motions are based on a scenario event with a moment magnitude of 7.35 and an epicentral distance of 18.7 km. The SEE motions are based on a scenario event with a moment magnitude of 7.36 and an epicentral distance of 17.3 km.

There are concerns with the Scenario PC scaled motions that have been previously discussed with the SCDOT which are also presented here. First, the scaled motions were noted to include a large permanent displacement offset or “drift.” The second concern is that the scaled motions contain unrealistic short period noise in the area of 50 Hz. We have used the baseline correction feature of DEEPSOIL to remove the drift, and we have used the DEEPSOIL linear interpolation calculation method for the time history interpolation in nonlinear analyses to reduce analytical problems created by the high frequency energy. The resulting output is included in Appendix I.

# 4.0 One Dimensional Soil Column

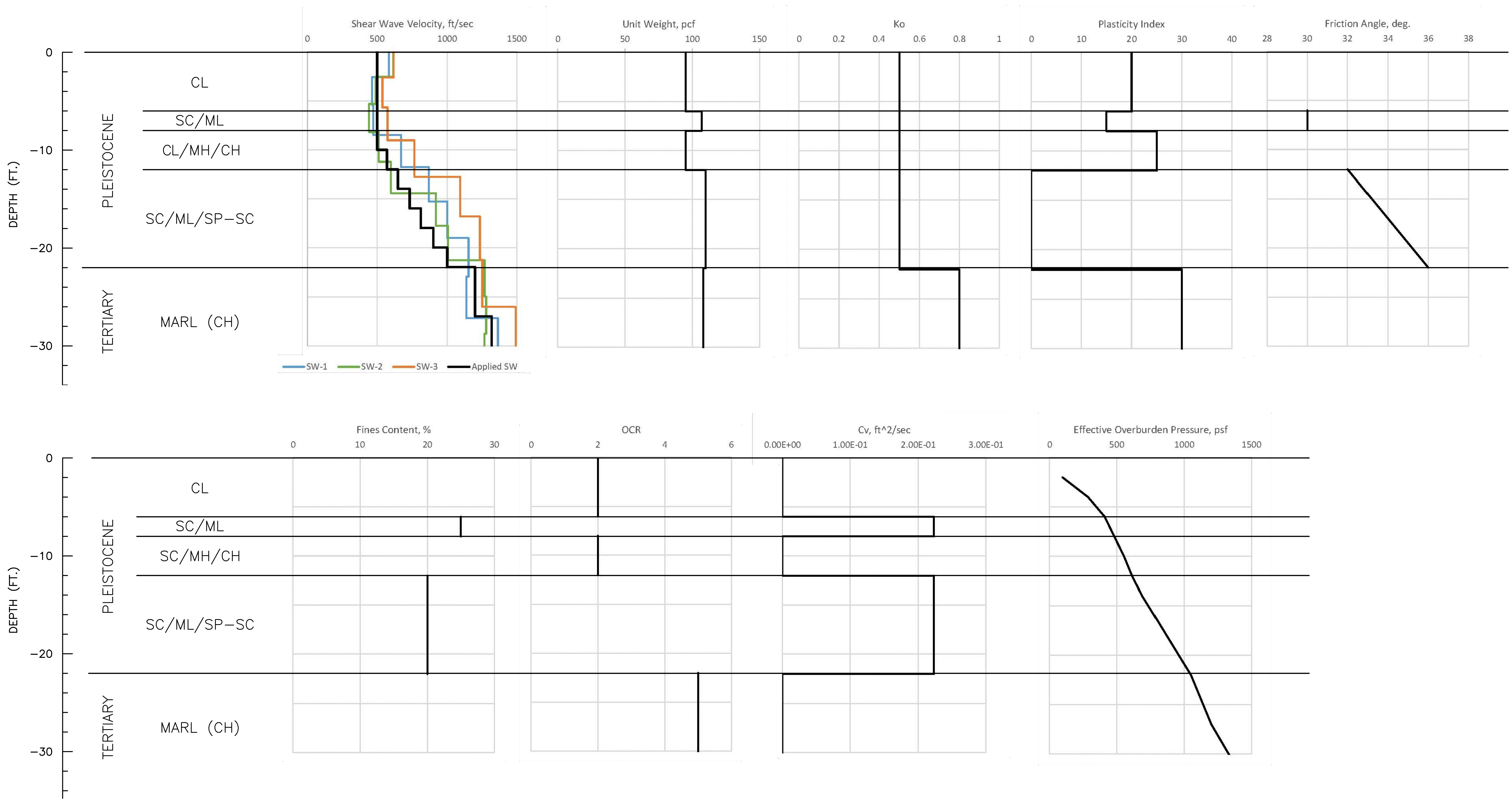
## 4.1 Regional Geology

The project site is located in the Atlantic Coastal Plain physiographic province, and specifically the lower Coastal Plain of South Carolina. The lower Coastal Plain consists of a wedge of late Cretaceous and younger sediments that have been deposited on Paleozoic/Mesozoic igneous and metamorphic rock. The crystalline rock gradually dips seawards or towards the southeast, but the overlying sediments dip at a lesser rate; consequently, the Coastal Plain sedimentary units thicken down dip (Heron, 1962). The thickness of the Coastal Plain sediments at the project site is likely on the order of 2,500 ft. The sediments within the Coastal Plain primarily consist of unconsolidated siliciclastic materials and carbonates with varying quantities of terrigenous matter (Horton and Zullo, 1991). With respect to geotechnical characterization, the materials generally classify as intermediate geomaterials (i.e., in between soil and rock), although some soft rock lenses may be encountered.

## 4.2 SSRA Profile: Shallow Layers (0 to 150 ft)

S&ME performed geotechnical site characterization for this project, and the data are summarized in the Geotechnical Base Line Report (GBLR). The site characterization included soil test borings (STB), cone penetration test (CPT) soundings, dilatometer soundings, MASW seismic testing, and laboratory testing. The data summarized in the GBLR are the basis for the model profile developed for the site-specific response analyses.

Figures 4-1(a) and 4-1(b) present the soil profile stratigraphy, $V_s$ profile, unit weight, Effective Stress Friction Angle (φ′), OCR, Atterberg Limits, $K_0$, Fines Content (FC) and coefficient of consolidation ($C_v$) for a 150 ft deep profile from the existing ground surface. Some of these data are direct measurements (e.g., $V_s$, Atterberg Limits), others are based on correlations (e.g., φ′, $C_v$), and others are estimates (e.g., $K_0$). The values used in our analyses are plotted in Figures 4-1(a) and 4-1(b) and are tabulated in Table 4-1.

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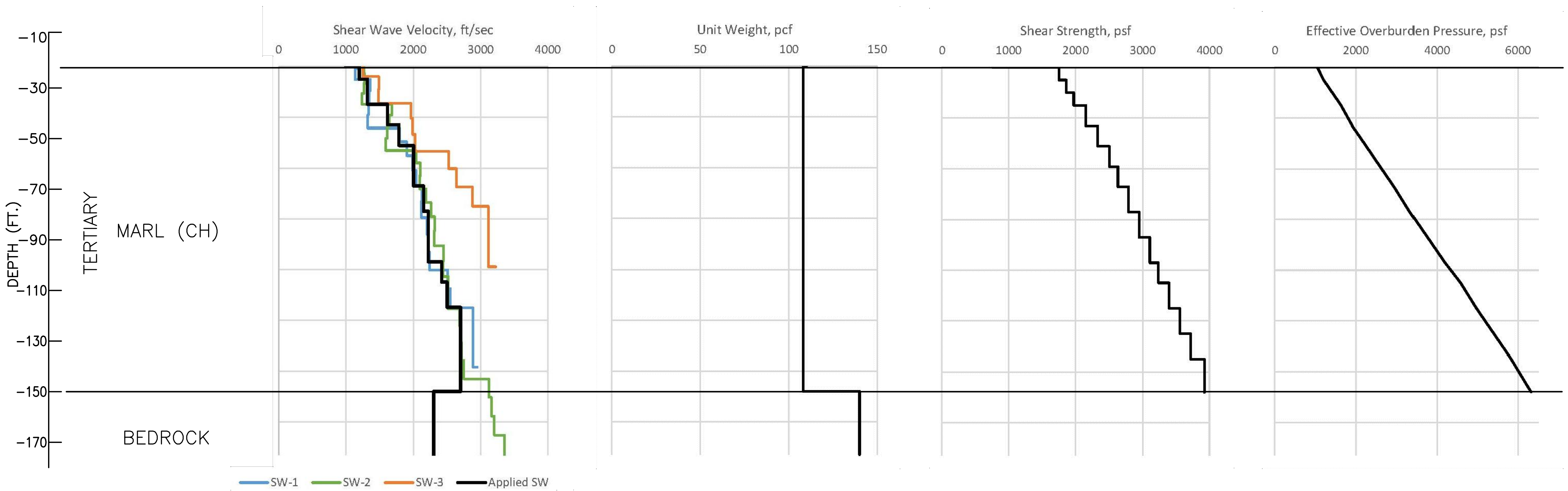
**PROFILE**

VOLVO I-26 INTERCHANGE SSRA
BERKELEY COUNTY, SOUTH CAROLINA

| SCALE: AS SHOWN | DRAWN BY: LAJ | APPROVED BY: MSU |
|---|---|---|
| PROJECT NO. 1413-15-114 | DATE: 12-03-2015 | FIGURE NO. 4-1(a) |

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**PROFILE**

VOLVO I–26 INTERCHANGE SSRA
BERKELEY COUNTY, SOUTH CAROLINA

| | | |
|---|---|---|
| SCALE: AS SHOWN | DRAWN BY: LAJ | APPROVED BY: MSU |
| PROJECT NO. 1413–15–114 | DATE: 12–03–2015 | FIGURE NO. 4-1(b) |

## Table 4-1 – Shallow SSRA profile (0 to 150 ft)

| Geologic Age | Layer No. | Layer H, ft | Depth, ft | USCS Soil Type | $V_s$, ft/sec | PI | Unit Weight, pcf | Ko | Friction Angle (⁰) | $S_u$, psf | OCR | FC (%) | $C_v$ (ft²/sec) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pleistocene | 1 | 2 | 2 | CL | 500 | 20 | 95 | 0.5 | - | 700 | 2 | - | 5.80E-07 |
| Pleistocene | 2 | 2 | 4 | CL | 500 | 20 | 95 | 0.5 | - | 850 | 2 | - | 5.80E-07 |
| Pleistocene | 3 | 2 | 6 | CL | 500 | 20 | 95 | 0.5 | - | 1000 | 2 | - | 5.80E-07 |
| Pleistocene | 4 | 2 | 8 | SC/ML | 500 | 15 | 107 | 0.5 | 30 | - | - | 25 | 2.23E-01 |
| Pleistocene | 5 | 2 | 10 | CL/MH/CH | 500 | 25 | 95 | 0.5 | - | 1500 | 2 | - | 5.80E-07 |
| Pleistocene | 6 | 2 | 12 | CL/MH/CH | 570 | 25 | 95 | 0.5 | - | 1800 | 2 | - | 5.80E-07 |
| Pleistocene | 7 | 2 | 14 | SC/ML/SP-SC | 650 | 0 | 110 | 0.5 | 32 | - | - | 20 | 2.23E-01 |
| Pleistocene | 8 | 2 | 16 | SC/ML/SP-SC | 730 | 0 | 110 | 0.5 | 33 | - | - | 20 | 2.23E-01 |
| Pleistocene | 9 | 2 | 18 | SC/ML/SP-SC | 810 | 0 | 110 | 0.5 | 34 | - | - | 20 | 2.23E-01 |
| Pleistocene | 10 | 2 | 20 | SC/ML/SP-SC | 900 | 0 | 110 | 0.5 | 35 | - | - | 20 | 2.23E-01 |
| Pleistocene | 11 | 2 | 22 | SC/ML/SP-SC | 1000 | 0 | 110 | 0.5 | 36 | - | - | 20 | 2.23E-01 |
| Tertiary | 12 | 5 | 27 | CH | 1200 | 30 | 108 | 0.8 | - | 1752 | 5 | - | 1.16E-05 |
| Tertiary | 13 | 5 | 32 | CH | 1320 | 30 | 108 | 0.8 | - | 1862 | 5 | - | 1.16E-05 |
| Tertiary | 14 | 5 | 37 | CH | 1320 | 30 | 108 | 0.8 | - | 1972 | 5 | - | 1.16E-05 |
| Tertiary | 15 | 8 | 45 | CH | 1620 | 30 | 108 | 0.8 | - | 2148 | 5 | - | 1.16E-05 |
| Tertiary | 16 | 8 | 53 | CH | 1790 | 30 | 108 | 0.8 | - | 2324 | 5 | - | 1.16E-05 |
| Tertiary | 17 | 8 | 61 | CH | 2000 | 30 | 108 | 0.8 | - | 2500 | 5 | - | 1.16E-05 |
| Tertiary | 18 | 8 | 69 | CH | 2000 | 30 | 108 | 0.8 | - | 2628 | 5 | - | 1.16E-05 |
| Tertiary | 19 | 10 | 79 | CH | 2150 | 30 | 108 | 0.8 | - | 2788 | 5 | - | 1.16E-05 |
| Tertiary | 20 | 10 | 89 | CH | 2220 | 30 | 108 | 0.8 | - | 2948 | 5 | - | 1.16E-05 |
| Tertiary | 21 | 10.00 | 99 | CH | 2220 | 30 | 108 | 0.8 | - | 3108 | 5 | - | 1.16E-05 |
| Tertiary | 22 | 8.00 | 107 | CH | 2420 | 30 | 108 | 0.8 | - | 3236 | 5 | - | 1.16E-05 |
| Tertiary | 23 | 10.00 | 117 | CH | 2500 | 30 | 108 | 0.8 | - | 3396 | 5 | - | 1.16E-05 |
| Tertiary | 24 | 10.00 | 127 | CH | 2700 | 30 | 108 | 0.8 | - | 3556 | 5 | - | 1.16E-05 |
| Tertiary | 25 | 10.00 | 137 | CH | 2700 | 30 | 108 | 0.8 | - | 3716 | 5 | - | 1.16E-05 |
| Tertiary | 26 | 13.00 | 150 | CH | 2700 | 30 | 108 | 0.8 | - | 3924 | 5 | - | 1.16E-05 |

***Notes:***

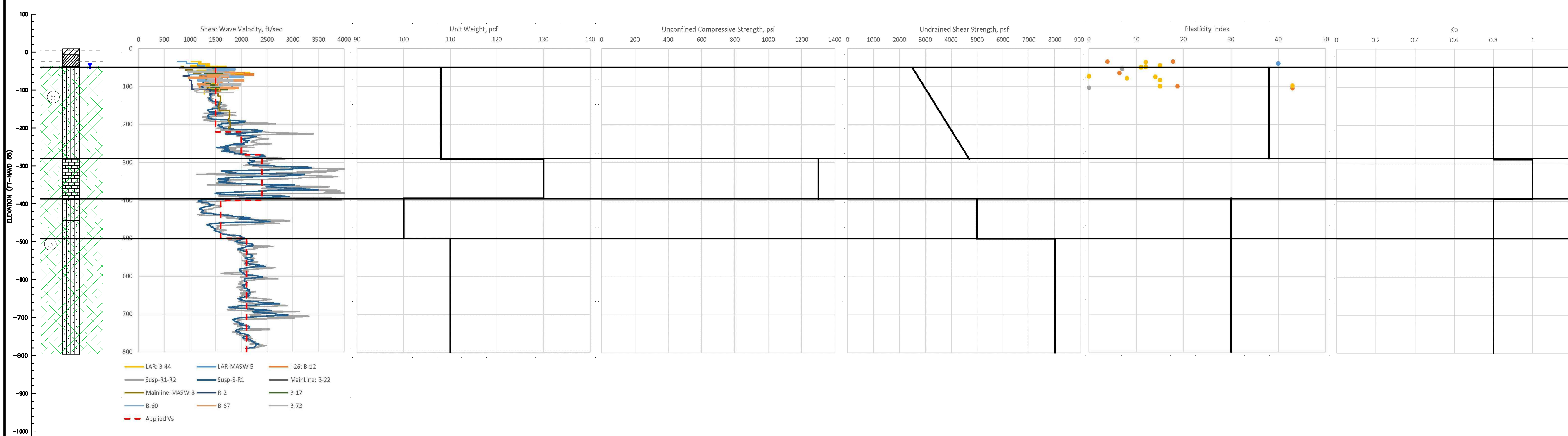
*Modulus & Damping Curves are based on Andrus et al. (2003) Pleistocene and Tertiary (Cooper Marl) Parameters*

*Groundwater location is considered at 4 ft below ground surface*

### 4.3 Basement Profile (150 to 1000 ft Below Ground Surface)

S&ME drilled a deep soil test boring in North Charleston, South Carolina, in October 2008 for the SCDOT. The boring was drilled and sampled to a depth of 500 ft and logged using geophysics (including P-S suspension logging) to a depth of 800 ft. The Basement Profile was inferred from the deep test hole data, and the Basement Profile was appended to the surface profile in Table 4-1 to generate 500 ft and 1000 ft deep profiles for our sensitivity analyses. The layer details of the Basement Profile from 150 ft below the existing ground surface used in our site response analyses are illustrated in Figure 4-2 and summarized in Table 4-2.

# BASEMENT PROFILE




LEGEND

| | |
|---|---|
| (1A) | ARTIFICIAL FILL |
| (1B) | HOLOCENE MARSH DEPOSITS EAST OF SHIPYARD CREEK |
| (1C) | HOLOCENE MARSH DEPOSITS WEST OF I-26 ALIGNMENT |
| (2A) | RECENT TO PLEISTOCENE "BARRIER" SANDS |
| (2B) | PLEISTOCENE FOSSILIFEROUS SHELL SAND AND SILT |
| (2C) | PLEISTOCENE CLAYS AND SILTS |
| (3) | PLEISTOCENE LOWER SANDS |
| (5) | OLIGOCENE/EOCENE COOPER GROUP, SANTEE LIMESTONE AND BLACK MINGO GROUP |

NOTE: OLIGOCENE/EOCENE COOPER GROUP, SANTEE LIMESTONE AND BLACK MINGO GROUP WERE ALL MODELED USING ANDRUS ET AL 2003 TERTIARY M/D CURVES.

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**BASEMENT PROFILE**
PORT ACCESS ROAD
CHARLESTON, SOUTH CAROLINA

| SCALE: AS SHOWN | DRAWN BY: LAJ | APPROVED BY: |
|---|---|---|
| PROJECT NO. 1413-15-075 | DATE: 7-24-2015 | FIGURE NO. 4-2 |

## Table 4-2 – Basement Profile Model (150 to 1000 ft)

| Geologic Age | Layer No. | Layer H, ft | Depth, ft | Formation | USCS Soil Type | $V_s$, fps | PI | Unit Weight, pcf | Ko | Friction Angle | $S_u$, psf |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Tertiary | 27* | 10 | 154 | Ashley | CH | 3200** | 30 | 108 | 0.8 | - | 3930 |
| Tertiary | 28 | 10 | 164 | Ashley | CH | 3200** | 30 | 108 | 0.8 | - | 4040 |
| Tertiary | 29 | 12 | 176 | Harleyville | CH | 3300** | 30 | 108 | 0.8 | - | 4150 |
| Tertiary | 30 | 12 | 188 | Harleyville | CH | 3300** | 30 | 108 | 0.8 | - | 4260 |
| Tertiary | 31 | 12 | 200 | Harleyville | CH | 1500 | 30 | 108 | 0.8 | - | 4370 |
| Tertiary | 32 | 10 | 210 | Harleyville | CH | 1500 | 30 | 108 | 0.8 | - | 4455 |
| Tertiary | 33 | 10 | 220 | Harleyville | CH | 1500 | 30 | 108 | 0.8 | - | 4540 |
| Tertiary | 34 | 12 | 232 | Harleyville | CH | 2000 | 30 | 108 | 0.8 | - | 4625 |
| Tertiary | 35 | 16 | 248 | Harleyville | CH | 2000 | 30 | 108 | 0.8 | - | 4710 |
| Tertiary | 36 | 16 | 264 | Harleyville | CH | 2000 | 30 | 108 | 0.8 | - | 4795 |
| Tertiary | 37 | 16 | 280 | Harleyville | CH | 2000 | 30 | 108 | 0.8 | - | 4700 |
| Tertiary | 38 | 20 | 300 | Cross | IGM | 2400 | 30 | 130 | 1.0 | 48 | 14686 |
| Tertiary | 39 | 20 | 320 | Cross | IGM | 2400 | 30 | 130 | 1.0 | 48 | 16187 |
| Tertiary | 40 | 20 | 340 | Chicora | IGM | 2400 | 30 | 130 | 0.8 | 48 | 17689 |
| Tertiary | 41 | 20 | 360 | Chicora | IGM | 2400 | 30 | 130 | 0.8 | 48 | 19190 |
| Tertiary | 42 | 20 | 380 | Chicora | IGM | 2400 | 30 | 130 | 0.8 | 48 | 20692 |
| Tertiary | 43 | 20 | 400 | Rhems | IGM | 2400 | 30 | 130 | 0.8 | 48 | 22193 |
| Tertiary | 44 | 9 | 409 | Rhems | IGM | 1600 | 30 | 100 | 0.8 | - | 7000 |
| Tertiary | 45 | 13 | 422 | Rhems | IGM | 1600 | 30 | 100 | 0.8 | - | 7625 |
| Tertiary | 46 | 13 | 435 | Rhems | IGM | 1600 | 30 | 100 | 0.58 | - | 8250 |
| Tertiary | 47 | 13 | 448 | Rhems | IGM | 1600 | 30 | 100 | 0.8 | - | 8875 |
| Tertiary | 48 | 13 | 461 | Rhems | IGM | 1600 | 30 | 100 | 0.8 | - | 9500 |
| Tertiary | 49 | 13 | 474 | Rhems | IGM | 1600 | 30 | 100 | 0.8 | - | 10125 |
| Tertiary | 50 | 13 | 487 | Rhems | IGM | 1600 | 30 | 100 | 0.8 | - | 10750 |
| Tertiary | 51 | 13 | 500 | Rhems | IGM | 1600 | 30 | 100 | 0.8 | - | 11375 |
| Tertiary | 52 | 7 | 507 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 12000 |
| Tertiary | 53 | 17 | 524 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 12172 |
| Tertiary | 54 | 17 | 541 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 12344 |
| Tertiary | 55 | 17 | 558 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 12516 |
| Tertiary | 56 | 17 | 575 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 12688 |
| Tertiary | 57 | 17 | 592 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 12860 |
| Tertiary | 58 | 17 | 609 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 13032 |
| Tertiary | 59 | 17 | 626 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 13204 |
| Tertiary | 60 | 17 | 643 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 13376 |
| Tertiary | 61 | 17 | 660 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 13548 |
| Tertiary | 62 | 17 | 677 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 13720 |
| Tertiary | 63 | 17 | 694 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 13892 |
| Tertiary | 64 | 17 | 711 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 14064 |
| Tertiary | 65 | 17 | 728 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 14236 |
| Tertiary | 66 | 17 | 745 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 14408 |
| Tertiary | 67 | 17 | 762 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 14580 |
| Tertiary | 68 | 17 | 779 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 14752 |
| Tertiary | 69 | 17 | 796 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 14924 |
| Tertiary | 70 | 17 | 813 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 15096 |

| Geologic Age | Layer No. | Layer H, ft | Depth, ft | Formation | USCS Soil Type | $V_s$, fps | PI | Unit Weight, pcf | Ko | Friction Angle | $S_u$, psf |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Tertiary | 71 | 17 | 830 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 15268 |
| Tertiary | 72 | 17 | 847 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 15440 |
| Tertiary | 73 | 17 | 864 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 15612 |
| Tertiary | 74 | 17 | 881 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 15784 |
| Tertiary | 75 | 17 | 898 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 15956 |
| Tertiary | 76 | 17 | 915 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 16128 |
| Tertiary | 77 | 17 | 932 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 16300 |
| Tertiary | 78 | 17 | 949 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 16472 |
| Tertiary | 79 | 17 | 966 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 16644 |
| Tertiary | 80 | 17 | 983 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 16816 |
| Tertiary | 81 | 17 | 1000 | - | IGM | 2100 | 30 | 105 | 0.8 | - | 17000 |

***Notes:***

**Layer number continued from the last layer of the profile in Table 4-1*

***Values are based on the MASW test, SW-2 performed in the project site*

*Light gray portion is the additional layers for generating 500 ft deep profile.*

*Entire table includes all layers for generating 1000 ft deep profile.*

*Modulus & Damping Curves for All Layers were generated using Andrus et al. (2003) Tertiary (Ashley or Cooper Marl) Parameters.*

## 4.4 SSRA Profiles for Sensitivity Analyses

Based on our subsurface exploration and associated laboratory testing on the site, we observed a general ±10 to 20% variability of $V_s$, SS and PI data, especially in the shallow layers above Cooper Marl although both SS and PI values were erratically distributed. Fines content (FC) in the sand layers were also erratic in nature and ranged from 9 to 48%.

### *4.4.1 Total Stress Analyses*

The following soil profile variations were considered:

- Input motions depth variations: three profiles were developed assuming the B/C boundary to be at 150, 500 and 1000 ft depth from the ground surface.
- Higher $V_s$ profile: $V_s$ values of the entire SSRA profile were increased by 20%. Corresponding SS values were also increased by 10%.
- Lower $V_s$ profile: $V_s$ values of the entire SSRA profile were decreased by 20%. Corresponding SS values were also decreased by 10%.
- Variations to the soil modulus reduction and damping curves: soil profiles were developed assuming a 20% increment and 20% reduction to the PI values.

### 4.4.2 *Effective Stress Analyses*

The following soil profile variations were considered:

- Input motions depth variations: two profiles were developed assuming the B/C boundary to be at 150 and 500 ft depth from the ground surface.
- Higher $V_s$ profile: $V_s$ values of the entire SSRA profile were increased by 20%. Corresponding SS values were also increased by 10%.
- Lower $V_s$ profile: $V_s$ values of the entire SSRA profile were decreased by 20%. Corresponding SS values were also decreased by 10%.
- The FC values of the sand layers were increased to 35% (i.e. FC = 35%) along with a 30% increment to the corresponding SS values of the sand layers.

# 5.0 Shear Modulus Reduction & Damping Curves

## 5.1 Andrus et al. (2003) Models

The dynamic soil properties were modeled using the normalized modulus and damping reduction curves following Andrus et al. (2003). These curves represent best fits to available laboratory data on South Carolina soils.

The modulus curves are based on three equations:

$$G/G_{max} = \frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}} \tag{5.1}$$

$$\gamma_r = \gamma_{r1}\left({\sigma'_m}/{P_a}\right)^{k} \tag{5.2}$$

$$\sigma'_m = \sigma'_v\left(\frac{1+2K'_o}{3}\right) \tag{5.3}$$

Where:

$G$ = shear modulus, and $G_{max}$ = low strain shear modulus.

$\gamma$ = shear strain, $\gamma_r$ = reference shear strain for hyperbolic fitting, and $\gamma_{r1}$ = reference shear strain at a mean effective confining pressure of 100 kPa.

$\sigma'_m$ = mean effective confining pressure, $\sigma'_v$ = vertical effective stress and $K'_o$ = coefficient of effective earth pressure at rest.

α and κ are curve fitting parameters.

The mean effective stress is calculated based on the applicable vertical effective stress and $K'_o$, and tabulated fitting parameters (α, κ and $\gamma_{r1}$) are provided for different values of Plasticity Index and geologic age or origin (Holocene, Pleistocene, Tertiary or Residual). The Tertiary soils are further discretized based on geologic formation.

The damping curves are based on three equations as well:

$$D - D_{min} = 12.2(G/G_{max})^2 - 34.2(G/G_{max}) + 22.0 \tag{5.4}$$

$$D_{min} = D_{min1}(\sigma'_m/P_a)^{-k/2} \tag{5.5}$$

$$D_{min1} = a(PI) + b \tag{5.6}$$

Where:

$D$ = damping, $D_{min}$ = low strain damping, and $D_{min1}$ = small strain damping at a mean effective confining pressure of 100 kPa.

a, b and k (same as in equation 5.2) are curve fitting parameters and PI is the soil plasticity index.

Values for the curve fitting parameters a, b, k and $D_{min1}$ are based on the geologic age or formation and for some of the parameters, PI.

The above equations with appropriate parameters were used to construct normalized modulus and damping curves for the midpoint of every layer. The resulting curves were entered into DEEPSOIL as a series of points.

## 5.2 Implied Shear Strength Correction

The shear modulus (G) is equal to the shear stress (τ) divided by the shear strain (γ); therefore, a normalized modulus curve results in an implied stress-strain curve for a given value of $G_{max}$. However, since the normalized curves are based only on small to mid-strain soil data, the resulting stress-strain curve at larger strains is not limited to the actual shear strength of the soil. Depending on the shape of the normalized curve and the value of $G_{max}$, the implied shear stress at large strains may be greater than or less than the actual shear strength. For stronger ground motions, such as those provided for this project, large shear strain behavior is critical to the site response, and the stress corresponding to the large strain (i.e. shear strength) must be appropriately captured in the soil models. This issue has been raised by a number of researchers including Hashash et al. (2010) and Yee et al. (2013). They both proposed manual correction procedures that are time consuming and not always feasible, especially when the implied and target strengths are quite different. More recently Prof. Hashash's research group has developed an efficient approach to address this issue through the introduction of a new soil model called the GQ/H (Generalized Quadratic/Hyperbolic) model (Groholski et al. 2015) and incorporated into DEEPSOIL. The model allows the user to capture both the small strain response and the desired shear strength.

In Figure 5.1, the GQ/H model was used to fit a curve through the points defined by the Andrus et al. (2003) procedure and the estimated maximum shear strength of each layer. The "Reference" curve is the Andrus et al. (2003) procedure, the "Target" strength is user-defined SS value, and the "Current" and "Curve Fit" lines illustrate the fitted curve using GQ/H model. The GQ/H model fits the 'Reference' line at the smaller strain levels while matched with the SS value at higher strain levels. Therefore, the GQ/H model option available in DEEPSOIL was used in this project.

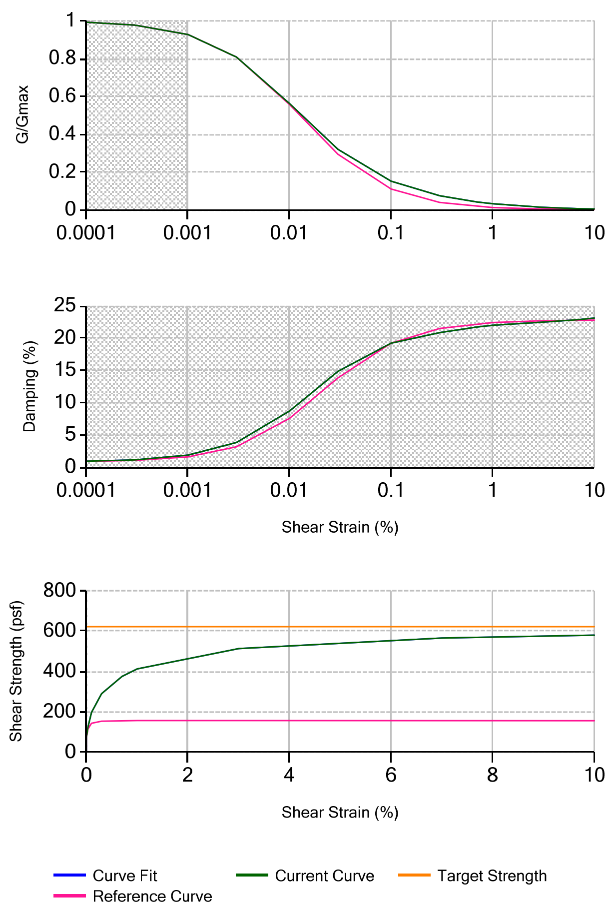



**Figure 5-1 – Example of Curve Fitting with the GQH Model.**

## 6.0 Results

A total of 406 individual site response simulations were performed using NL and EL, and both total stress and effective stress analysis options. A total of 11 soil profile variations were considered in total stress analysis and seven profile variations were considered in effective stress analysis option. Each of the model scenarios was analyzed for each of the 14 acceleration time histories (i.e., 7 FEE motions, 7 SEE motions) using both EL (total stress option only) and NL methods.

### Table 6-1 – Summary of Analyses

| Scenario | FEE | | SEE | | Total |
|---|---|---|---|---|---|
| | Equivalent-Linear | Nonlinear | Equivalent-Linear | Nonlinear | |
| Total Stress Analysis | | | | | |
| Input at 150 ft | 7 | 7 | 7 | 7 | **28** |
| Input at 500 ft | 7 | 7 | 7 | 7 | **28** |
| Input at 1000 ft | 7 | 7 | 7 | 7 | **28** |
| Higher Vs for both 150 and 500 ft Profiles | 14 | 14 | 14 | 14 | **28** |
| Lower Vs for both 150 and 500 ft Profiles | 14 | 14 | 14 | 14 | **28** |
| Higher PI for both 150 and 500 ft Profiles | 14 | 14 | 14 | 14 | **28** |
| Lower PI for both 150 and 500 ft Profiles | 14 | 14 | 14 | 14 | **28** |
| *Sub-total* | *77* | *77* | *77* | *77* | ***308*** |
| Effective Stress Analysis | | | | | |
| Input at 150 ft | - | 7 | - | 7 | **14** |
| Input at 500 ft | - | 7 | - | 7 | **14** |
| Higher Fines for both 150 and 500 ft Profiles | - | 14 | - | 14 | **28** |
| Higher Vs for 500 ft Profile | - | 7 | - | 7 | **14** |
| Lower Vs for both 150 and 500 ft Profiles | - | 14 | - | 14 | **28** |
| *Sub-total* | - | *49* | - | *49* | ***98*** |
| **Grand Total** | **77** | **126** | **77** | **126** | **406** |

DEEPSOIL allows multiple analyses to be performed with a single input file. For a given scenario, the NL and EL analyses were performed using all 14 acceleration time histories with a single input file and execution. The resulting output was extensive, and we used a custom built MATLAB program to process it.

The results from total stress and effective stress analyses are plotted in Appendices II and III, respectively. Each appendix is organized in a consistent manner. Summary plots are provided in the beginning followed by individualized output. The general organization of each of the appendices are as follows:

1. Four pages of *summary plots* of averaged response of the 7 FEE motions for each of the sensitivity variations (e.g., Input at different depths, Higher/lower Vs, PI, etc.). These plots present the comparison of all sensitivity variations for the FEE motion cases:

   a. Plots of PGA, maximum shear strain, and maximum cyclic stress ratios versus depth for NL and EL analyses. (2 pages)
   b. Plots of EL and NL response spectra at the ground surface and the top of marl. (2 pages)
2. Four pages of *summary plots* of average response of the 7 SEE motions for each of the sensitivity variations (e.g., Input at different depths, Higher/lower Vs, PI, etc.). These plots present the comparison of all sensitivity variations for the SEE motion cases:
   a. Plots of PGA, maximum shear strain, and maximum cyclic stress ratios versus depth for NL and EL analyses. (2 pages)
   b. Plots of EL and NL response spectra at the ground surface and the top of marl. (2 pages)
3. Four pages of *Individualized plots* for each sensitivity variations (e.g., Input at different depths, Higher/lower Vs, PI, etc.) consist responses from each of the 7 FEE and 7 SEE motions including the corresponding average responses:
   a. Plots of PGA, maximum shear strain, and maximum cyclic stress ratios versus depth for NL and EL analyses. (2 pages)
   b. Plots of EL and NL response spectra at the ground surface and the top of marl. (2 pages)

# 7.0 Synthesis of Results and Design Spectra

Our general observations with respect to the output are summarized as follows:

- The NL analyses generally yield smaller motions at the ground surface than do the EL analyses for both the FEE and SEE acceleration time histories; although, the differences between EL and NL analyses are more prominent in SEE cases.
- With respect to the top of marl response spectra, the EL and NL analyses produce similar results, especially in the FEE cases.
- Higher shear wave velocities and the use of higher-PI modulus and damping curves generally produced larger surface accelerations.
- Smaller shear wave velocities and the use of smaller-PI modulus and damping curves generally produced smaller surface accelerations.
- For all analyses, the maximum shear strain occurs within the Pleistocene sand layers between depths of about 6 to 22 ft. The magnitudes of shear strain within these sand layers are significant, in many cases beyond 2%, which is the threshold set by the SCDOT GDM (version 2010) beyond which EL analyses are not reliable. Strain accumulation in the sand layers are lesser for the deeper profile variations i.e. 500 ft and 1000 ft profiles.
- Large localized shear strains in the sand layers dampened out a significant portion of the wave energy; consequently, de-amplified surface responses are observed, especially in the SEE cases.
- For the deeper profile cases (i.e. 500 ft and 1000 ft cases), more localized shear strains are observed at the base and at some of the deeper layers. Thus, further dampening of the propagating wave energy occurred, and significantly smaller responses were observed at the ground surface, especially for the SEE cases.

- The effective stress analysis cases for the 150 ft deep model cases show greater shear strain accumulation in the sand layers which produce smaller surface acceleration than the total stress responses, especially for SEE cases.
- The effective stress analysis cases for the 500 and 1000 ft deep model cases produce similar surface accelerations as compared to the total stress analysis responses.
- Top of Marl responses are effectively similar from both total and effective stress analyses cases.
- Excess pore pressure ratios indicate the sand layers are liquefiable, especially during an earthquake event similar to the SEE scenarios.

The 150 ft deep profile cases produced significantly higher spectral accelerations than the 500 ft and 1000 ft profile cases, both at ground surface and at top of Marl. All the sensitivity variations for the 150 ft and 500 ft model profiles produced similar responses. Based on the known geology of the area, we expect the deeper profiles (i.e. the 500 ft and 1000 ft profiles) better represent the actual subsurface condition. However, if the deeper strata (i.e. much below than 150 ft) are stiffer than what has been assumed in our 500 ft and 1000 ft model profiles, then accelerations computed based on 150 ft model are also possible but not equally probable. It is our opinion the large accelerations computed based on 150 ft base model would produce significant, unrealistic structural demand. Unless any future attempt is planned to obtain the deep profile strata for this specific project site, we believe the accelerations computed using the deeper profiles (i.e. 500 ft and 1000 ft models) are more practicable. Additionally the responses computed using 500 ft and 1000 ft models were observed to be very close. Therefore, the 500 ft model was selected as our 'Base' SSRA model for performing our recommended ADRS curve computations.

With respect to the sensitivity analyses of all the variations of our 'Base' SSRA model profile (i.e. 500 ft profile), the 'Base' model reflects our estimate of the actual subsurface conditions and soil column properties based on the exploration data and our experience with soils in the general area of the site. While the alternative scenarios that were considered in the sensitivity analyses are all possible, they are not equally probable. Therefore, a simple arithmetic average of the base model result with all of the sensitivity analyses may introduce an unknown bias. Furthermore, the base model analyses generally fell within the midrange of the results. As the effective stress analyses show practically similar surface responses, it is our opinion that the total stress NL 'Base' model response spectra should be used as the basis for the site-specific ADRS. However, in recognition of the fact that NL analyses can under-predict the short period motions, we used the PGA values based on the EL analysis outcomes.

The site-specific ADRS was constructed in accordance with the procedures specified in the Scope of Services (i.e., Table 12-33). Four site-specific ADRS curves were developed for: 1) FEE ground surface, 2) FEE top of marl, 3) SEE ground surface, and 4) SEE top of marl. The resulting ADRS curves are presented in Figures 7-1 through 7-4. Each figure includes the following:

- The response spectra developed using the SCDOT "three-point method".
- The 'recommended site specific acceleration response spectra' curve for the 'Base' profile with NL total stress analysis.
- A 'Smoothed Site-Specific ADRS' curve based on the Scope of Services (i.e., Table 12-33).
- Tabulated values of the 'Smoothed Site-Specific ADRS'.
- Tabulated ancillary values as required by the Scope of Services.

For comparison purpose, the FEE and SEE 'Smoothed Site-Specific ADRS' curves for both at ground surface and top of Marl have been plotted together in Figure 7-5.

The ADRS curves presented herein are applicable for the current free-field conditions. The ADRS curves do not consider the consequences of ground improvement or the potential stiffening effects of deep foundations.

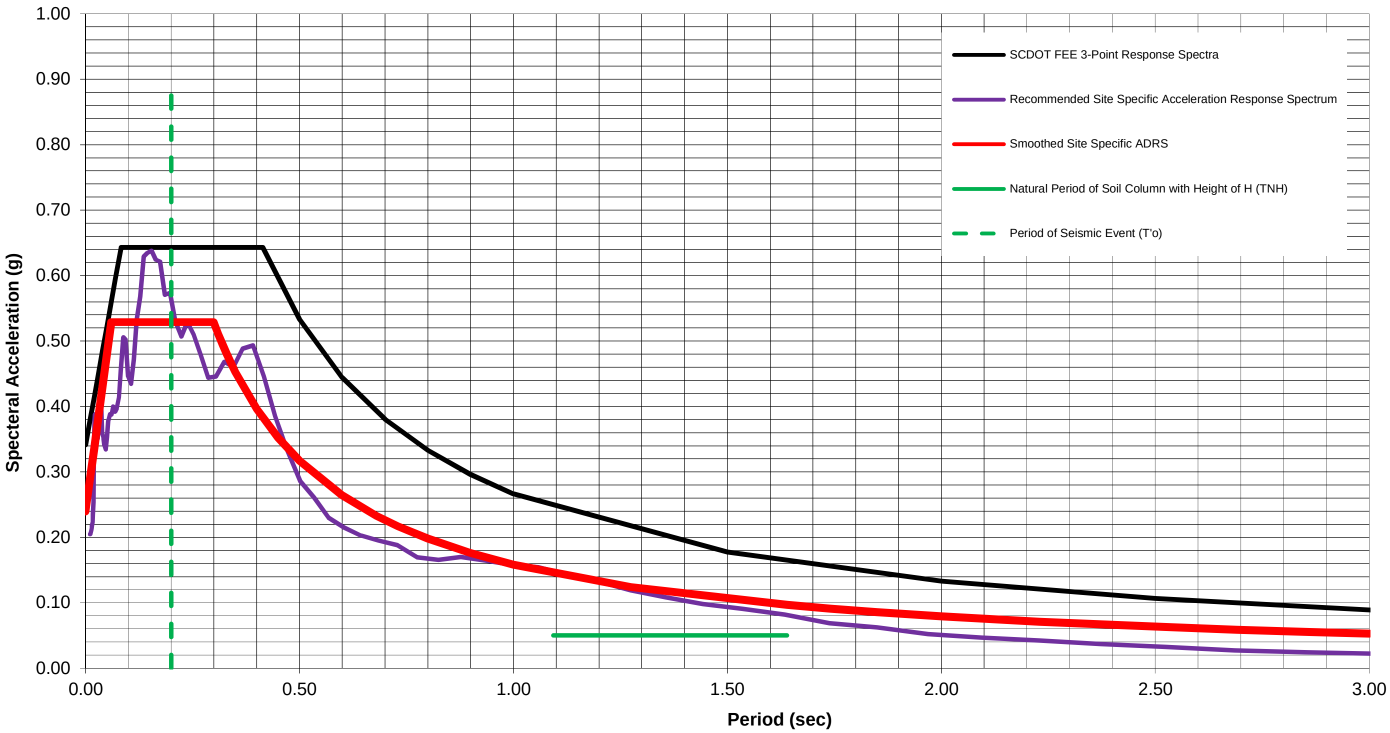
| | | | | | |
|---|---|---|---|---|---|
| **Project ID:** | 1413-15-114 | | | Latitude: | 33.121009 |
| **Route:** | | County: | Berkeley | Longitude: | -80.277809 |
| **Project:** | Volvo I-26 Interchange | | | | |

Designer: S&ME, Inc.
Date: December 2015

| Design EQ | PGA | $S_{DS}$ | $S_{D1}$ | $M_w$ | R | PGV | $D_{a5-95}$ | $T'_o$ |
|---|---|---|---|---|---|---|---|---|
| | g | g | g | | km | ft/sec | sec | sec |
| FEE | 0.240 | 0.529 | 0.158 | 7.35 | 19 | 8.7 | 22.31 | 0.20 |

| | | | |
|---|---|---|---|
| OC= | I | Damping: | 5% |
| Geologic Condition: | Geologically Realistic (Q=100) | | |
| ADRS Location within Soil Column: | Ground Surface | | |

| **Fundamental Period of Structure $T_o$** | **Range of Interest Sec** | | **V*s,H** | **H** | **$T_{NH}$ sec** | |
|---|---|---|---|---|---|---|
| **sec** | $0.5*T_o$ | $2.0*T_o$ | ft/sec | ft | (4*H)/V*s,H | (6*H)/V*s,H |
| - | - | - | 1830 | 500 | 1.093 | 1.639 |




Tabulated Values (Smoothed ADRS Curve)

| | T (sec) | Sa (g) |
|---|---|---|
| | 0.00 | 0.24 |
| | 0.01 | 0.29 |
| | 0.02 | 0.34 |
| | 0.03 | 0.38 |
| $\sim T_o$ | 0.06 | 0.53 |
| | 0.10 | 0.53 |
| | 0.15 | 0.53 |
| | 0.17 | 0.53 |
| $\sim T_s$ | 0.30 | 0.53 |
| | 0.31 | 0.51 |
| | 0.32 | 0.49 |
| | 0.33 | 0.48 |
| | 0.35 | 0.45 |
| | 0.40 | 0.40 |
| | 0.45 | 0.35 |
| | 0.50 | 0.32 |
| | 0.60 | 0.26 |
| | 0.68 | 0.23 |
| | 0.73 | 0.22 |
| | 0.80 | 0.20 |
| | 0.90 | 0.18 |
| | 1.00 | 0.16 |
| | 1.27 | 0.12 |
| | 1.64 | 0.10 |
| | 1.74 | 0.09 |
| | 1.85 | 0.09 |
| | 2.00 | 0.08 |
| | 2.23 | 0.07 |
| | 2.52 | 0.06 |
| | 2.69 | 0.06 |
| | 2.86 | 0.06 |
| | 3.00 | 0.05 |

| | | |
|---|---|---|
| **Project No.: 1413-15-114** | **S&ME** | **Site-Specific Design Response Spectra**<br>**FEE Design Event**<br>**Response Spectra at Ground Surface** | **Figure 7-1** |
| **Date: December 11, 2015** | | | |
| **Volvo I-26 Interchange** | | | |

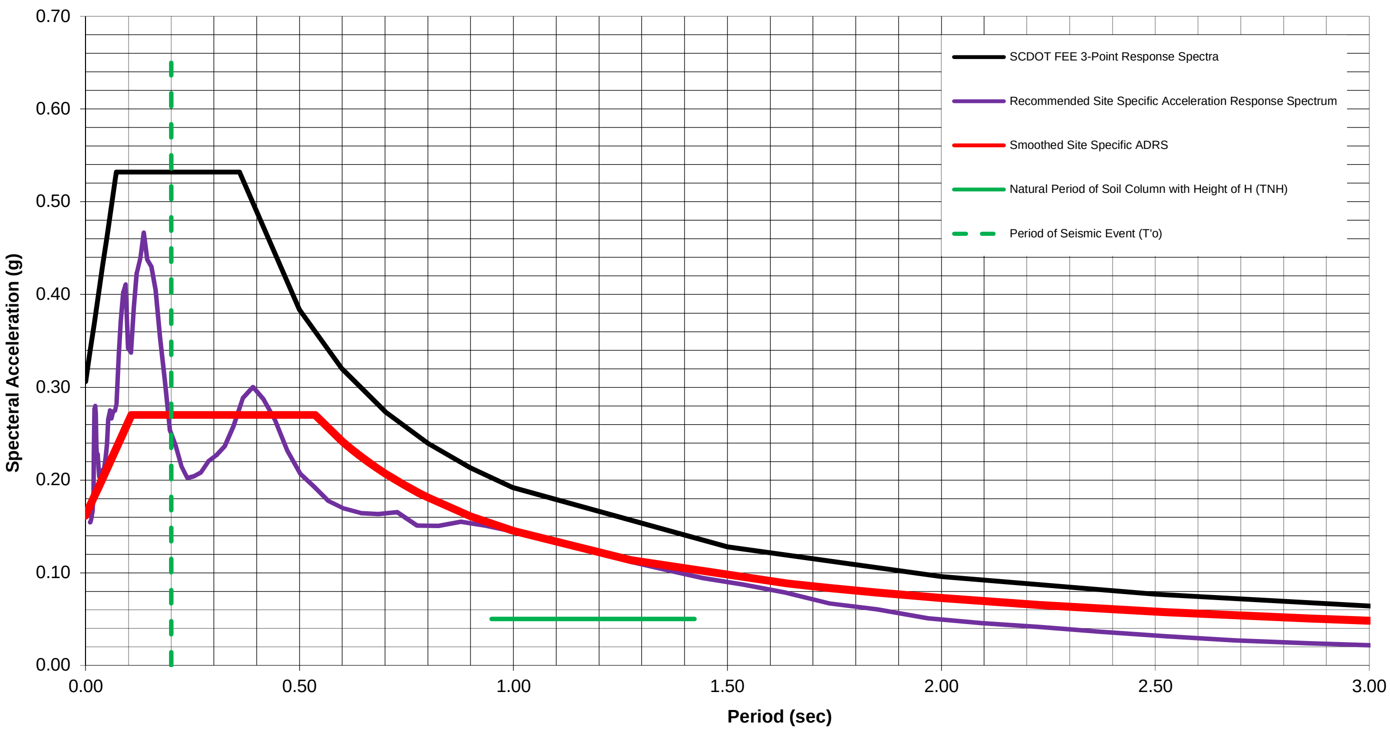
| | | | | | |
|---|---|---|---|---|---|
| **Project ID:** | 1413-15-114 | | | Latitude: | 33.121009 |
| **Route:** | | County: | Berkeley | Longitude: | -80.277809 |
| **Project:** | Volvo I-26 Interchange | | | | |

Designer: S&ME, Inc.
Date: December 2015

| Design EQ | PGA | $S_{DS}$ | $S_{D1}$ | $M_w$ | R | PGV | $D_{a5-95}$ | $T'_o$ |
|---|---|---|---|---|---|---|---|---|
| | g | g | g | | km | ft/sec | sec | sec |
| FEE | 0.162 | 0.270 | 0.145 | 7.35 | 19 | 8.0 | 22.31 | 0.20 |

| | | | |
|---|---|---|---|
| OC= | I | Damping: | 5% |
| Geologic Condition: | Geologically Realistic (Q=100) | | |
| ADRS Location within Soil Column: | Top of Marl | | |

| Fundamental Period of Structure $T_o$ | Range of Interest | | $V^*s,H$ | H | $T_{NH}$ | |
|---|---|---|---|---|---|---|
| | Sec | | | | sec | |
| sec | $0.5^*T_o$ | $2.0^*T_o$ | ft/sec | ft | $(4^*H)/V^*s,H$ | $(6^*H)/V^*s,H$ |
| - | - | - | 2015 | 478 | 0.949 | 1.423 |




Tabulated Values (Smoothed ADRS Curve)

| | T (sec) | Sa (g) |
|---|---|---|
| | 0.00 | 0.16 |
| | 0.01 | 0.17 |
| | 0.02 | 0.18 |
| | 0.03 | 0.19 |
| $\sim T_o$ | 0.11 | 0.27 |
| | 0.13 | 0.27 |
| | 0.15 | 0.27 |
| | 0.25 | 0.27 |
| $\sim T_s$ | 0.54 | 0.27 |
| | 0.60 | 0.24 |
| | 0.62 | 0.23 |
| | 0.64 | 0.23 |
| | 0.66 | 0.22 |
| | 0.68 | 0.21 |
| | 0.70 | 0.21 |
| | 0.72 | 0.20 |
| | 0.74 | 0.20 |
| | 0.76 | 0.19 |
| | 0.78 | 0.19 |
| | 0.80 | 0.18 |
| | 0.90 | 0.16 |
| | 1.00 | 0.14 |
| | 1.27 | 0.11 |
| | 1.64 | 0.09 |
| | 1.74 | 0.08 |
| | 1.85 | 0.08 |
| | 2.00 | 0.07 |
| | 2.23 | 0.07 |
| | 2.52 | 0.06 |
| | 2.69 | 0.05 |
| | 2.86 | 0.05 |
| | 3.00 | 0.05 |

| | | | |
|---|---|---|---|
| **Project No.:** **1413-15-114** | S&ME | **Site-Specific Design Response Spectra**<br>**FEE Design Event**<br>**Response Spectra at Top of Marl** | **Figure**<br>**7-2** |
| **Date: December 11, 2015** | | | |
| **Volvo I-26 Interchange** | | | |

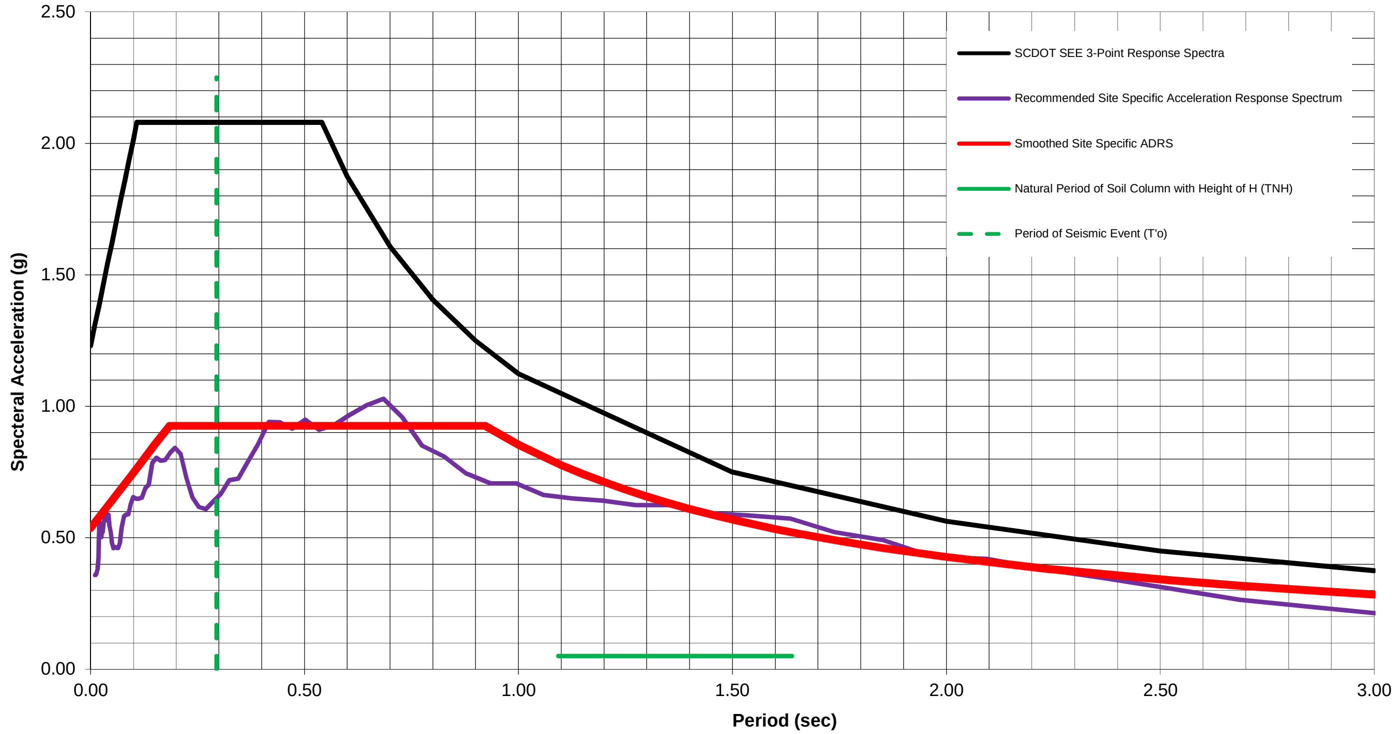
| | | | | | |
|---|---|---|---|---|---|
| **Project ID:** | 1413-15-114 | | | Latitude: | 33.121009 |
| **Route:** | | County: | Berkeley | Longitude: | -80.277809 |
| **Project:** | Volvo I-26 Interchange | | | | |

Designer: S&ME, Inc.
Date: December 2015

| Design EQ | PGA | $S_{DS}$ | $S_{D1}$ | $M_w$ | R | PGV | $D_{a5-95}$ | $T'_o$ |
|---|---|---|---|---|---|---|---|---|
| | g | g | g | | km | ft/sec | sec | sec |
| SEE | 0.537 | 0.926 | 0.854 | 7.36 | 17 | 47.0 | 21.52 | 0.29 |

| | | | |
|---|---|---|---|
| OC= | I | Damping: | 5% |
| Geologic Condition: | Geologically Realistic (Q=100) | | |
| ADRS Location within Soil Column: | Ground Surface | | |

| Fundamental Period of Structure $T_o$ | Range of Interest Sec | | V*s,H | H | $T_{NH}$ sec | |
|---|---|---|---|---|---|---|
| sec | $0.5*T_o$ | $2.0*T_o$ | ft/sec | ft | (4*H)/V*s,H | (6*H)/V*s,H |
| - | - | - | 1830 | 500 | 1.093 | 1.639 |




Tabulated Values (Smoothed ADRS Curve)

| | T (sec) | Sa (g) |
|---|---|---|
| | 0.00 | 0.54 |
| | 0.01 | 0.56 |
| | 0.10 | 0.75 |
| | 0.15 | 0.85 |
| ~$T_o$ | 0.18 | 0.93 |
| | 0.25 | 0.93 |
| | 0.50 | 0.93 |
| | 0.75 | 0.93 |
| ~$T_s$ | 0.92 | 0.93 |
| | 1.00 | 0.85 |
| | 1.10 | 0.78 |
| | 1.15 | 0.74 |
| | 1.20 | 0.71 |
| | 1.25 | 0.68 |
| | 1.30 | 0.66 |
| | 1.35 | 0.63 |
| | 1.40 | 0.61 |
| | 1.45 | 0.59 |
| | 1.50 | 0.57 |
| | 1.55 | 0.55 |
| | 1.60 | 0.53 |
| | 1.65 | 0.52 |
| | 1.70 | 0.50 |
| | 1.75 | 0.49 |
| | 1.80 | 0.47 |
| | 1.85 | 0.46 |
| | 2.00 | 0.43 |
| | 2.23 | 0.38 |
| | 2.52 | 0.34 |
| | 2.69 | 0.32 |
| | 2.86 | 0.30 |
| | 3.00 | 0.28 |

| | | | |
|---|---|---|---|
| **Project No.:** 1413-15-114 | S&ME | **Site-Specific Design Response Spectra**<br>**SEE Design Event**<br>**Response Spectra at Ground Surface** | **Figure**<br>**7-3** |
| **Date: December 11, 2015** | | | |
| **Volvo I-26 Interchange** | | | |

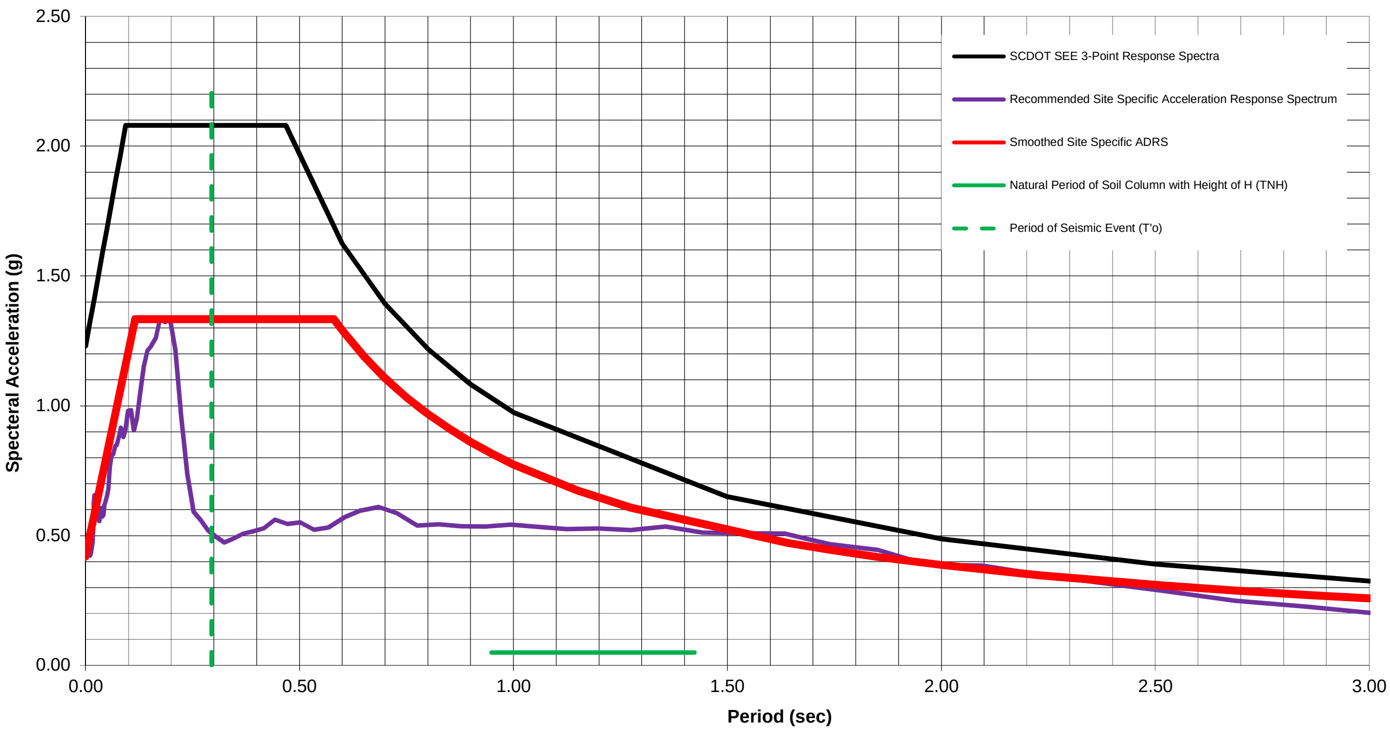
| | | | | | |
|---|---|---|---|---|---|
| **Project ID:** | 1413-15-114 | | | Latitude: | 33.121009 |
| **Route:** | | County: | Berkeley | Longitude: | -80.277809 |
| **Project:** | Volvo I-26 Interchange | | | | |

Designer: S&ME, Inc.
Date: December 2015

| Design EQ | PGA | $S_{DS}$ | $S_{D1}$ | $M_w$ | R | PGV | $D_{a5\text{-}95}$ | $T'_o$ |
|---|---|---|---|---|---|---|---|---|
| | g | g | g | | km | ft/sec | sec | sec |
| SEE | 0.420 | 1.334 | 0.775 | 7.36 | 17 | 42.6 | 21.52 | 0.29 |

| OC= | I | Damping: | 5% |
|---|---|---|---|
| | | Geologic Condition: | Geologically Realistic (Q=100) |
| | | ADRS Location within Soil Column: | Top of Marl |

| Fundamental Period of Structure $T_o$ | Range of Interest | | V*s,H | H | $T_{NH}$ | |
|---|---|---|---|---|---|---|
| | Sec | | | | sec | |
| sec | $0.5{*}T_o$ | $2.0{*}T_o$ | ft/sec | ft | (4*H)/V*s,H | (6*H)/V*s,H |
| - | - | - | 2015 | 478 | 0.949 | 1.423 |




Tabulated Values (Smoothed ADRS Curve)

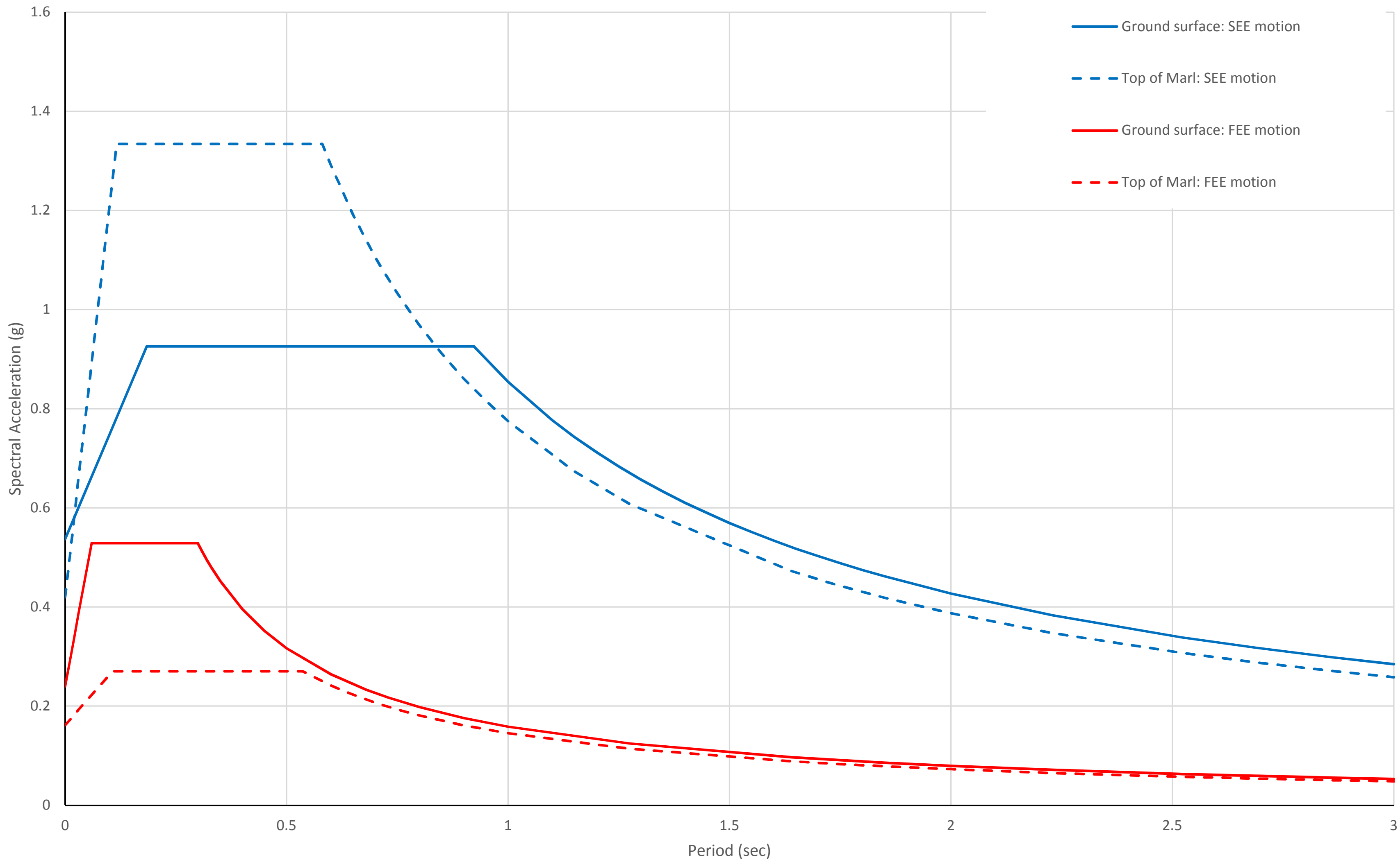
| | T (sec) | Sa (g) |
|---|---|---|
| | 0.00 | 0.42 |
| | 0.01 | 0.50 |
| | 0.02 | 0.58 |
| | 0.03 | 0.66 |
| ~$T_o$ | 0.12 | 1.33 |
| | 0.13 | 1.33 |
| | 0.15 | 1.33 |
| | 0.17 | 1.33 |
| ~$T_s$ | 0.58 | 1.33 |
| | 0.60 | 1.29 |
| | 0.65 | 1.19 |
| | 0.68 | 1.14 |
| | 0.70 | 1.11 |
| | 0.72 | 1.08 |
| | 0.75 | 1.03 |
| | 0.78 | 0.99 |
| | 0.80 | 0.97 |
| | 0.85 | 0.91 |
| | 0.90 | 0.86 |
| | 0.95 | 0.82 |
| | 1.00 | 0.77 |
| | 1.15 | 0.67 |
| | 1.27 | 0.61 |
| | 1.64 | 0.47 |
| | 1.74 | 0.45 |
| | 1.85 | 0.42 |
| | 2.00 | 0.39 |
| | 2.23 | 0.35 |
| | 2.52 | 0.31 |
| | 2.69 | 0.29 |
| | 2.86 | 0.27 |
| | 3.00 | 0.26 |

**Project No.:** **1413-15-114**
**Date: December 11, 2015**
**Volvo I-26 Interchange**

S&ME

**Site-Specific Design Response Spectra**
**SEE Design Event**
**Response Spectra at Top of Marl**

**Figure 7-4**




| | | | |
|---|---|---|---|
| **Project No.:** 1413-15-114 | S&ME | **Site-Specific Design Response Spectra** | **Figure** |
| **Date: December 11, 2015** | | **SEE and FEE Design Events** | **7-5** |
| **Volvo I-26 Interchange** | | **Response Spectra at Ground Surface and Top of Cooper Marl** | |

# 8.0 References

Groholski, D., Hashash, Y.M.A., Kim, Byungmin, Musgrove, M., Harmon, J., and Stewart, J.P., (2015) "Simplified Model for Small-Strain Nonlinearity and Strength in 1-D Seismic Site Response Analysis," Journal of Geotechnical and Geoenvironmental Engineering, Submitted for review.

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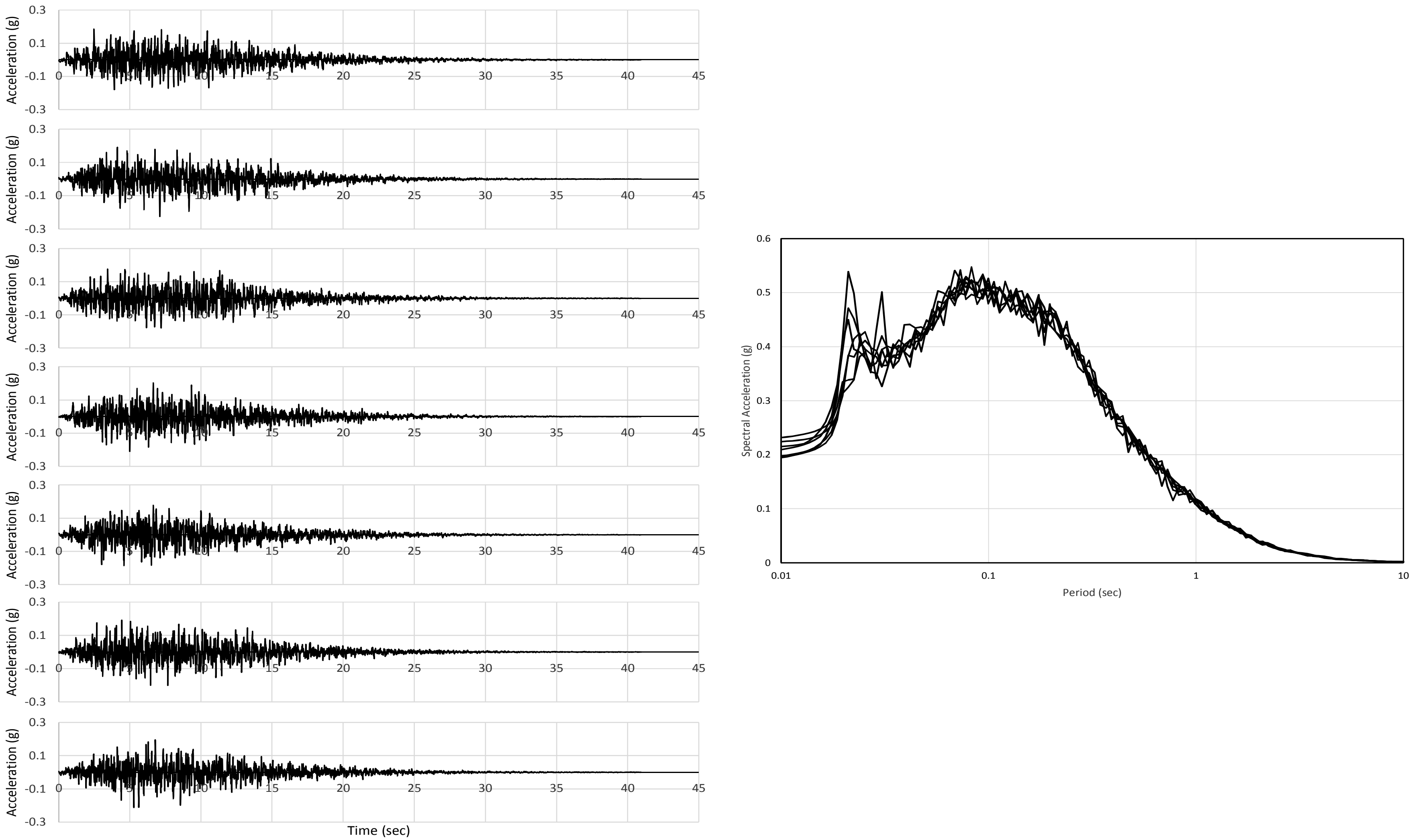
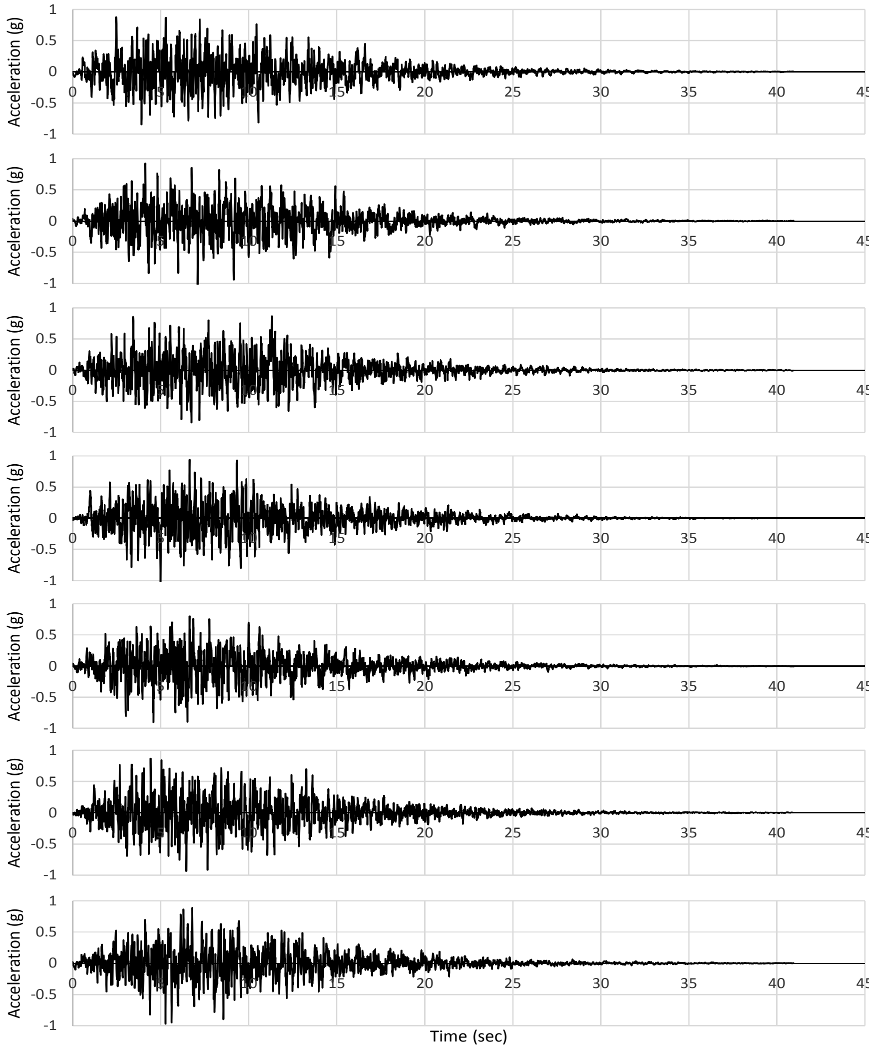
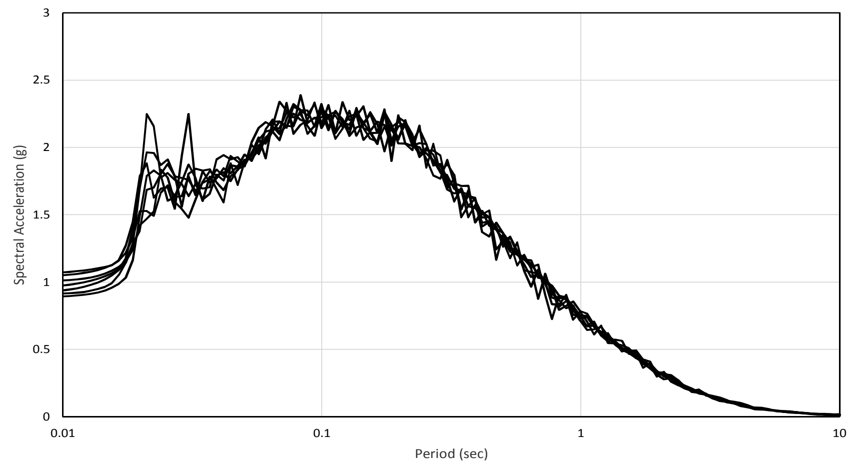
# Appendix I – Ground Motions

FEE and SEE Scaled Time Histories and Corresponding Response Spectra




| **Project No. 1413-15-114** | S&ME | **FEE Motions from SCENARIO_PC**<br>I-26 Volvo Interchange<br>Berkeley County, South Carolina | **Figure I-1** |
|---|---|---|---|
| **Date: December 4, 2015** | | | |



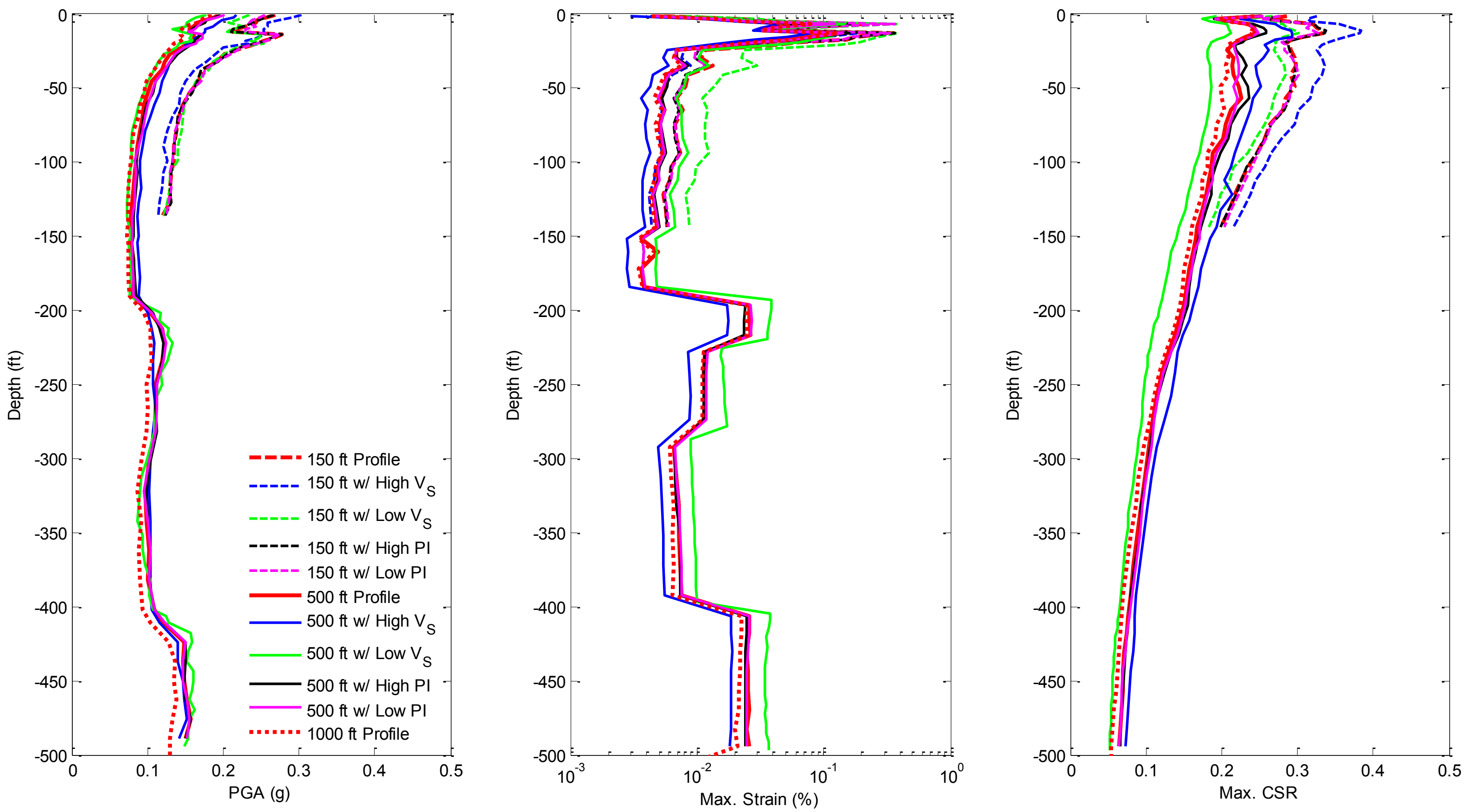




| | | | |
|---|---|---|---|
| **Project No. 1413-15-114**<br>**Date: December 4, 2015** | S&ME | **SEE Motions from SCENARIO_PC**<br>I-26 Volvo Interchange<br>Berkeley County, South Carolina | **Figure I-2** |

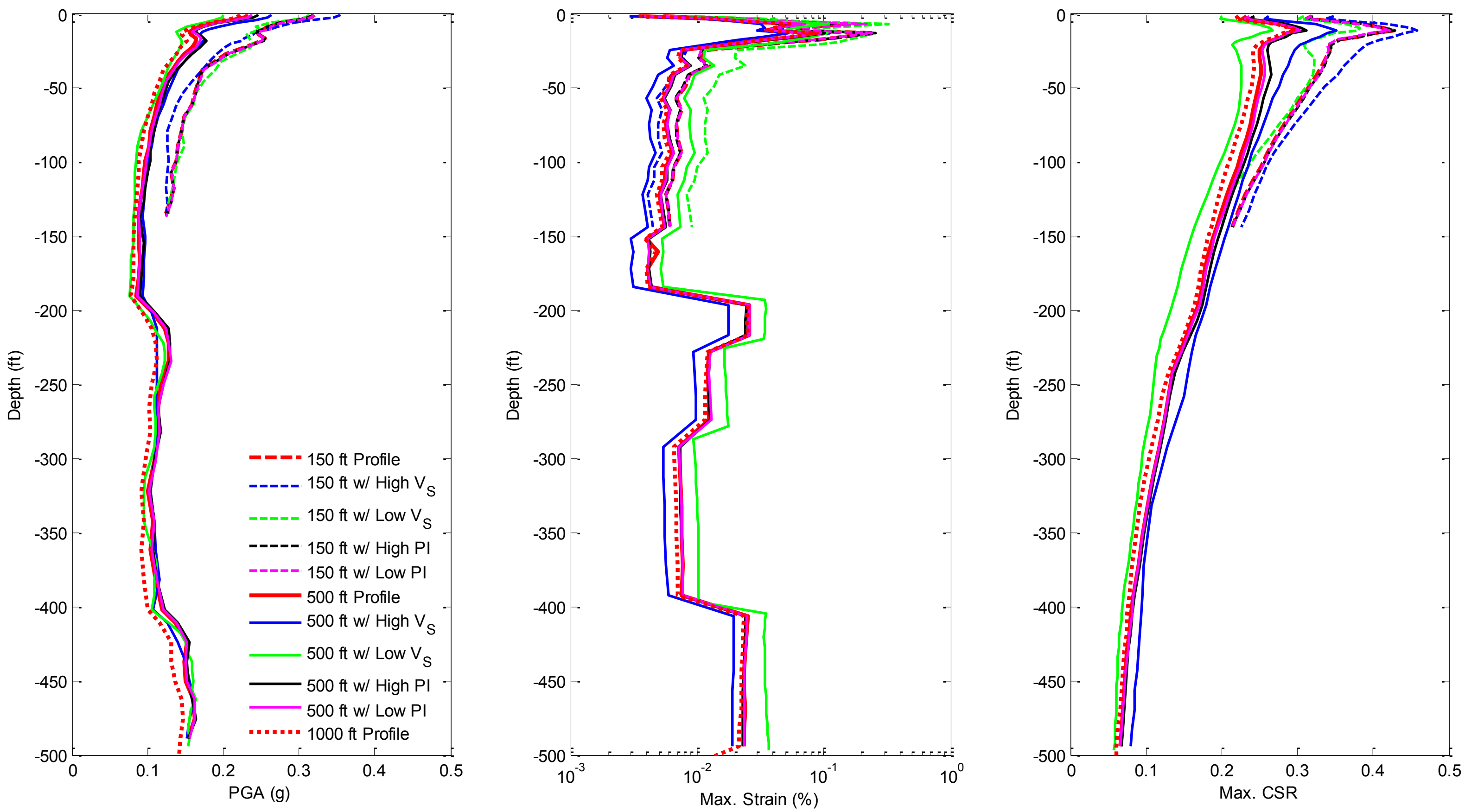
# Appendix II– Total Stress Site Response Analysis Outputs

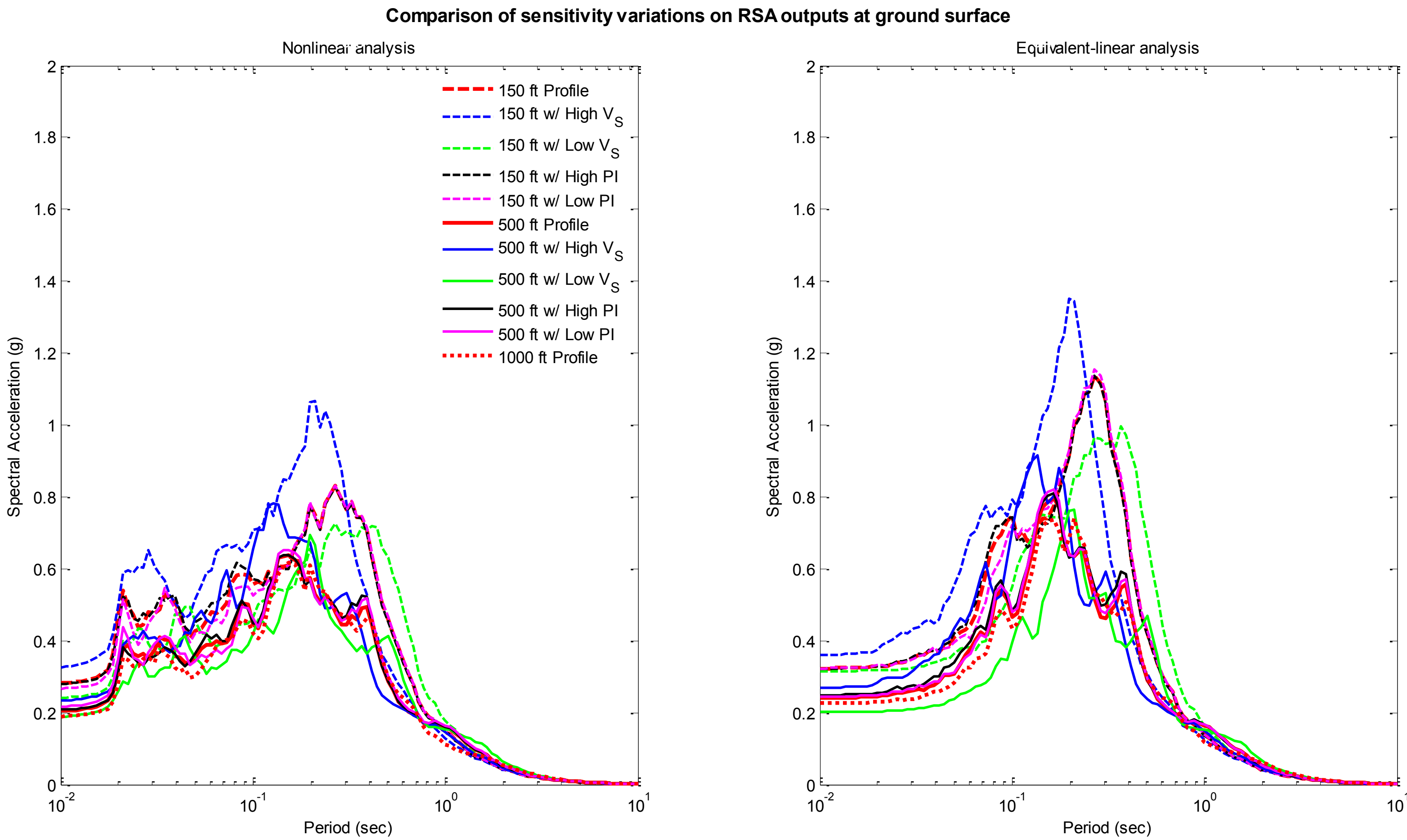
# SUMMARY PLOTS: FEE Motions

## Comparison of sensitivity variations on profile outputs based on Nonlinear analysis

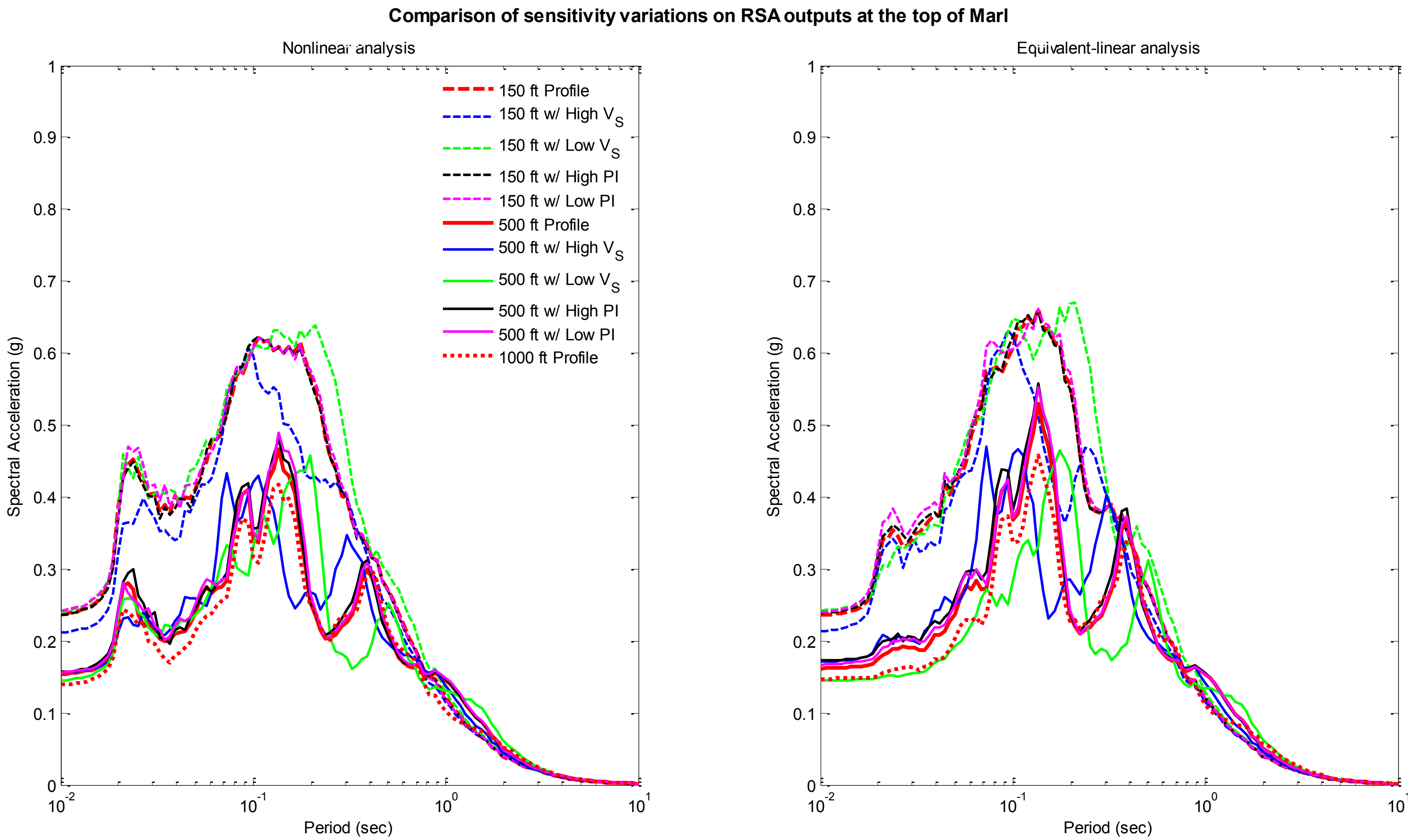


**Comparison of sensitivity variations on profile outputs based on Equivalent-linear analysis**



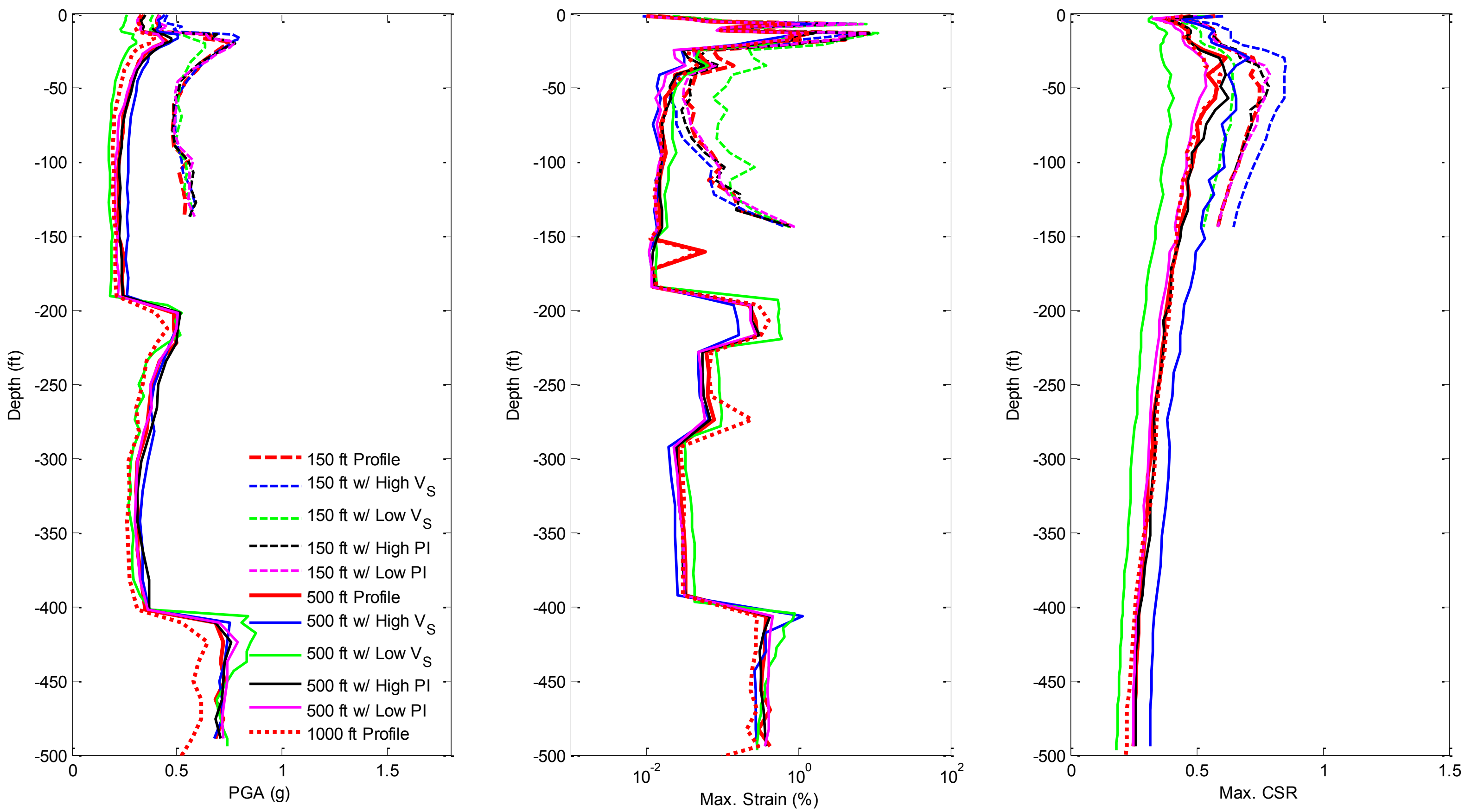


Comparison of sensitivity variations on RSA outputs at ground surface
Nonlinear analysis
Equivalent-linear analysis
150 ft Profile
150 ft w/ High V_S
150 ft w/ Low V_S
150 ft w/ High PI
150 ft w/ Low PI
500 ft Profile
500 ft w/ High V_S
500 ft w/ Low V_S
500 ft w/ High PI
500 ft w/ Low PI
1000 ft Profile
Spectral Acceleration (g)
Period (sec)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
10^-2
10^-1
10^0
10^1

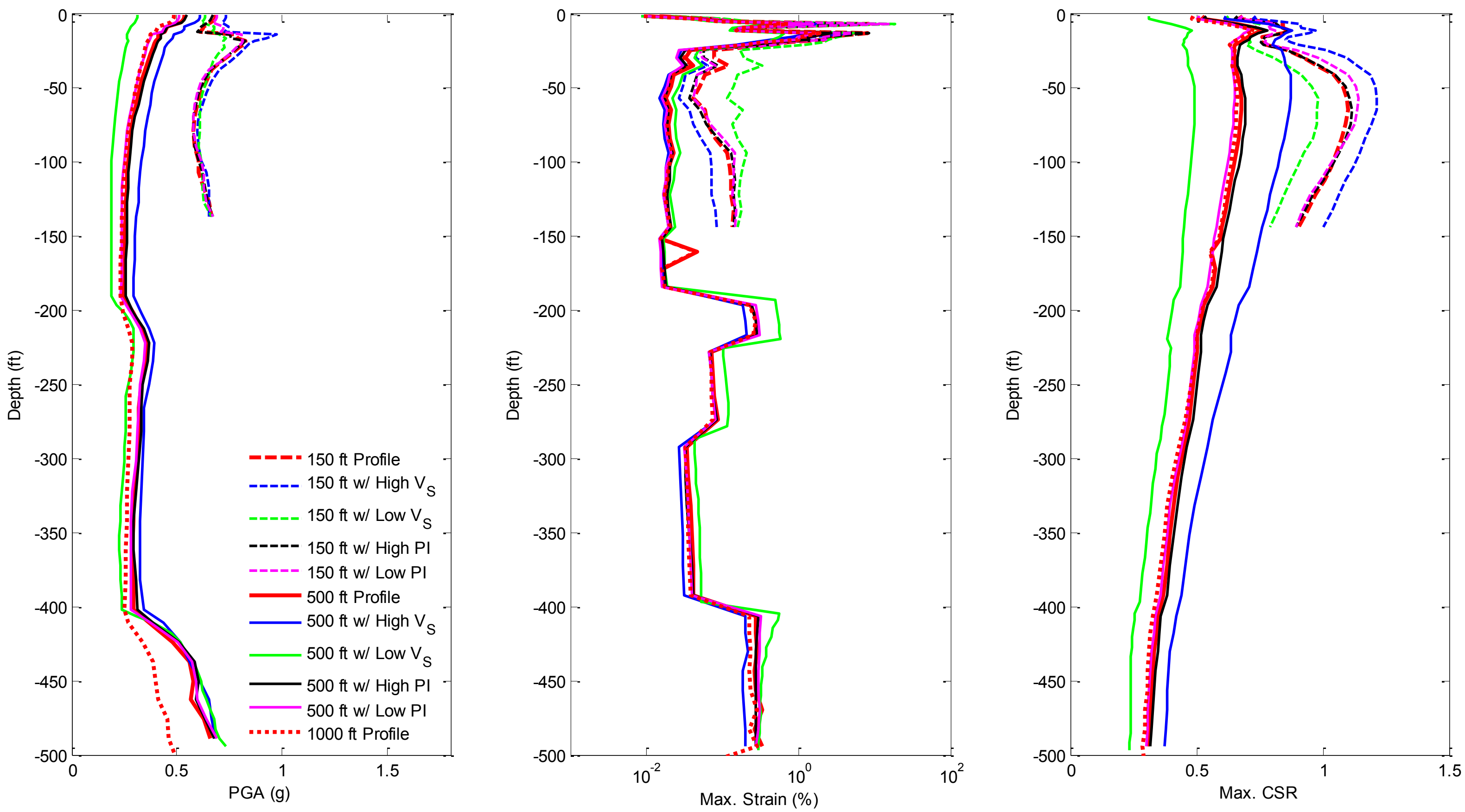

Comparison of sensitivity variations on RSA outputs at the top of Marl
Nonlinear analysis
Equivalent-linear analysis
150 ft Profile
150 ft w/ High $V_S$
150 ft w/ Low $V_S$
150 ft w/ High PI
150 ft w/ Low PI
500 ft Profile
500 ft w/ High $V_S$
500 ft w/ Low $V_S$
500 ft w/ High PI
500 ft w/ Low PI
1000 ft Profile
Spectral Acceleration (g)
Period (sec)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
$10^{-2}$
$10^{-1}$
$10^{0}$
$10^{1}$

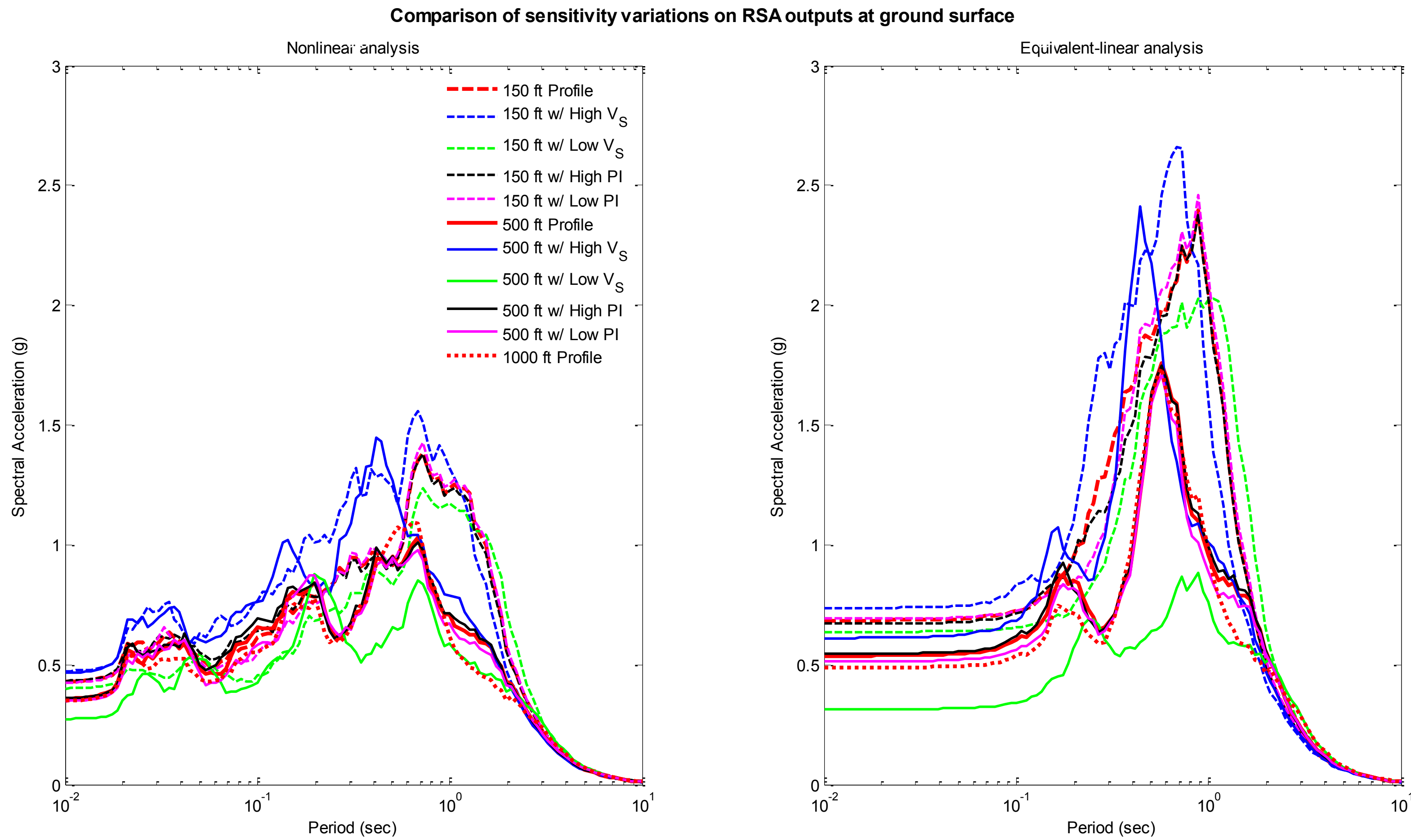
# SUMMARY PLOTS: SEE Motions

Comparison of sensitivity variations on profile outputs based on Nonlinear analysis

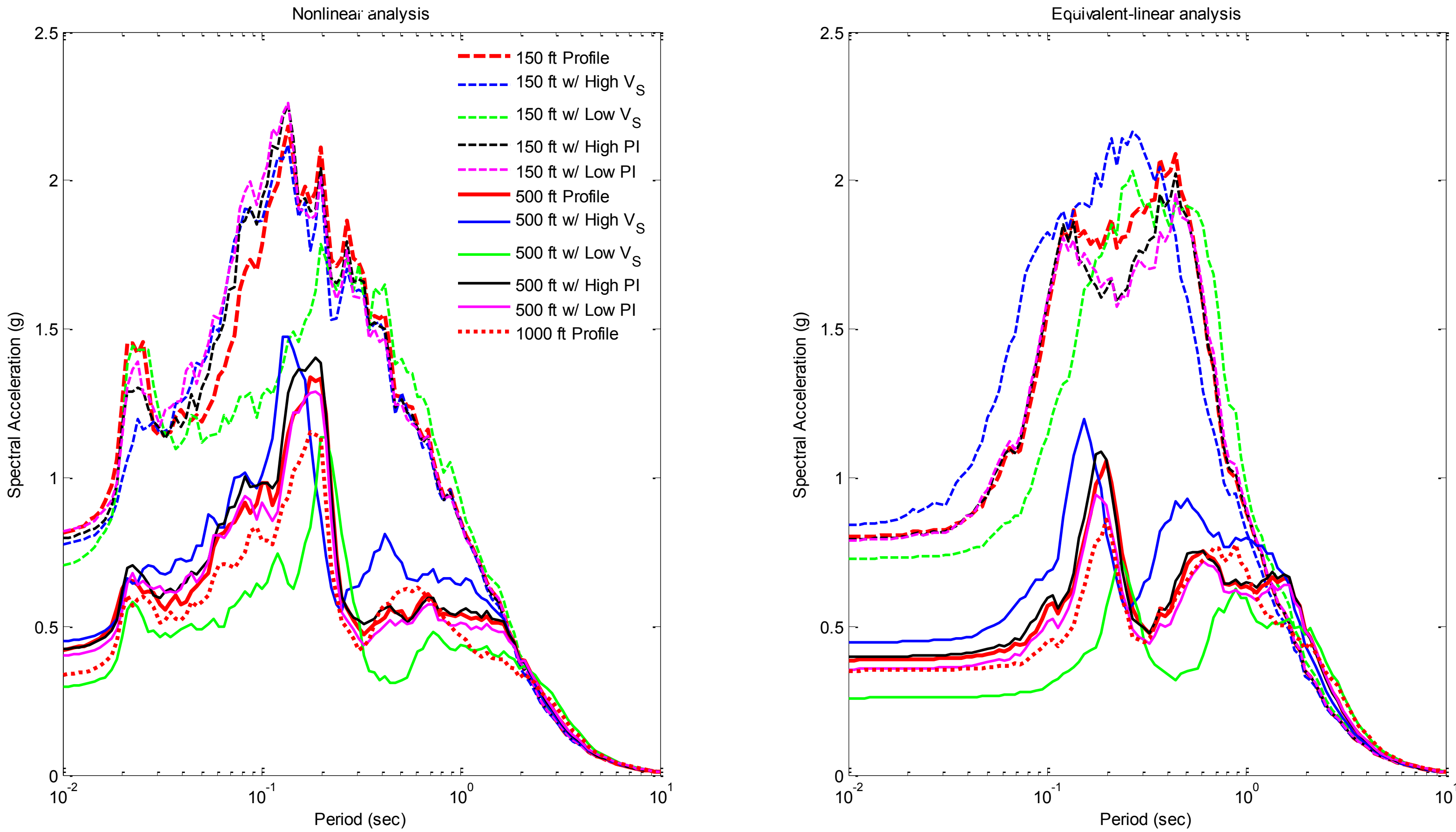


Comparison of sensitivity variations on profile outputs based on Equivalent-linear analysis



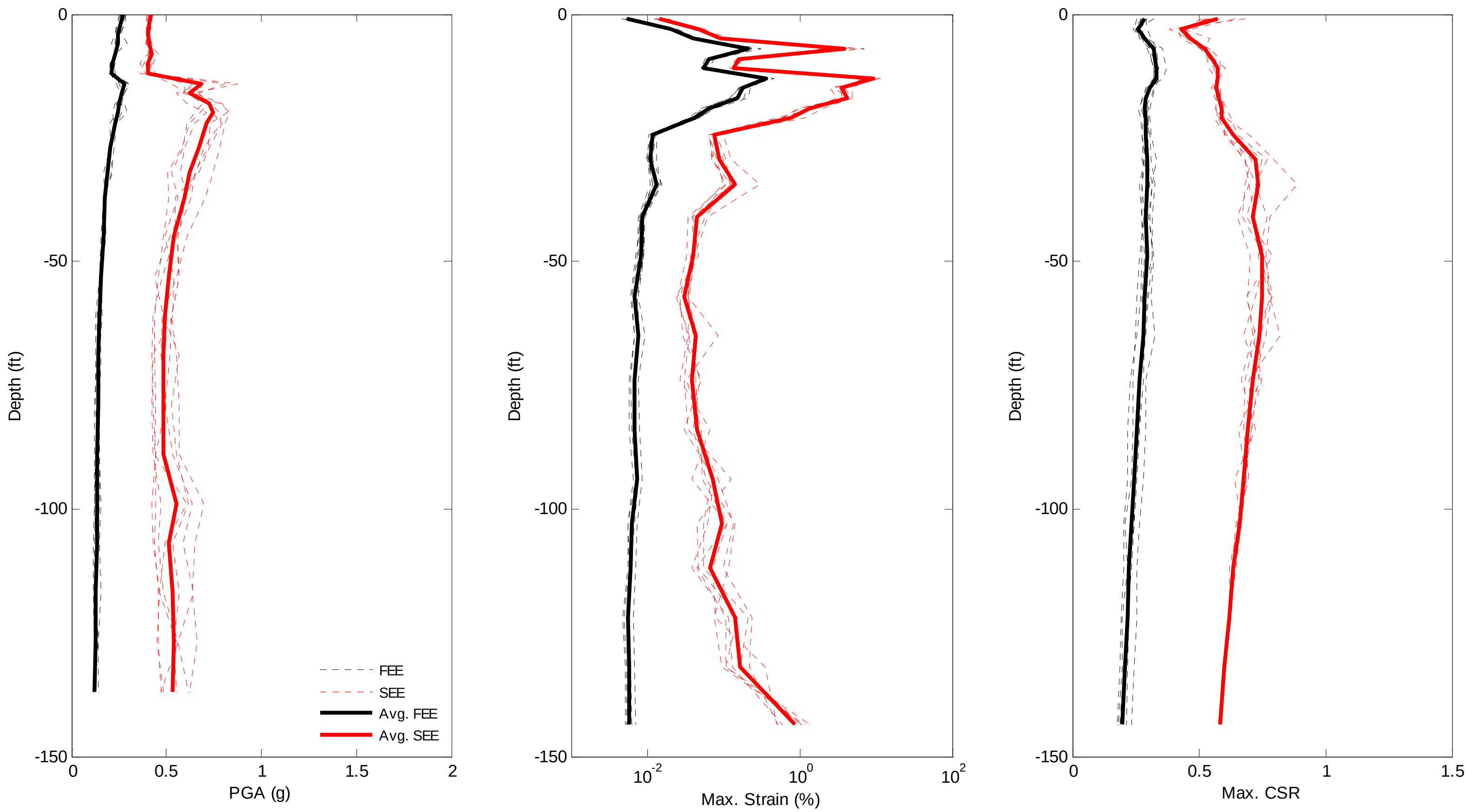

Comparison of sensitivity variations on RSA outputs at ground surface
Nonlinear analysis
Equivalent-linear analysis
Spectral Acceleration (g)
Period (sec)
150 ft Profile
150 ft w/ High V_S
150 ft w/ Low V_S
150 ft w/ High PI
150 ft w/ Low PI
500 ft Profile
500 ft w/ High V_S
500 ft w/ Low V_S
500 ft w/ High PI
500 ft w/ Low PI
1000 ft Profile

**Comparison of sensitivity variations on RSA outputs at the top of Marl**

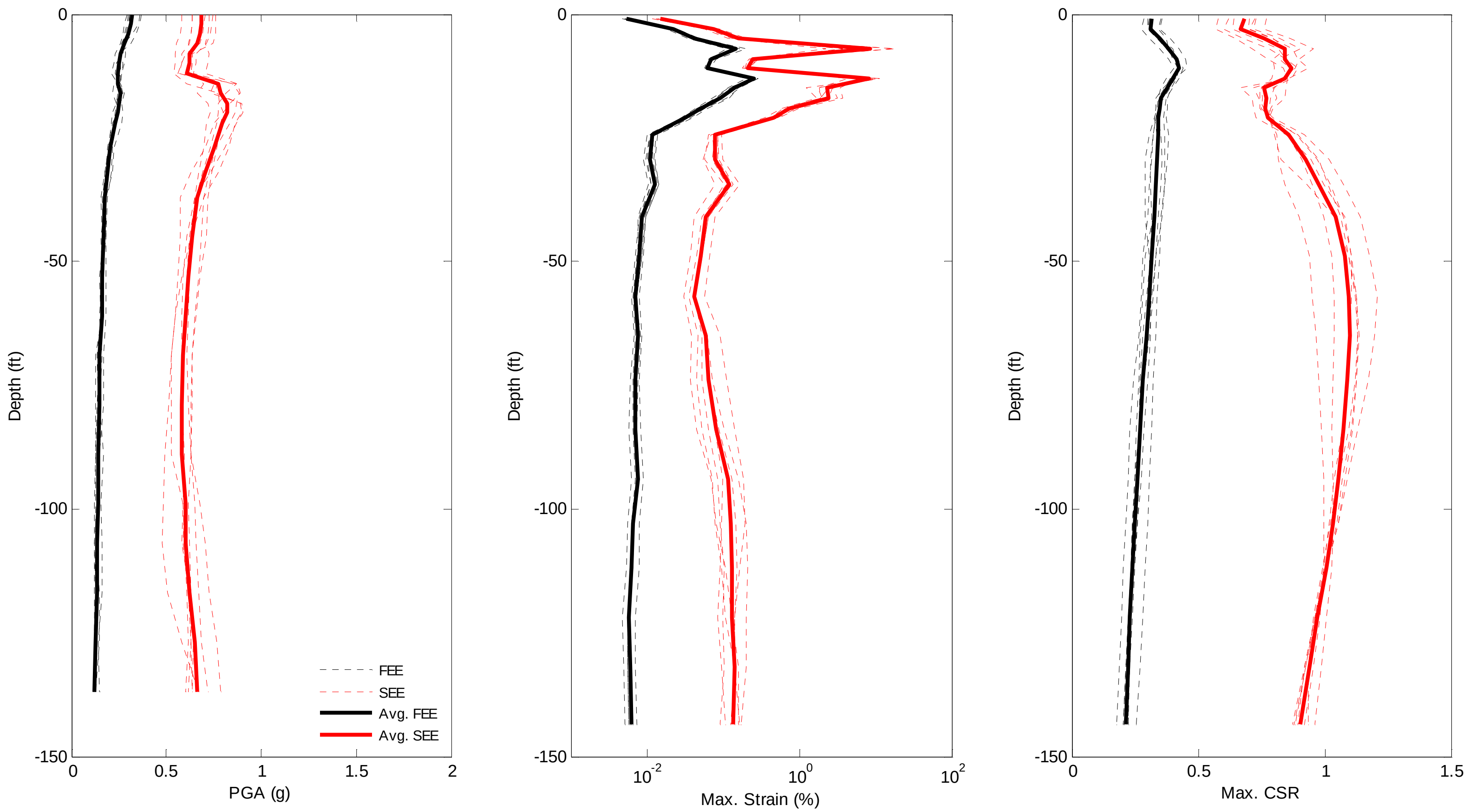


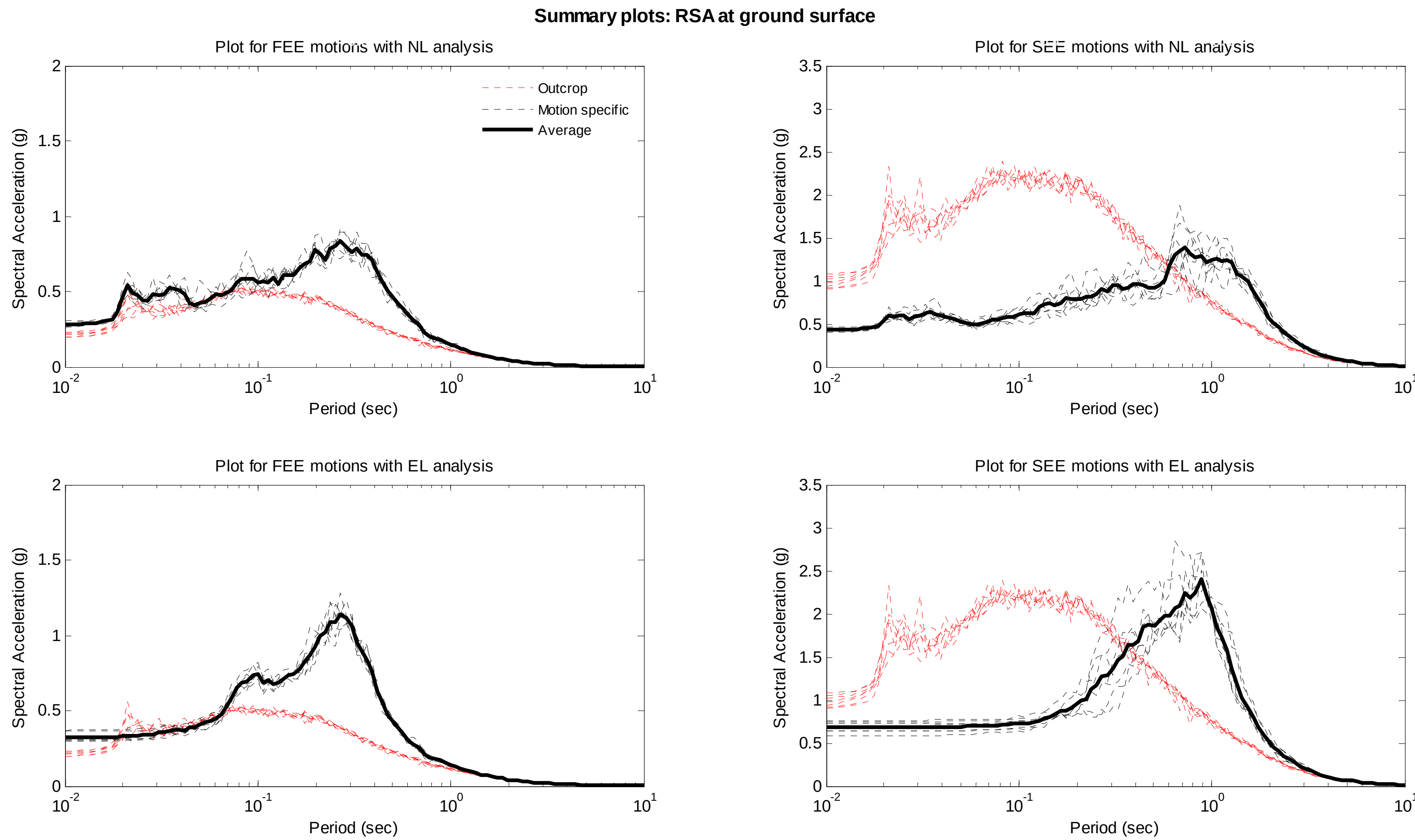
# ANALYSIS OUTPUTS:
# 150 Ft Deep Profile

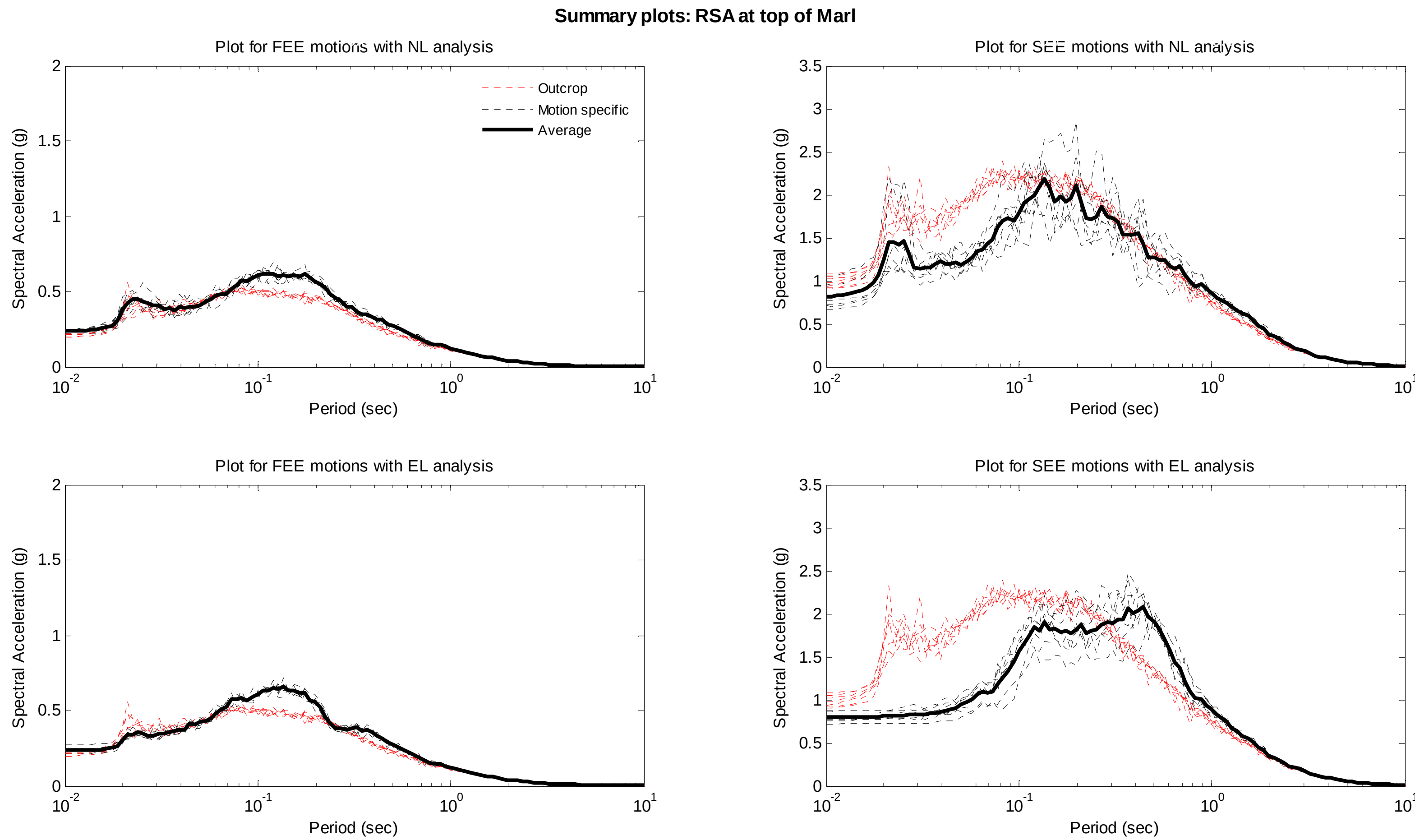
**Summary of the profile outputs for Nonlinear analysis**



**Summary of the profile outputs for Equivalent-linear analysis**

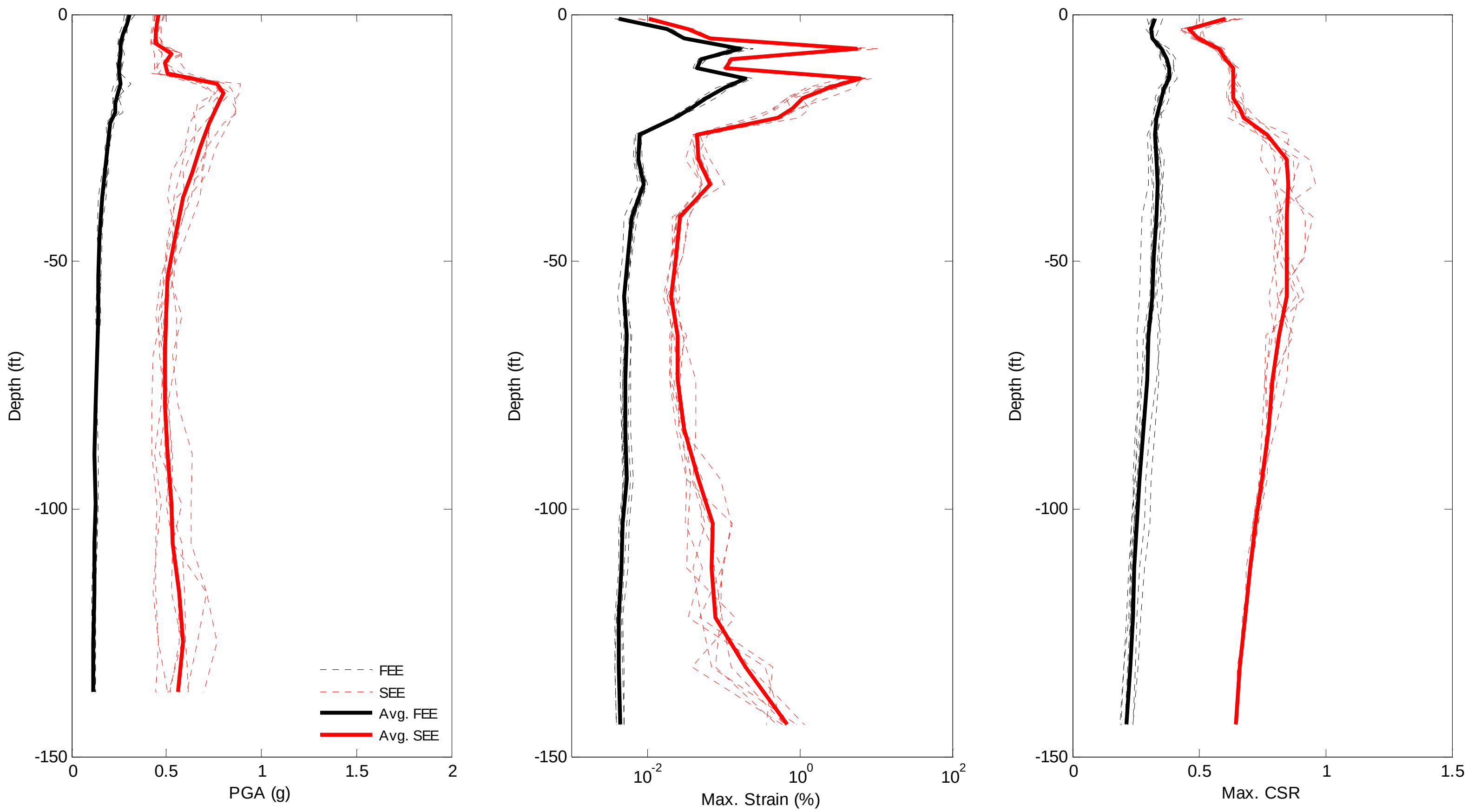



Summary plots: RSA at ground surface
Plot for FEE motions with NL analysis
Plot for SEE motions with NL analysis
Plot for FEE motions with EL analysis
Plot for SEE motions with EL analysis
Outcrop
Motion specific
Average
Spectral Acceleration (g)
Period (sec)
$10^{-2}$
$10^{-1}$
$10^{0}$
$10^{1}$

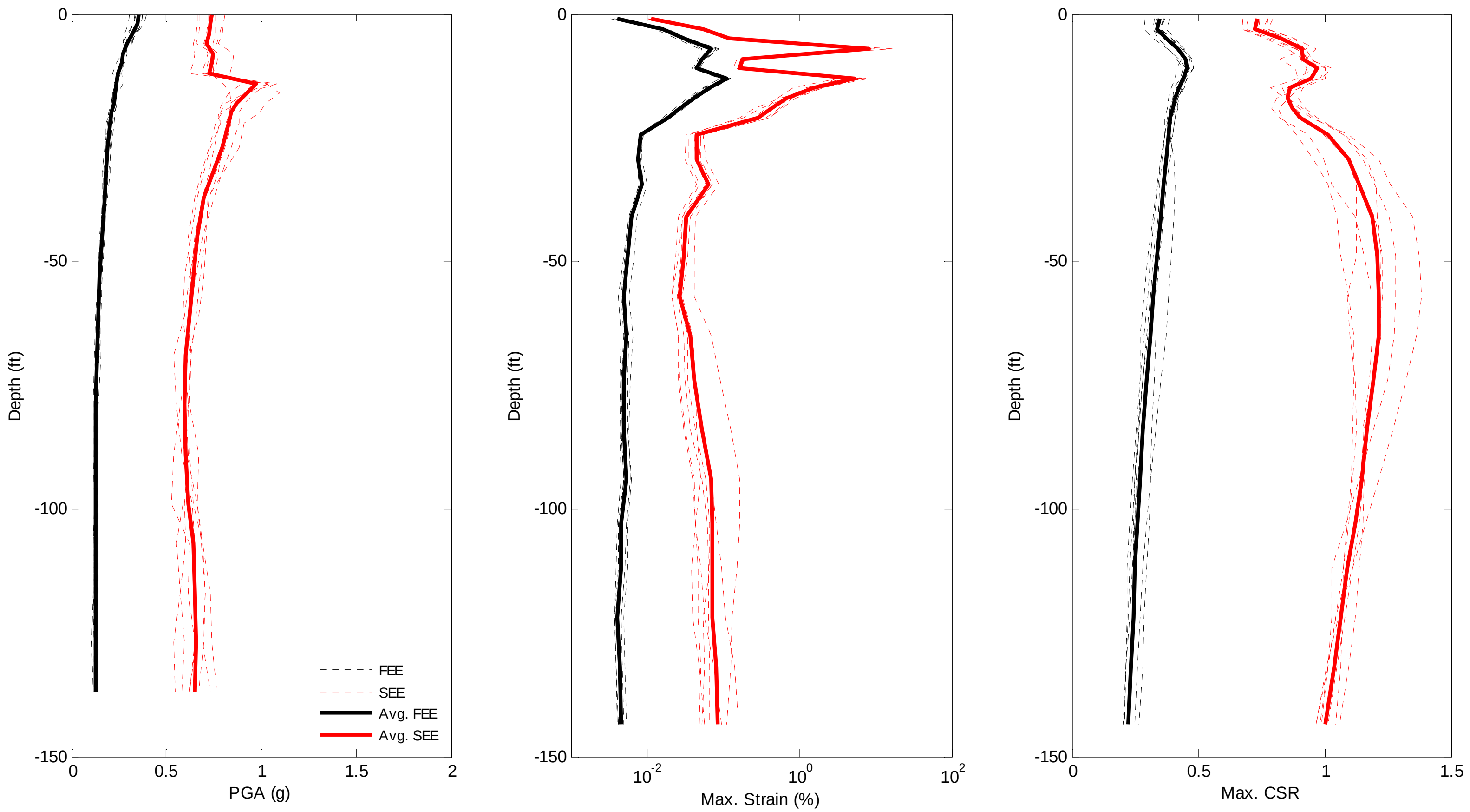

Summary plots: RSA at top of Marl
Plot for FEE motions with NL analysis
Plot for SEE motions with NL analysis
Plot for FEE motions with EL analysis
Plot for SEE motions with EL analysis
Outcrop
Motion specific
Average
Spectral Acceleration (g)
Period (sec)
0
0.5
1
1.5
2
2.5
3
3.5
$10^{-2}$
$10^{-1}$
$10^{0}$
$10^{1}$

# ANALYSIS OUTPUTS:
# 150 Ft Deep Profile w/ High $V_S$ and SS

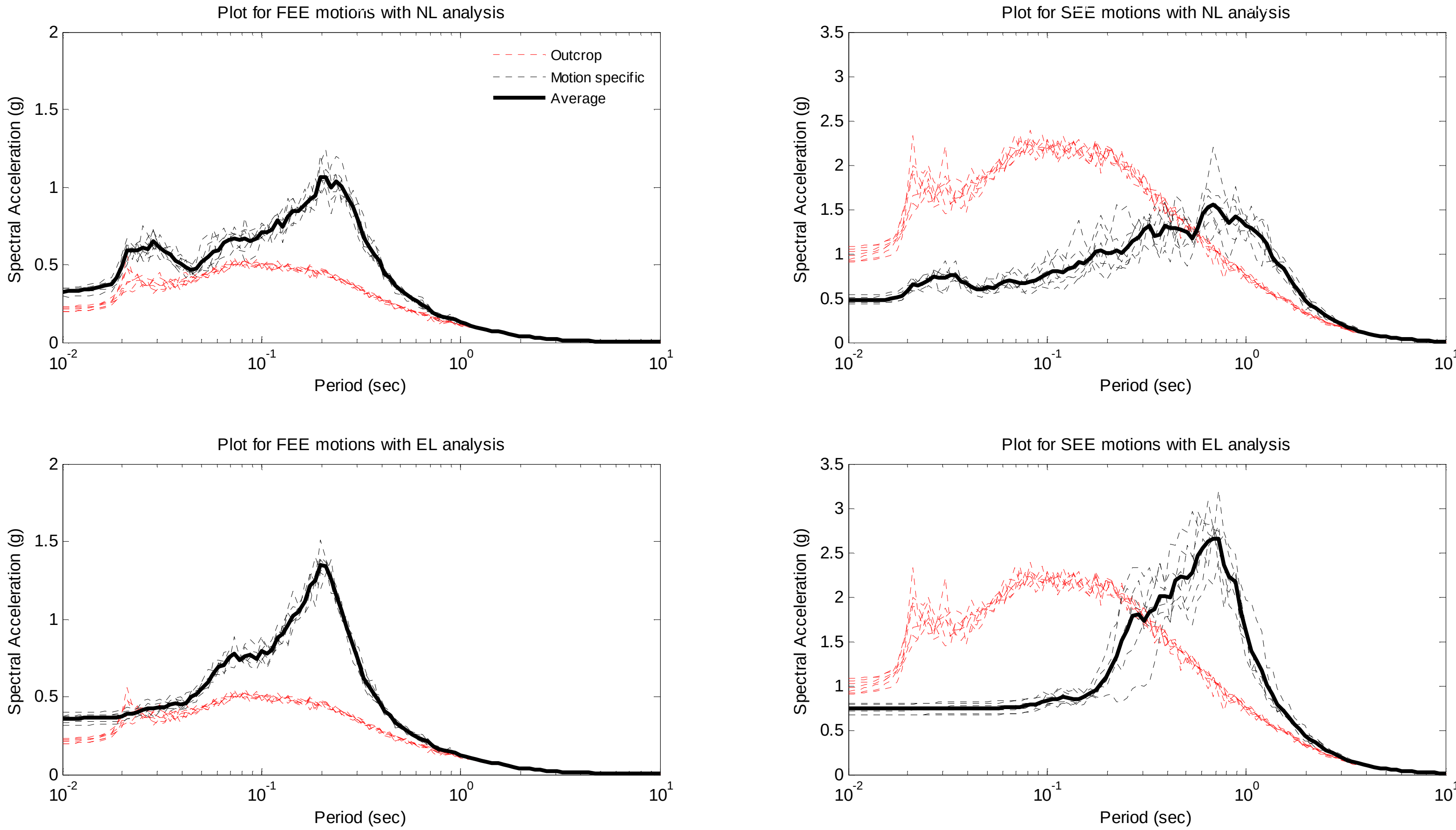
**Summary of the profile outputs for Nonlinear analysis**



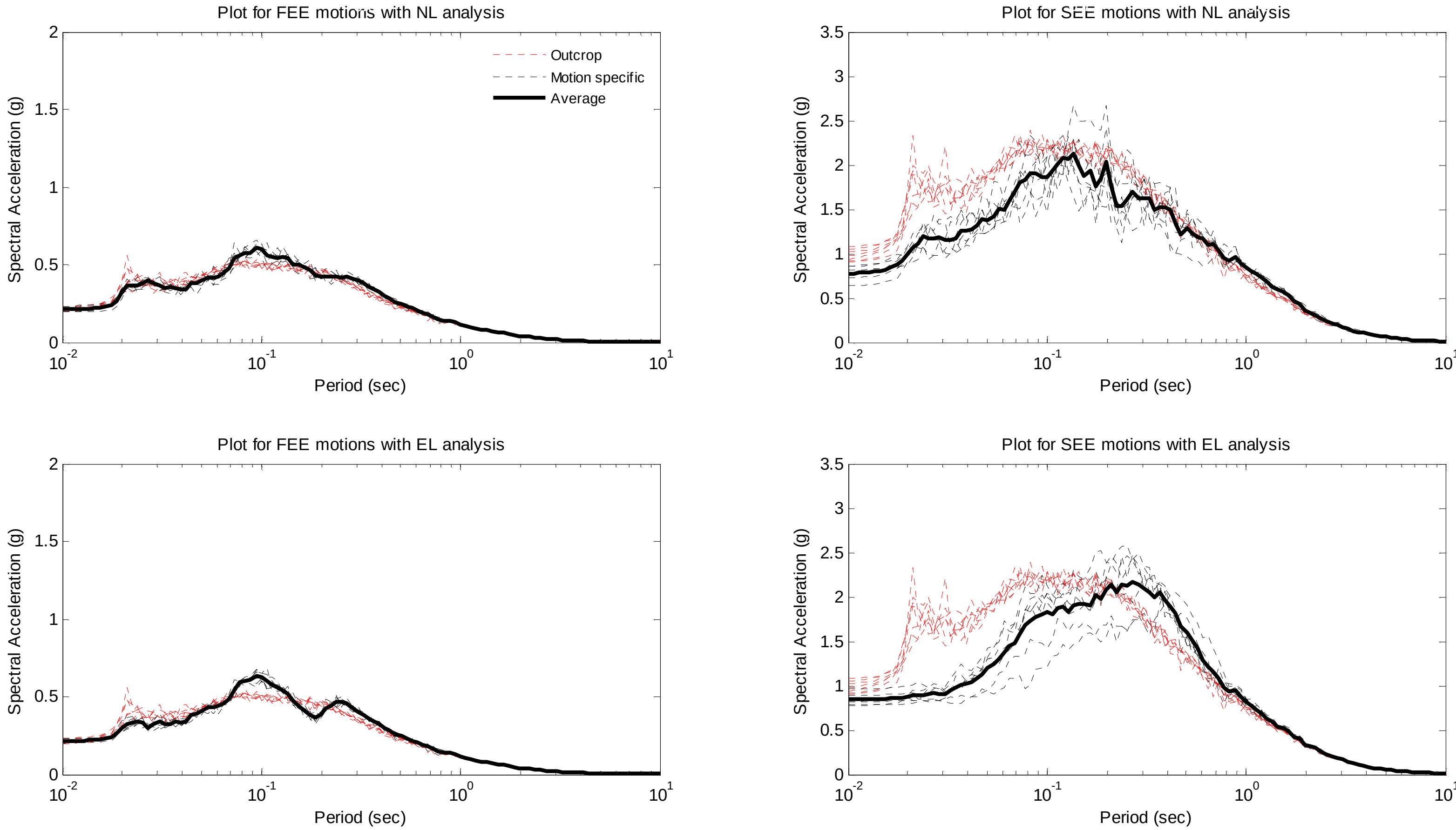
**Summary of the profile outputs for Equivalent-linear analysis**



**Summary plots: RSA at ground surface**

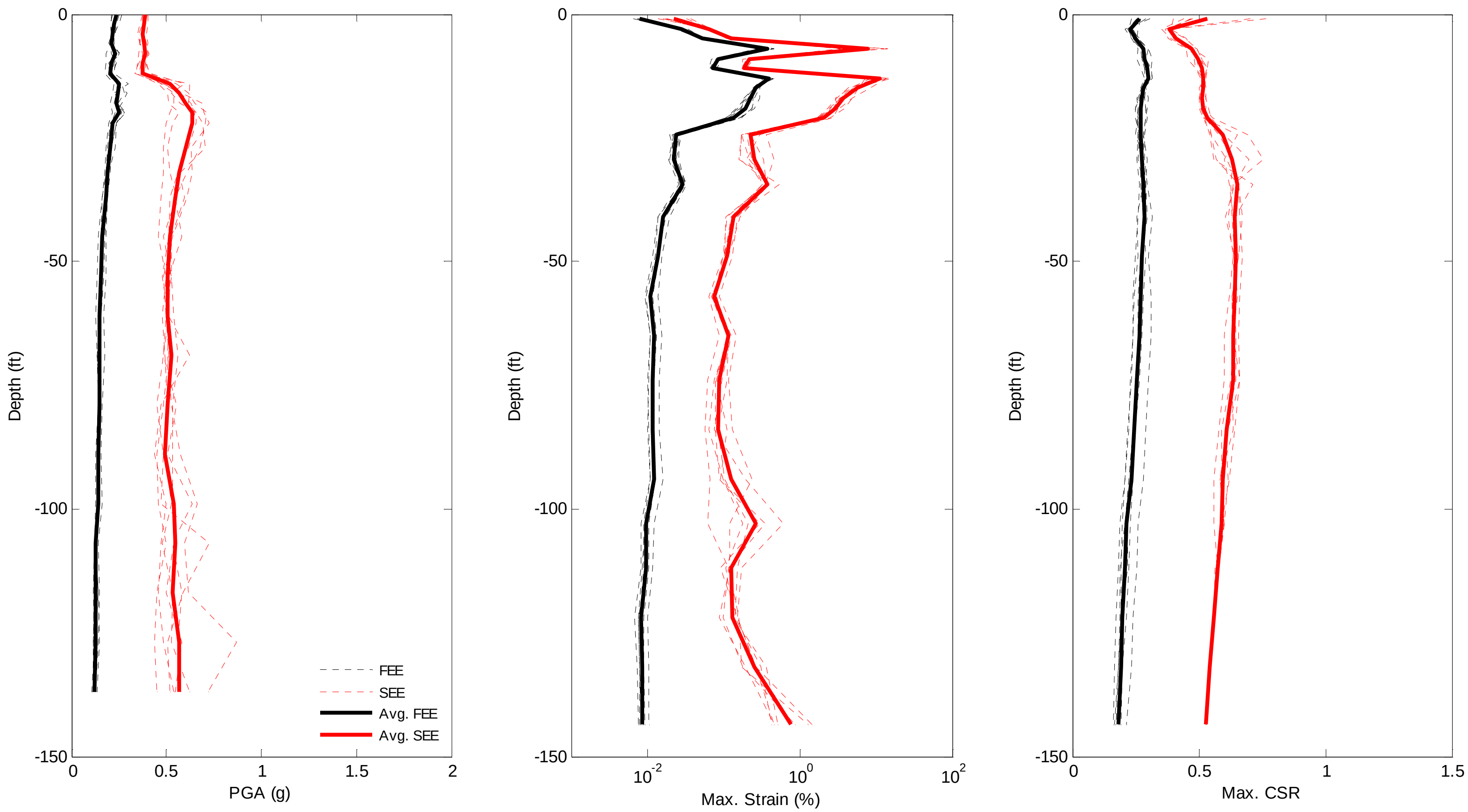


**Summary plots: RSA at top of Marl**

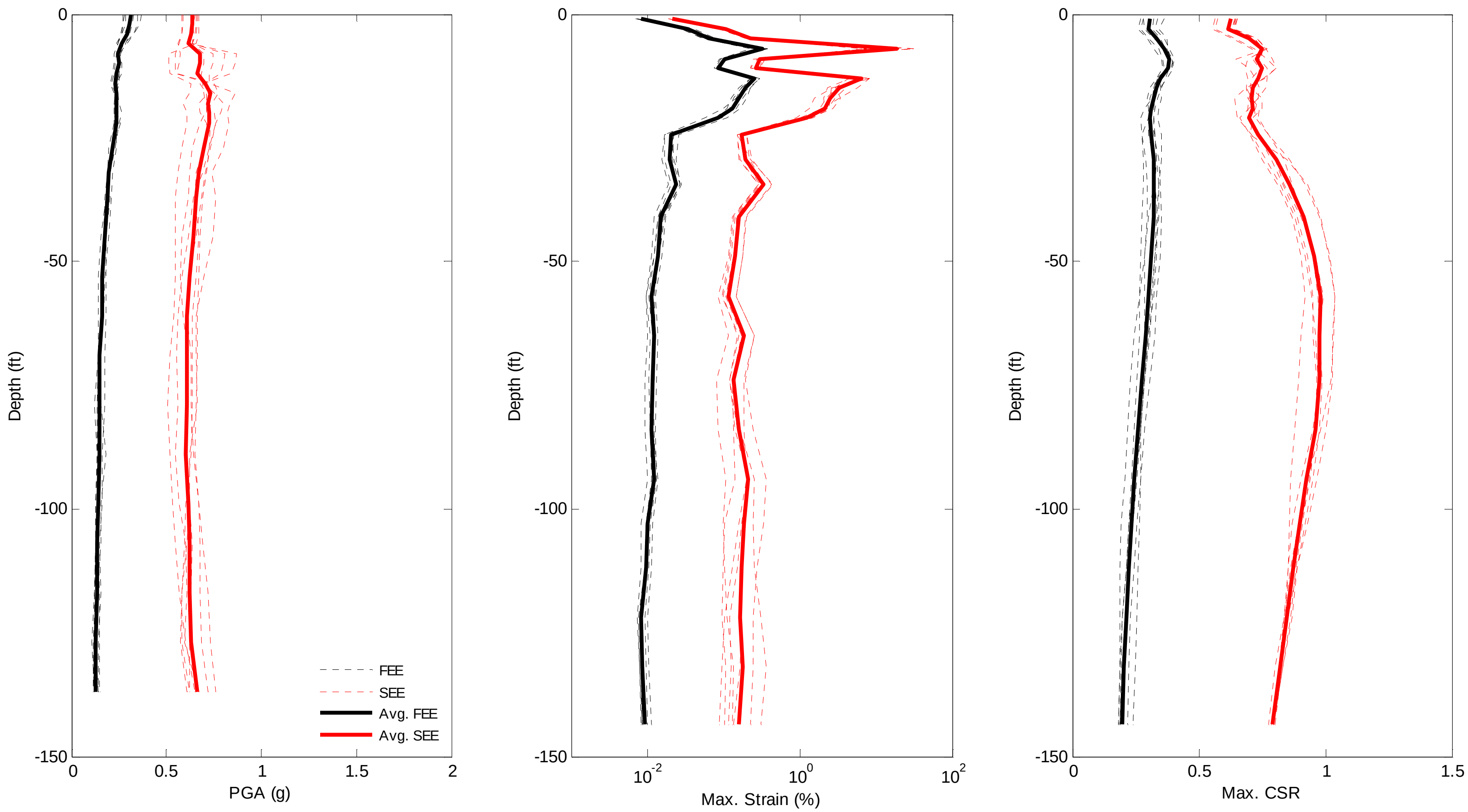


# ANALYSIS OUTPUTS:
# 150 Ft Deep Profile w/ Low $V_S$ and SS

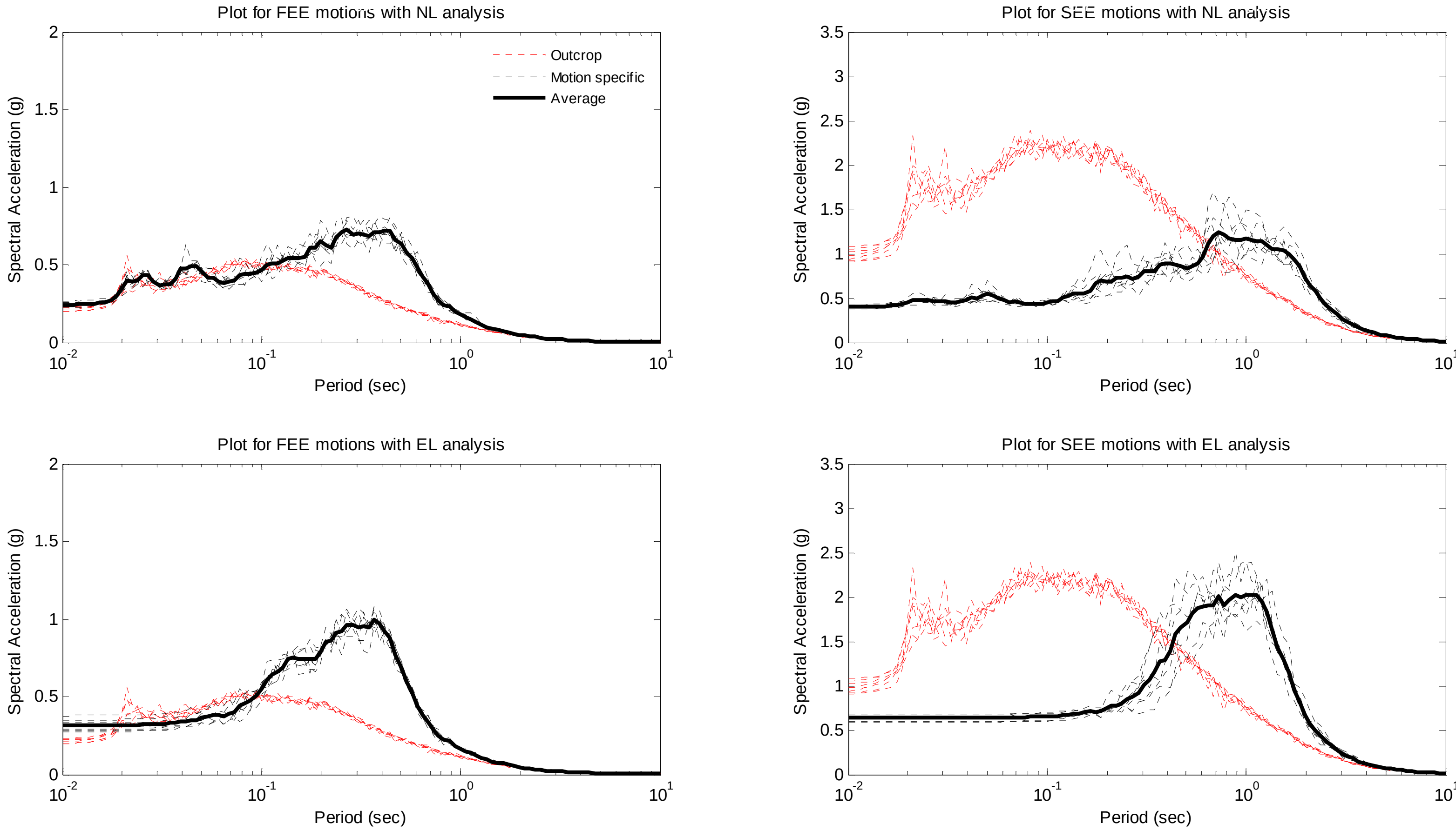
**Summary of the profile outputs for Nonlinear analysis**

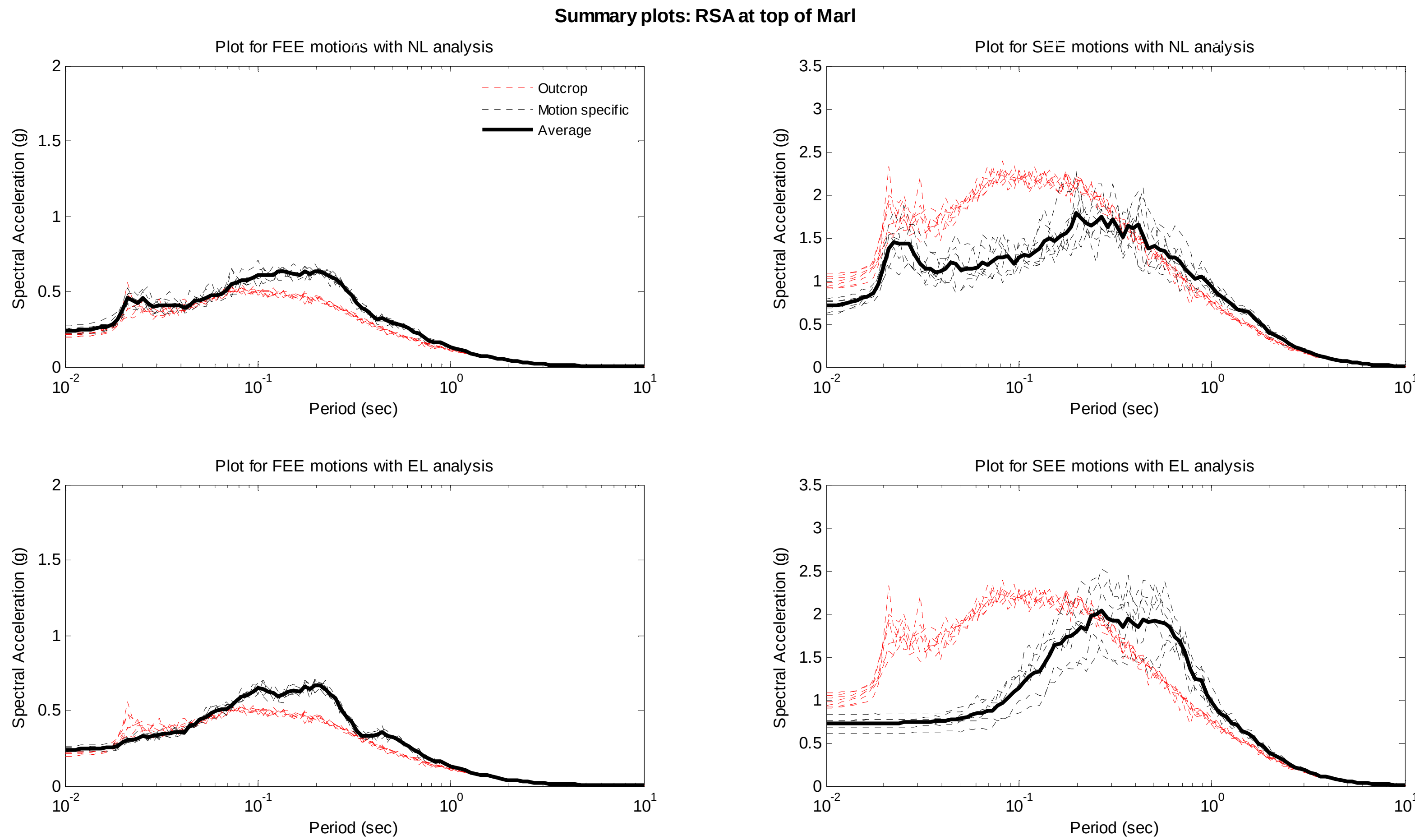


**Summary of the profile outputs for Equivalent-linear analysis**

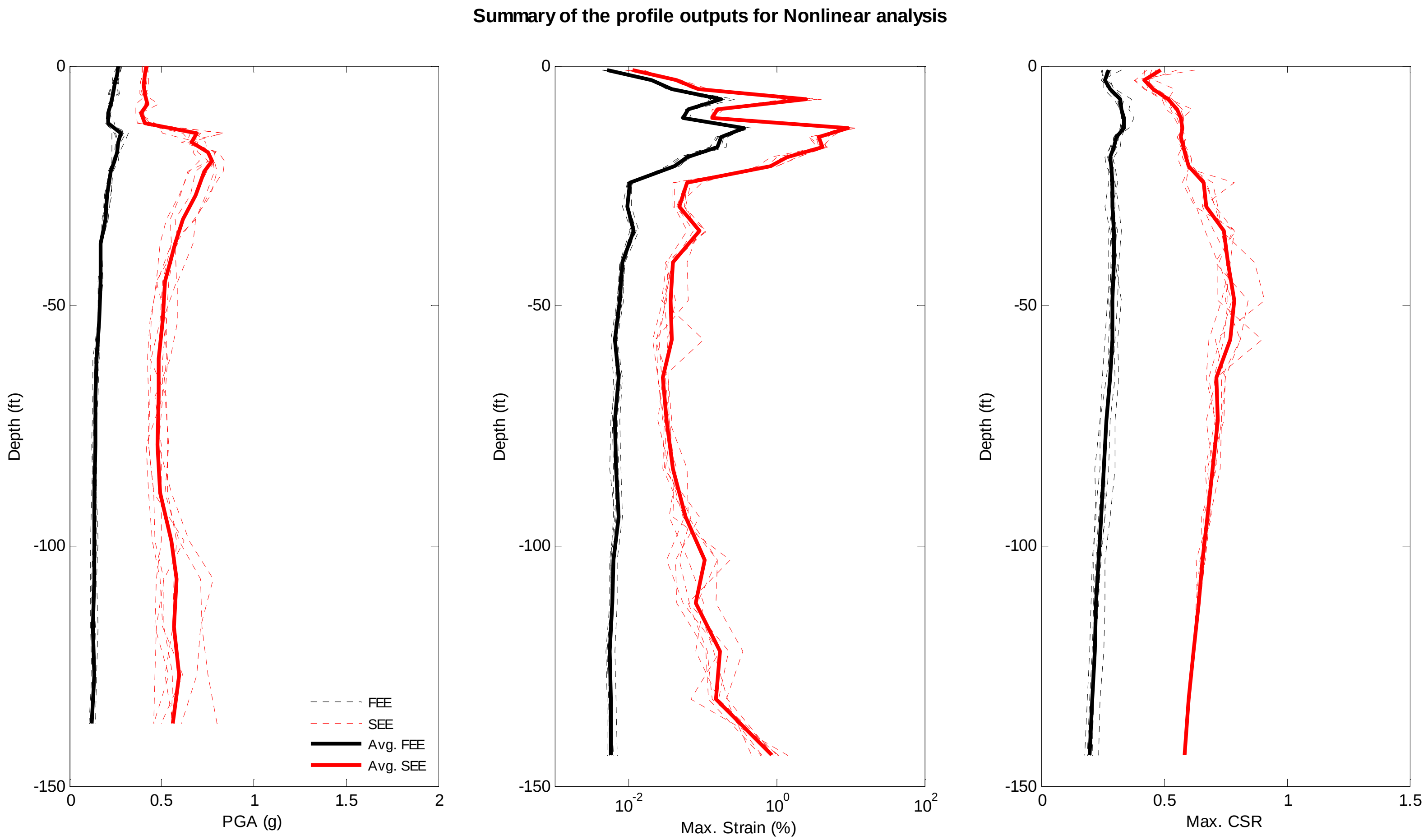


**Summary plots: RSA at ground surface**

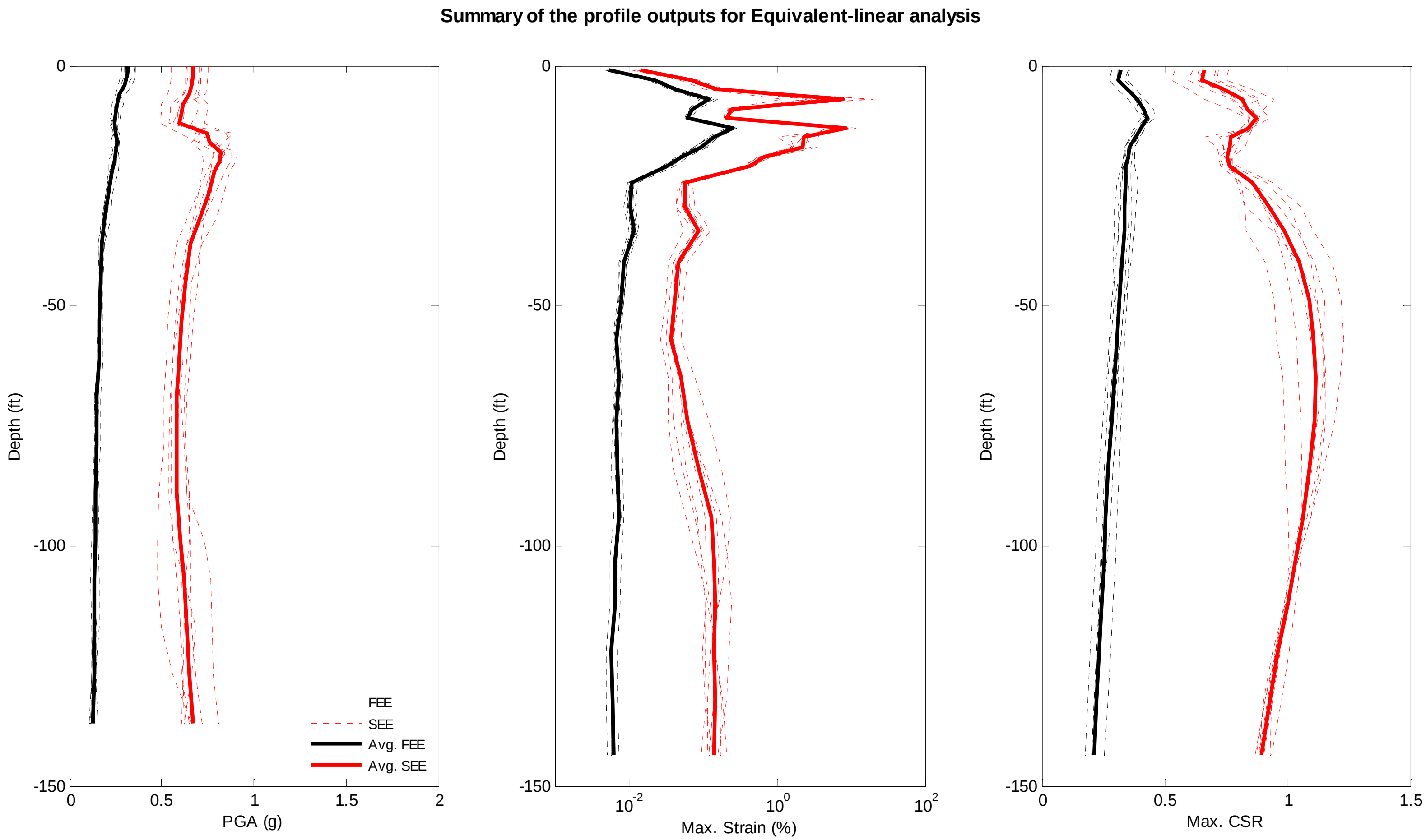



Summary plots: RSA at top of Marl
Plot for FEE motions with NL analysis
Plot for SEE motions with NL analysis
Plot for FEE motions with EL analysis
Plot for SEE motions with EL analysis
Outcrop
Motion specific
Average
Spectral Acceleration (g)
Period (sec)

# ANALYSIS OUTPUTS:
# 150 Ft Deep Profile w/ High PI

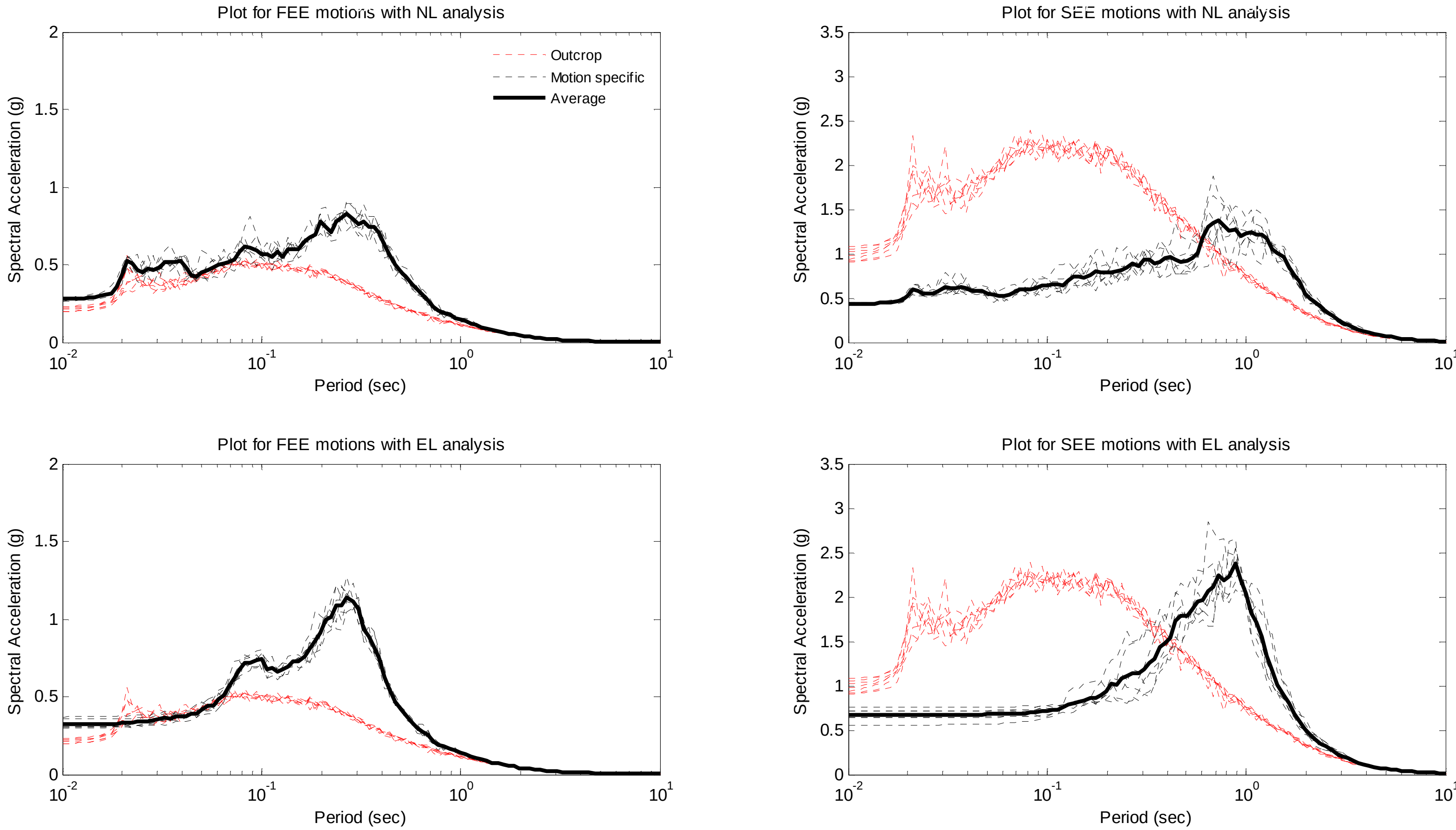


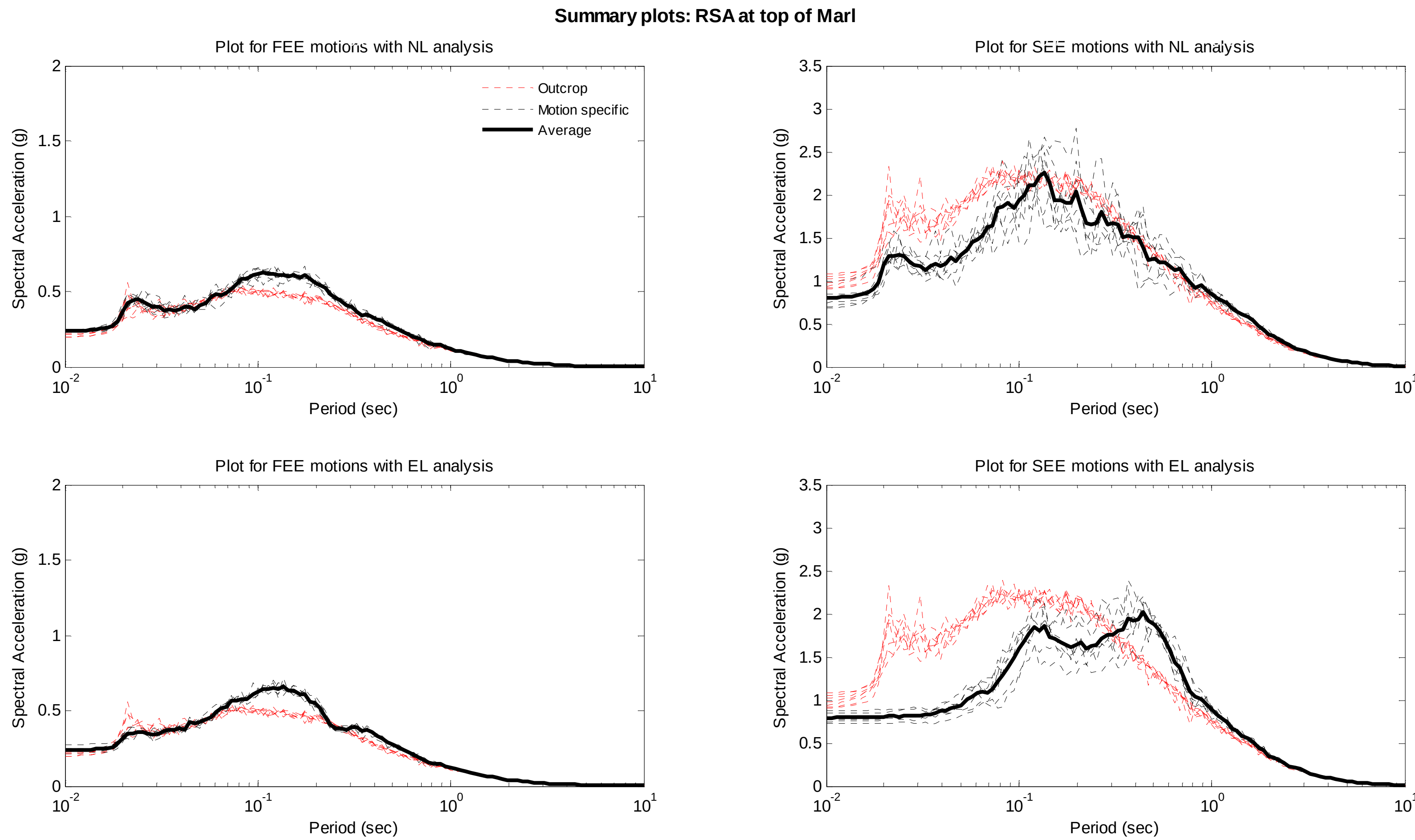
Summary of the profile outputs for Nonlinear analysis
Depth (ft)
PGA (g)
Max. Strain (%)
Max. CSR
FEE
SEE
Avg. FEE
Avg. SEE
0
-50
-100
-150
0
0.5
1
1.5
2
$10^{-2}$
$10^{0}$
$10^{2}$



Summary of the profile outputs for Equivalent-linear analysis
Depth (ft)
PGA (g)
Max. Strain (%)
Max. CSR
FEE
SEE
Avg. FEE
Avg. SEE

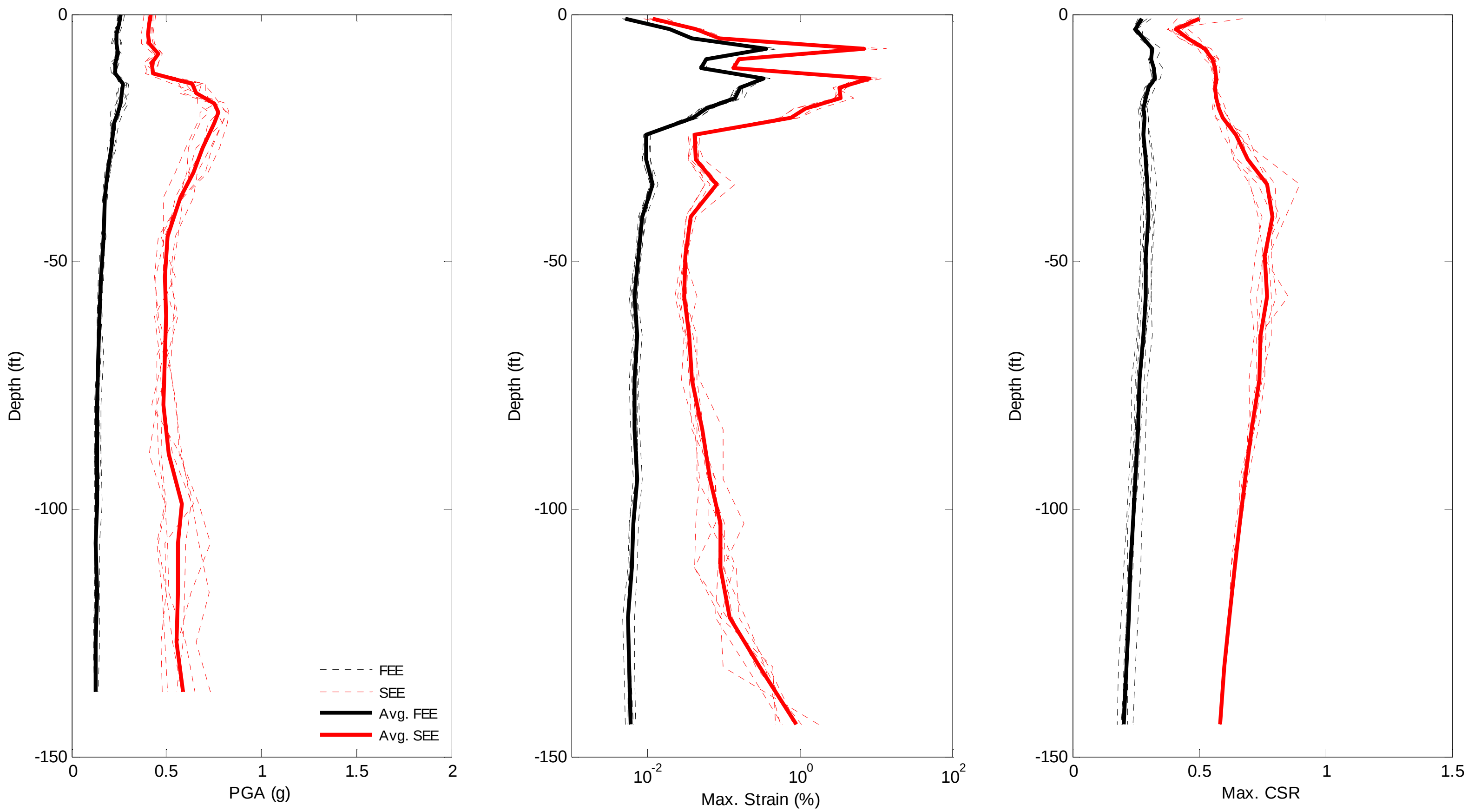
**Summary plots: RSA at ground surface**



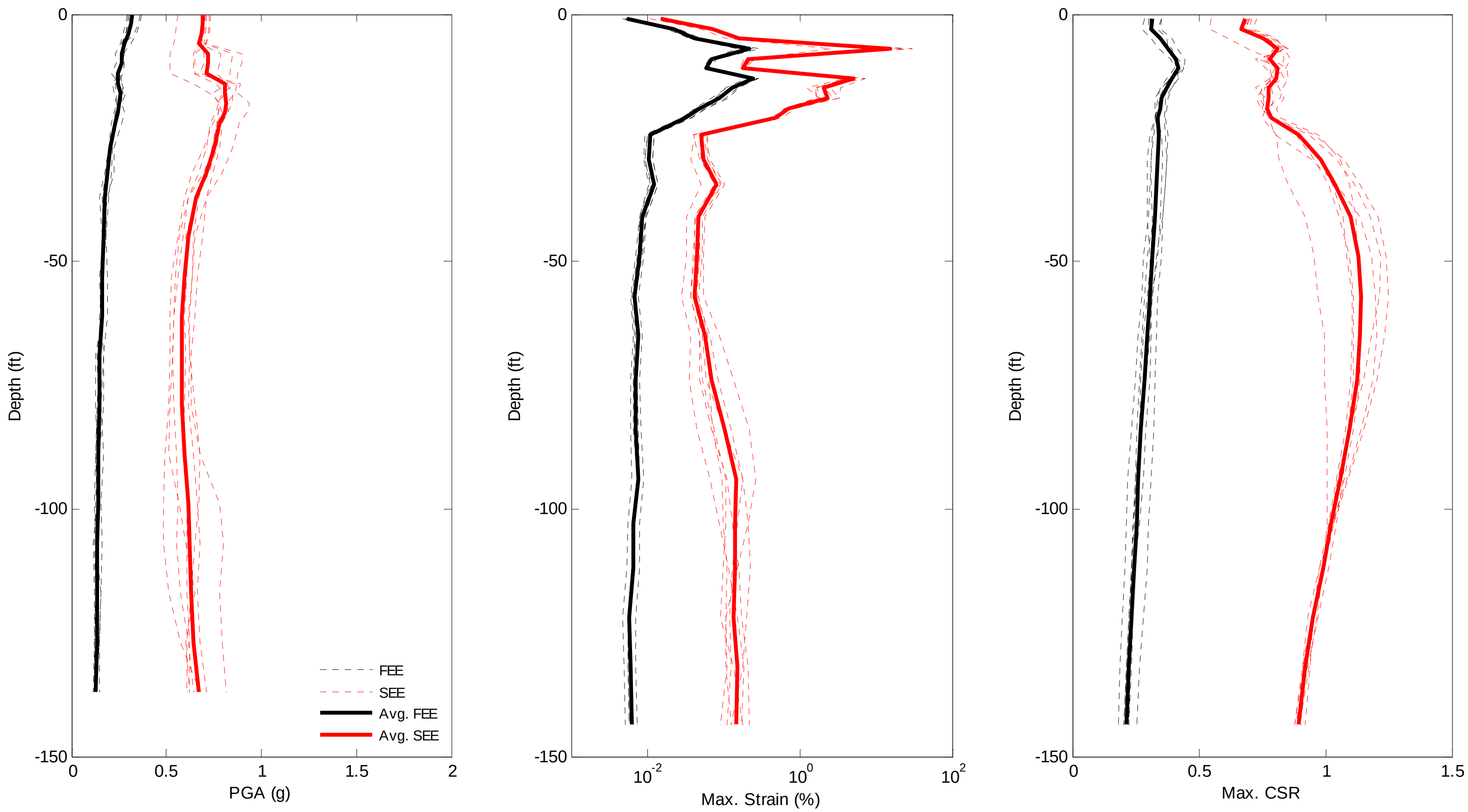

Summary plots: RSA at top of Marl
Plot for FEE motions with NL analysis
Plot for SEE motions with NL analysis
Plot for FEE motions with EL analysis
Plot for SEE motions with EL analysis
Outcrop
Motion specific
Average
Spectral Acceleration (g)
Period (sec)
0
0.5
1
1.5
2
2.5
3
3.5
$10^{-2}$
$10^{-1}$
$10^{0}$
$10^{1}$

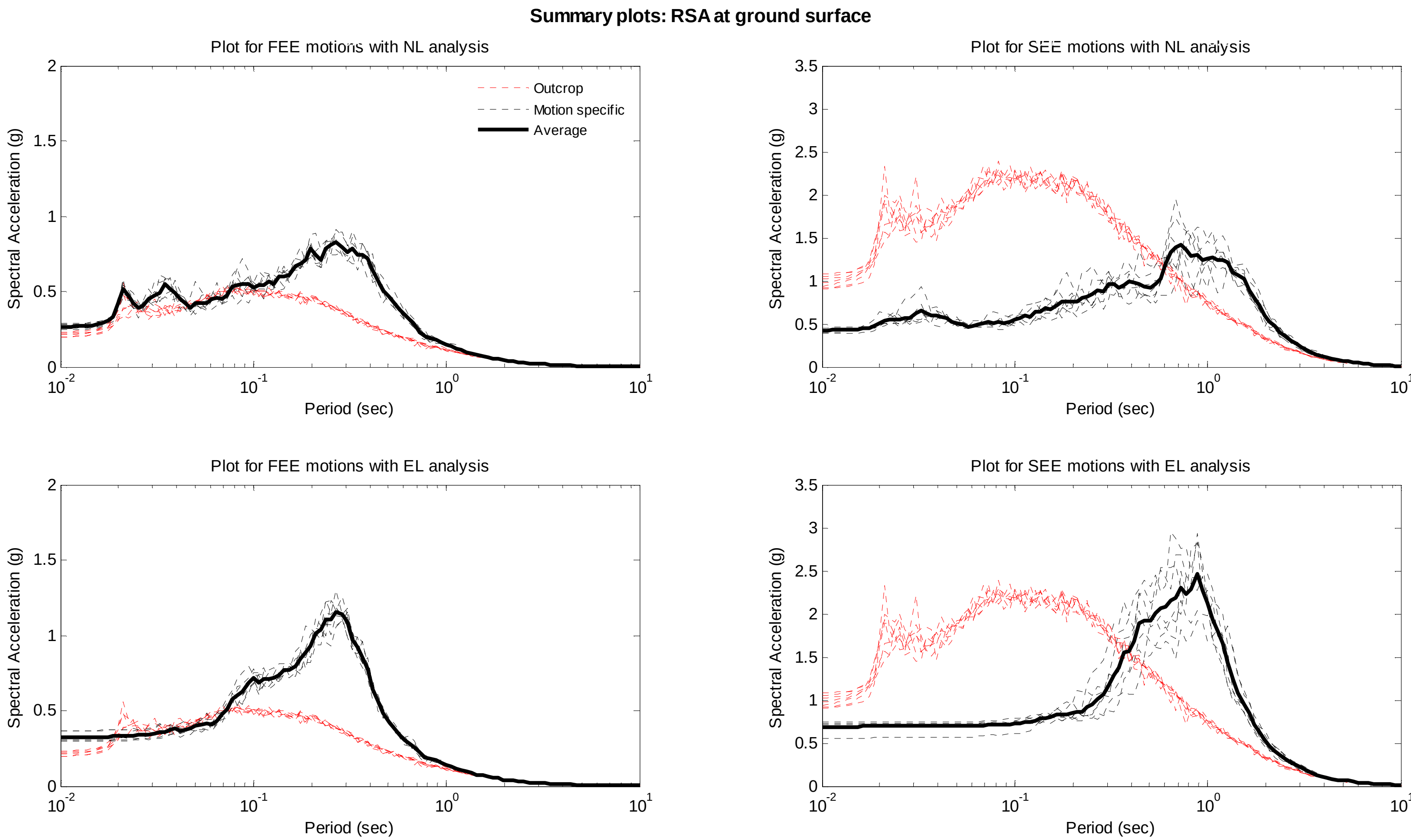
# ANALYSIS OUTPUTS:
# 150 Ft Deep Profile w/ Low PI

**Summary of the profile outputs for Nonlinear analysis**

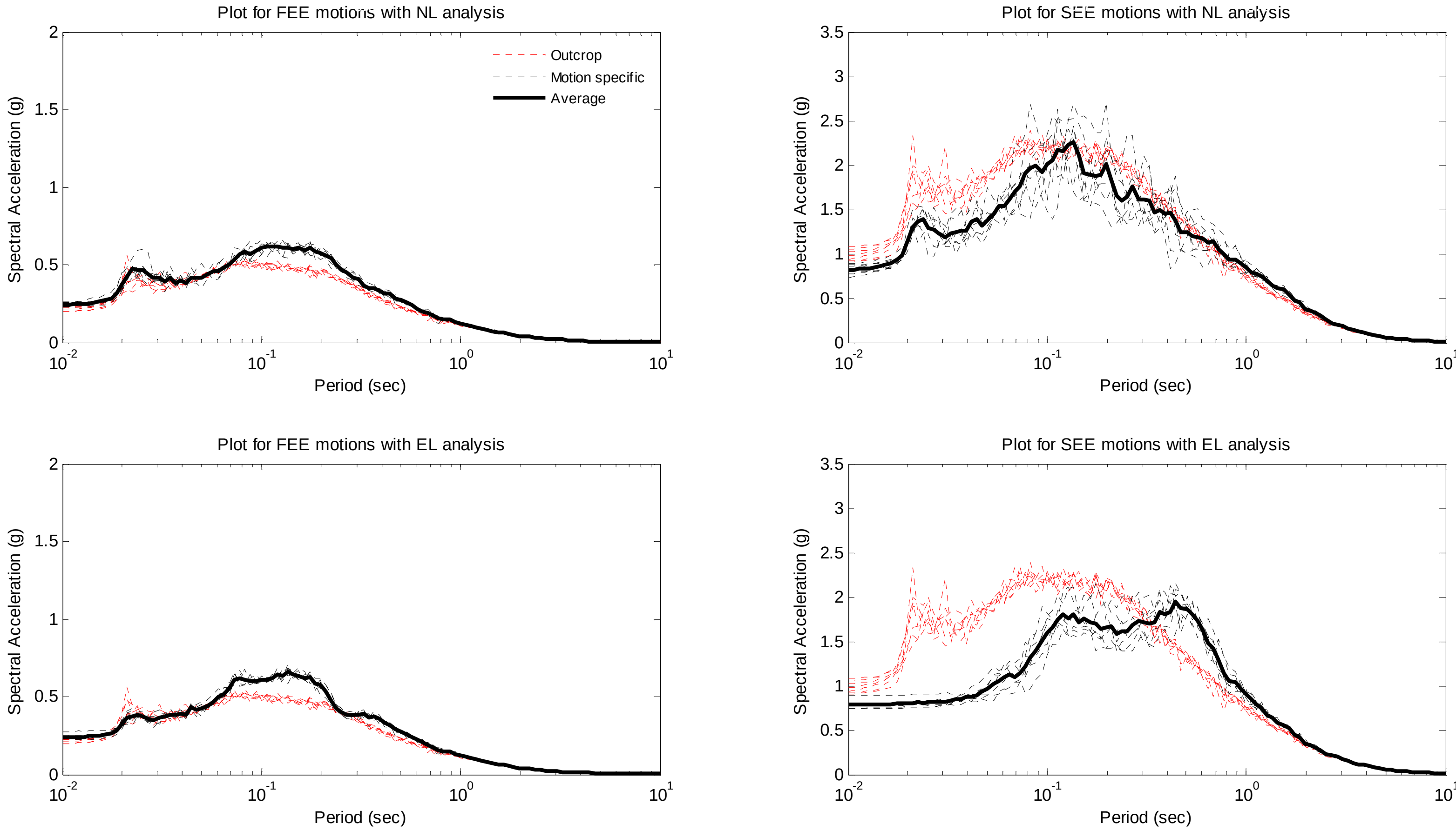


## Summary of the profile outputs for Equivalent-linear analysis



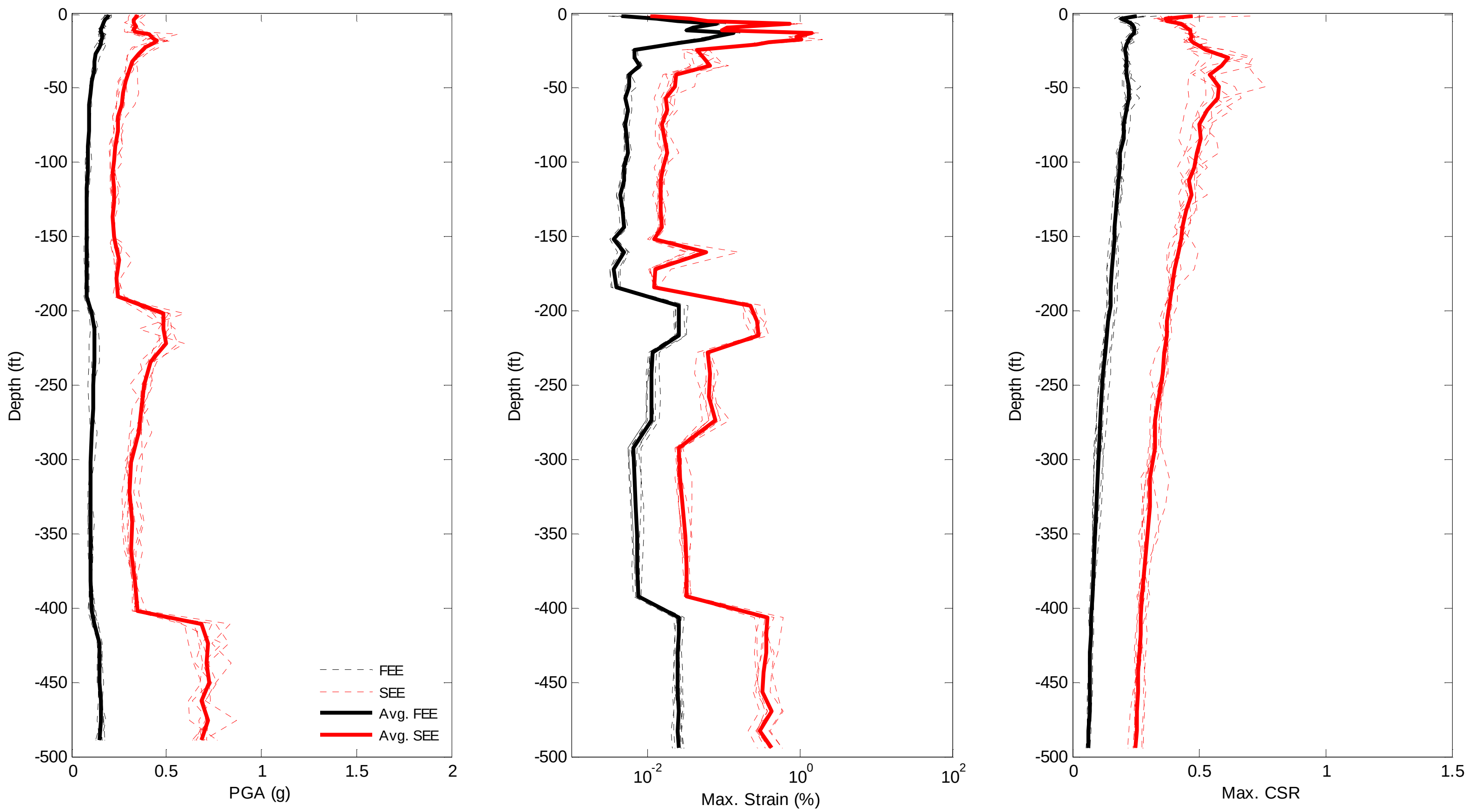

Summary plots: RSA at ground surface
Plot for FEE motions with NL analysis
Plot for SEE motions with NL analysis
Plot for FEE motions with EL analysis
Plot for SEE motions with EL analysis
Outcrop
Motion specific
Average
Spectral Acceleration (g)
Period (sec)
$10^{-2}$
$10^{-1}$
$10^{0}$
$10^{1}$

**Summary plots: RSA at top of Marl**

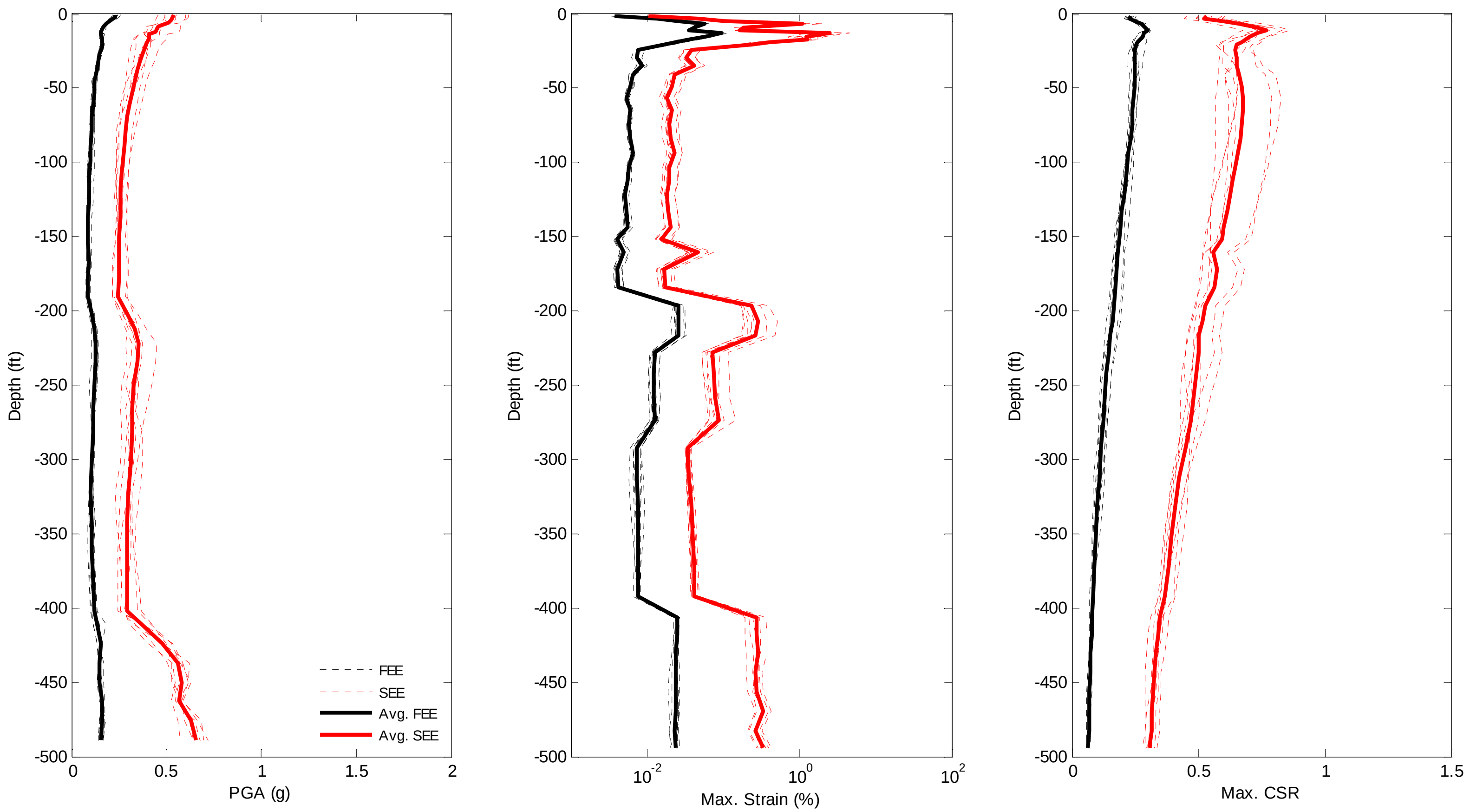


# ANALYSIS OUTPUTS:
# 500 Ft Deep Profile

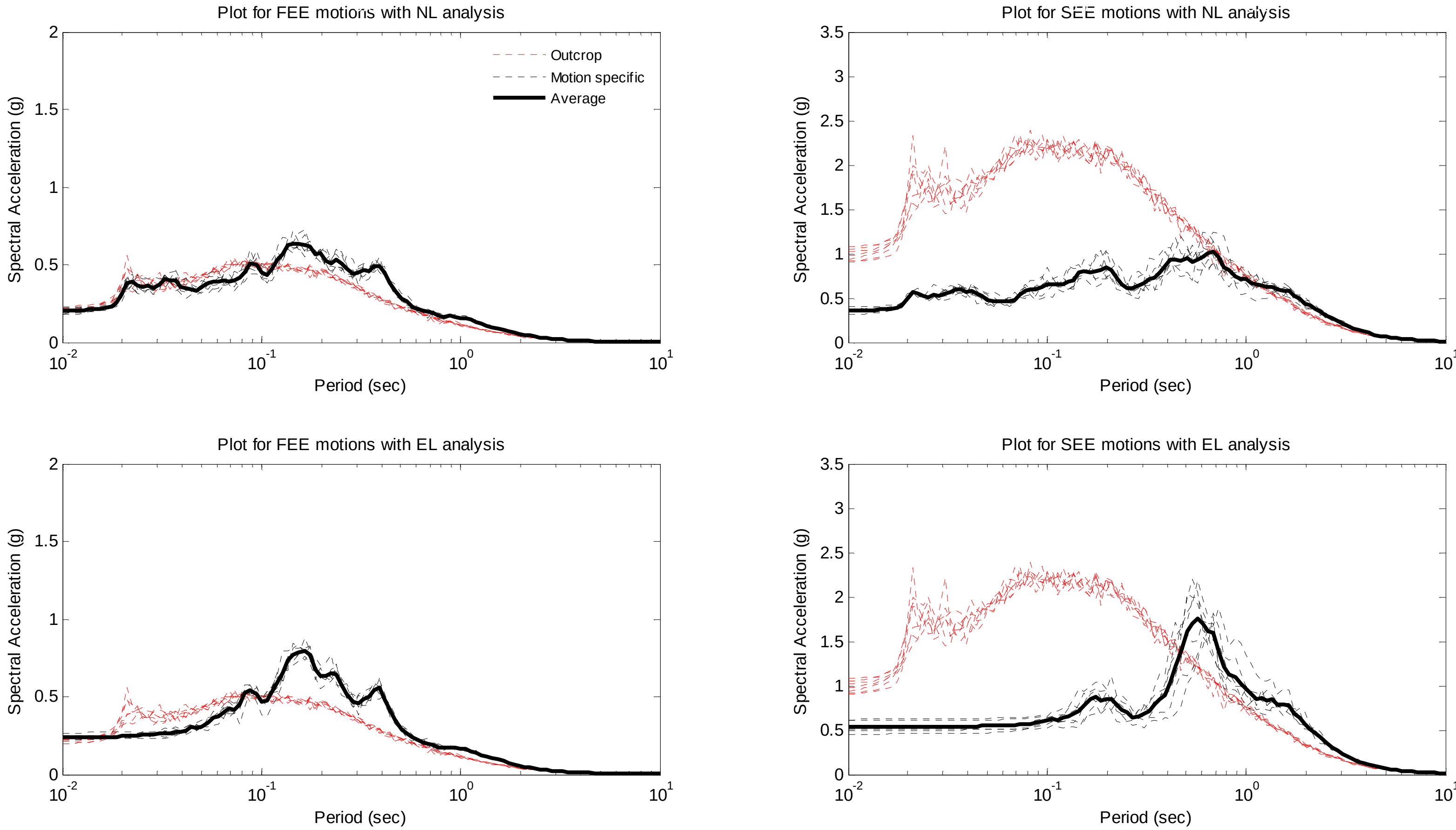
**Summary of the profile outputs for Nonlinear analysis**

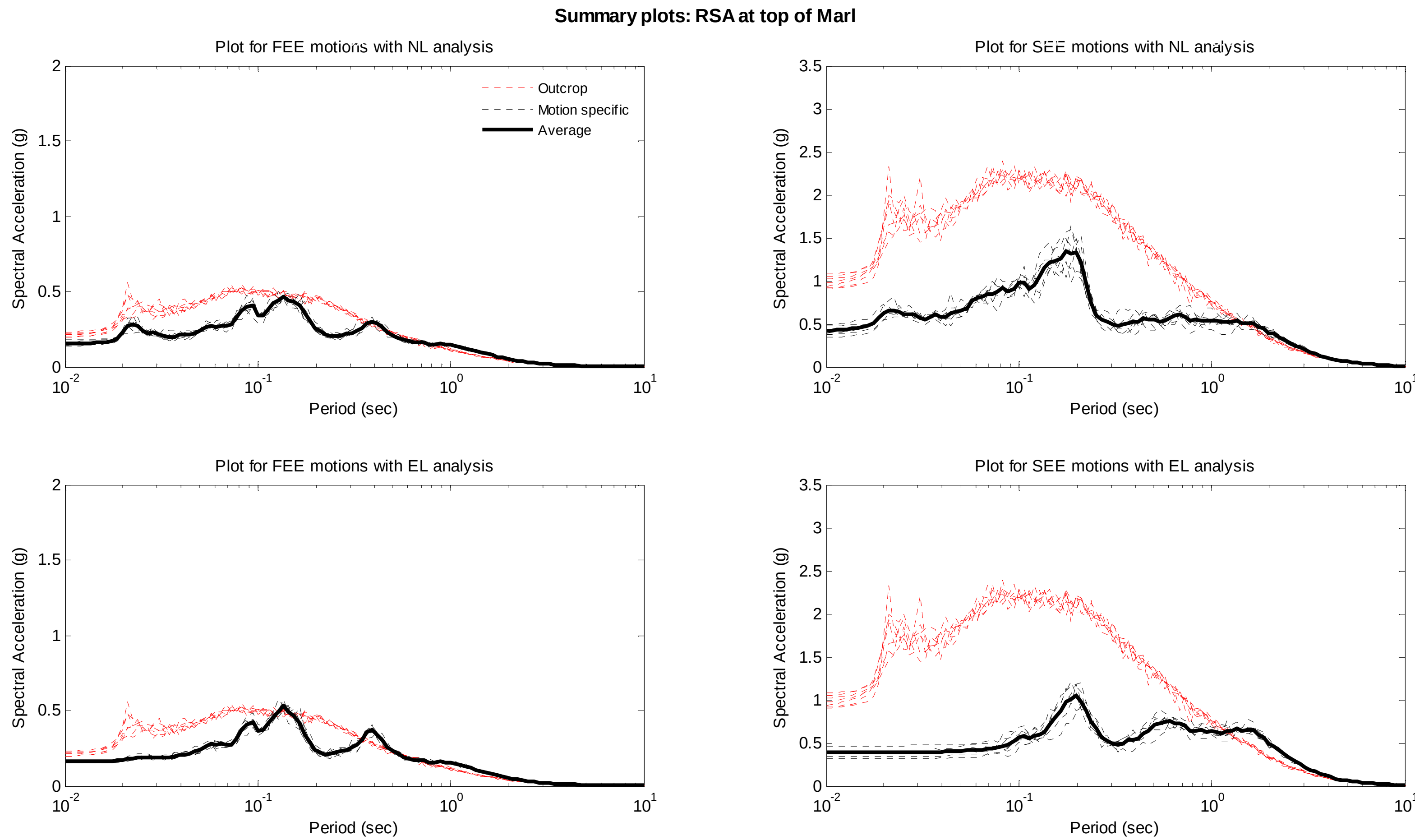


**Summary of the profile outputs for Equivalent-linear analysis**



**Summary plots: RSA at ground surface**

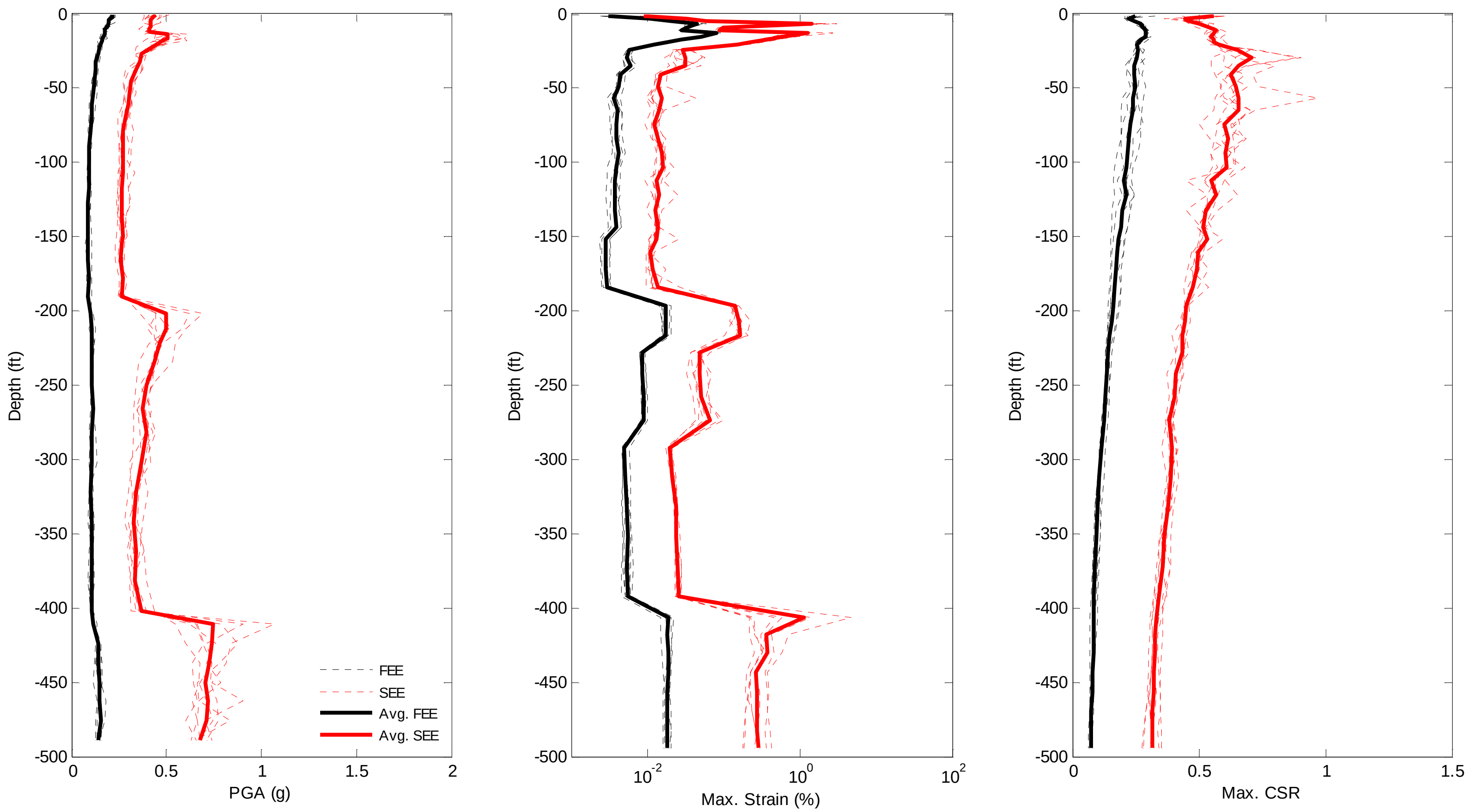




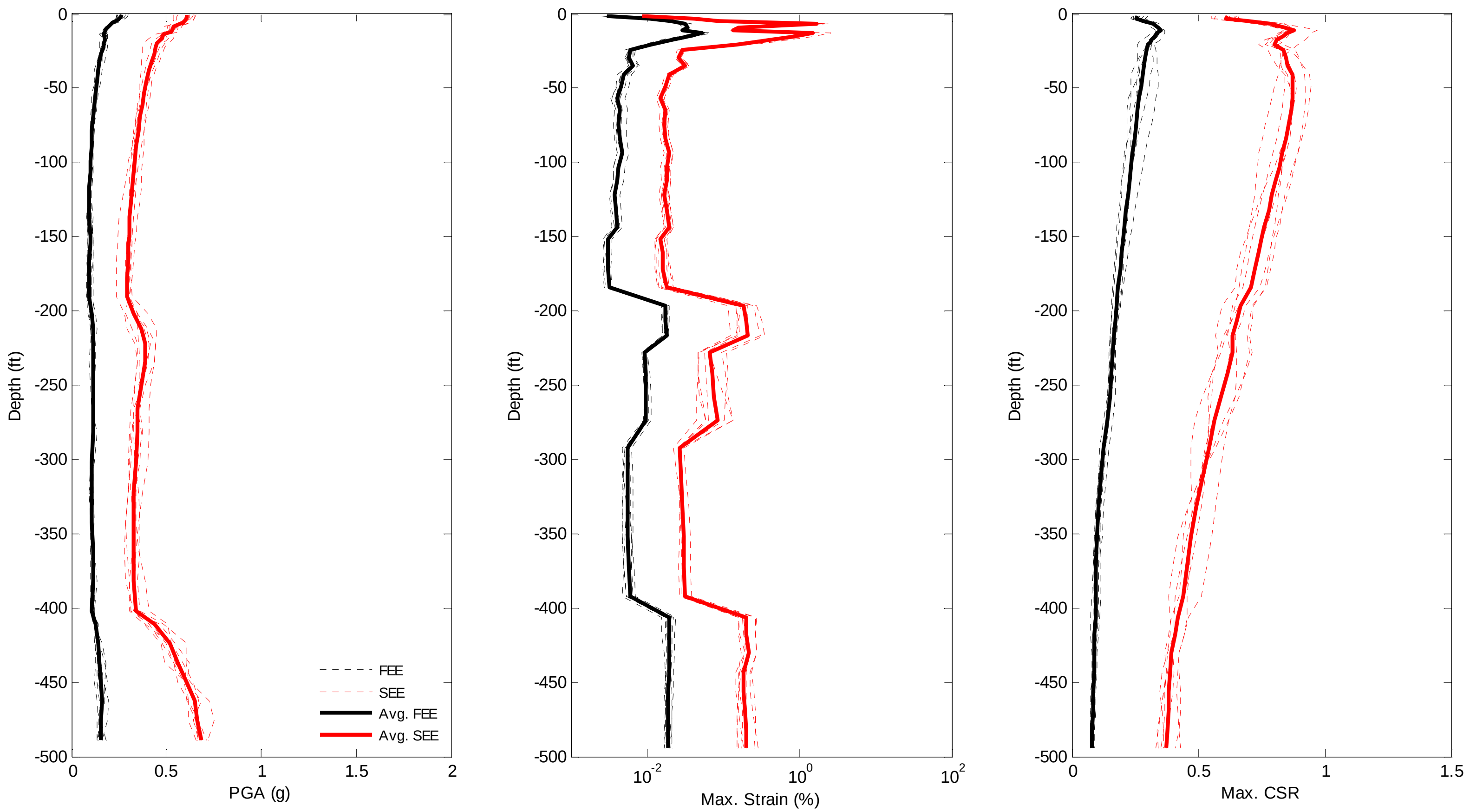
Summary plots: RSA at top of Marl
Plot for FEE motions with NL analysis
Plot for SEE motions with NL analysis
Plot for FEE motions with EL analysis
Plot for SEE motions with EL analysis
Outcrop
Motion specific
Average
Spectral Acceleration (g)
Period (sec)

# ANALYSIS OUTPUTS:
# 500 Ft Deep Profile w/ High $V_S$ and SS

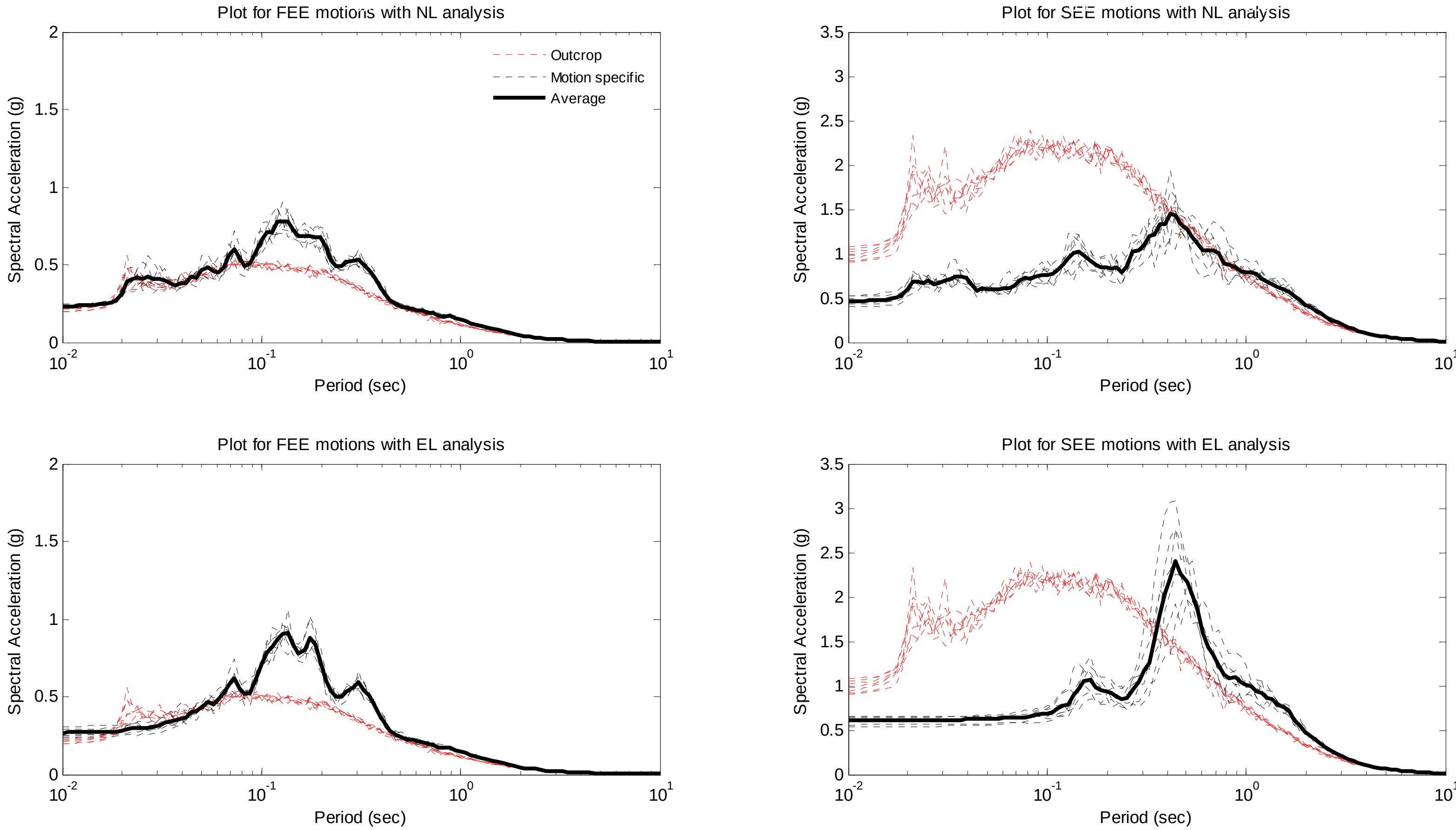
**Summary of the profile outputs for Nonlinear analysis**



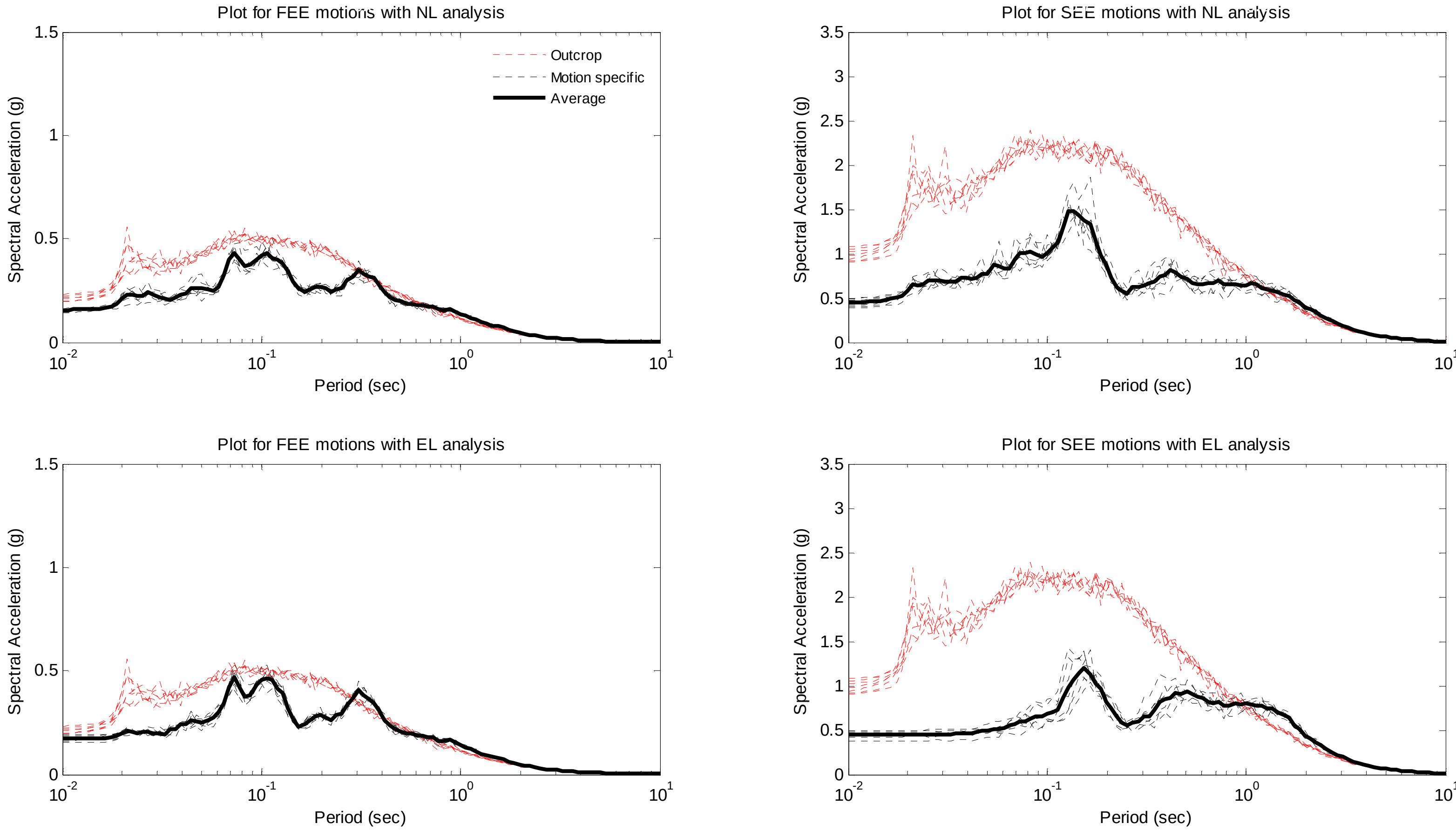
**Summary of the profile outputs for Equivalent-linear analysis**



**Summary plots: RSA at ground surface**

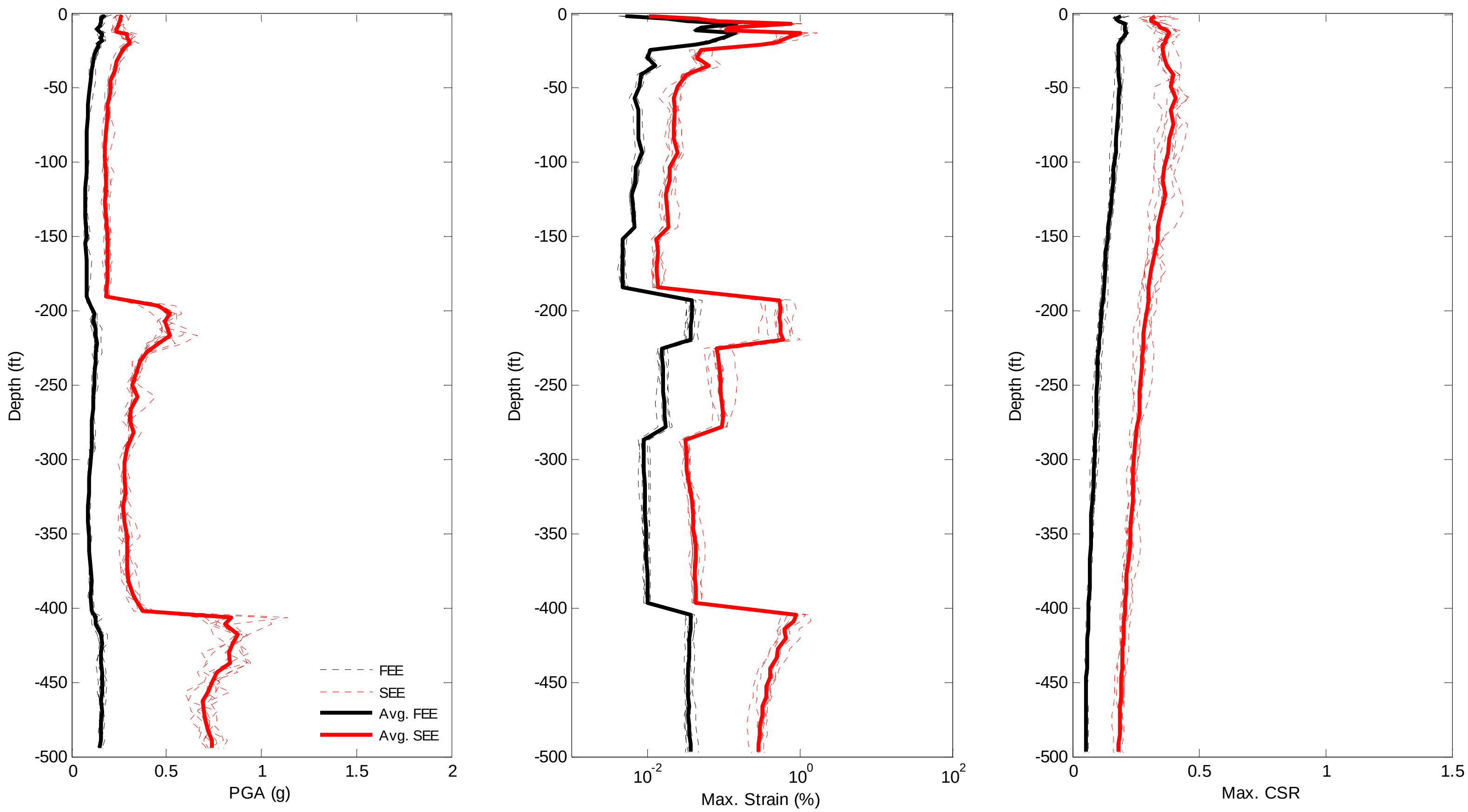


**Summary plots: RSA at top of Marl**

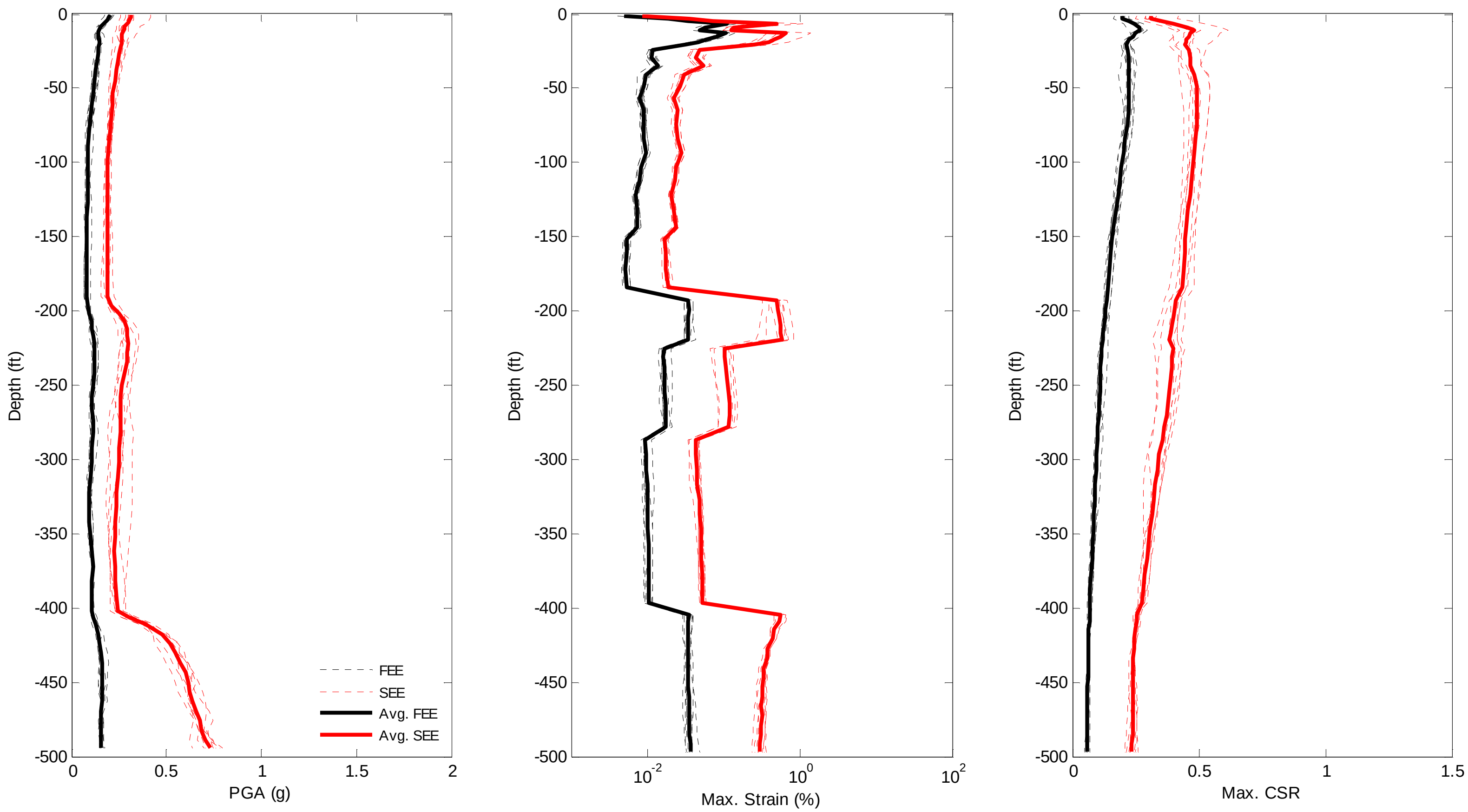


# ANALYSIS OUTPUTS:
# 500 Ft Deep Profile w/ Low $V_S$ and SS

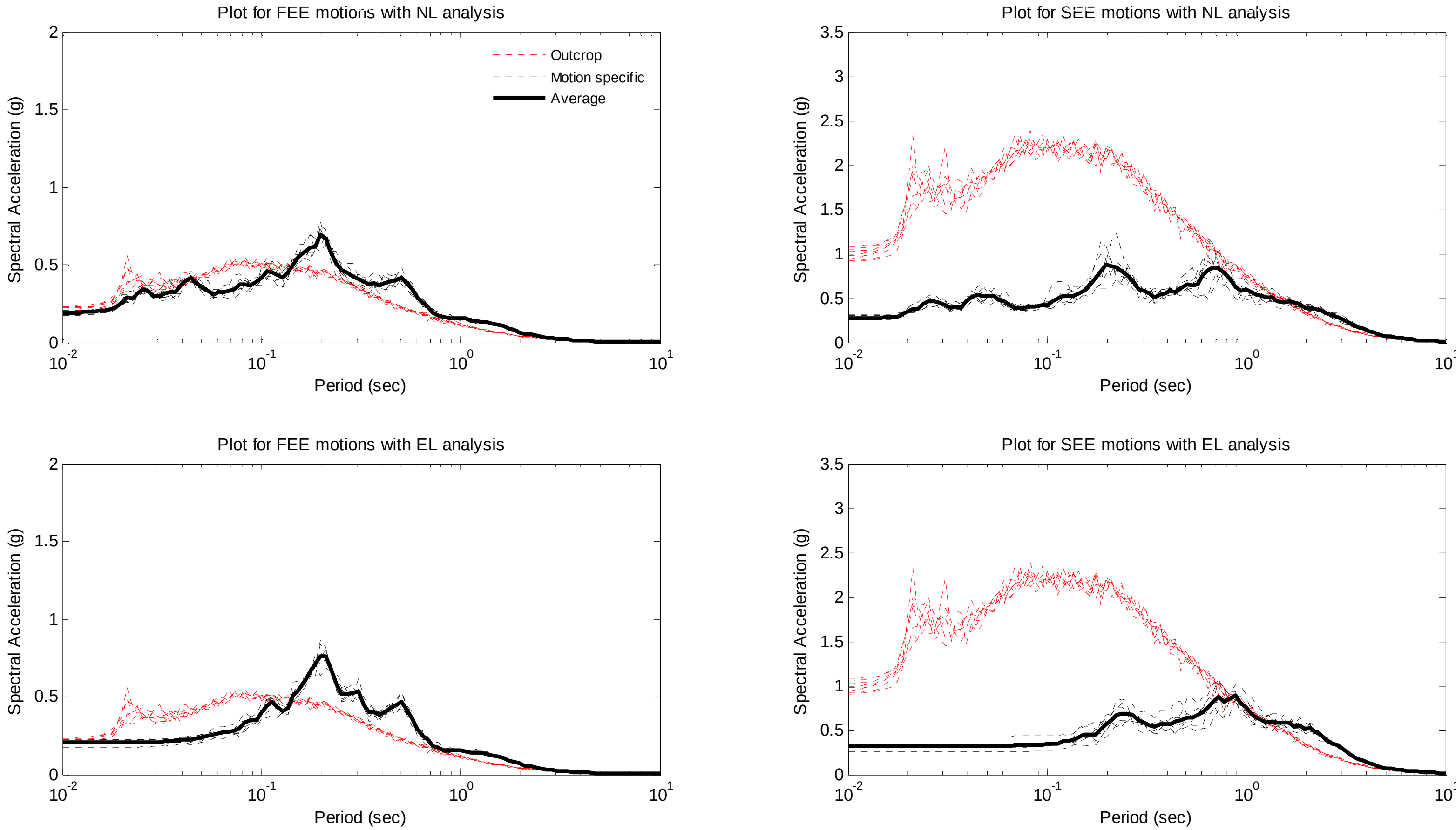
**Summary of the profile outputs for Nonlinear analysis**



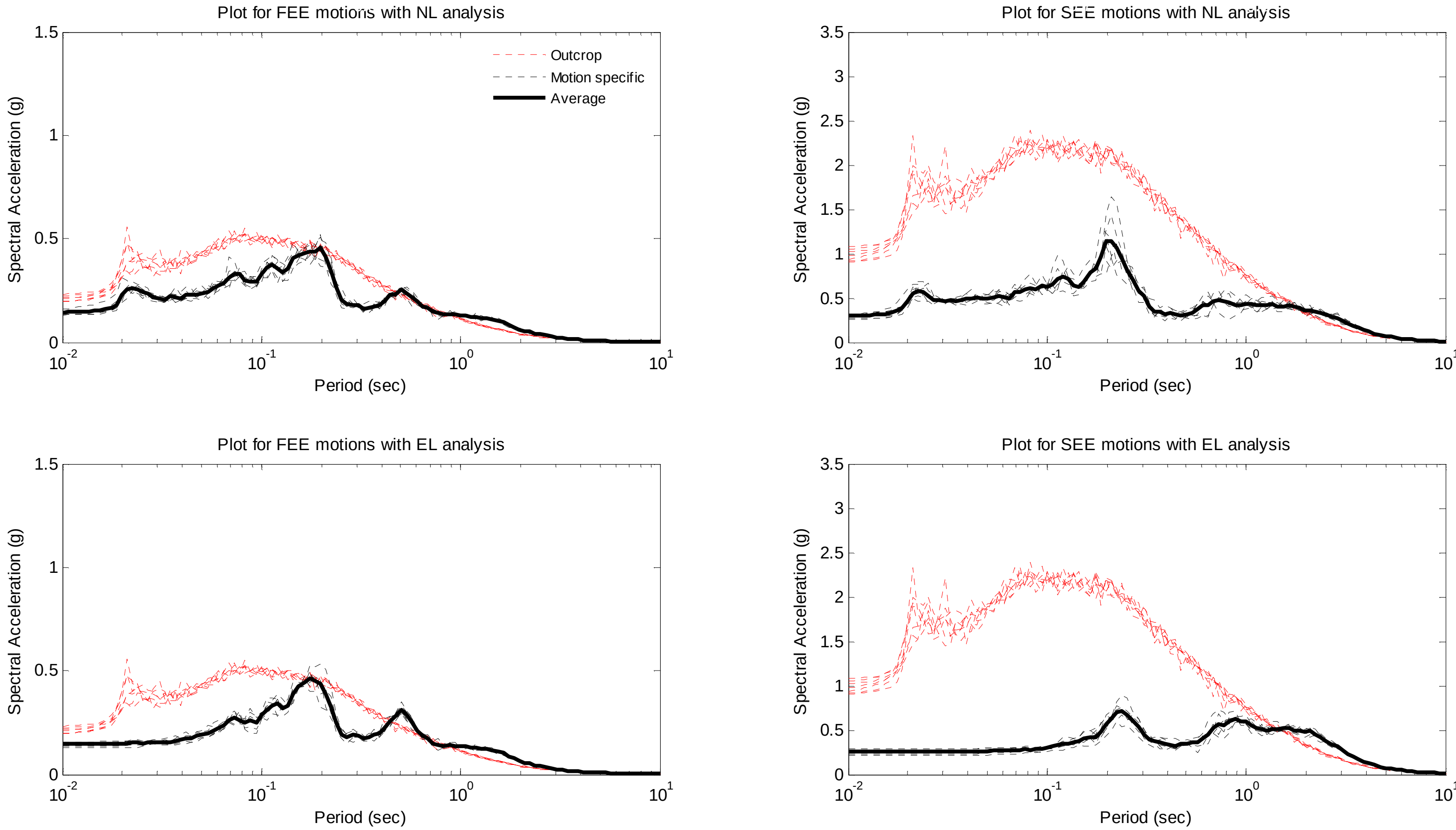
**Summary of the profile outputs for Equivalent-linear analysis**



**Summary plots: RSA at ground surface**

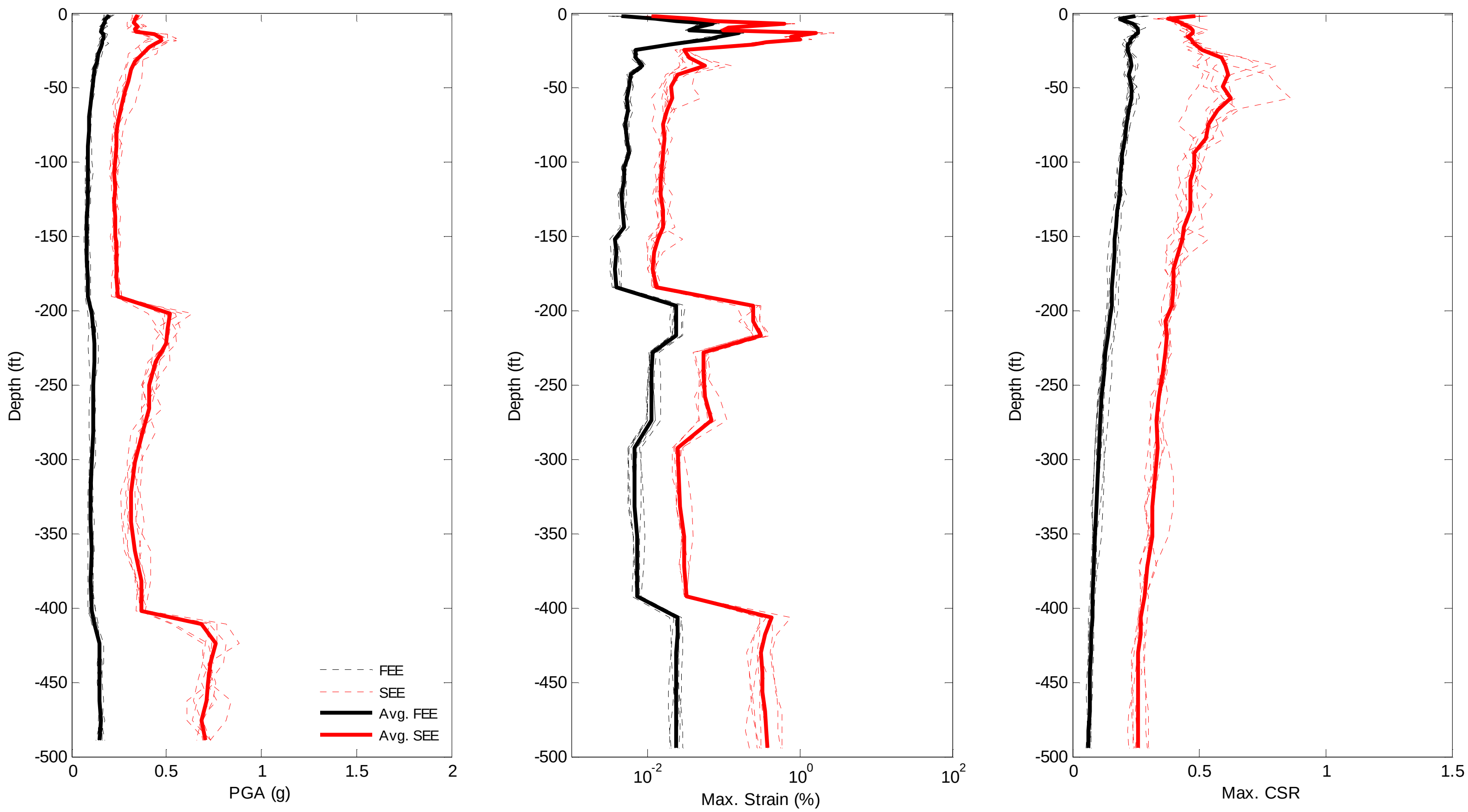


**Summary plots: RSA at top of Marl**

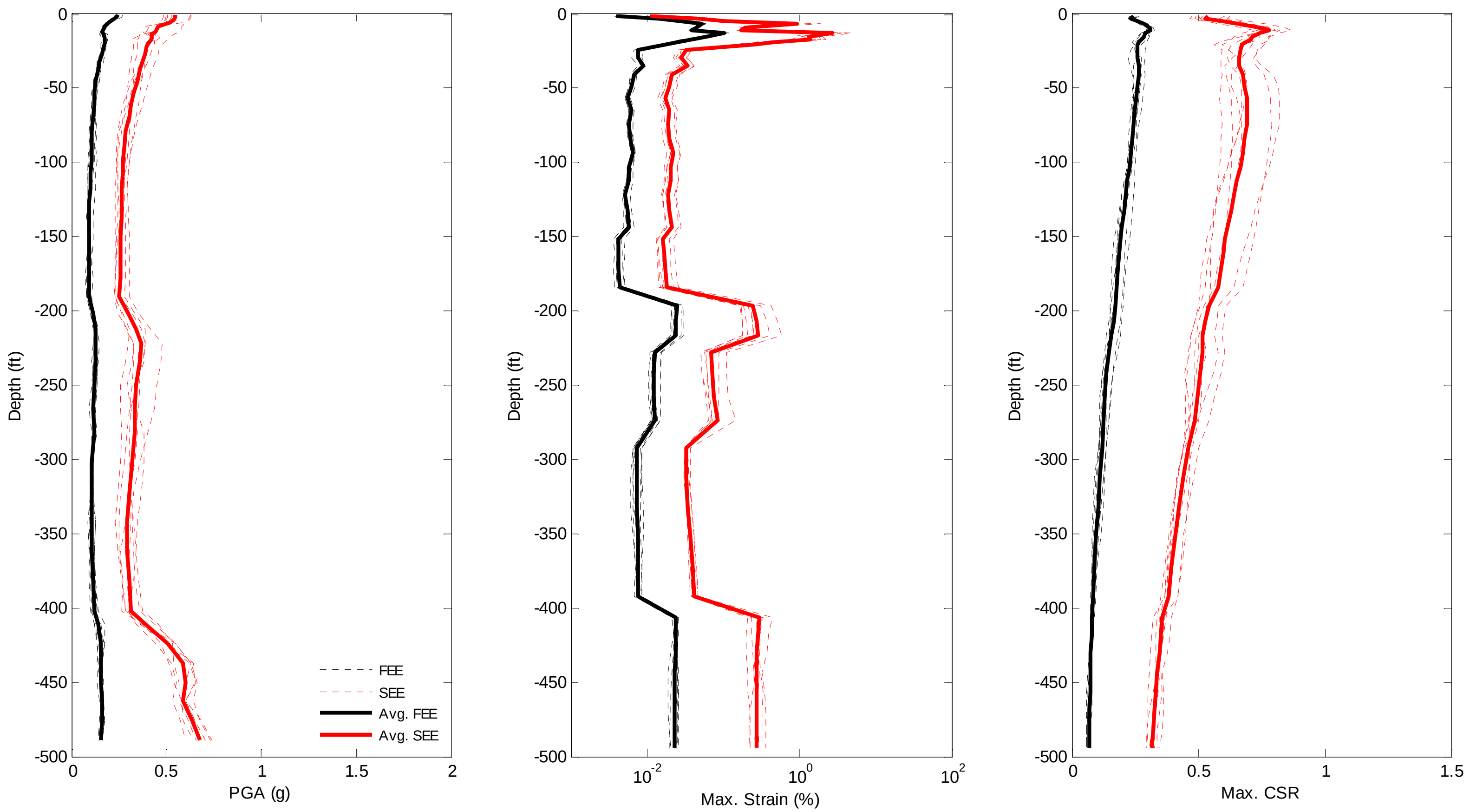


# ANALYSIS OUTPUTS:
# 500 Ft Deep Profile w/ High PI

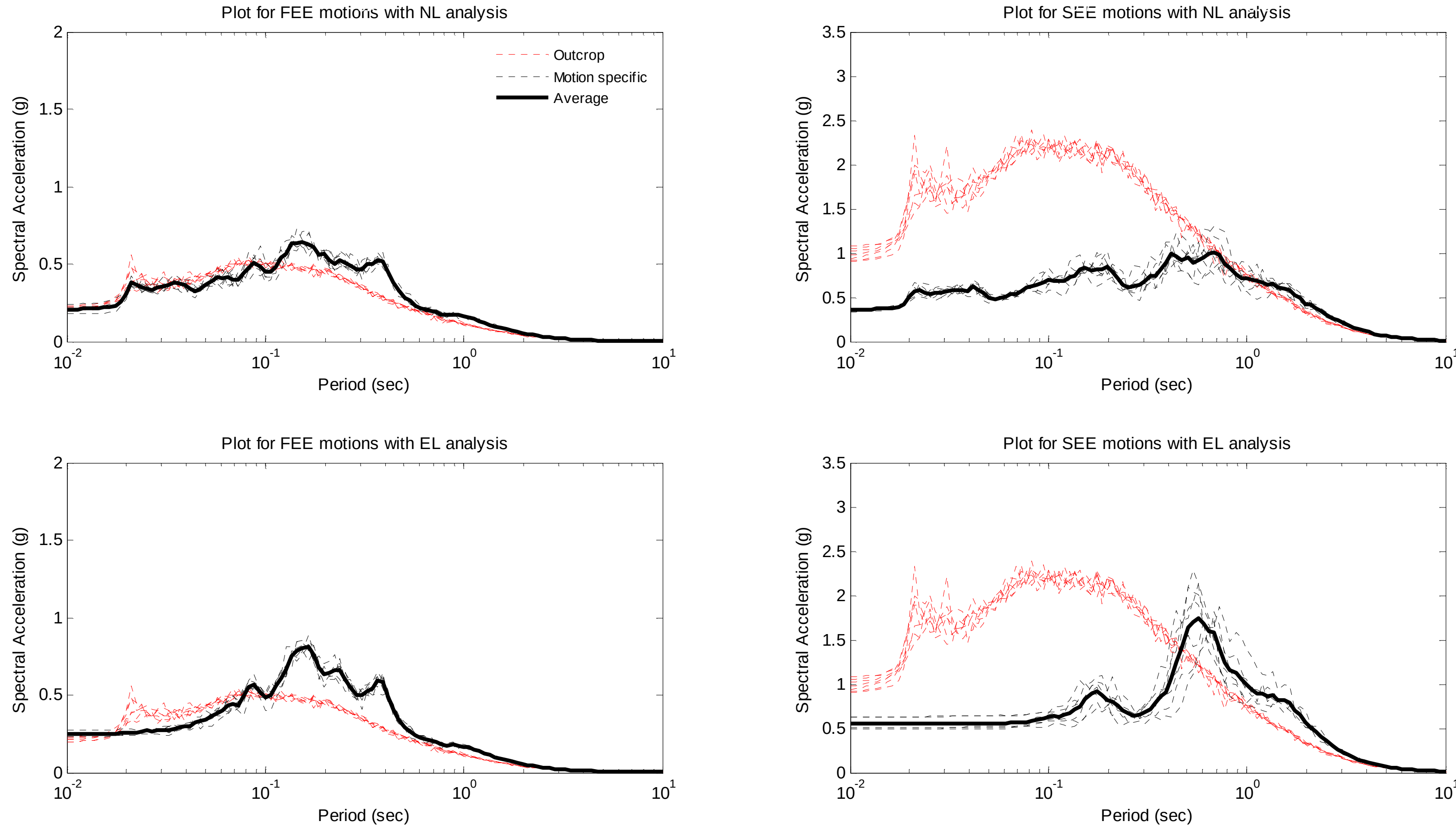
## Summary of the profile outputs for Nonlinear analysis



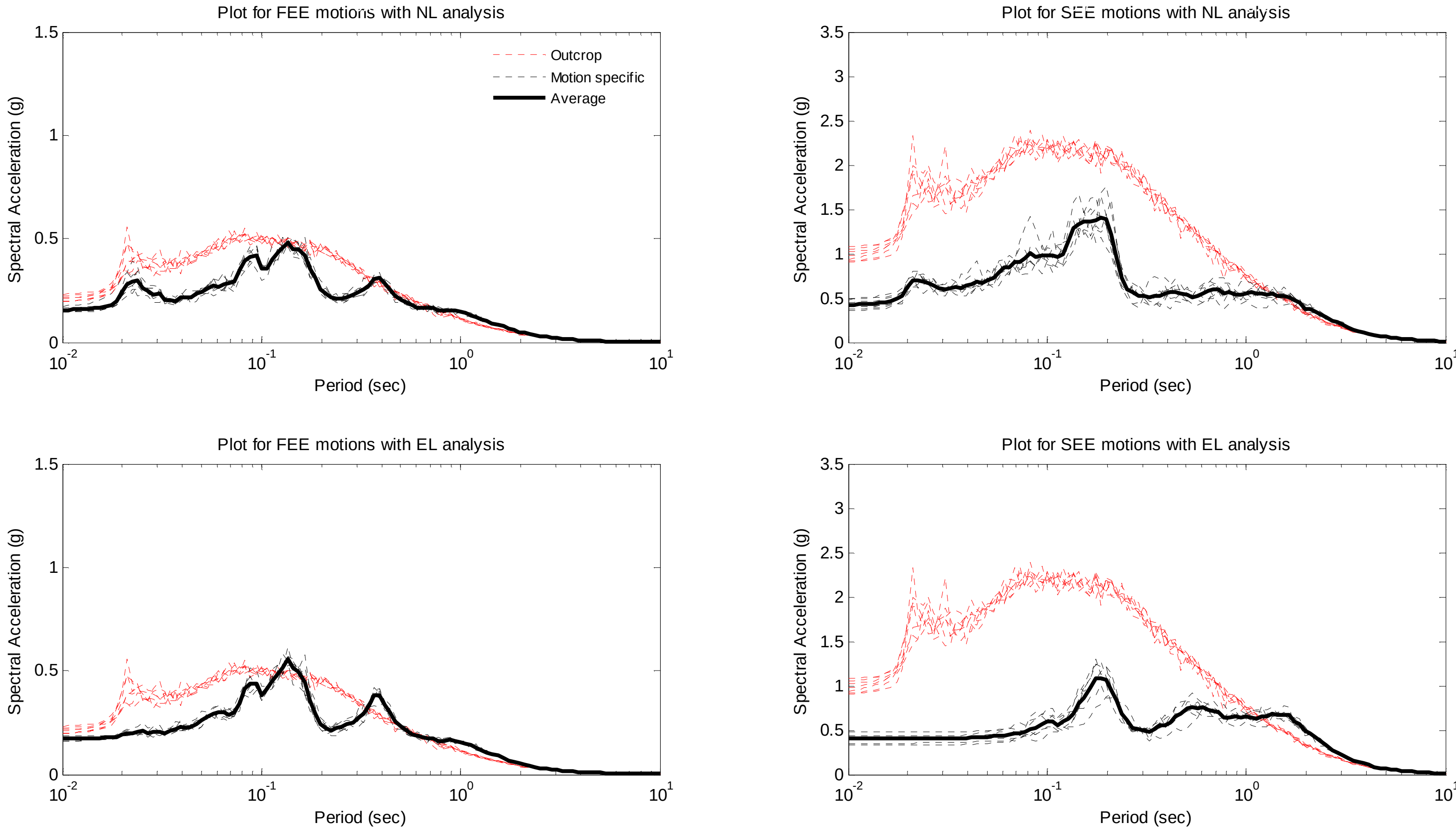
**Summary of the profile outputs for Equivalent-linear analysis**



## Summary plots: RSA at ground surface

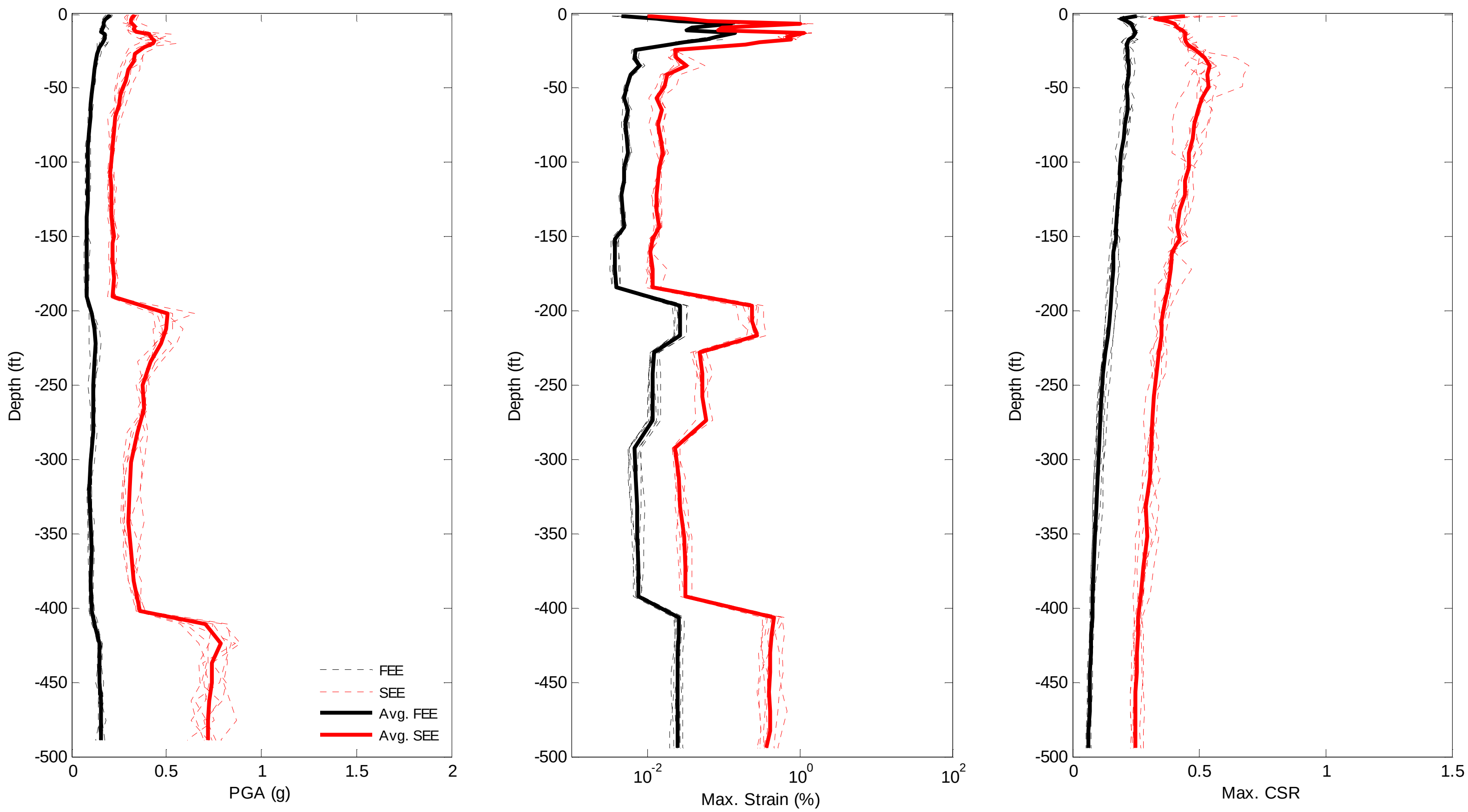


## Summary plots: RSA at top of Marl

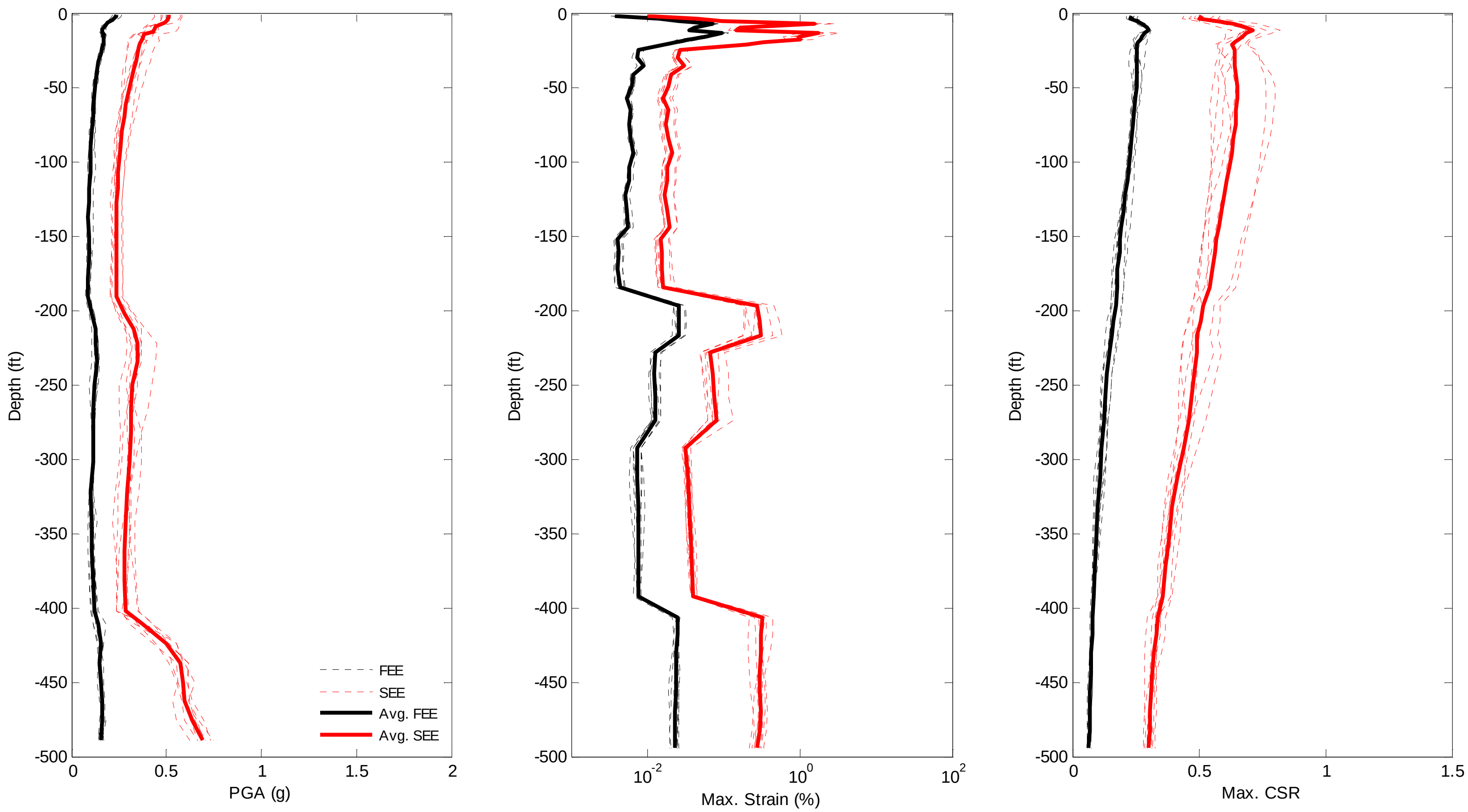


# ANALYSIS OUTPUTS:
# 500 Ft Deep Profile w/ Low PI

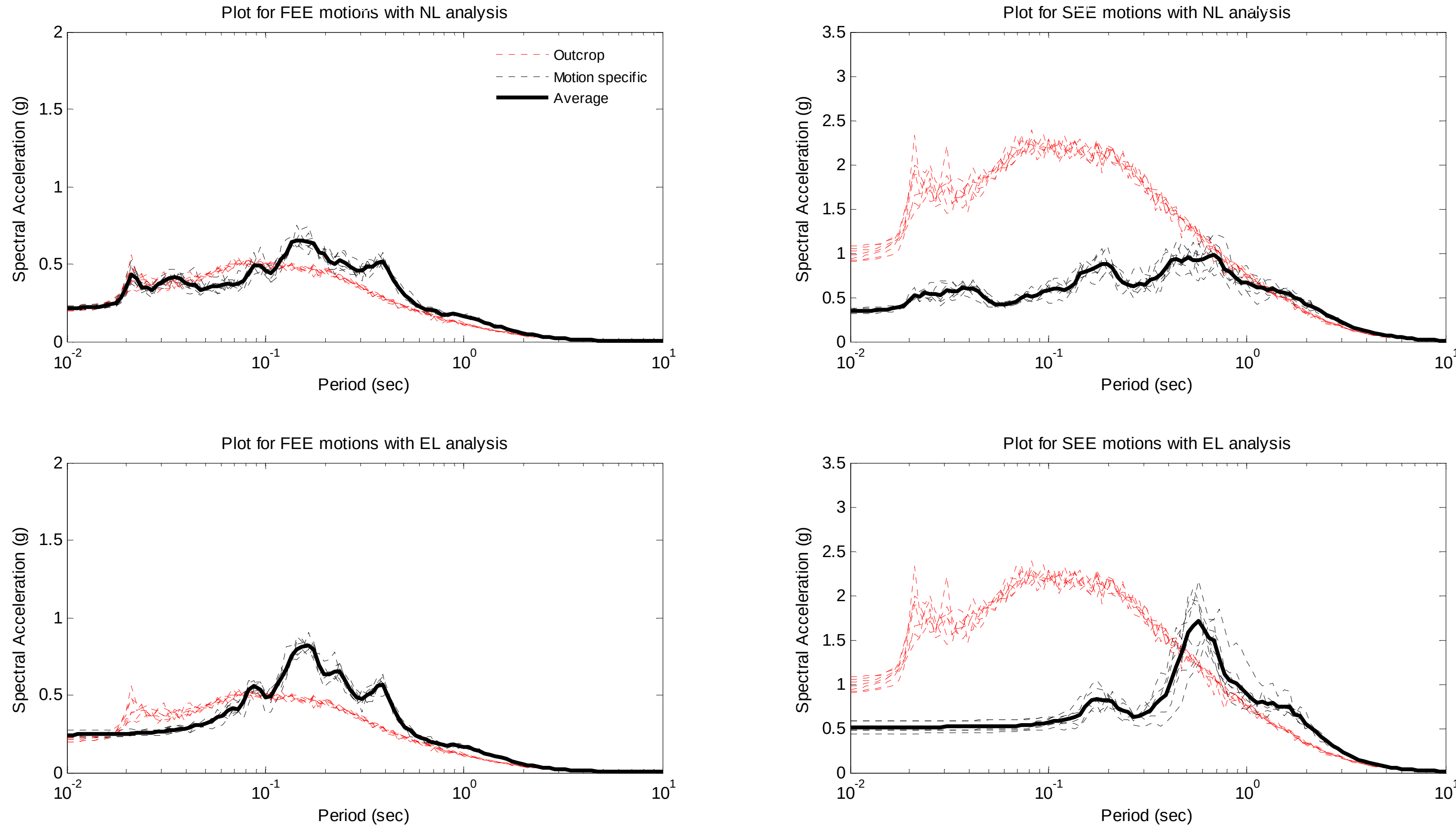
**Summary of the profile outputs for Nonlinear analysis**

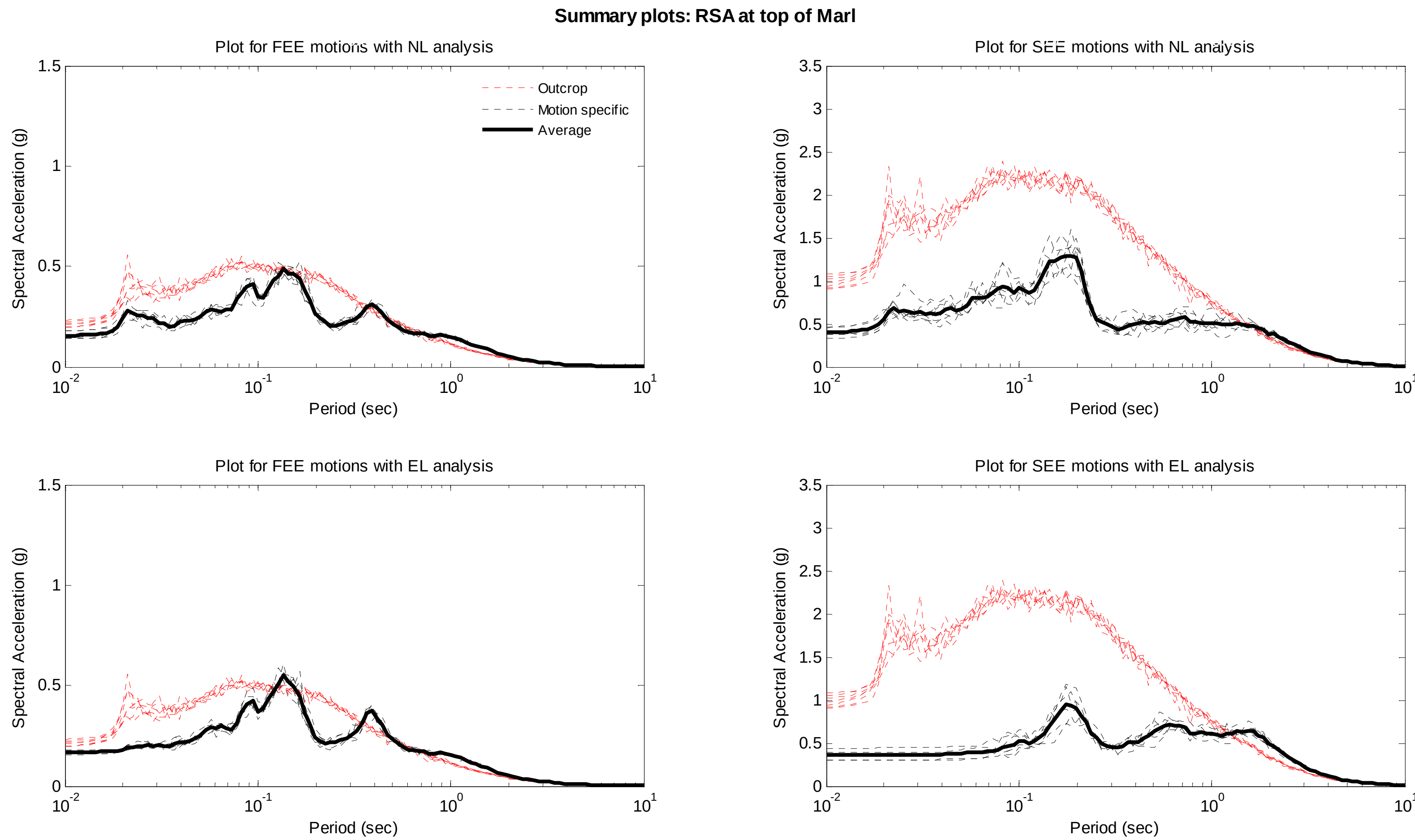


**Summary of the profile outputs for Equivalent-linear analysis**



## Summary plots: RSA at ground surface

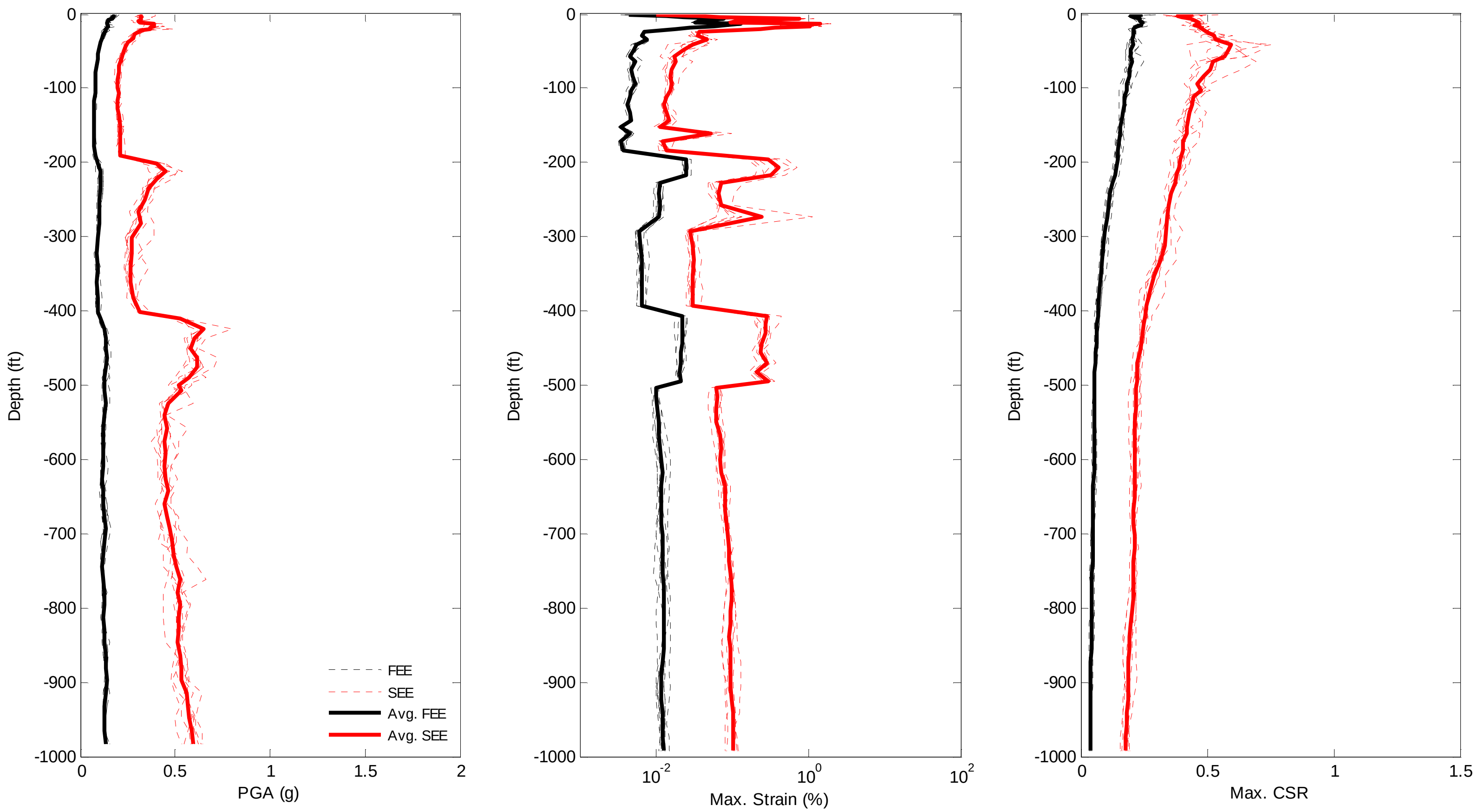




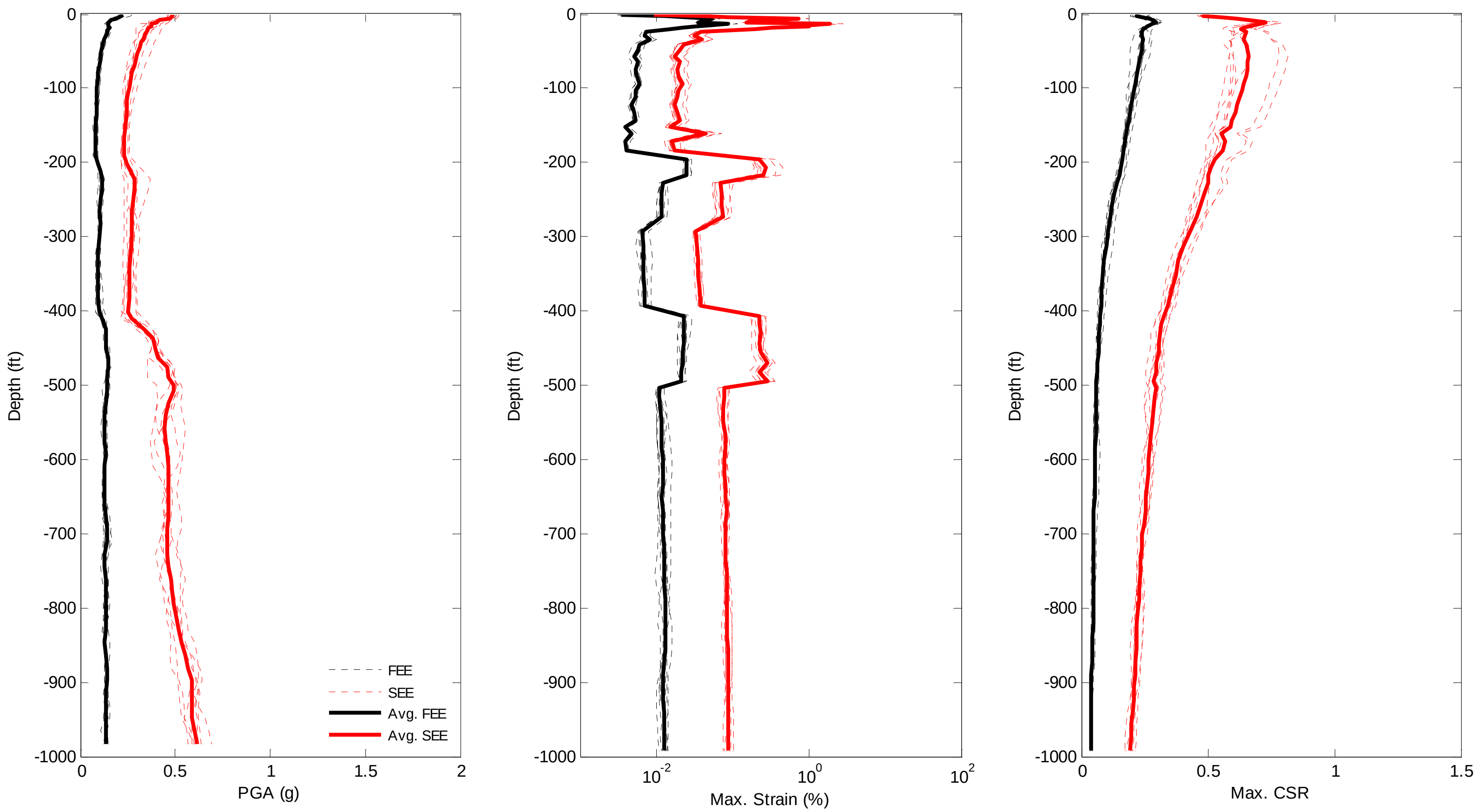
Summary plots: RSA at top of Marl
Plot for FEE motions with NL analysis
Plot for SEE motions with NL analysis
Plot for FEE motions with EL analysis
Plot for SEE motions with EL analysis
Outcrop
Motion specific
Average
Spectral Acceleration (g)
Period (sec)
0
0.5
1
1.5
2
2.5
3
3.5
$10^{-2}$
$10^{-1}$
$10^{0}$
$10^{1}$

# ANALYSIS OUTPUTS:
# 1000 Ft Deep Profile

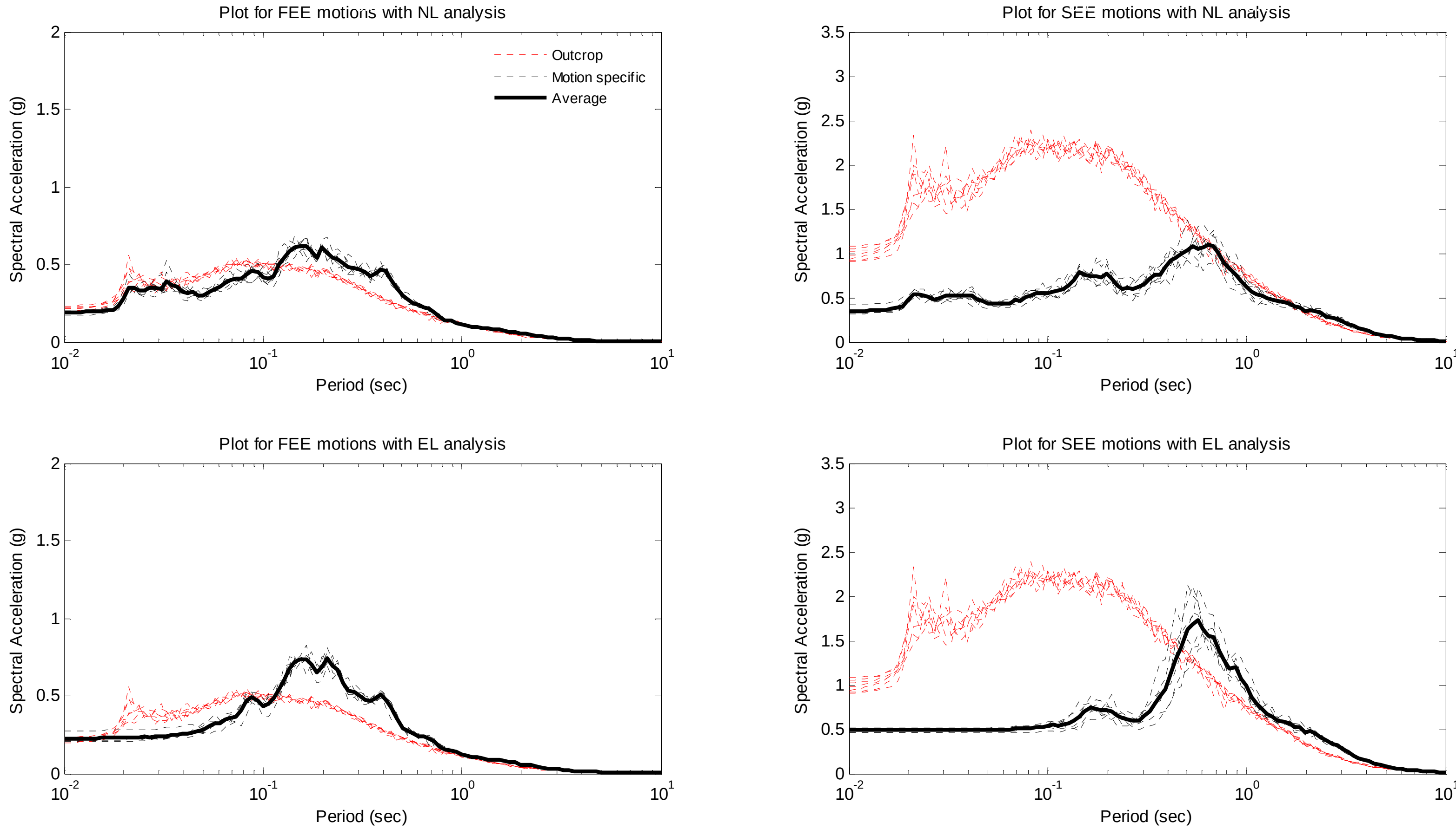
**Summary of the profile outputs for Nonlinear analysis**



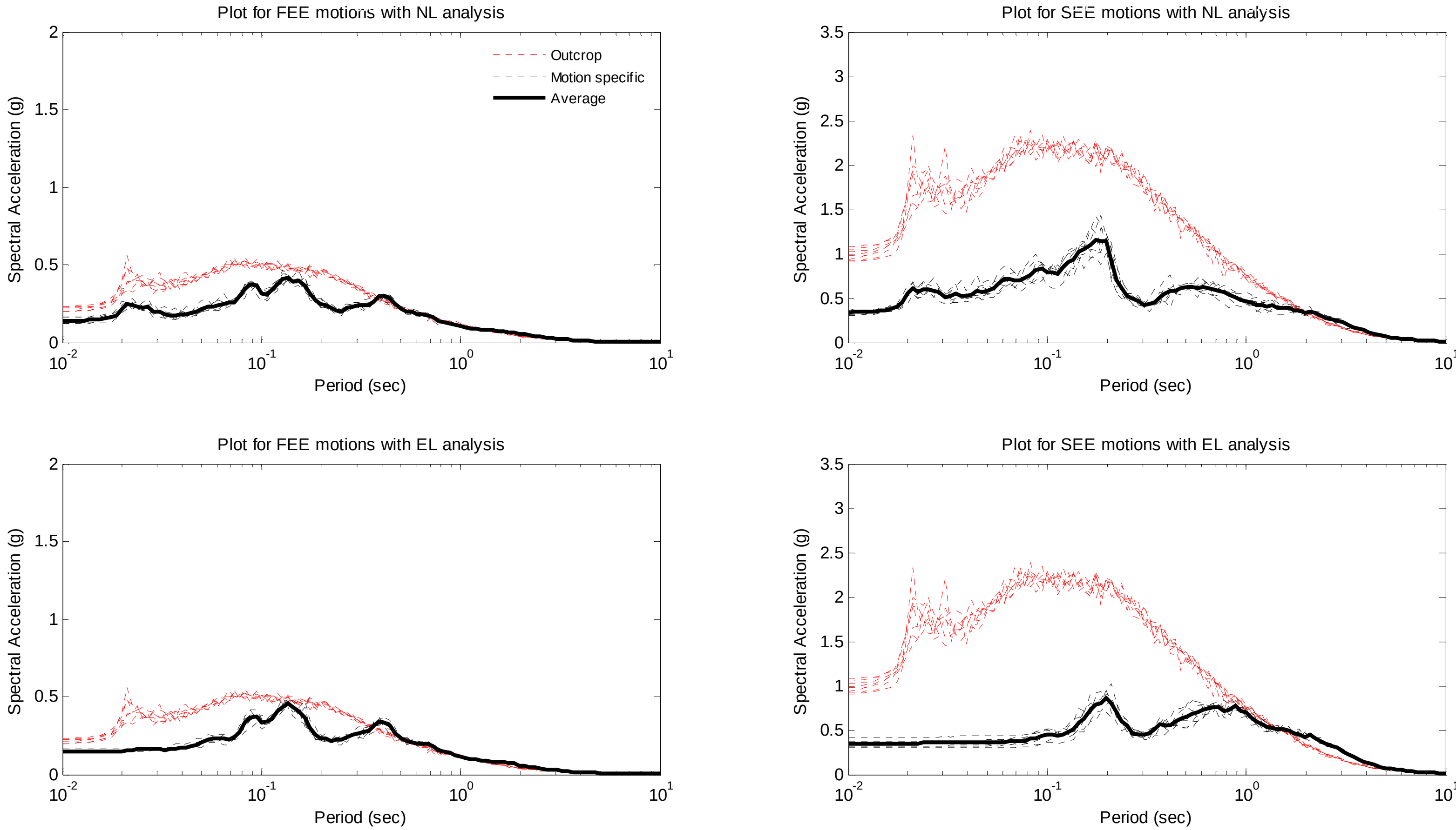
## Summary of the profile outputs for Equivalent-linear analysis



**Summary plots: RSA at ground surface**

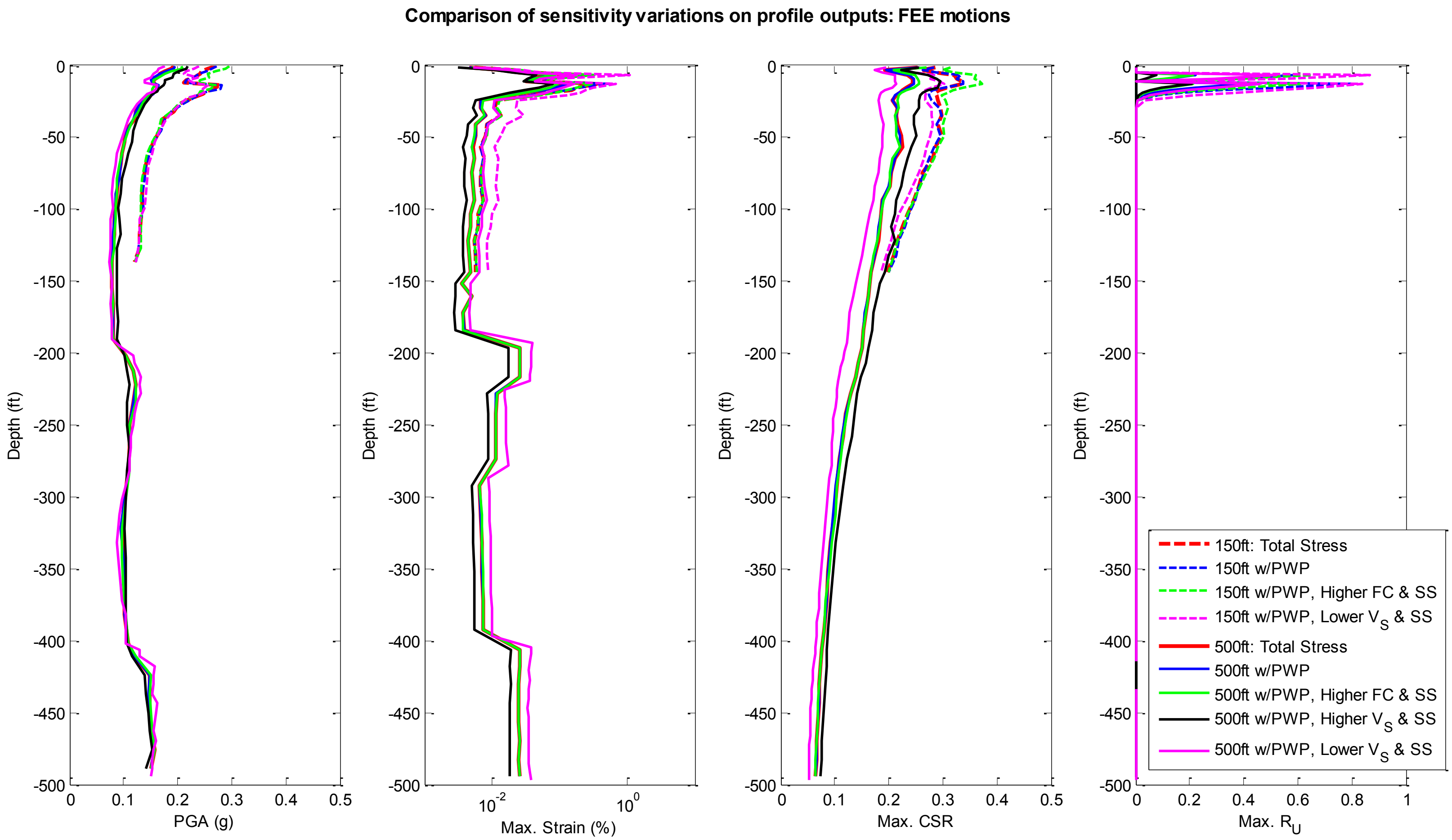


**Summary plots: RSA at top of Marl**

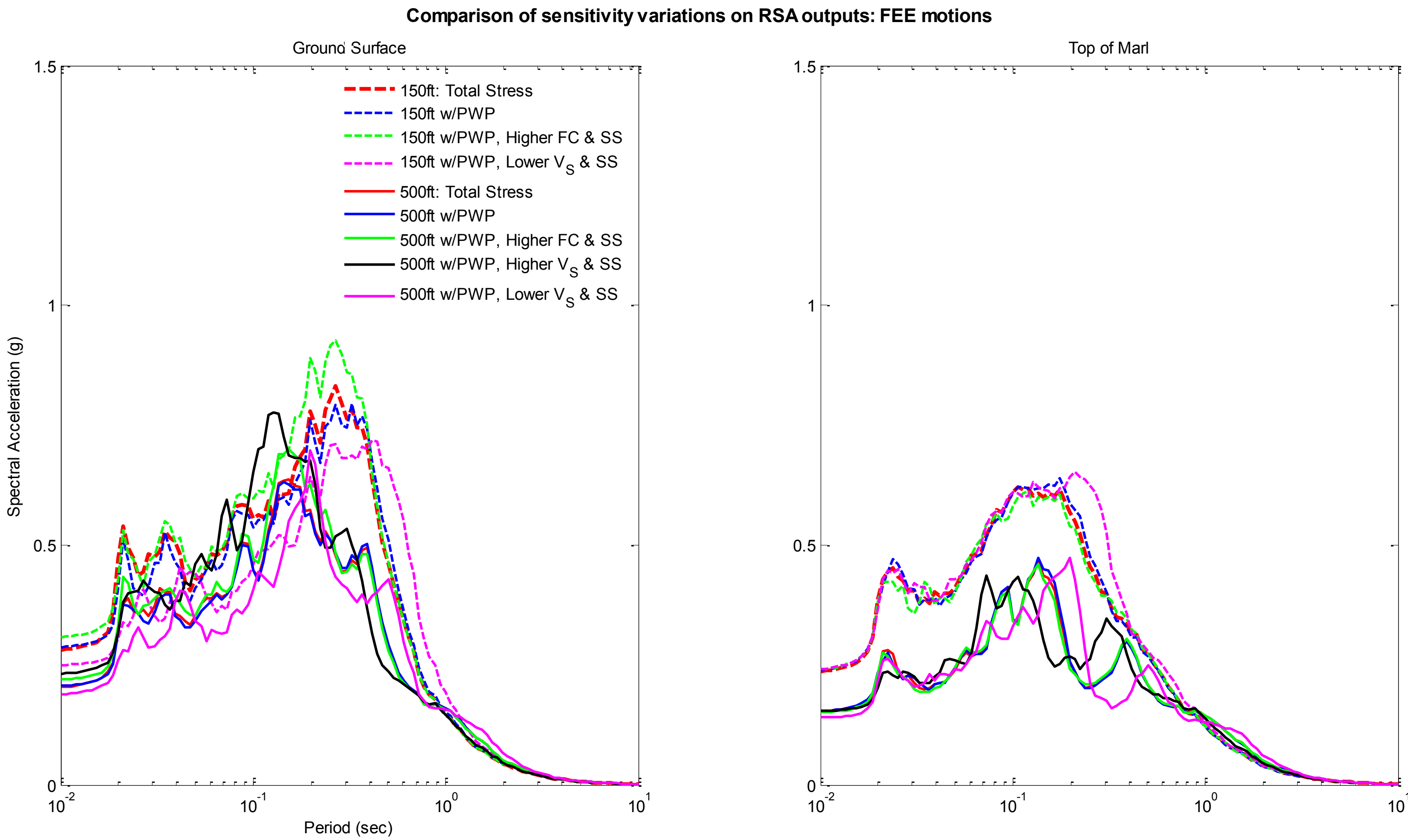


# Appendix III– Effective Stress Site Response Analysis Outputs

# SUMMARY PLOTS: FEE Motions

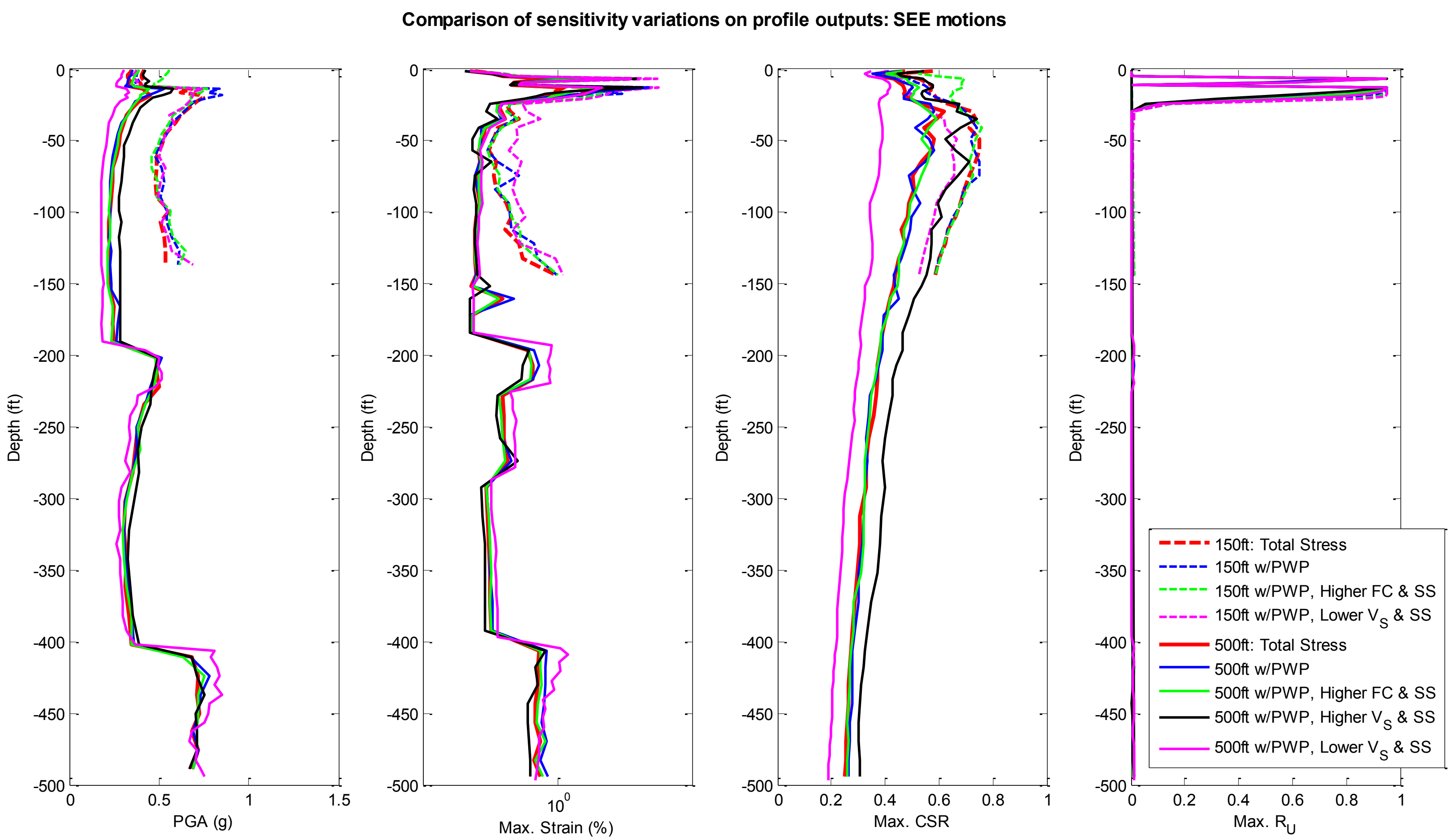


Comparison of sensitivity variations on profile outputs: FEE motions
Depth (ft)
PGA (g)
Max. Strain (%)
Max. CSR
Max. $R_U$
150ft: Total Stress
150ft w/PWP
150ft w/PWP, Higher FC & SS
150ft w/PWP, Lower $V_S$ & SS
500ft: Total Stress
500ft w/PWP
500ft w/PWP, Higher FC & SS
500ft w/PWP, Higher $V_S$ & SS
500ft w/PWP, Lower $V_S$ & SS

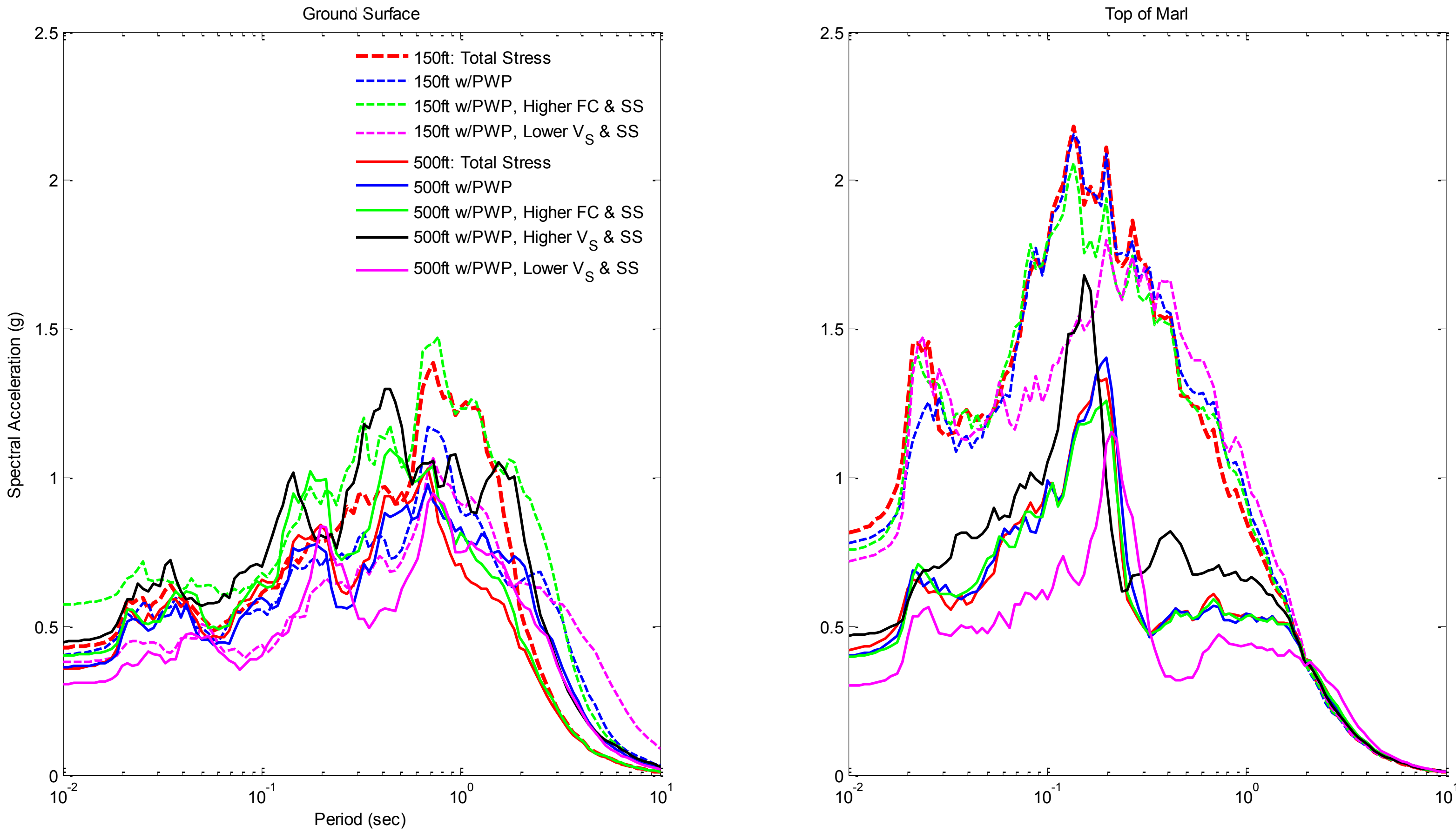


Comparison of sensitivity variations on RSA outputs: FEE motions
Ground Surface
Top of Marl
150ft: Total Stress
150ft w/PWP
150ft w/PWP, Higher FC & SS
150ft w/PWP, Lower V_S & SS
500ft: Total Stress
500ft w/PWP
500ft w/PWP, Higher FC & SS
500ft w/PWP, Higher V_S & SS
500ft w/PWP, Lower V_S & SS
Spectral Acceleration (g)
Period (sec)
1.5
1
0.5
0
10^-2
10^-1
10^0
10^1

# SUMMARY PLOTS: SEE Motions

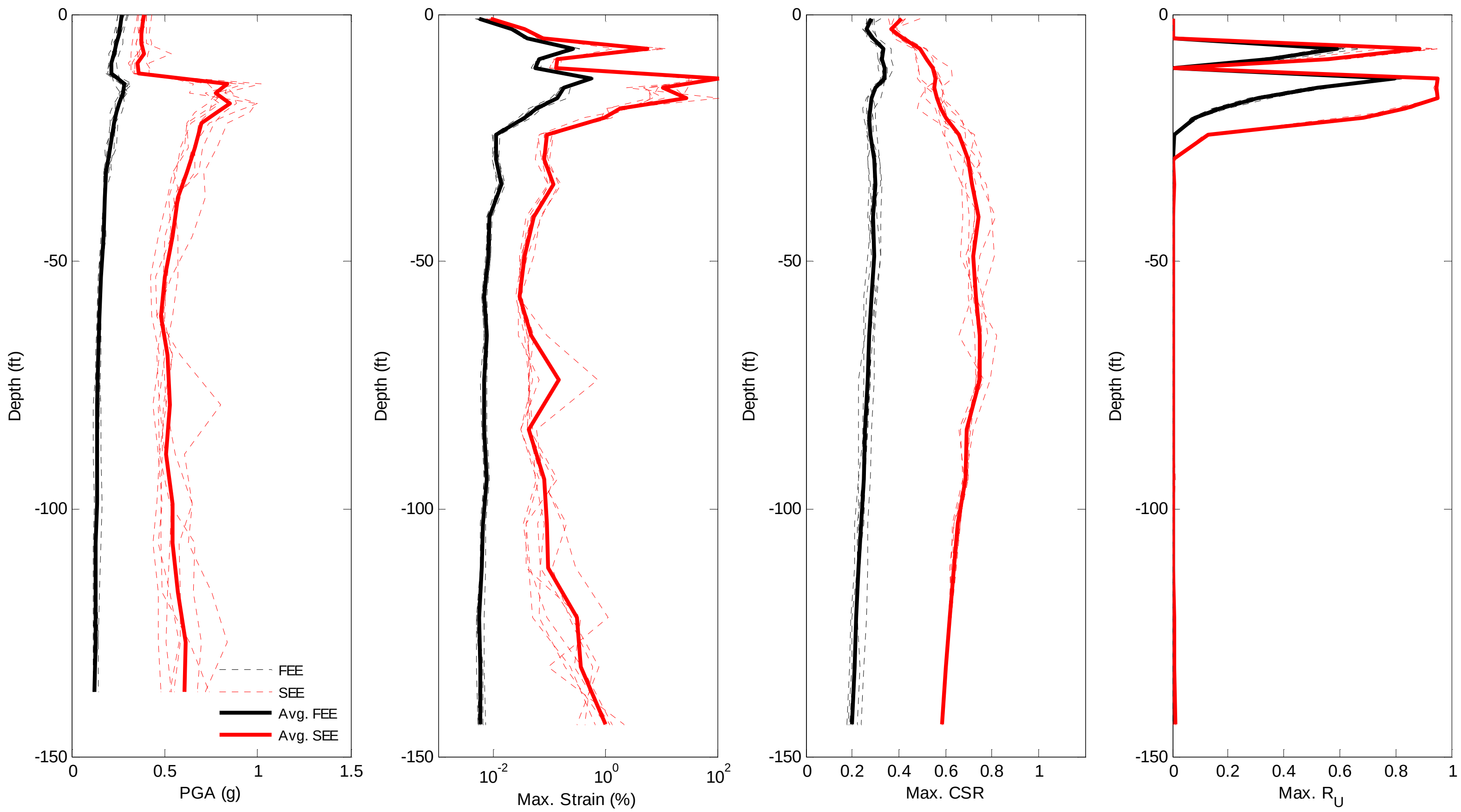

Comparison of sensitivity variations on profile outputs: SEE motions
Depth (ft)
PGA (g)
Max. Strain (%)
Max. CSR
Max. $R_U$
150ft: Total Stress
150ft w/PWP
150ft w/PWP, Higher FC & SS
150ft w/PWP, Lower $V_S$ & SS
500ft: Total Stress
500ft w/PWP
500ft w/PWP, Higher FC & SS
500ft w/PWP, Higher $V_S$ & SS
500ft w/PWP, Lower $V_S$ & SS

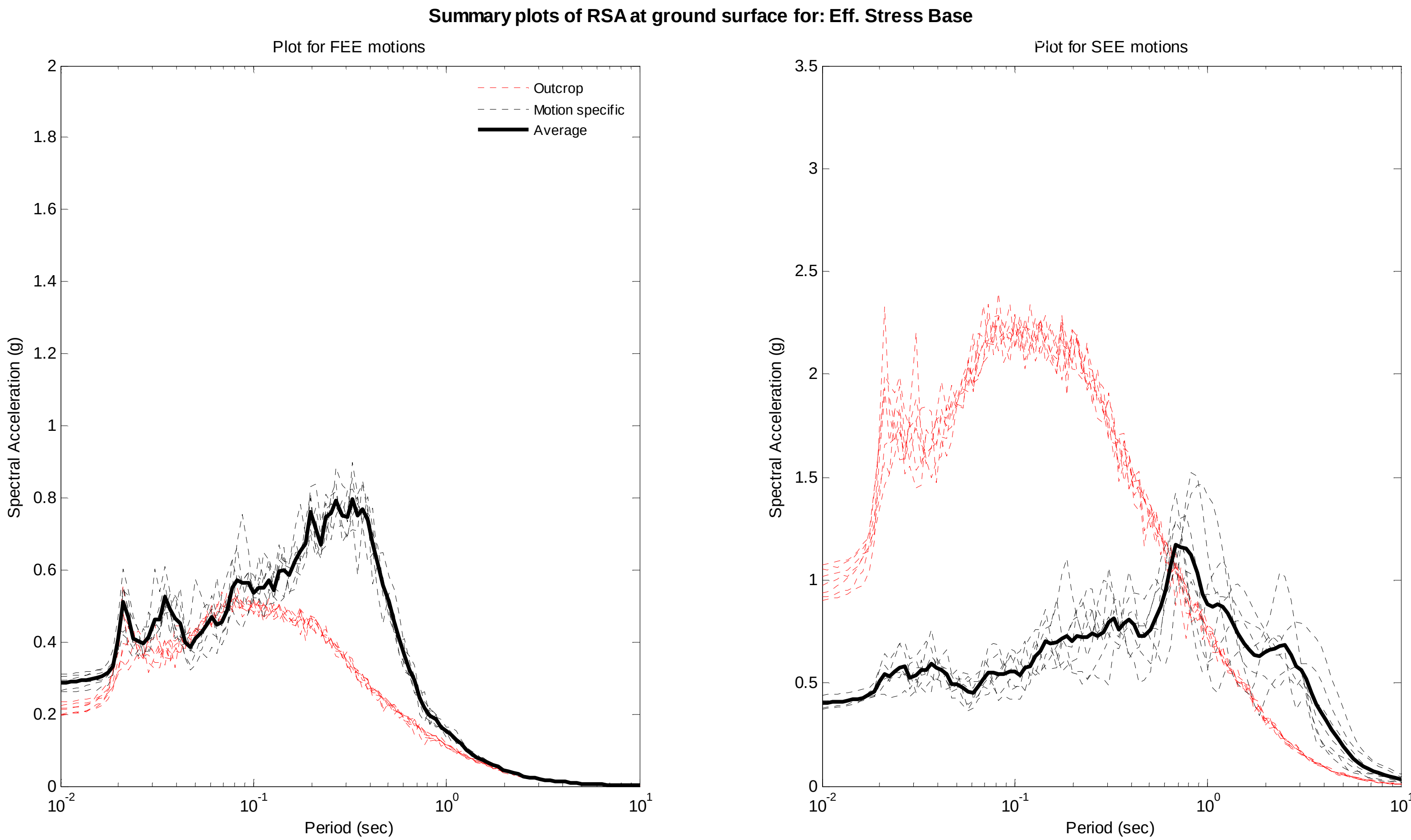
**Comparison of sensitivity variations on RSA outputs: SEE motions**



# ANALYSIS OUTPUTS:
# 150 Ft Deep Profile

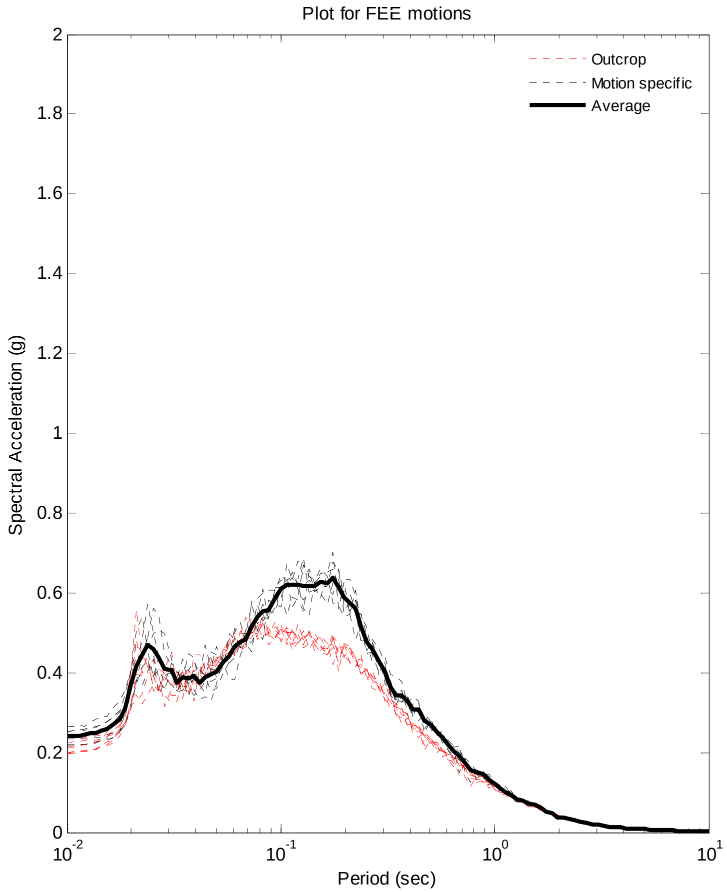
**Summary of profile outputs for: Eff. Stress Base**

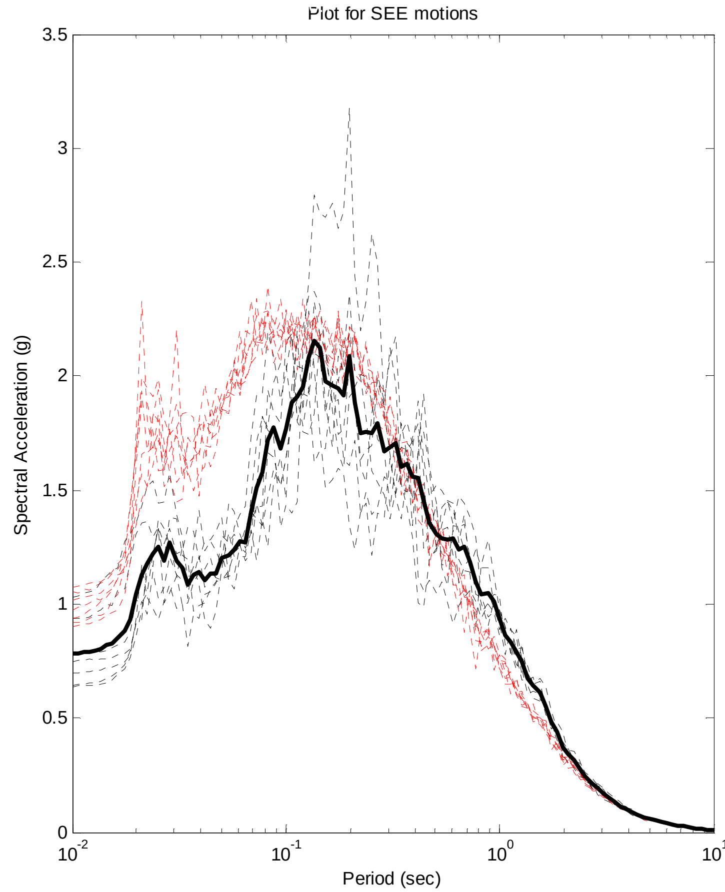




Summary plots of RSA at ground surface for: Eff. Stress Base
Plot for FEE motions
Plot for SEE motions
Outcrop
Motion specific
Average
Spectral Acceleration (g)
Period (sec)

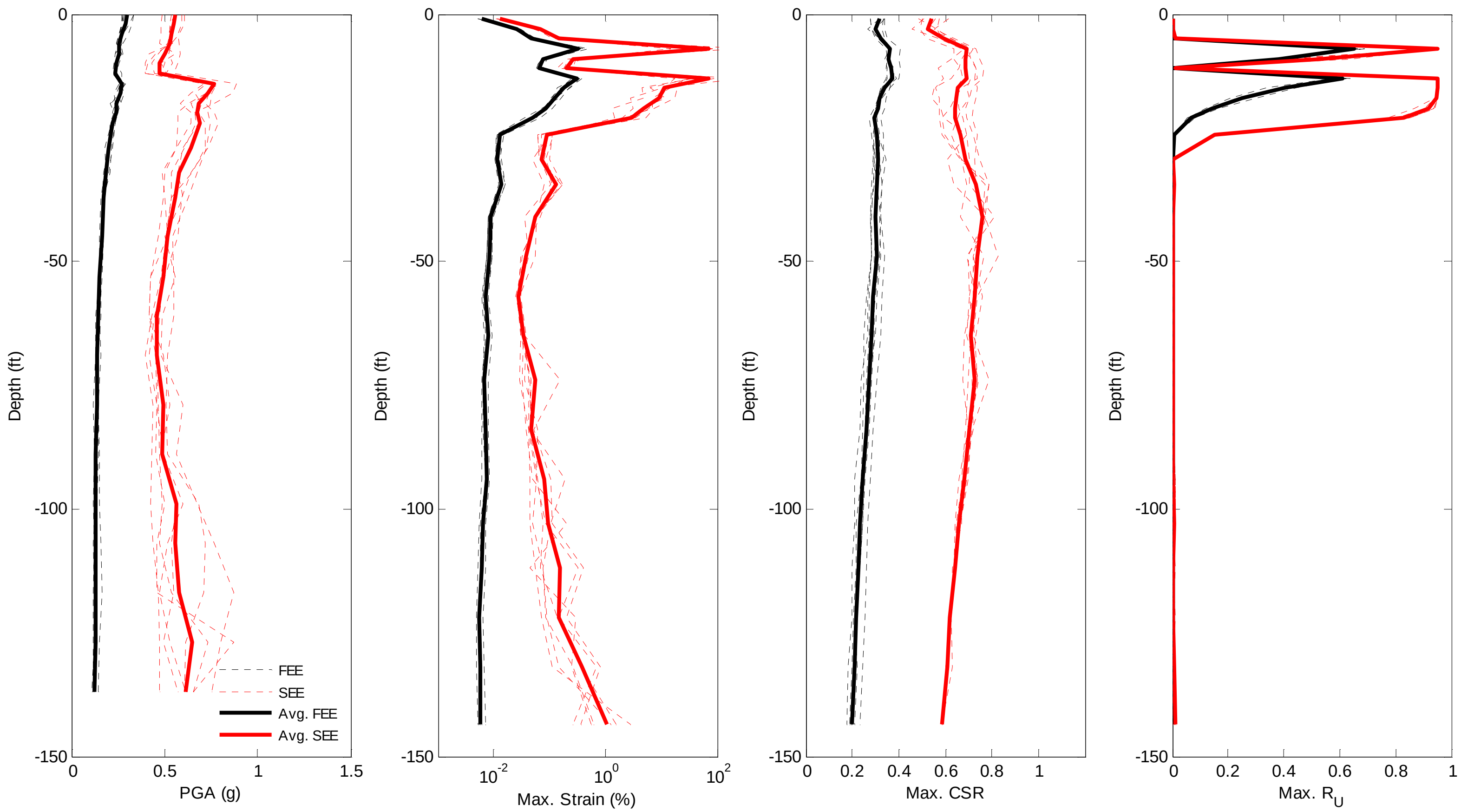
**Summary plots of RSA at top of Marl for: Eff. Stress Base**

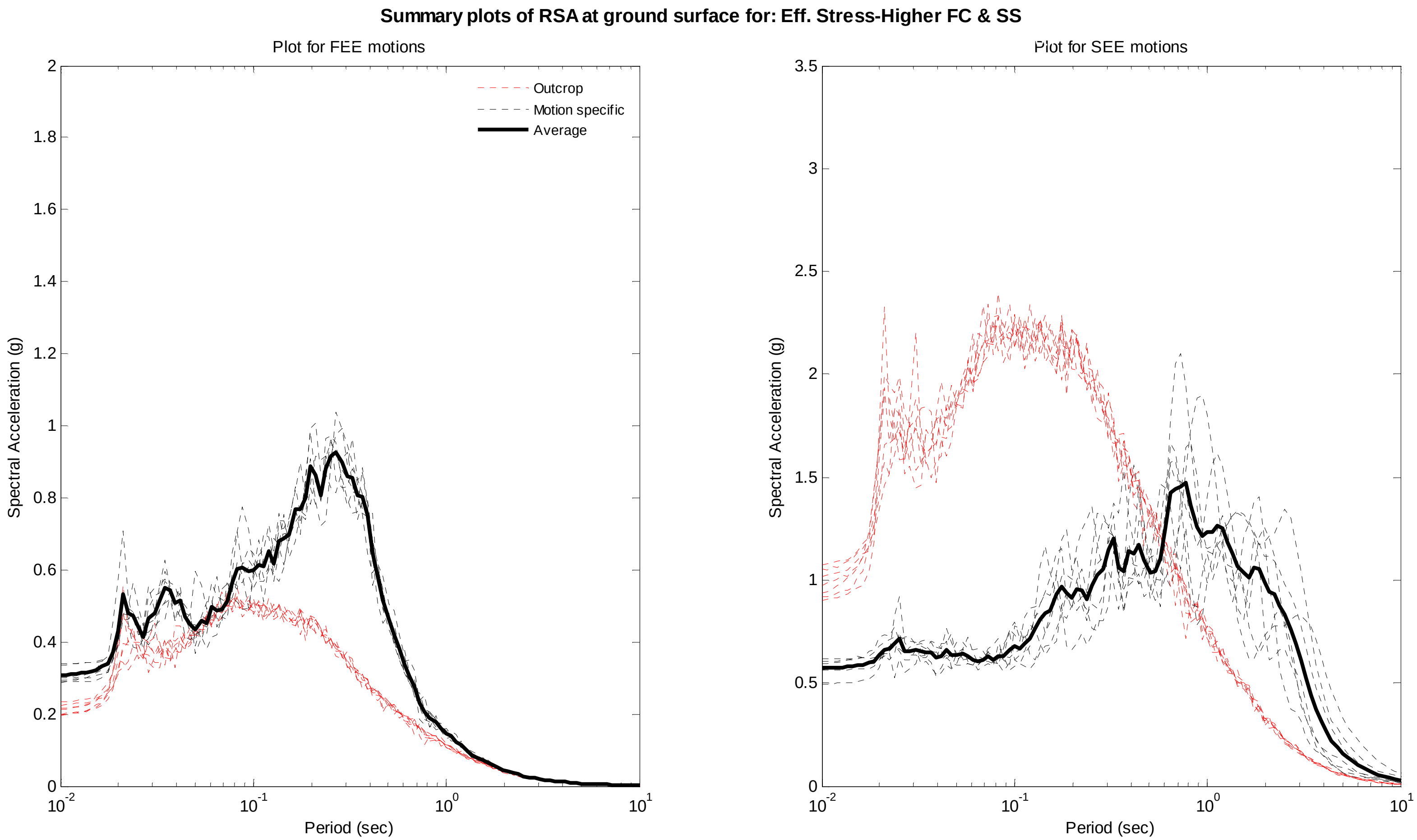


# ANALYSIS OUTPUTS:
# 150 Ft Deep Profile w/ High FC and SS in Sand Layers

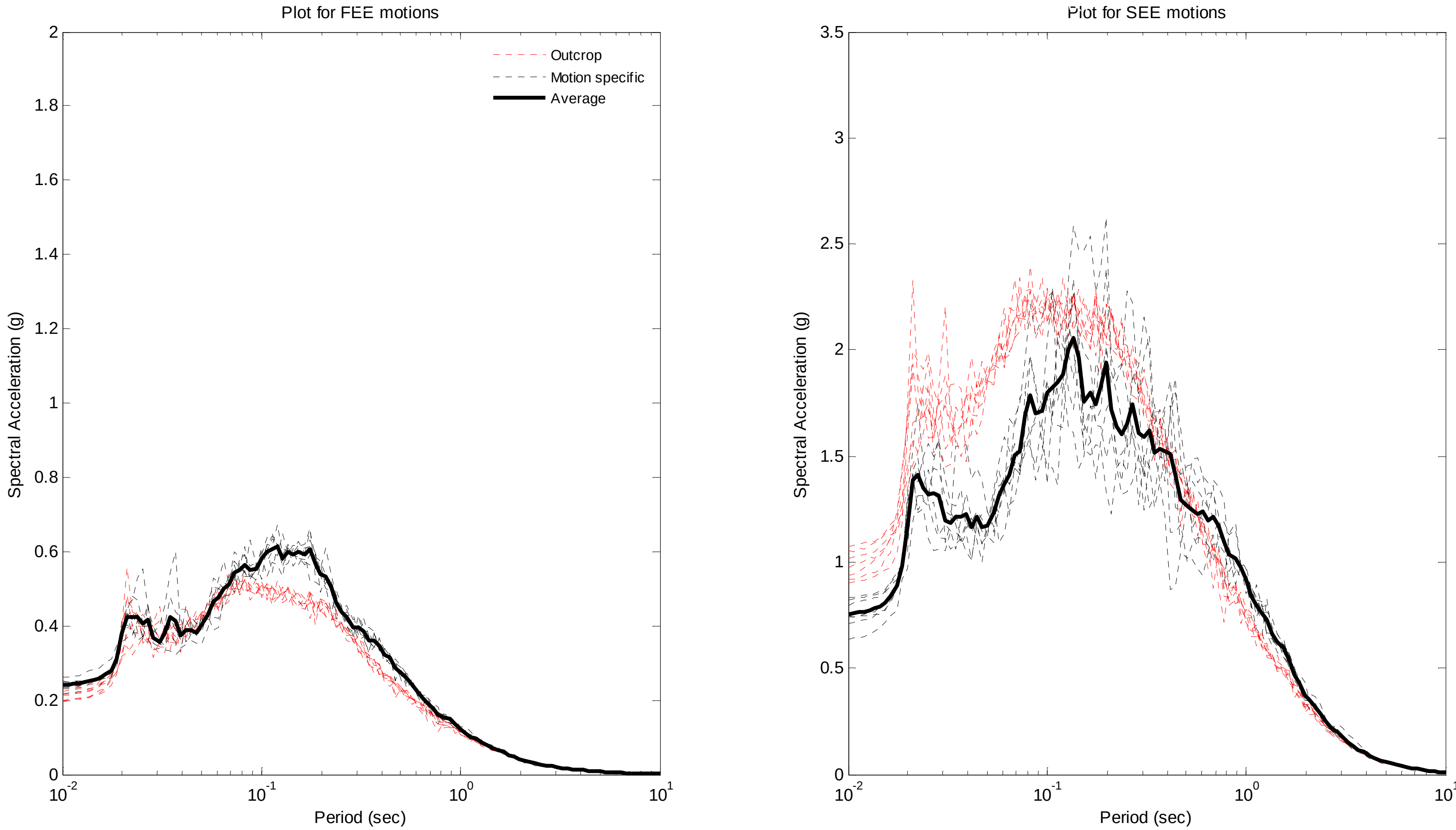
**Summary of profile outputs for: Eff. Stress-Higher FC & SS**





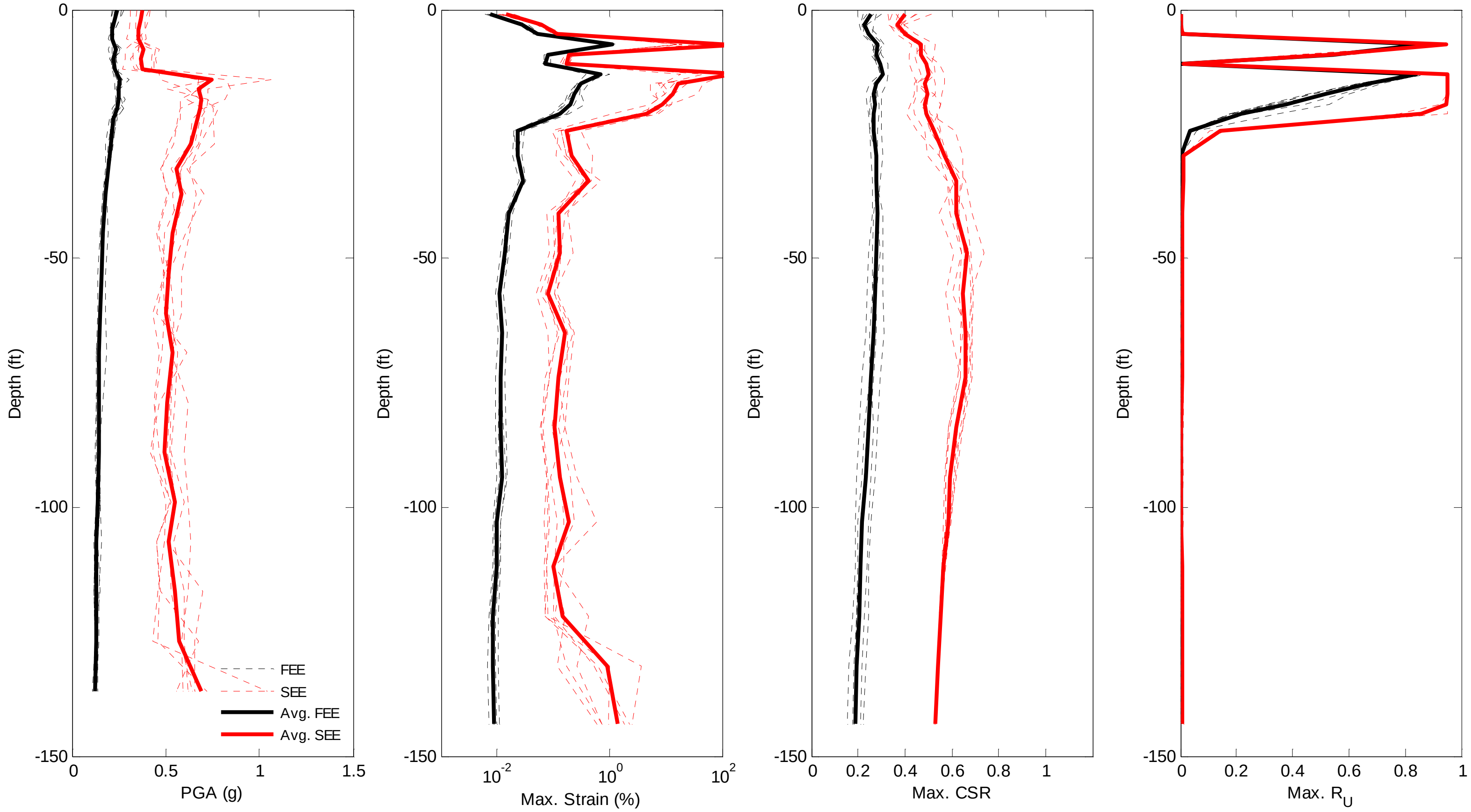
Summary plots of RSA at ground surface for: Eff. Stress-Higher FC & SS
Plot for FEE motions
Plot for SEE motions
Outcrop
Motion specific
Average
Spectral Acceleration (g)
Period (sec)
$10^{-2}$
$10^{-1}$
$10^{0}$
$10^{1}$

**Summary plots of RSA at top of Marl for: Eff. Stress-Higher FC & SS**

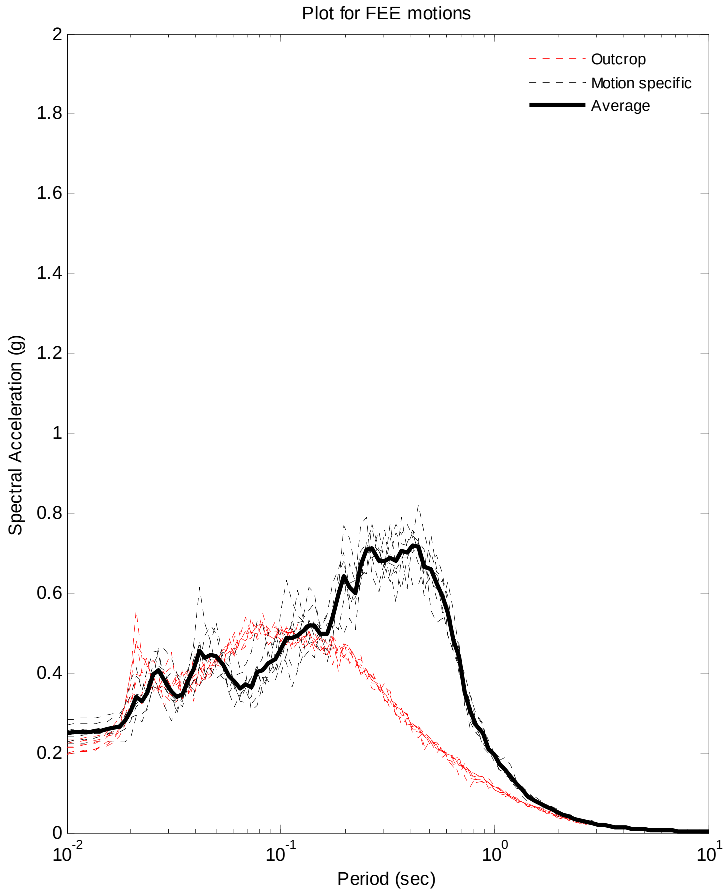


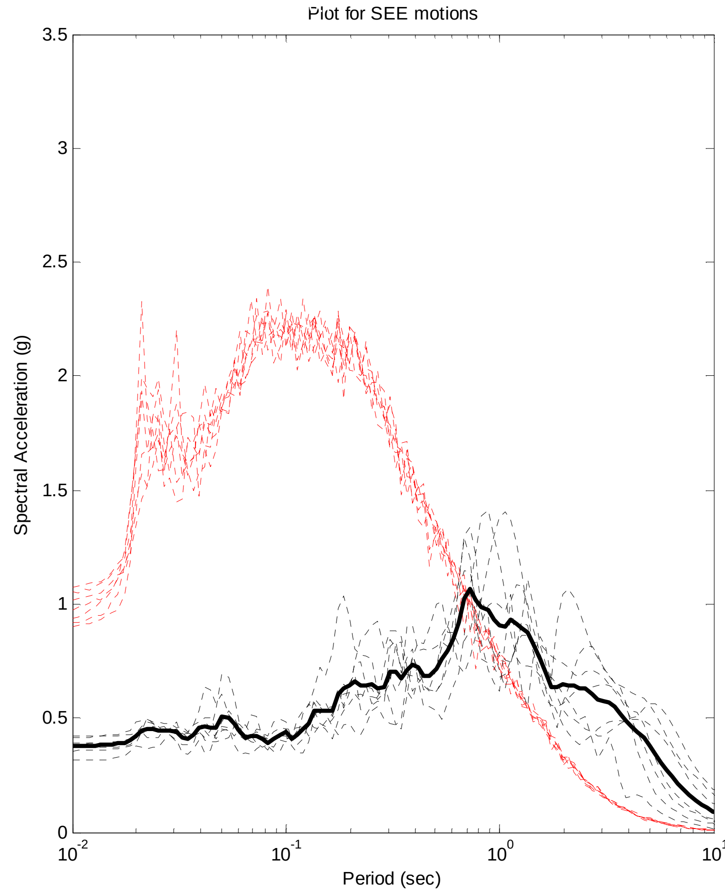
# ANALYSIS OUTPUTS:
# 150 Ft Deep Profile w/ Low $V_S$ and SS

**Summary of profile outputs for: Eff. Stress-Lower $V_S$ & SS**

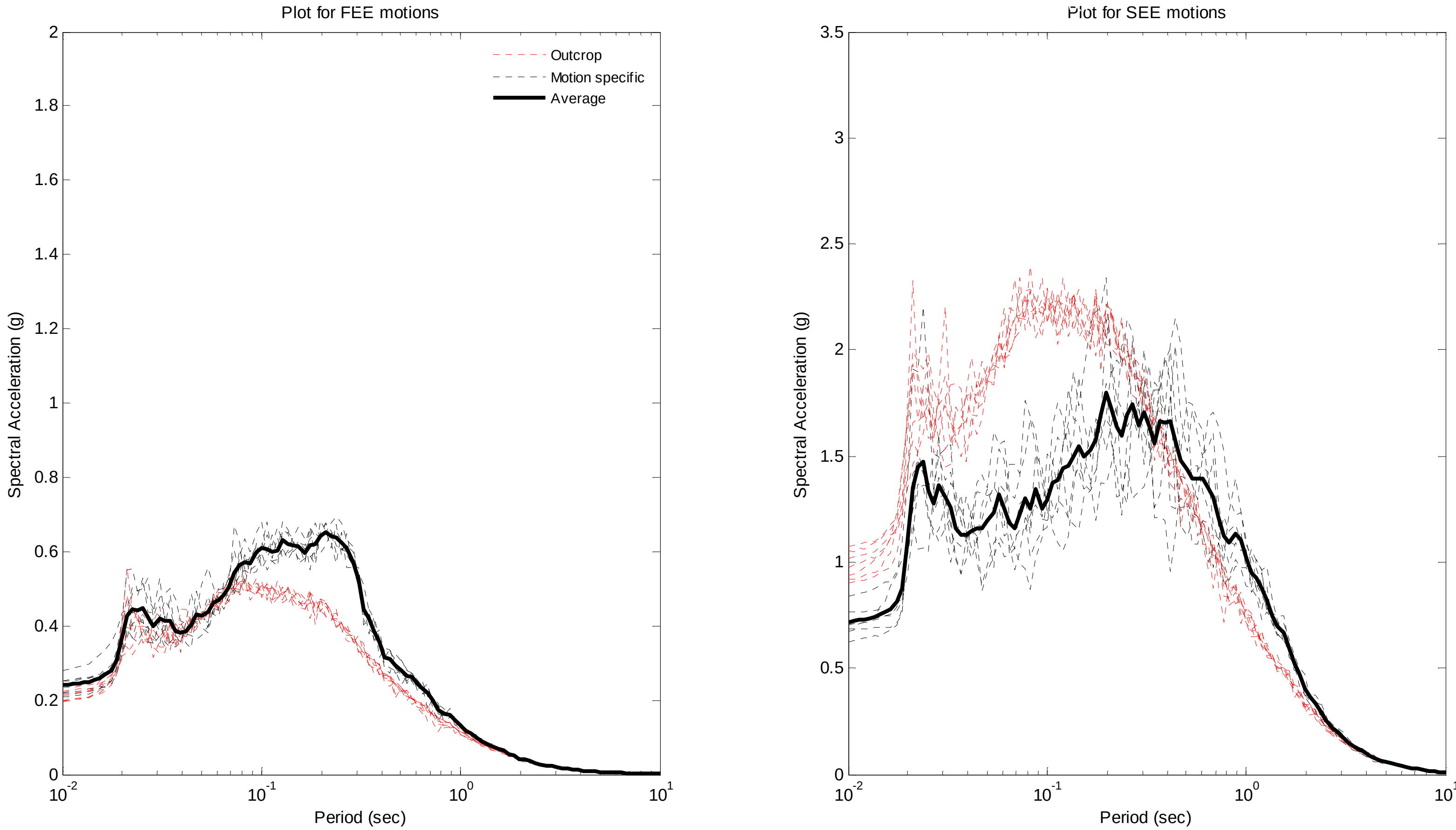


**Summary plots of RSA at ground surface for: Eff. Stress-Lower $V_S$ & SS**

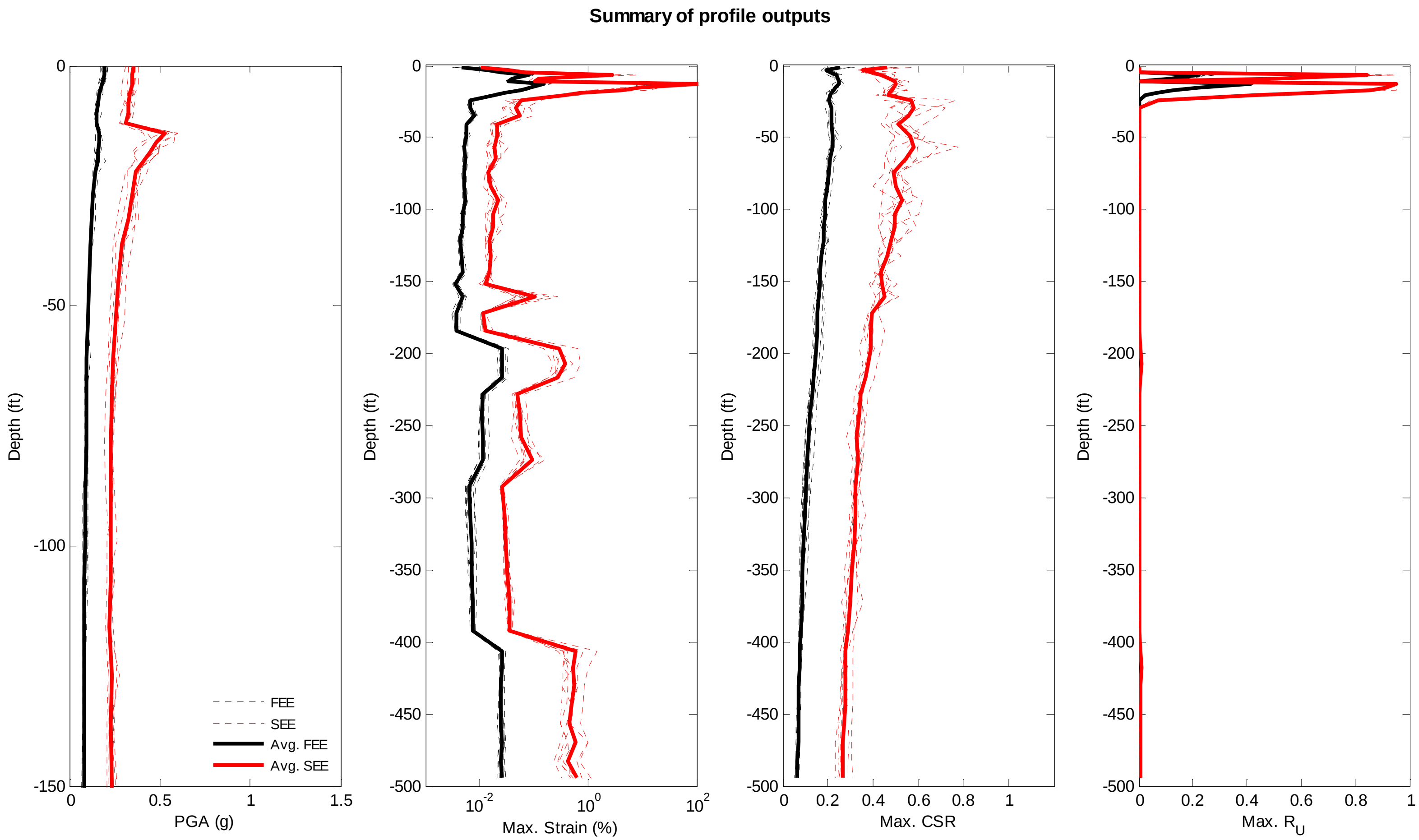


**Summary plots of RSA at top of Marl for: Eff. Stress-Lower $V_S$ & SS**

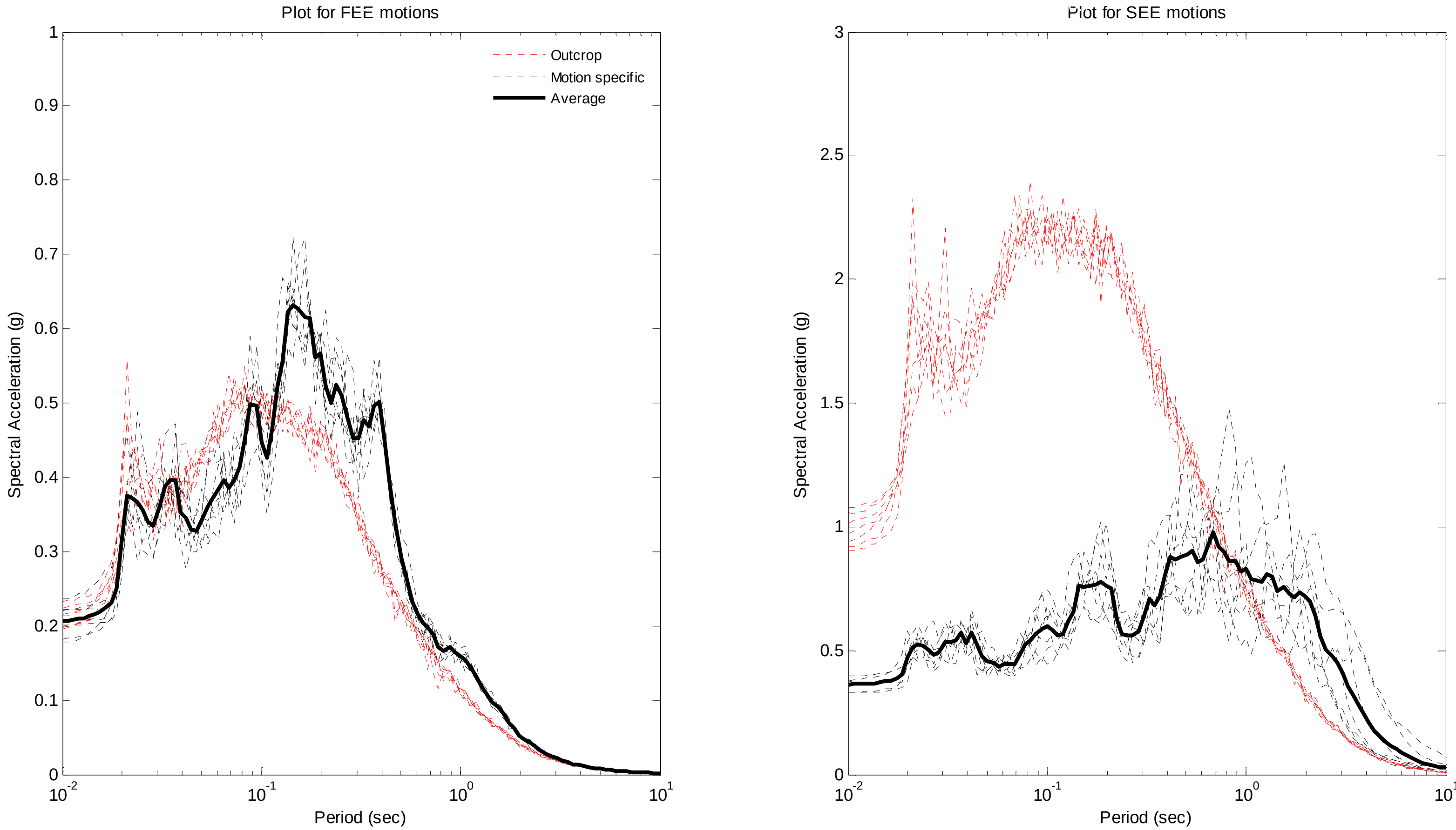


# ANALYSIS OUTPUTS:
# 500 Ft Deep Profile

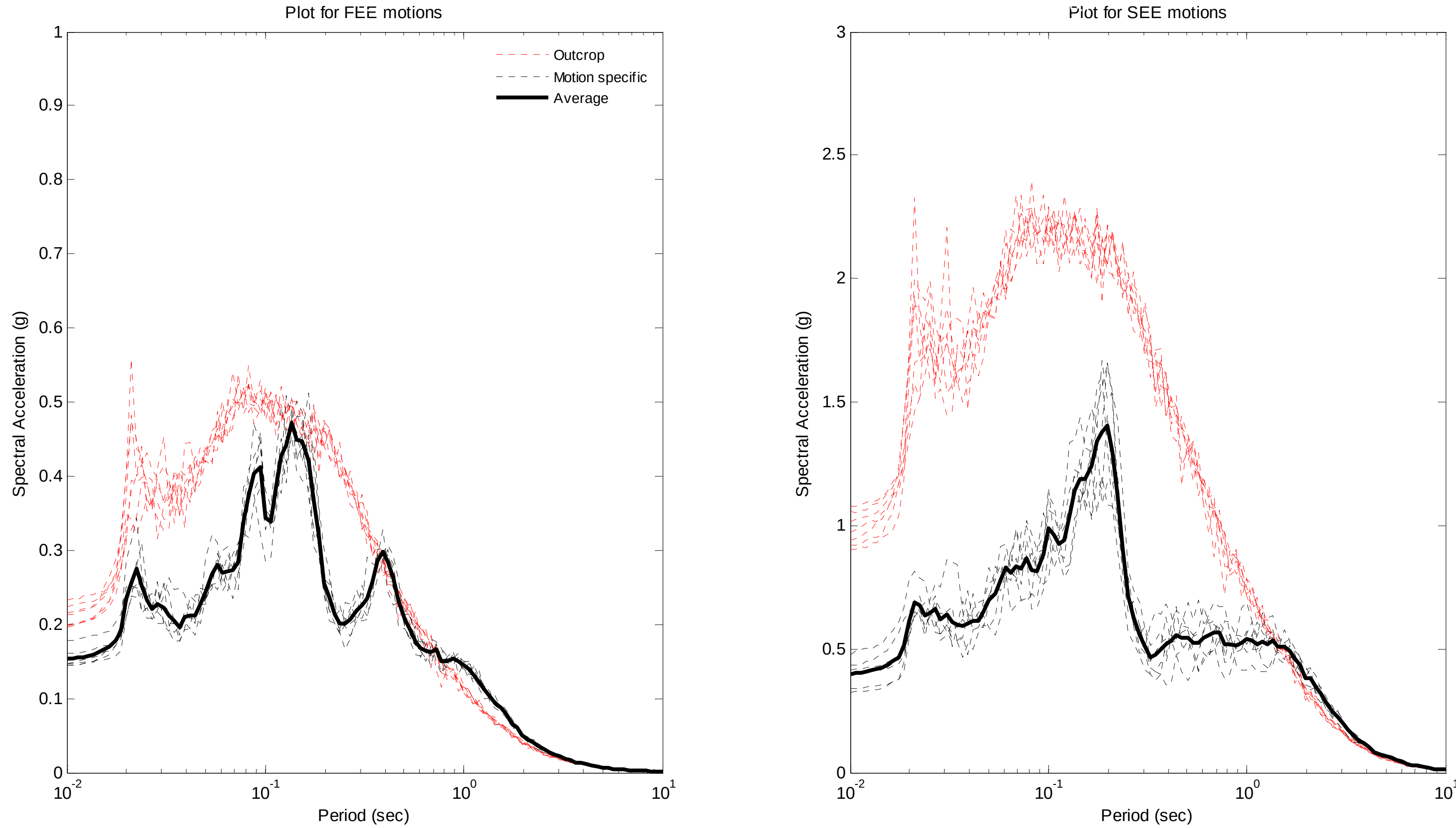


Summary of profile outputs
Depth (ft)
PGA (g)
Max. Strain (%)
Max. CSR
Max. $R_U$
FEE
SEE
Avg. FEE
Avg. SEE

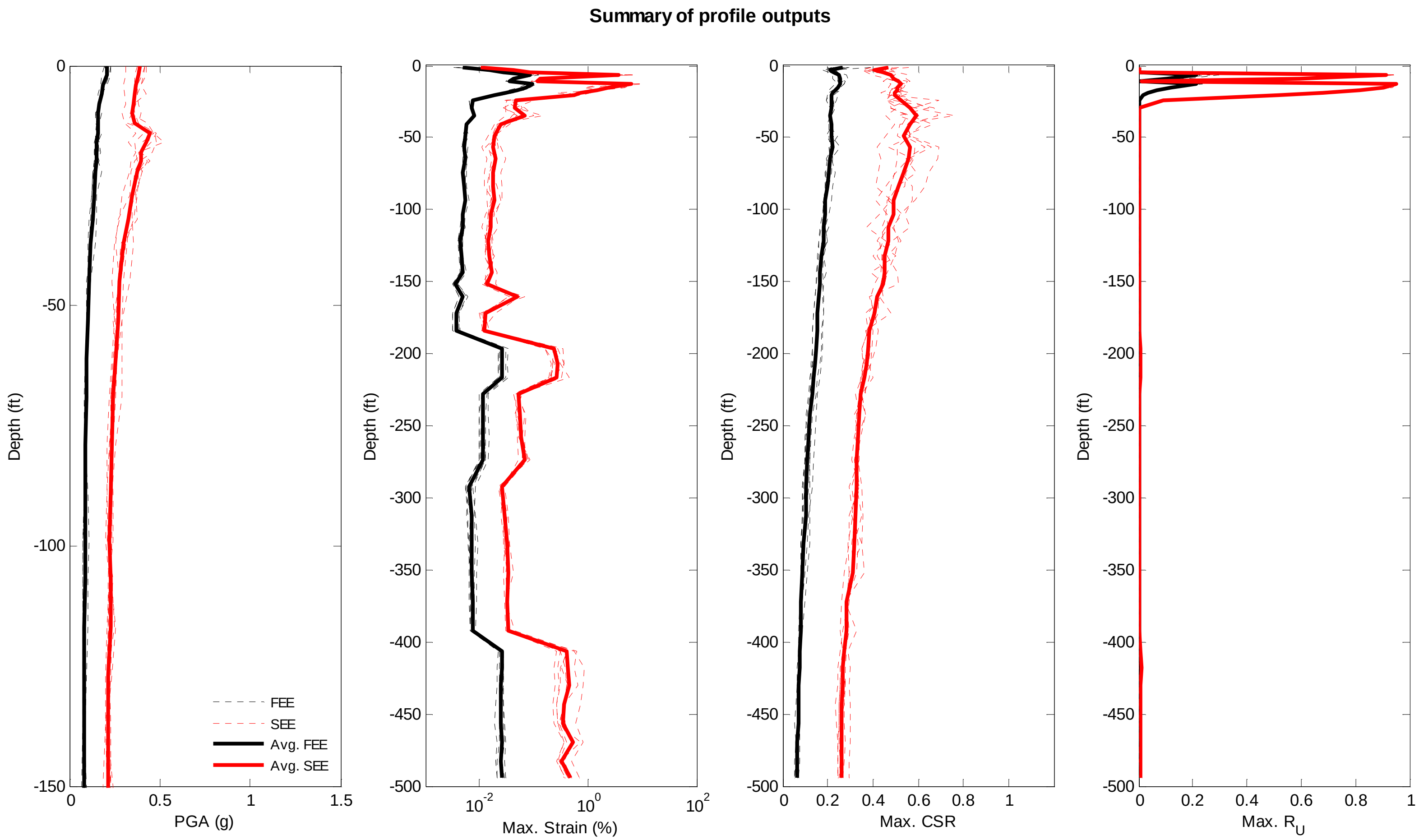
**Summary plots of RSA at ground surface**



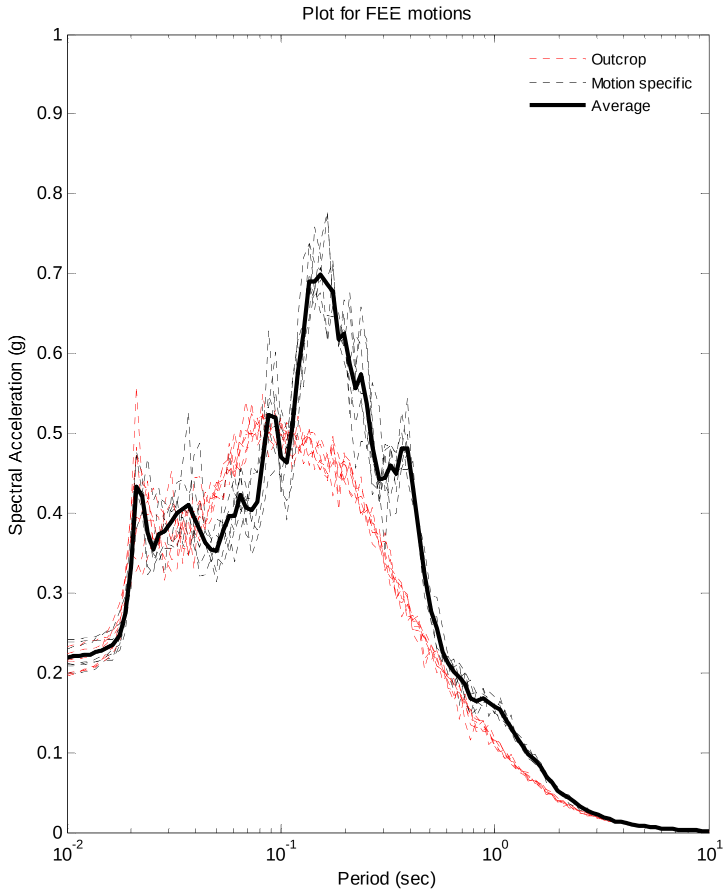
## Summary plots of RSA at top of Marl

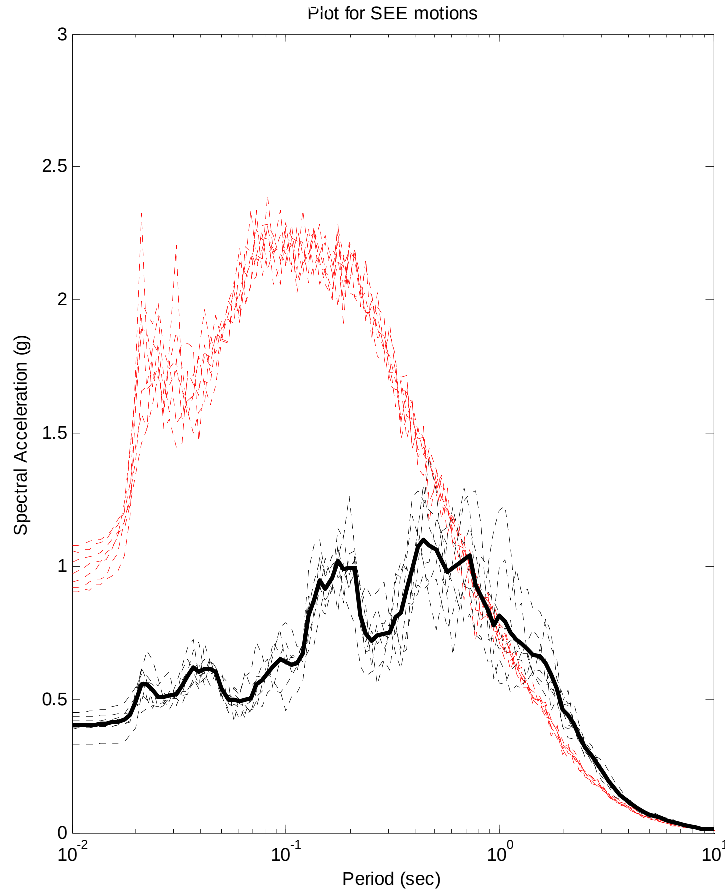


# ANALYSIS OUTPUTS:
# 500 Ft Deep Profile w/ High FC and SS in Sand Layers

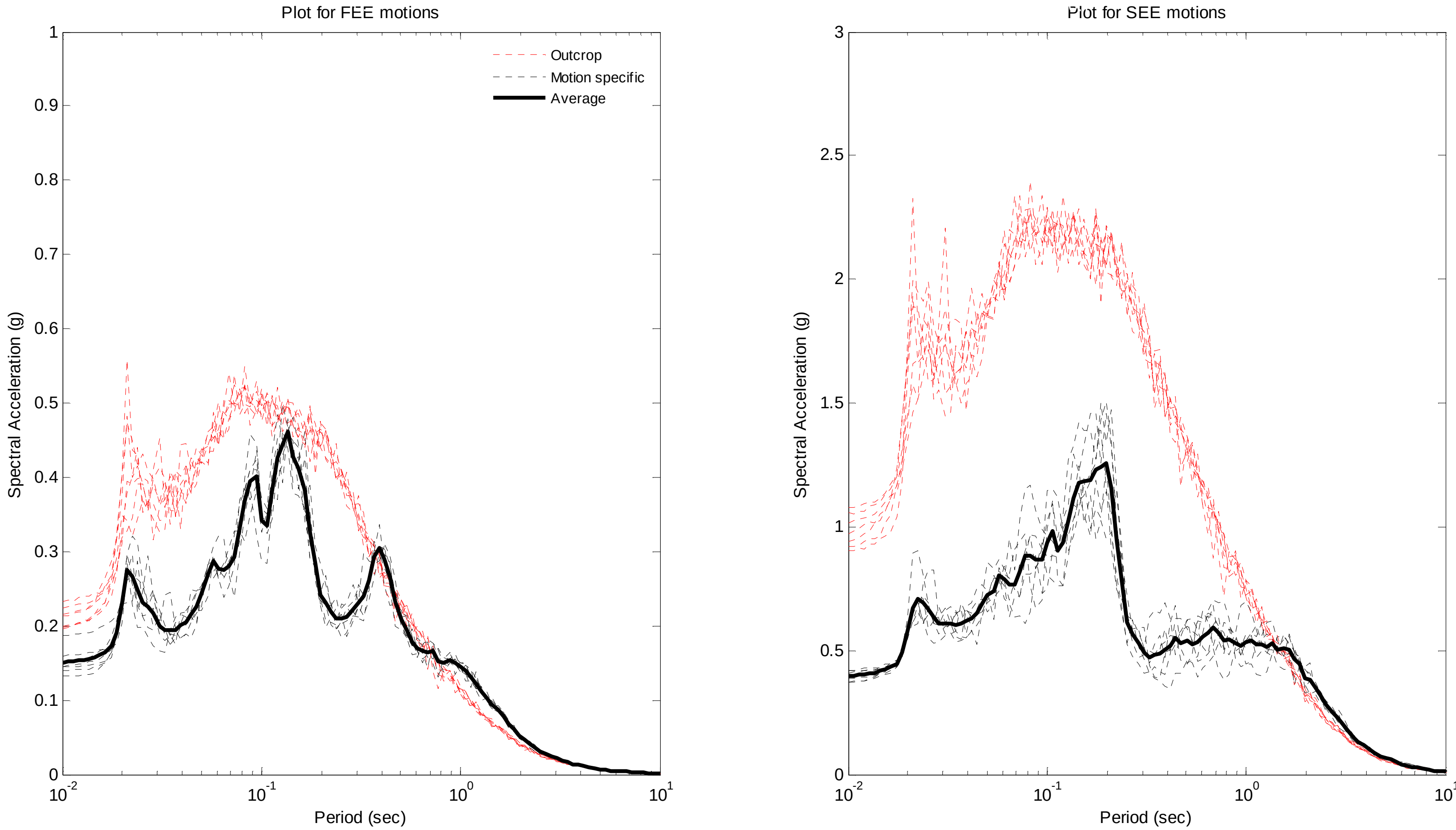

Summary of profile outputs
Depth (ft)
PGA (g)
FEE
SEE
Avg. FEE
Avg. SEE
Max. Strain (%)
Max. CSR
Max. $R_U$

**Summary plots of RSA at ground surface**



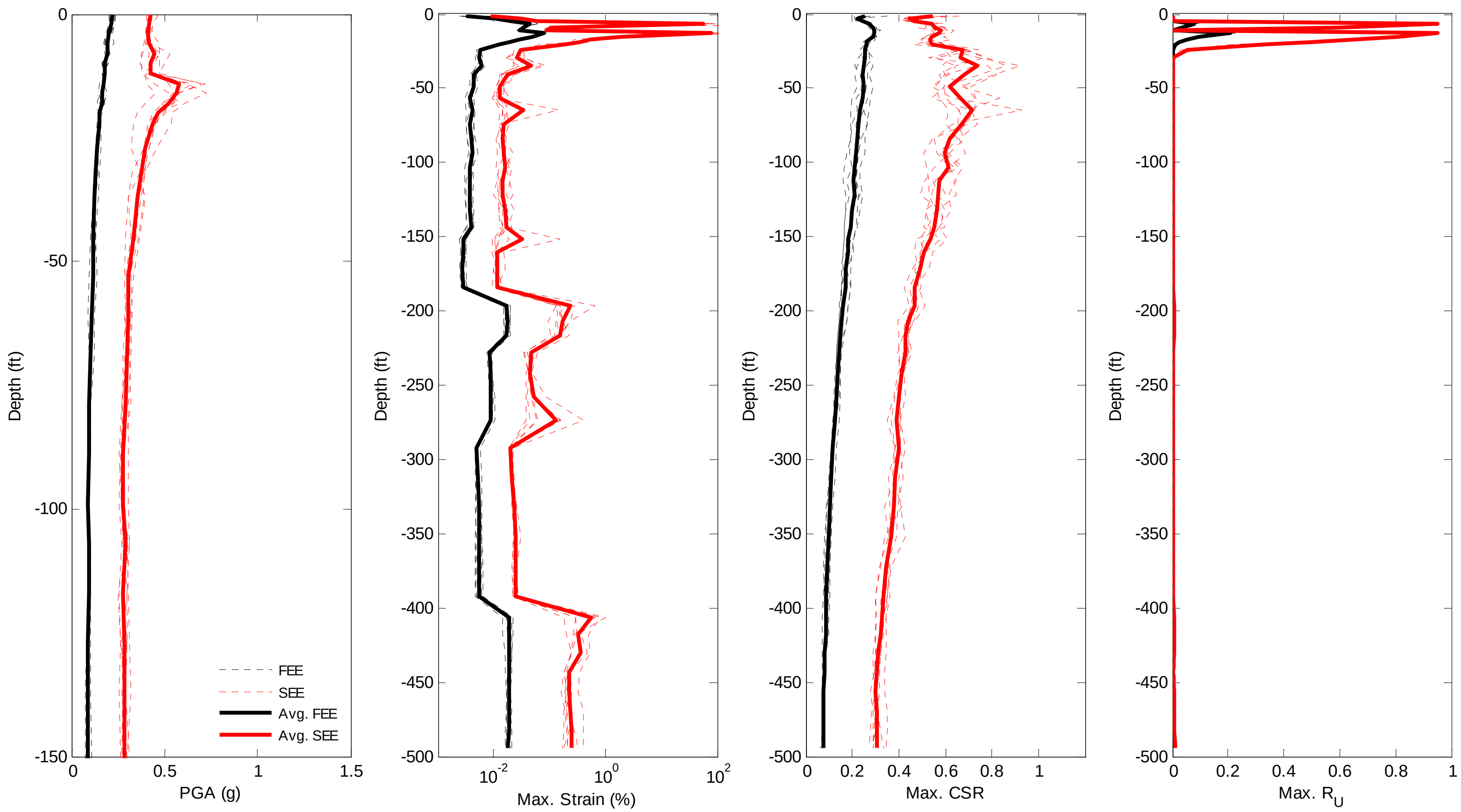



## Summary plots of RSA at top of Marl

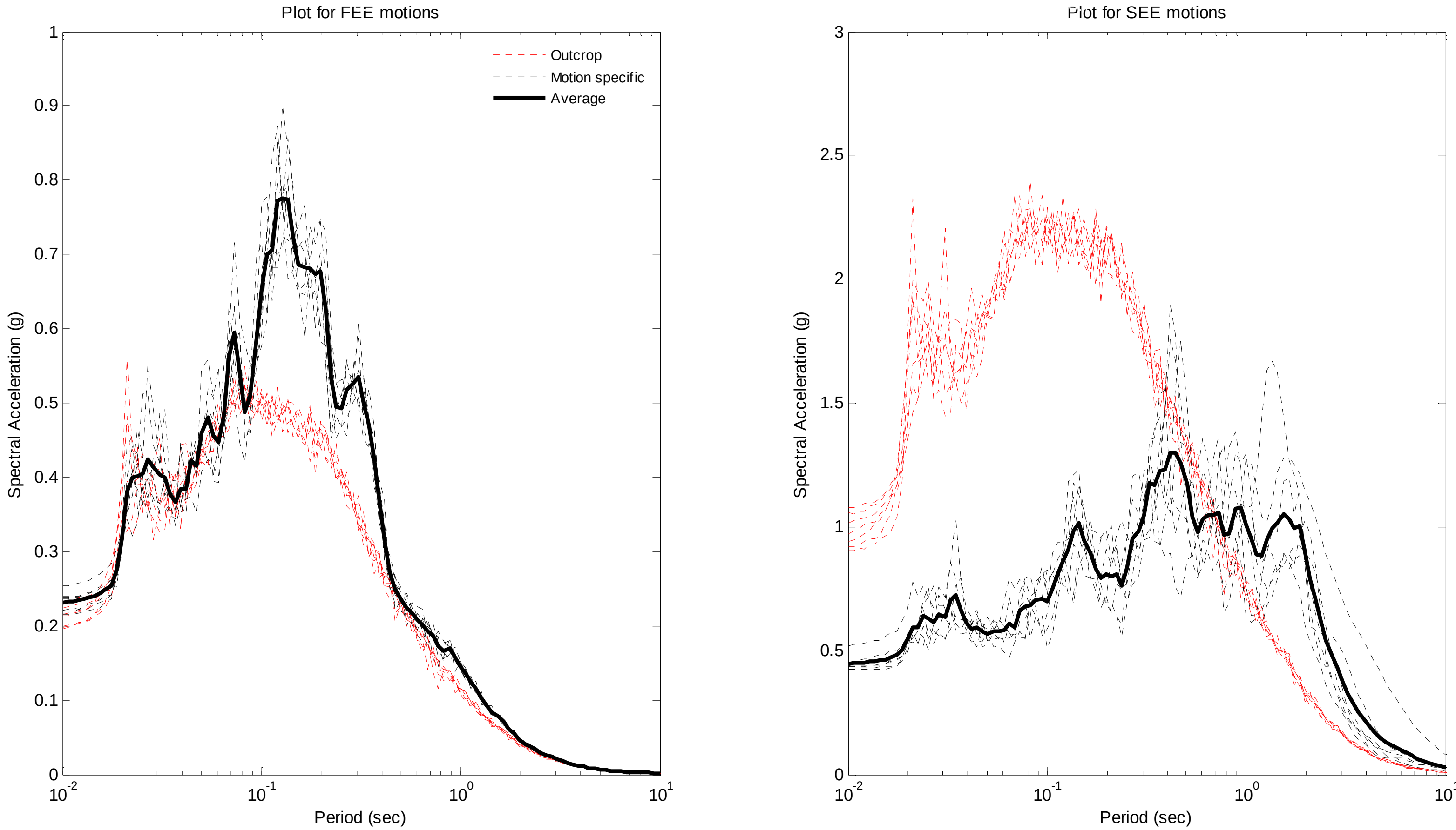


# ANALYSIS OUTPUTS:
# 500 Ft Deep Profile w/ High $V_S$ and SS

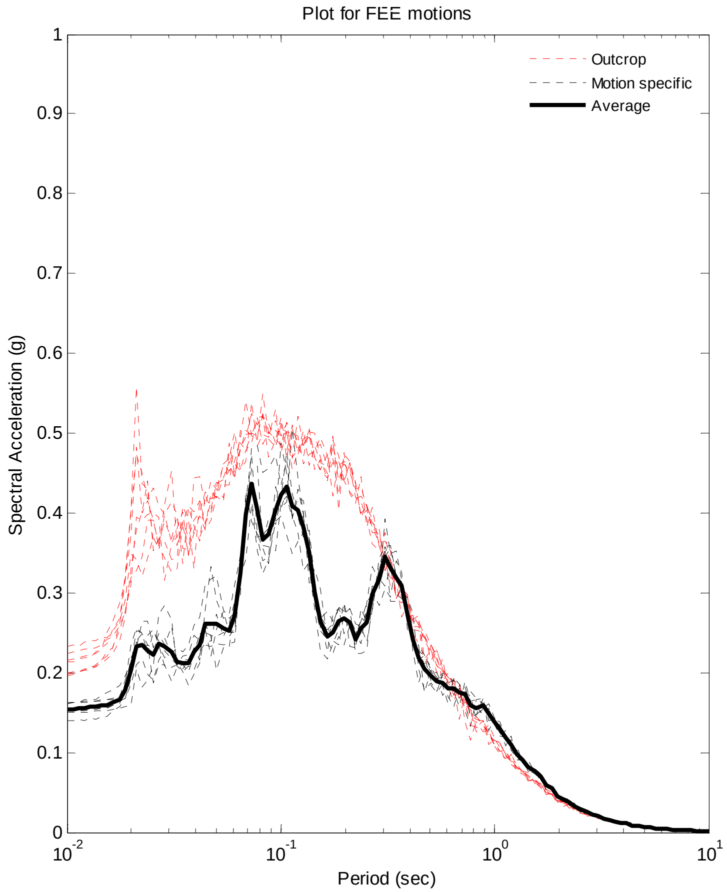
**Summary of profile outputs**

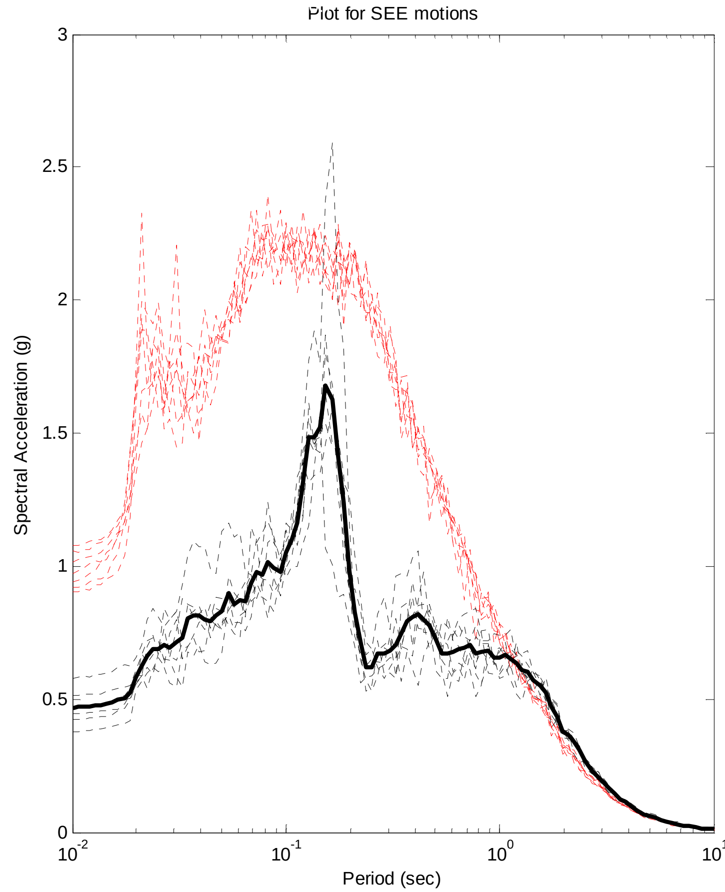


Summary plots of RSA at ground surface



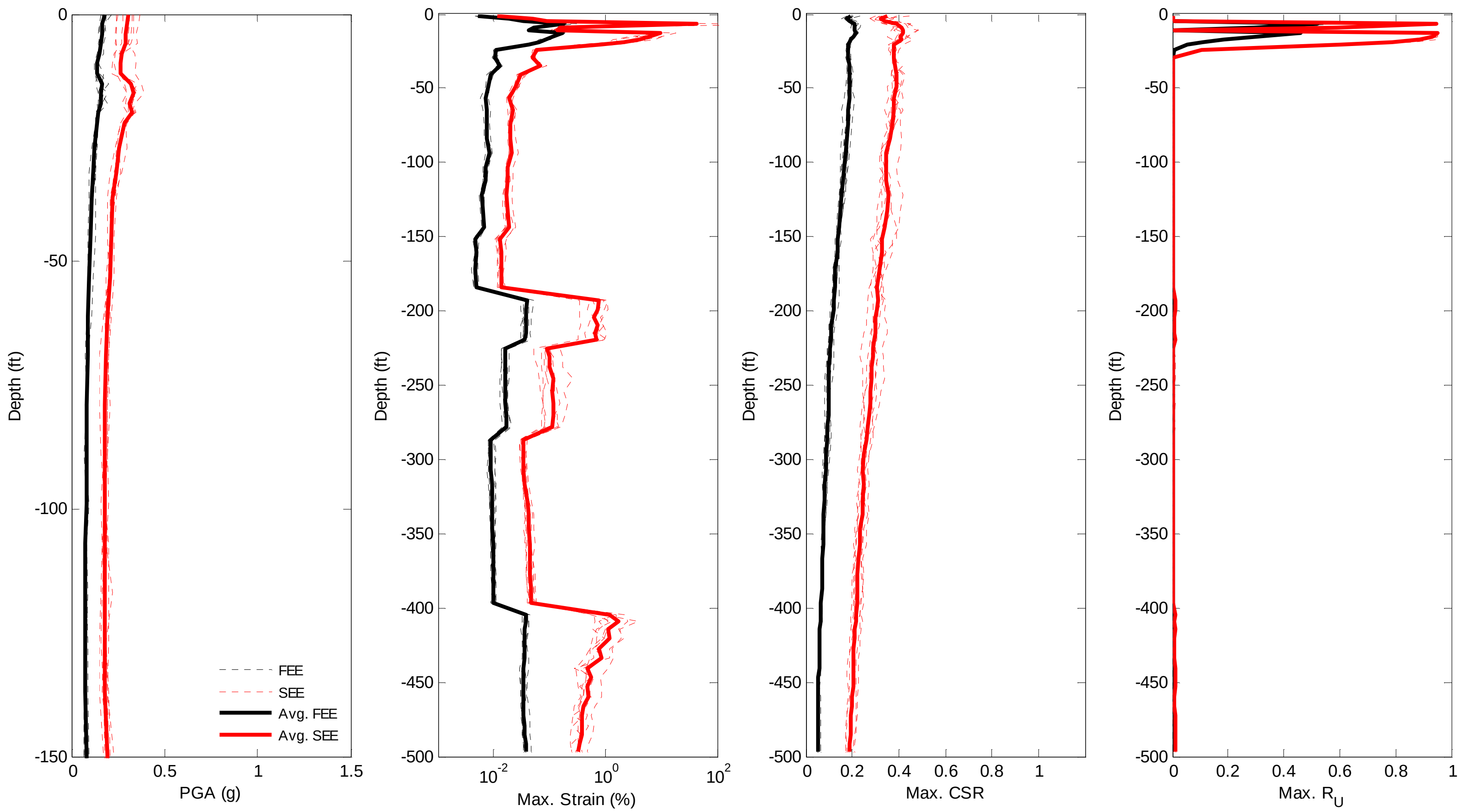
**Summary plots of RSA at top of Marl**



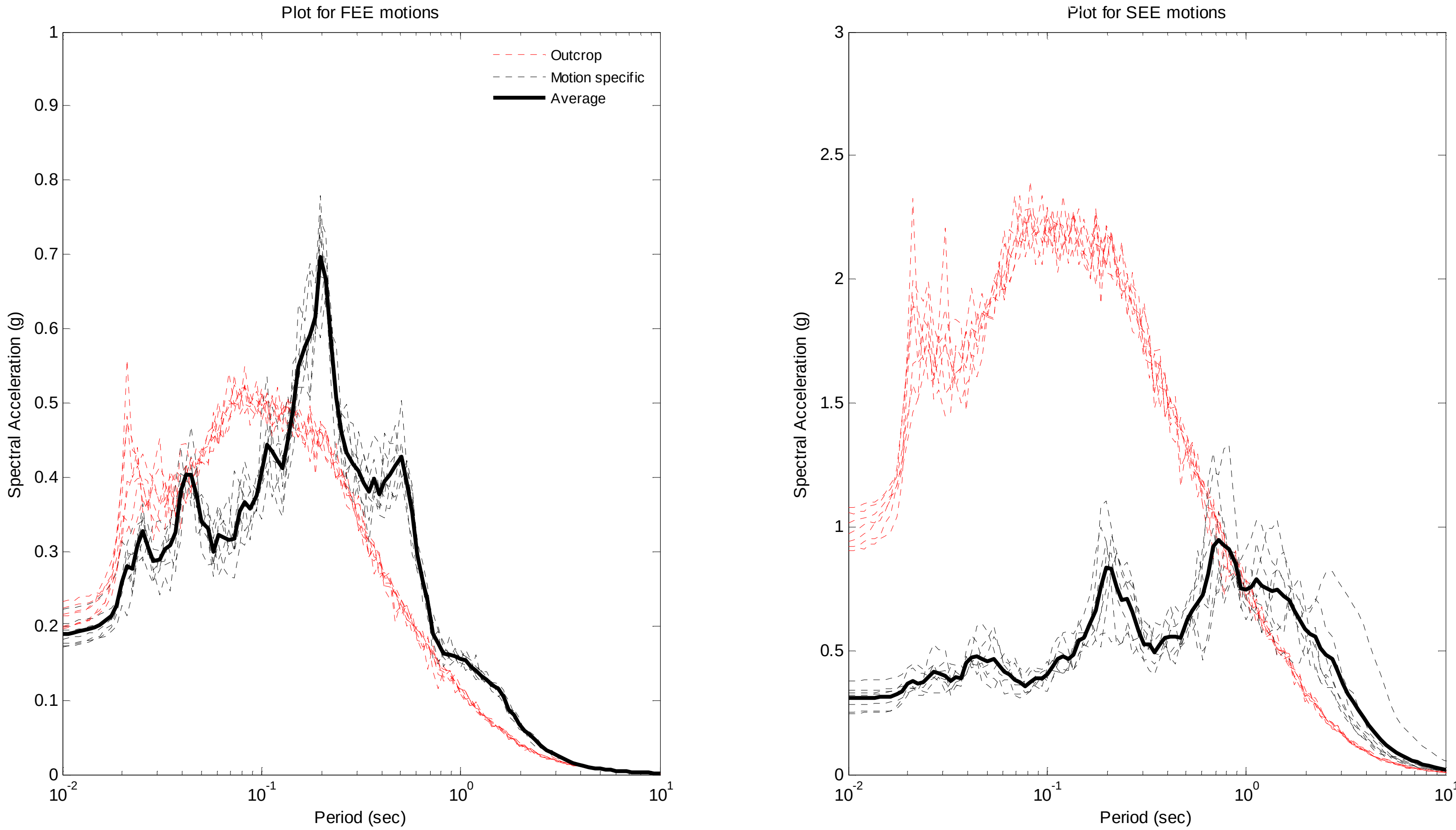



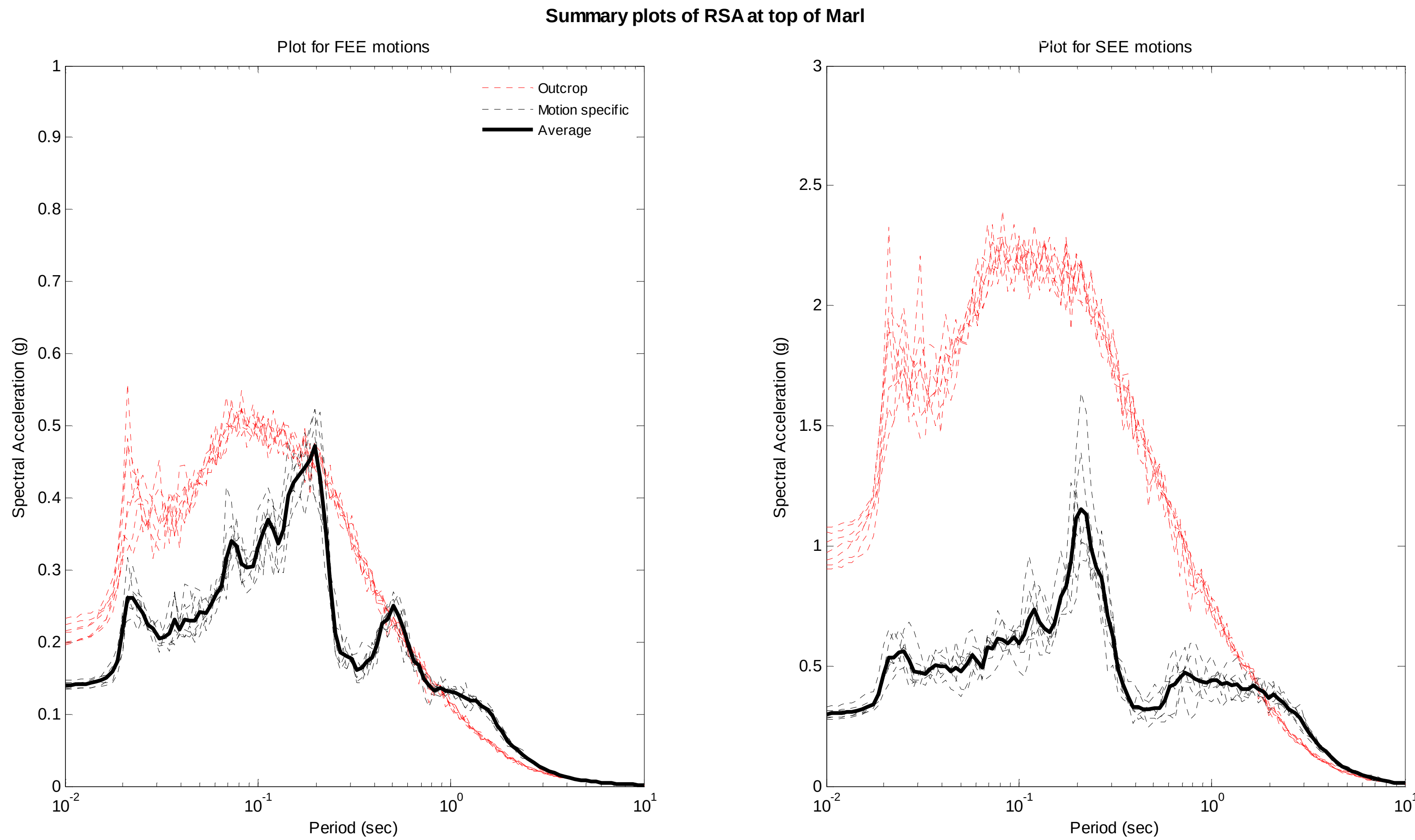
# ANALYSIS OUTPUTS:
# 500 Ft Deep Profile w/ Low $V_S$ and SS

**Summary of profile outputs**



**Summary plots of RSA at ground surface**




Summary plots of RSA at top of Marl
Plot for FEE motions
Plot for SEE motions
Outcrop
Motion specific
Average
Spectral Acceleration (g)
Period (sec)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0.5
1.5
2
2.5
3
$10^{-2}$
$10^{-1}$
$10^{0}$
$10^{1}$

# Appendix IV– Peer Review

1441 Main Street, Suite 1000
Columbia, South Carolina 29201
tel: 803-758-4500

December 11, 2015

Michael Ulmer, P.E.
S&ME
620 Wando Park Boulevard
Mount Pleasant, SC 29464

Subject: I-26 Volvo Interchange SSRA Peer Review

Dear Michael:

As requested, we have performed a peer review of the Site-specific Seismic Response Analyses (SSRA) report prepared by S&ME for the I-26 Volvo Interchange project, dated December 11, 2015. Our review was conducted in accordance with our proposal dated July 1, 2015. A draft of the SSRA report was provided for our review previously, and our comments related to the review of the draft report were submitted to S&ME in a letter dated December 4, 2015. We have also conducted discussions with S&ME via telephone and email to clarify our comments related to the draft report.

Based on our review of the revised (final) report, it is our opinion that the SSRA were conducted in conformance with the standards developed by SCDOT for this project. We also concur with the development of the smoothed site-specific ADRS based on a "Base Model" with the B-C Boundary located at a depth of 500 feet.

We appreciate the opportunity to be of service to you on this project. Please let us know if you have any questions or comments.

Sincerely,

Sanjoy Chakraborty

Sanjoy Chakraborty, Ph.D., P.E.

Senior Geotechnical Engineer

CDM Smith Inc.

cc: Jennifer Humphreys (CDM Smith)