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ESTABLISHING OUTCROP TIME SERIES AT VARIOUS DEPTHS IN A NONLINEAR SOIL COLUMN: AN ITERATIVE APPROACH

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Abstract ---- *Seismic soil-structure interaction (SSI) analysis of nuclear facilities is an important consideration during design and retrofit. SSI tools used in the nuclear industry are currently based on an equivalent linear approach. Procedures for developing input ground motion for equivalent linear approaches are well established. However, the procedures for establishing input ground motion for Nonlinear Soil-Structure Interaction (NLSSI) analysis of nuclear facilities is not well established. A collaborative research group at Idaho National Laboratory (INL) has recently developed analytical methods and numerical tools for using NLSSI analysis for nuclear facility seismic calculations. NLSSI analysis for a nuclear facility allows for calculation of seismic wave motion through a near field soil domain using either, (1) vertically propagating shear and compressive waves, which is the current industry practice, or (2) a three-dimensional non-vertical wave field. This paper presents an iterative procedure for establishing outcrop motion at a depth in the soil column for NLSSI analysis that uses vertically propagating shear waves.*

The approach presented in this paper starts with a known ground motion at the surface that is deconvolved to a depth, and then the obtained motion is convolved up to a different desired location of input for the NLSSI model. To demonstrate the validity of the approach a finite element soil column, that is representative of a nuclear facility site in the US, is used to produce compatible outcrop seismic time series for reduced nonlinear soil mesh depths. The developed approach for reducing the nonlinear soil column model depth is a two-step iterative method; 1) the first step is establishing an outcrop time series at the lowest depth considered that produces the top-of-soil response spectrum of an actual recorded ground motion, and 2) the second step is providing compatible outcrop time series at a shallower depth based on the information from the first step.

The comparison of the 5% damped response spectrum from the resulting acceleration time series based on the iterated outcrop motions and the original acceleration time series are conducted. The study showed that the proposed iterative approach produced comparable results within 1% range of the original recorded time series results when sufficient iterations were performed.

Keywords: Nuclear facilities, nonlinear, site response analysis

I. INTRODUCTION

Nuclear facilities and other safety-critical structures need to be designed to withstand the displacements and forces resulting from earthquakes. Earthquake resistant design of nuclear facilities and other safety-critical facilities requires professionals from multi-disciplinary fields working together; such as seismologists, geotechnical earthquake engineers, and structural engineers. There has been significant research conducted related to seismic analysis, design, and assessment of nuclear facilities in the last six decades since nuclear energy illuminated the first light bulbs at EBR-I in Idaho, USA. The research since then, paved the road of knowledge on how to analyze and design nuclear power plants (NPPs) and nuclear facilities to resist the damaging effects of earthquakes.

Nuclear regulatory commission (NRC) is the government agency in the US responsible for protecting public health and safety related to nuclear energy. Title 10, code of federal regulations (10 CFR) covers the requirements binding on all persons and organizations who receive a license from NRC to use nuclear materials or operate nuclear facilities. 10 CFR Part 50 is the document related to the domestic licensing of production and utilization facilities. 10 CFR Part 50 Appendix A is the general design criteria for NPPs. 10 CFR Part 50 Appendix A states that the structures, systems, and components (SSC) need to be designed against the natural phenomena including earthquakes. 10 CFR Part 50 Appendix S includes the earthquake engineering criteria for NPPs. 10 CFR Part 100 covers the siting factors and criteria for proposed sites for stationary power and testing reactors subject to part 50. 10 CFR Part 100 Appendix A states that the seismic design consideration including the effects of soil-structure interaction.

Seismic design of a nuclear facility starts with defining the seismic hazard at a location in the site specific soil domain, typically at the location of competent rock. The site specific seismic hazard curves are calculated at multiple frequencies. Then, a uniform hazard response spectrum (UHRS), given an earthquake ground motion return period, is developed at the competent rock location. Finally rock outcrop motions are developed by fitting the UHRS. Guidelines for developing seismic hazards for nuclear facilities are provided in NUREG-Series.[1]

Next, site response analyses are conducted to calculate the transfer of the rock outcrop motion to a location where it will be input into the NLSSI model. The depth of a nonlinear soil domain is dependent on the vertical soil stiffness when the model is in static or dynamic equilibrium. The number of degrees of freedom and elements in the nonlinear soil column is directly related to the depth of the soil column. It is important to limit the size of the meshed soil domain to reduce model run times. Hence, the soil properties from the bedrock to the free surface have influence on the characteristics of the motions experienced at the free surface. The influences may be amplifications or degradations of the ground motions at the free surface depending on the frequency content of the ground motion and site soil under consideration.

A case study is provided in this paper to demonstrate an approach for defining acceleration time series at a location other than the input location (typically the level of input for NLSSI analysis). The case study uses a nonlinear time domain one-dimensional site response approach and a horizontal acceleration time series input motion. Figure 1 shows a sketch of the steps for establishing the outcrop motions at different elevations during the case study. In the first step of the case study, an approximate outcrop acceleration time series is iteratively established at 486 feet deep in the soil column provided that the free-field surface motion is known. In the second step of the case study, an approximate outcrop acceleration time series is iteratively established at 220 feet deep in the soil column based on results from the 486 feet soil column model. The case study only considers one horizontal direction but it is applicable to all three directions. The case study uses a surface acceleration time series recorded during the 5.8 magnitude, Mineral, VA earthquake of August 23, 2011 as measured at Corbin, VA.[2]

II. BACKGROUND

Research in mid-1950s indicated that there were differences in the accelerations recorded on the basemat of structures with those recorded in the free-field.[3] The observations led to the conclusion that there existed an interaction between the surrounding soil and the structure thus changing the accelerations experienced by the structure compared to the accelerations present in the free-field. The interaction between the foundation and the soil, i.e. the transfer of the forces and the resulting displacements, results in modifications on the free field motion. This phenomenon is usually referred to as soil-structure interaction (SSI).

Measurements and observations from different cases showed that the coupling between the structure and ground is negligible if the relative stiffness of the structure is comparable to or less than the stiffness of the soil. This observation is mainly related to the mobilization of the center of mass during shaking of the structure. Flexible structures may deform without appreciable changes of the center of its mass during the displacements of the foundation. However, the extreme case of a structure being rigid requires moving the total mass of the structure in conjunction with the foundation. The general design philosophy for the NPPs mostly relies on keeping the safety-related structures in the elastic range which requires the structures to be stiff. Hence, soil-structure interaction has potential influences on the seismic behavior and design of heavy and stiff structures, such as NPPs, sitting on soils with relatively less stiffness.

Two methods are used to solve the SSI problem: (1) the one-step direct method, and (2) the two-step equivalent linear method. The direct method includes the soil continuum, the structure and foundation, and boundaries at the ends of the soil medium. The direct method addresses: i) the inertial effects caused by the mass of the structure and soil, ii) the kinematic effects at the interface between the structure and the soil, and iii) the resulting forces at the foundation due to the deformations of the foundation and soil. The acceleration time series is applied to the base of the soil domain as outcrop motions in this approach.

Direct method is preferred for addressing the possible nonlinearities such as nonlinear soil or structure and uplift of the foundation. The direct method is practically conducted in the time domain, whereas the two-step method is usually preferred for linear analysis in the frequency domain for the sake of computational efficiency. In the two-step method, the soil-structure interface coincides with the interaction horizon, where the outwardly propagating waves are defined.[4] On the other hand, the outwardly waves in the direct method are defined at the artificial boundary, where a high absorbing boundary condition is formulated to prevent the reflecting waves from entering the soil domain at the boundary.

The two-step method uses an equivalent linear site response analysis to establish strain-compatible soil properties that are then applied to a linear frequency domain tool to calculate the SSI response. The frequency domain SSI tool uses a superposition calculation process that subtracts the effects of the excavated soil volume for the nuclear facility, and then adds the structure into the system to evaluate the interaction effects between the site soil and the superstructure. The equivalent linear method has been used in several frequency domain analysis software including SASSI.[5]

Site response analysis has an important role in the SSI analysis either modelled explicitly as in the linear frequency domain analysis or implicitly as in the nonlinear time domain analysis. Details on modeling the site response analysis in the frequency domain and time domain are presented in the literature.[6] Site response analysis for the equivalent linear SSI calculation is used to: (1) develop strain compatible soil properties, and (2) establish ground motion at a location different than the control location. Transfer functions are used in the frequency domain analysis to calculate soil response. Transfer functions are affected by the dynamic soil properties (such as damping, or shear velocity) of the layers of soil between the bedrock and free field surface. The equivalent linear (EL) method is an accepted method for solving the response of the horizontal soil layers against seismic excitations.[7] The EL method is an iterative solution to the equation of motion to approximate the true nonlinearity observed in soils. The EL method along with frequency domain solutions are capable of approximating the nonlinear behavior of soils when the strains are small to moderate level, but the method has significant limitations in moderate to large strain problems. The EL method was integrated in the frequency-domain solution software SHAKE.[8]

Given an input acceleration time series, establishing an acceleration time series at any depth (other than the input location) in the soil column is straightforward, when using an equivalent linear solver such as SHAKE. This is true whether the acceleration time series is inlayer (the summation of upward and downward traveling waves) or outcrop (an upward traveling wave). Iterations may be necessary with equivalent linear analysis if the soil column material properties need to be calibrated to the time series being used.[7]

NPPs and other safety critical structures are designed for low probability and high magnitude ground motion seismic hazards. It was documented that the soil experiences nonlinear behavior during such high magnitude shakings.[9,10] Using the nonlinear hysteretic soil model that is currently being studied at Idaho National Laboratory (INL), a different approach is needed for defining appropriate acceleration time series at different soil depths.[11] Nonlinear soil-structure interaction (NLSSI) analysis methods are currently being developed at INL.[12] ASCE 4 also includes a framework for performing NLSSI analysis.[13] For NLSSI analysis to be widely implemented in industry, it is important to demonstrate that accurate response in a dynamically loaded, nonlinear soil column can be produced.

Solutions including the nonlinear behavior in the site response analysis are conducted in the time domain analysis. Yet, there does not exist a commonly accepted procedure for site response analysis in the time domain.[14] DEEPSOIL is one of the nonlinear time domain software for 1D site response analysis.[15] LS-DYNA is a commercially available finite element software package that is suitable for nonlinear 3D site response analysis.[16] Three dimensional site response analysis methods are gaining attention in the industry in order to overcome the limitations of one-dimensional analysis.[17] A detailed study has been developed on the nonlinear site response analysis.[18] As stated above, this paper is investigating development of acceleration time series at a location of interest (typically the location of input for the NLSSI analysis), within the soil column, using a nonlinear site response analysis procedures.

III. SOIL COLUMN FINITE ELEMENT MODELS

Three dimensional finite element models (FEMs) for the soil columns were developed in the commercially available FE software LS-DYNA. Figure 2 shows the developed FE models for this study. Figure 2a. shows different layers of soils for the 220 and 486 feet FE models. Figure 2b. presents the shear velocities and densities at different depths. The FE models and material properties are representative of a deep soil site in the Eastern U.S. [19] The developed models are 486 and 220 feet, respectively. The 486 feet FEM is defined with 37 nonlinear soil material properties. The 220 feet FEM is defined with 29 nonlinear soil material properties. The solid elements in the models are defined with reduced integration formulation. The dimensioning for the solid element layers is based on approximating the maximum expected strain, and calculating the speed of sound at the expected strain. It is recommended that the ratio of element length to the wavelength to be a maximum of one-eight, which is based on the slowest elastic body wave propagating in the material.[20] An element length is selected that provides ten elements per wavelength for the highest considered frequency. Dynamic relaxation is applied at the initial step of the analysis for appropriately imposing the static gravity loads on the soil columns. Both FEMs are free at the top and have a single elastic element at the base. The elastic element has the elastic material properties of the soil layer where it is placed and is used primarily as a boundary condition. The load time histories are applied at the top of the elastic element. Steps necessary for converting the acceleration time history to load time history input are discussed at the later stages of this section. Non-reflecting boundary conditions are assigned at the bottom of the elastic element. The non-reflecting boundary condition at the base of the models additionally supplies a static pressure to oppose the static gravitational loads and also absorbs the waves traveling down the soil column.

The nonlinear soil constitutive model used in the FEMs is the hysteretic soil constitutive model (*MAT_HYSTERETIC_SOIL in LS-DYNA). Figure 3 shows examples of the shear stress versus shear strain curves used for the case study material properties. The constitutive model for the soil is of a form that includes pressure dependency, which shows a reasonable nonlinear behavior for soil. The material model is capable of capturing the nonlinear hysteretic soil behavior by superimposing elasto-plastic nested surfaces. The model can be represented by shear type parallel-series distributed nested components (springs and sliders) in one dimensional shear stress space.[21] Each curve in Figure 3 is a collection of layers that are represented by elastic/perfectly plastic behavior with different elastic stiffness and yield stress values. The response of the layers is summed together to produce the post yielding shear stress versus shear strain curve, such as the curves shown in Figure 3.

The shear stress at peak shear strain can be modified with changes to the shear modulus versus shear strain curve in the linear approaches, which are based on linear soil constitutive models. Energy absorbed per cycle can be separately modified with changes to the damping ratio versus shear strain curve in a linear model. However, the post yielding shear stress versus shear strain curve of the nonlinear models dictates the shear stress at peak shear strain and energy absorbed per cycle in the hysteresis loop.

Consequently, changes in the shear stress versus shear strain curve affect both the peak shear strain and energy absorbed per cycle in the hysteresis loop. Another aspect of this nonlinear soil constitutive model is that the energy absorbed per cycle must go to zero as plasticity goes to zero. In the linear constitutive model any damping ratio may be defined at any shear strain amplitude. However, the nonlinear model may provide better estimates in modeling actual soil behavior.

Shear modulus versus shear strain and damping ratio versus shear strain data obtained from soil testing is usually used for linear SSI analysis. The nonlinear time domain analysis techniques discussed in the paper require soil material properties in a different form than is used for linear frequency domain analysis techniques. Therefore, calibrations on the data intended for use in linear analysis are necessary for implementing them into the nonlinear models. The shear stress at peak shear strain, and energy absorbed per cycle are the key considerations when calibrating the material properties for nonlinear models. The nonlinear hysteretic models used in the case studies for this paper follow the recommendations and procedures described in a detailed study on how to calibrate nonlinear soil material properties for seismic analysis using soil material properties intended for linear analysis.[22]

The input for the FEMs was an outcrop shear load time series on top of the elastic element at the bottom of the FEMs. The density, modulus of elasticity and Poisson's ratio for the elastic element at the base of the 486 feet model are 0.004 kip*sec^2/ft^4, 80400 ksf and 0.45, respectively. The density, modulus of elasticity and Poisson's ratio for the elastic element at the base of the 220 feet model are 0.004 kip*sec^2/ft^4, 32100 ksf and 0.45, respectively. The elastic element shear wave speed of sound is the square root of the shear modulus of the element divided by its soil density. The velocity time series is obtained by integrating the recorded acceleration time series in order to derive the shear load time series for the input. The shear strain in the elastic element is calculated by dividing the velocity time series to the elastic element shear wave speed of sound. Following the calculation of the shear strain in the element, the shear load time series is derived by multiplying the shear strain by the shear modulus of the element. If the nonlinear soil elements were deleted from one of the FEMs (leaving just the elastic element in the model), the resulting motion on the top of the elastic element would be the same outcrop motion that produced the load time series. However, if the FEMs have the nonlinear soil included, the motion at the top of the elastic element is an inlayer motion including the upward and downward traveling waves. The non-reflective boundary condition at the bottom of the elastic element ensures that downward traveling waves leave the model.

The following sections provide the details of the developed FEMs for the study based on the modeling techniques described above. The first step is establishing the deep outcrop time series using the recorded ground motion at the free-field surface. Then, the FEM from the first step is reduced to a shallower soil column with the same properties as in the first model. An outcrop motion at this shallow location is established based on the motion obtained from the first step. Such a reduction in the size of soil model is necessary for two main reasons; i) establishing the input motion for a site response or SSI analysis at the bottom of the reduced soil model, and ii) reducing significantly the run times for NLSSI time domain analysis. The calculated input motions from the two models reasonably match the recorded ground motion response spectra at the surface.

Quantitative comparisons are provided for response spectra at the free-field surface using calculated input motions from the two models and the response spectra from the recorded ground motion.

III.A. DEEP SOIL OUTRCROP TIME SERIES (First Step)

For the first step of this case study, an approximate outcrop acceleration time series is iteratively established at 486 feet deep in the soil column. When used with the 486 foot deep FEM, the resulting FEM, top-of-soil motion approximately matches that of the recorded Corbin, VA, top-of-soil motion.

ASCE 43 provides recommendations for developing modified recorded time histories that are generated to match the desired response spectrum.[23] The general objective is to generate a modified motion that achieves frequency-to-frequency ratio of the modified motion and the desired (target) motion to be slightly greater than one. One of the criteria for the modified motion is that spectral accelerations at 5% damping shall be computed at a minimum of 100 points per frequency decade. The evaluated spectral accelerations need to be uniformly spaced over the log frequency scale. Another criterion is that the modified motions have strong durations, which are defined by the 5% to 75% Arias intensity.

The recorded ground motion in this study lasts over 190 seconds although much of the time series is relatively low amplitude. Consequently, only about 20 seconds of the recorded ground motion is used. The resulting time series is transitioned to a second of quiet time at the start and end, is oversampled (using fast Fourier techniques) from a 0.005 second time step to a 0.00125 second time step, and is drift corrected using frequencies less than 0.5 Hz. The comparison of the 5% damped response spectrum from the resulting acceleration time series based on the iterated outcrop motion, and the entire original acceleration time series is conducted.

The approach in this study is based on applying a similar time series at 486 feet deep as is desired at the top-of-soil except the frequency content is scaled. By scaling the frequency content, adjustments can be made to counteract amplification and damping that occurs as the waves pass through the soil column. The decision for how much to scale a given frequency amplitude is based on comparing the desired top-of-soil response with the FEM top-of-soil response.

Frequency content is found by taking the fast Fourier transform of the desired top-of-soil time series. The fast Fourier transform provides amplitude and phase angle information for all the sine waves that produce a smooth curve through all of the time series data points when all the sine waves are summed. Consequently, if a given input time series produces a top-of-soil response that is too high at a given frequency, the frequency amplitude at that frequency could be scaled down. After scaling the motions at different frequencies, the inverse fast Fourier transform could be taken to produce an adjusted time series.

In this simplest way of scaling, the adjusted time series may drift badly during integration. The drifted time series may be difficult to manage. Consequently, the approach used in this study is a variation on this concept. For this study, a fast Fourier transform is performed on the desired top-of-soil time series. Next, the resulting sine waves are defined. There are more sine waves defined than the frequencies evaluated in a response spectrum. The spectral accelerations of the response spectrum are represented with evenly spaced intervals of 100 points per decade. The defined sine waves in the vicinity of a single response spectrum frequency are summed. Finally, the ends of the sine waves are modified with a smooth polynomial so that they are drift corrected and smoothly transition to zero, as shown in Figure 4. The polynomial is defined as an acceleration curve applied over a set time interval, which is less than one cycle in time span for the acceleration wave being drift corrected. Displacement, velocity, acceleration, and jerk are defined at one end of the polynomial. The displacement, velocity, acceleration, and jerk are all defined to be zero at the other end of the polynomial. For drift correction at the start of an acceleration wave, the first step is to establish the initial displacement, velocity, acceleration, and jerk. The second step is to define a polynomial acceleration with identical initial displacement, velocity, acceleration, and jerk. The polynomial acceleration is then subtracted from the first part of the acceleration wave. The modified acceleration wave has initial displacement, velocity, acceleration, and jerk are all defined to be zero and no discontinuity where the polynomial acceleration time span ends. The third step is to repeat the process in reverse at the end of the modified acceleration wave.

In this study, the time period where the smooth polynomial is added is limited by the time period where less than 5% of the energy in the full time series is realized. This process produces a set of modified sine wave time series where each represents a frequency. ASCE 43-05, Section 2.4 (b) discusses criteria for developing synthetic or modified recorded time histories and it defines response spectra as having a minimum of 100 points per frequency decade, uniformly spaced over the log frequency scale. Consequently, response spectra from 0.1 Hz to 100 Hz would have 300 points distributed evenly on a log frequency scale. Taking the fast Fourier transform of a typical seismic acceleration time history produces thousands of points evenly distributed on a linear frequency scale. Because modifications to the fast Fourier transform points are based on the 300 points, fast Fourier transform data in the vicinity of each of the 300 response spectra points are grouped together. Their grouping is dependent on having each fast Fourier transform data point being grouped with any other fast Fourier transform point that shares the same response spectra point that they are nearest based on a log frequency scale.

The response spectrum data points are evaluated based on the obtained frequencies. Summing these unscaled modified sine waves approximates the desired top-of-soil time series (with it being close to matching at the ends but exactly matching in the middle). The advantage of this approach is that scaling one of the modified sine waves and then summing them all together produces a modified time series that has adjusted frequency content but doesn't drift.

After the sine waves are modified for drift correction, iterations for calculating the desired top of the soil motion (original motion) are performed. Initially, a scale factor of one is defined for each modified sine wave. A 5% damped acceleration response spectrum is generated for the Corbin, VA, top-of-soil acceleration (at the frequencies represented with the modified sine waves). For an iteration, the modified sine waves are multiplied by their scale factors and summed, the resulting acceleration time series is integrated to a velocity time series. The calculated velocity time series is converted to a load time series for the 486 feet FEM input at the base. Upon running the FEM, a 5% damped acceleration response spectrum is generated at top-of-soil. Finally, each scale factor is modified to equal its previous value multiplied by the Corbin, VA, top-of-soil response and divided by the FEM response.

Judgement is required in establishing the optimal number of iterations for this method because this approach does not converge to an exact solution. As guidance for convergence, the fast Fourier transform amplitudes versus frequency of the input acceleration time history should be considered along with the response spectra convergence. The initial iteration should provide insights as to a reasonable curve shape for the fast Fourier transform amplitudes. With few further iterations (possibly only one), a reasonable response spectra fit should be achieved. As the number of iterations is increased, the response spectra fit may slowly converge while the fast Fourier transform amplitudes curve may become very noisy and drastically different than the initial iteration. A good result for this method is to have a reasonable response spectra fit and a fast Fourier transform amplitudes curve that is similar to that of the initial iteration. Figure 5 shows the scale factors after two iterations. Figure 6 shows the resulting top-of-soil motion comparison between the Corbin, VA time series and the 486 feet FEM (noting that the Corbin, VA time series are offset by 0.294 seconds to accommodate the time that is required to pass the waves up through the FEM). The acceleration, velocity and displacement histories match reasonably for the Corbin time series and FE model. Figure 7 shows the top-of-soil, 5% damped response spectra comparison between the Corbin, VA time series and the 486 foot FEM. The comparison shows that the obtained motion produces reasonable comparison with the top of the soil response spectrum. The FEM response spectrum is only plotted to 67 Hz because that is as high of a frequency as the FEM is intended to accurately model. Energy content of the ground motion above 67 Hz motions is considered negligible.

The response spectrum for this study had a maximum error of 10.6% at one frequency point of 31.6 Hz. Although some point of the obtained response spectrum from the iterated ground motion at the base, the iterative procedure produced reasonable comparison with the response spectrum of the target acceleration time series. When comparing the accuracy of this step to a similar process with linear analysis, an important consideration is necessary. This consideration is that a single frequency input to the base of a linear soil model produces a single frequency response at the top-of-soil due to the elliptical shape of the viscous damping hysteresis loop.[24] The numerical and the analytical solutions show that any deviation of the hysteresis shape from a perfect ellipse introduces frequencies higher than the input frequency in the shear stress. Similarly, a single frequency input to the base produces a multi-frequency response at the top-of-soil due to the non-elliptical shape of the hysteresis loop for a nonlinear analysis as is the case in this study. Considering that neither the linear nor nonlinear models exactly reproduce the stiffness of the real soil, the input outcrop time series at the base is not exactly correct for either model. This inexact stiffness may make it necessary to increase (or decrease) the frequency content at various frequencies in the input time series to produce the desired top-of-soil motion. With linear analysis, the added (or subtracted) frequency content can be accommodated without negative consequence because each frequency content modification only affects the top-of-soil response at that frequency. With nonlinear analysis, adding frequency content causes a top-of-soil response at that frequency and other frequencies. Having the frequency content added to the other frequencies can make it difficult to maintain reasonable input frequency content.

As an example, a situation could occur where an actual outcrop and low frequency input to a FEM does not produce as much top-of-soil response as the actual soil column due to the stiffness of the FEM not being exactly correct. To reduce the top-of-soil discrepancy, more input low frequency content can be added (for either linear or nonlinear FEMs). For a linear FEM, the inaccuracy with such an approach is limited to the low frequency content that has been modified. For the nonlinear model, this could additionally cause inaccuracies at higher frequencies. Adding additional high frequency content to correct the higher frequency inaccuracies could lead to inaccuracies at yet higher frequencies. Given the nature of the changes to phasing, the best accuracy of the procedure should occur at low frequencies and the worst accuracy should occur at high frequencies. Attempting to address all the resulting inaccuracies could produce an unreasonably inaccurate input time series. Considering this example, few iterations are performed on the nonlinear model and an inexact result that is close to the real top-of-soil motion is considered acceptable.

III.B SHALLOW OUTCROP TIME SERIES (Second Step)

For the second step of this case study, an outcrop acceleration time series is iteratively established at 220 feet deep in the soil column. In this step, the inlayer acceleration at 220 feet is compared between the two FEMs to make adjustments instead of comparing top-of-soil response spectra. This step is relatively simple but can become unstable. Considering the nonlinear soil constitutive model used, there is a possibility for some erroneous high frequency noise. In reasonably defined soil material properties, this should be at frequencies higher than are of concern and not of significant magnitude. However, the iterative approach described in this study can magnify the high frequency noise issues to the point of making the approach unstable. To reduce the possibility of this instability, an initial modification is performed on the 220 feet, which is the inlayer acceleration time series taken from the 486 feet deep FEM. The first study involves taking the fast Fourier transform of the time series. Second, the frequency amplitudes above 100 Hz are ramped to near zero (considering that this is well above the 67 Hz cut off for frequencies that these FEMs can evaluate accurately). Third, the inverse fast Fourier transform is performed to produce a time series with less high frequency content. This modification is likely to cause a small amount of drift. For this study, this was corrected by adding a quarter second smooth correction in the shape of a sine wave to the start of the acceleration time series. The resulting modified acceleration time series is used as the desired inlayer motion for the 220 feet deep FEM.

Having a target inlayer acceleration time series, iterations can be performed. Initially, the target inlayer acceleration time series is integrated to velocity and used in the 220 feet deep FEM as though it is an input outcrop motion. For an iteration, the 220 feet deep FEM is run and the inlayer acceleration time series is output at 220 feet. Next, this inlayer acceleration is subtracted from the desired inlayer motion and the result is added to the acceleration used as outcrop for the 220 foot deep FEM. Because of the instability risk, the modified outcrop acceleration has the high frequency (above 100 Hz) ramped down and drift correction performed similar to the adjustment originally performed on the desired inlayer motion. Finally, the new outcrop acceleration time series is integrated to velocity for the next iteration.

Results shown in Figures 8 and 9 are for the second step of the case study performed with 125 iterations. Figure 8 shows the top-of-soil motion comparison between the 220 feet FEM and the 486 feet FEM. The maximum calculated accelerations of the two time series are around 5 ft/sec^2. The acceleration, velocity and displacement histories at the top for two different depths of soil columns match within 1% of each other. Figure 9 shows the top-of-soil, 5% damped response spectra comparison for the 220 feet FEM and the 486 feet FEM. Similarly, the top-of-soil response comparison is within 1% of each other. The response spectrum for this study had a maximum error of 1.5%. Figure 10 demonstrates the convergence of the iterations (based on the velocity time series curves). At 10 iterations, the percent difference for inlayer velocity between the 220 feet FEM and the 486 foot FEM was less than 0.2% until almost 2 seconds and the maximum percent error was 7.4%. At 50 iterations, the percent difference was less than 0.2% until a little more than 8 seconds and the maximum percent error was 1.7%. At 100 iterations, the percent difference was less than 0.2% until a little less than 16 seconds and the maximum percent error was 0.7%. At 125 iterations, the percent difference was less than 0.2% for the whole time series.

IV. CONCLUSION

Using the nonlinear hysteretic soil model that is currently being studied at the INL, an approach has been applied to a case study for defining appropriate acceleration time series at different soil depths. An approximate outcrop acceleration time series is iteratively established at 486 feet deep in the modeled soil column. This study showed that the proposed approach for the first step produced reasonable results compared to the original time series. In the second step of the case study, an approximate outcrop acceleration time series is iteratively established at 220 feet deep in the soil column based on results from the 486 foot soil column model. This study showed that the proposed approach for the second step produced accurate results when sufficient iterations were performed.

References

1. United States Nuclear Regulatory Commission (NRC) "Practical Implementation Guidelines for SSHAC Level 3 and 4 Hazard Studies", NUREG-2117, US Nuclear Regulatory Commission, Washington D.C., USA. (2012)
2. United States Geological Survey (USGS) "Strong Motion Virtual Data Center (VDC), Station: VA:Corbin (Fredericksburg Obs)" Acceleration Data for Mineral, VA Earthquake, August 23, 2011 (2011). http://www.strongmotioncenter.org/vdc/scripts/stnpage.plx?stations=5218, Web page last visited on October 2, 2017.
3. Housner, George W. "Interaction of building and ground during an earthquake." Bulletin of the Seismological Society of America 47, no. 3 (1957)
4. Wolf, John P. "Classification of Analysis Methods for Dynamic Soil-Structure Interaction" International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics. 4, (1991).
5. Lysmer, J., Ostadan, F., and Chin, C. "SASSI2000, User Manual, " Revision 1, A System for Analysis of Soil-Structure Interaction, University of California, Berkeley, CA. (1999).
6. Bolisetti C., Whittaker A. S., Mason H. B., Almufti I., and Willford M. "Equivalent linear and nonlinear site response analysis for design and risk assessment of safety related nuclear structures" Nuclear Engineering and Design, 274:107–121, (2014)
7. Idriss, Izzat M., and H. Bolton Seed. "Seismic response of horizontal soil layers." American Society of Civil Engr J Soil Mech (1968).

8. Shnabel, P., J. Lysmer, and H. B. Seed. "SHAKE: A computer program for earthquake response analysis of horizontal layer sites." UCB 72/12-EERC (1972).
9. Beresnev, Igor A., and Kuo-Liang Wen. "Nonlinear soil response—A reality?." Bulletin of the Seismological Society of America 86, no. 6 (1996): 1964-1978.
10. Régnier, Julie, Héloïse Cadet, and Pierre-Yves Bard. "Empirical Quantification of the Impact of Nonlinear Soil Behavior on Site Response." Bulletin of the Seismological Society of America 106, no. 4 (2016): 1710-1719.
11. Idaho National Laboratory "Finite element software for Multi-hazard Analysis for STOchastic time-DOmaiN phenomena." 2525 N. Fremont Ave, Idaho Falls, ID 83415 (https://moose.inl.gov/mastodon/SitePages/Home.aspx) (2017).
12. Coleman, Justin L., Chandrakanth Bolisetti, and Andrew S. Whittaker. "Time-domain soil-structure interaction analysis of nuclear facilities." Nuclear Engineering and Design 298 (2016): 264-270.
13. American Society of Civil Engineers. "Seismic Analysis of Safety-Related Nuclear Structures and Commentary" ASCE 4–16; ASCE Publications: Reston, VA, (2016)
14. Kwok, AOL., Stewart JP., Hashash YMA., Matasovic N., Pyke R., Wang Z., and Yang Z. "Use of exact solutions of wave propagation problems to guide implementation of nonlinear seismic ground response analysis procedures." Journal of Geotechnical and Geoenvironmental Engineering 133, no. 11 (2007)
15. Hashash, Y. M. A., D. R. Groholski, C. A. Phillips, D. Park, and M. Musgrove. "DEEPSOIL 5.1, user manual and tutorial." Department of Civil and Environmental Engineering–University of Illinois at Urbana-Champaign (2012).

16. LSTC "LS-DYNA, Version smp s R7.00, " LS-DYNA Keyword User's Manual, Volume II, Livermore Software Technology Corporation, Livermore, CA, USA (2013).

17. United States Nuclear Regulatory Commission (NRC) "NRC Seismic Research Program Plan FY 2008–2011", Structural, Geotechnical & Seismic Engineering Branch Division of Engineering Office of Nuclear Regulatory Research, Rockville, MD, (2008).

18. Stewart, J. P., A. O. L. Kwok, Y. M. A. Hashash, N. Matasovic, R. Pyke, Z. Wang, and Z. Yang. "Benchmarking of Nonlinear Geotechnical Ground Response Analysis Procedures" Pacific Earthquake Engineering Research Center. University of California, Berkeley (2008).

19. Southern Nuclear Operating Company "Seismic Hazard and Screening Report for Vogtle Electric Generating Plant – Units 1 and 2", https://www.nrc.gov/docs/ML1409/M14092A019.pdf (2014)

20. Kuhlemeyer, Roger L., and John Lysmer. "Finite element method accuracy for wave propagation problems." Journal of Soil Mechanics & Foundations Div 99.Tech Rpt (1973).

21. Coleman, Justin, Andrew Slaughter, Swetha Veeraraghavan, Chandrakanth Bolisetti, Ozgun Alp Numanoglu, Robert Spears, William Hoffman, and Efe Kurt. "MASTODON theory manual." No. INL/EXT-17-41930. Idaho National Lab.(INL), Idaho Falls, ID (United States), 2017.

22. Spears, R. E., & Coleman, J. L. "Calibrating Nonlinear Soil Material Properties for Seismic Analysis Using Soil Material Properties Intended for Linear Analysis" INL/CON-14-33615, Idaho National Laboratory (INL), Idaho Falls, ID (United States) (2015).

23. American Society of Civil Engineers (ASCE). "Seismic design criteria for structures, systems, and components in nuclear facilities." ASCE/SEI 43-05, ASCE, Reston, VA., (2005).
24. Veeraraghavan S., Spears RE., and Coleman JL. "High frequency content in nonlinear soil response: A numerical artifact or a reality? ", submitted to Earthquake Engineering and Structural Dynamics for review in October 2017.

Figures

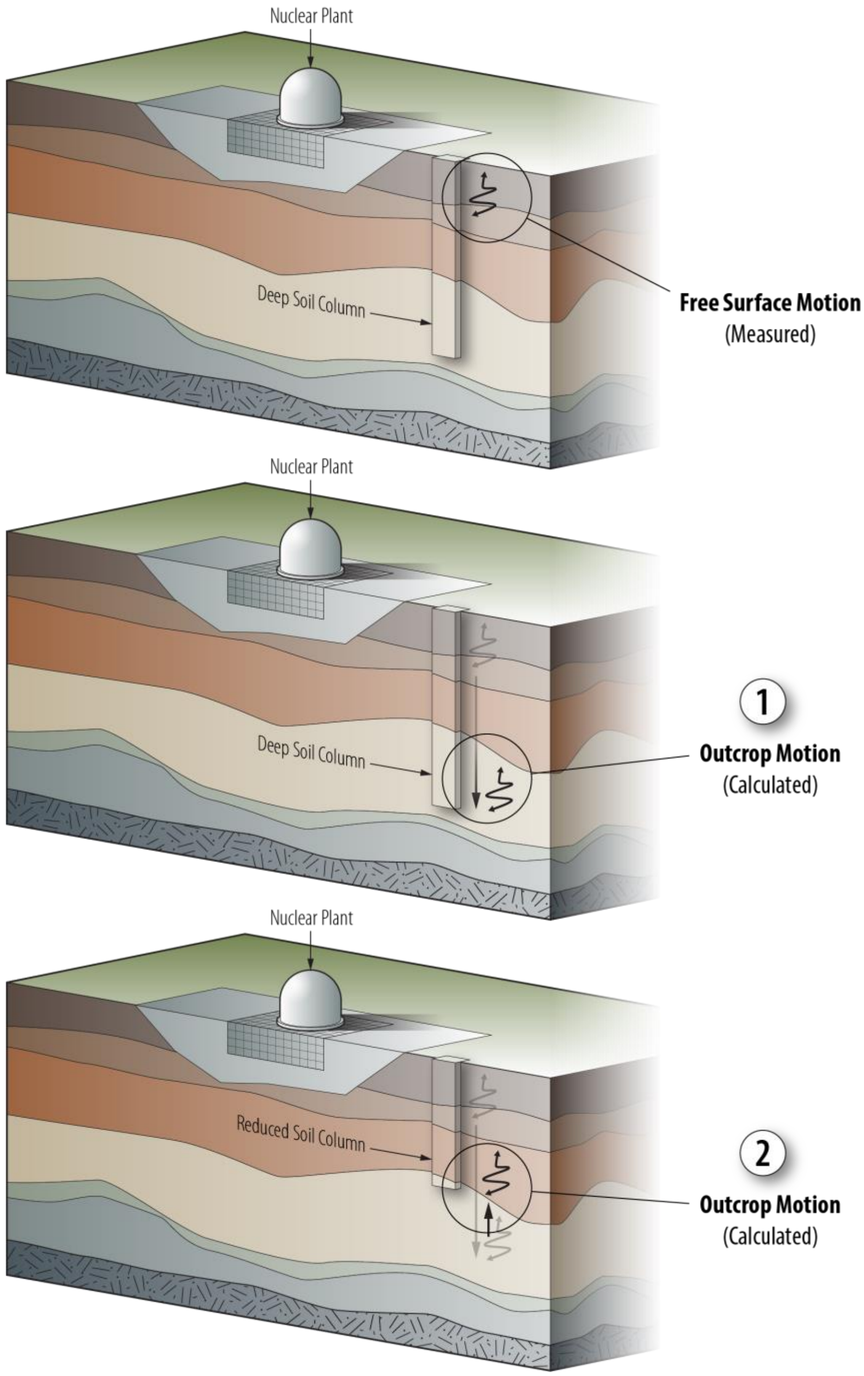
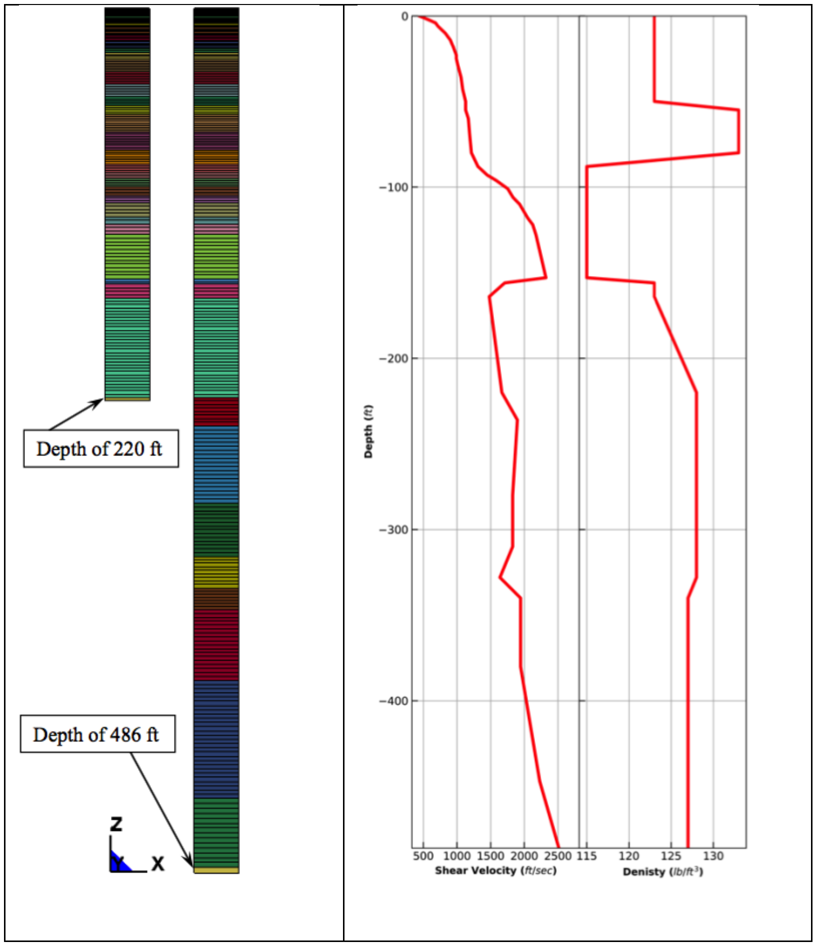


Figure 1. First step: establishing outcrop motion at the deep soil base, and Second step: establishing outcrop motion from the inlayer motion calculated from the first step



a) b)

Figure 2. a) Three dimensional soil column FEMs, b) shear velocity and density corresponding to the soil column models

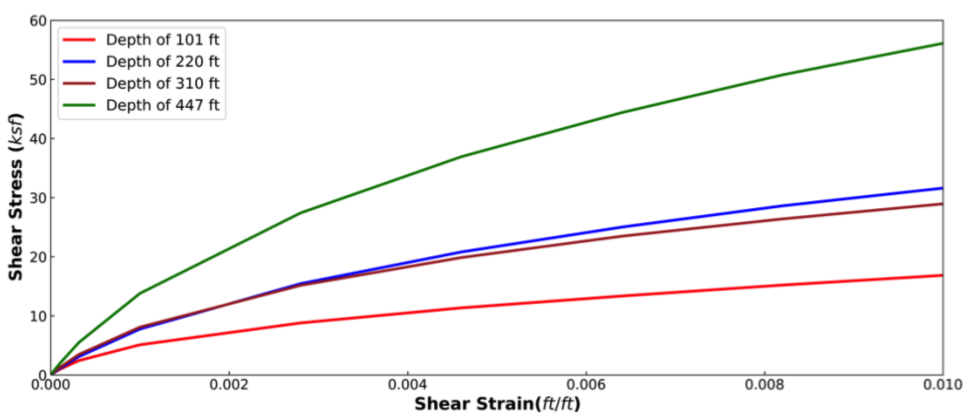


Figure 3. Example shear stress versus shear strain curves use in the soil column FEMs at different depths

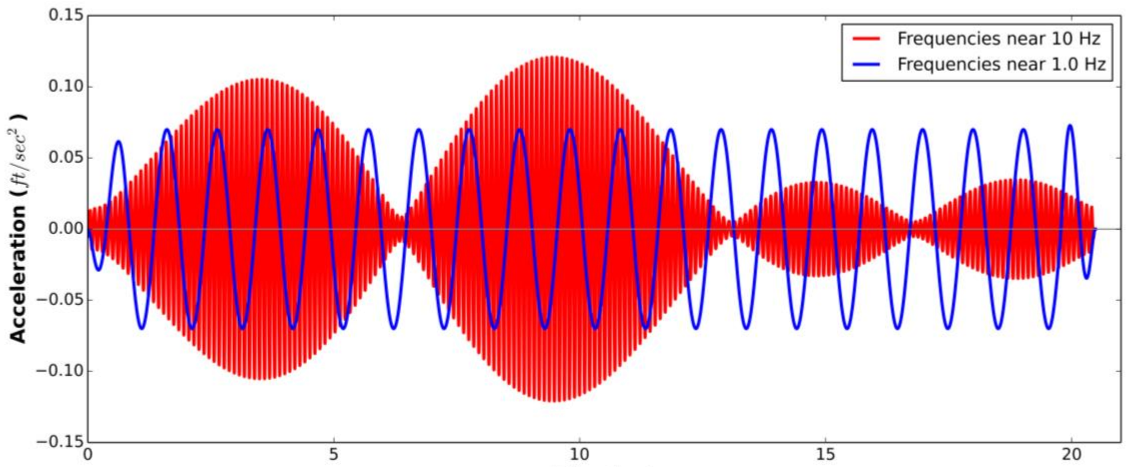


Figure 4. Modified sine waves with frequencies near two of those defined in a response spectrum.

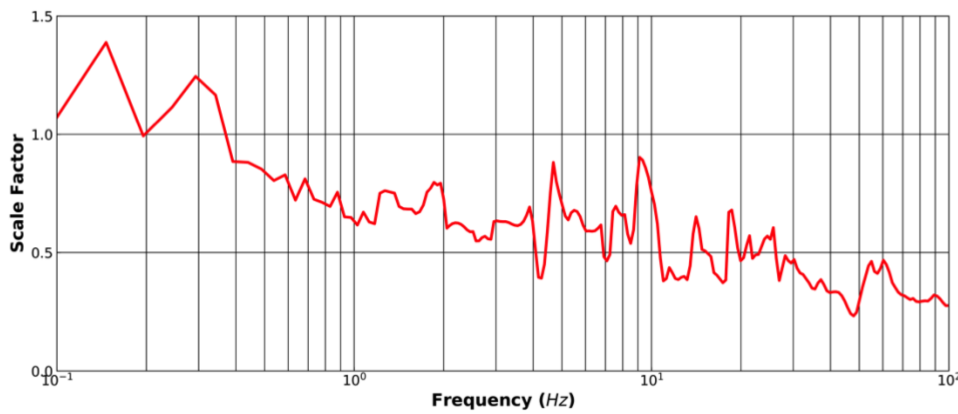


Figure 5. Scale factors after two iterations for the 486 feet deep soil column

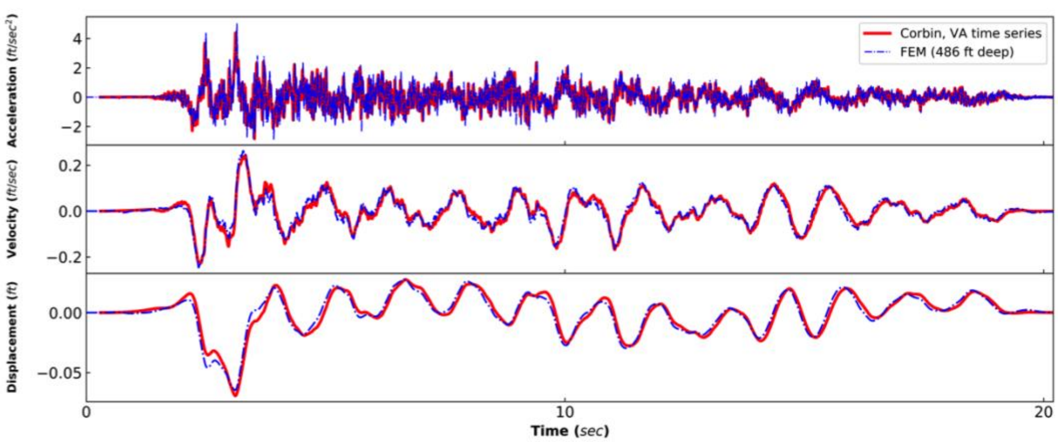


Figure 6. Top-of-soil acceleration, velocity, and displacement history comparison between the Corbin, VA time series and the 486 feet FEM results

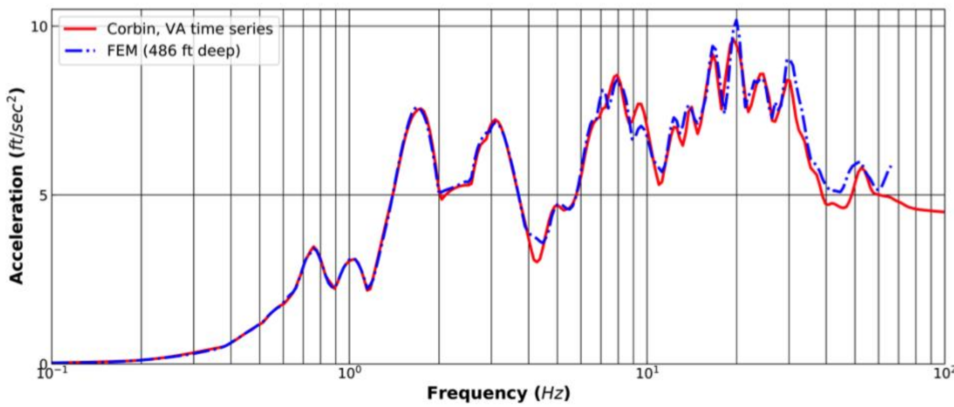


Figure 7. Top-of-soil acceleration history comparison between the Corbin, VA time series and the 486 feet FEM.

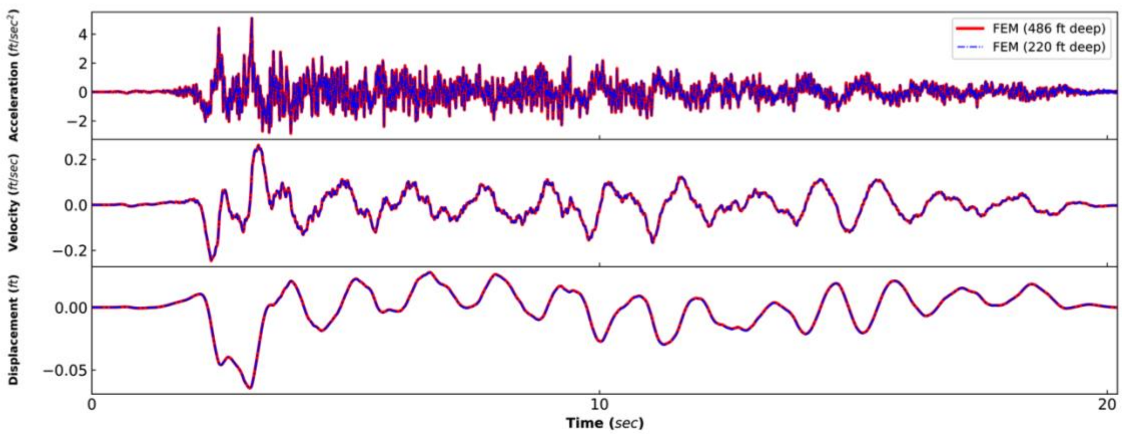


Figure 8. Top-of-soil acceleration, velocity and displacement history comparison between the 220 feet FEM and the 486 feet FEM.

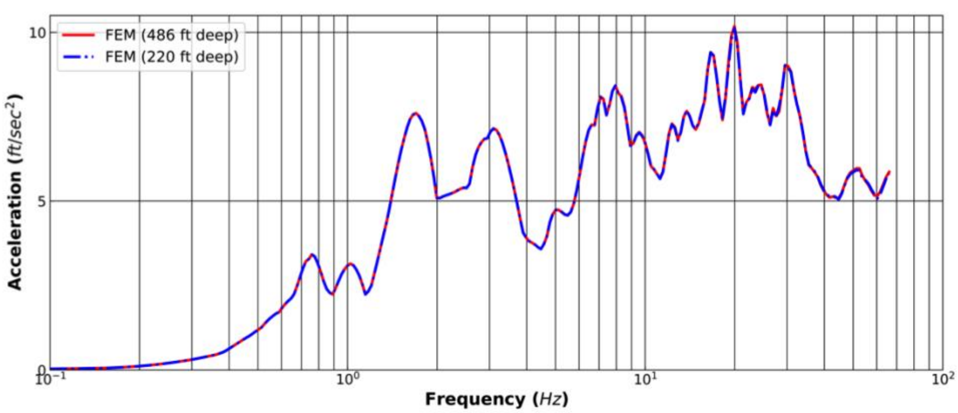


Figure 9. Top-of-soil acceleration history comparison between the 220 feet FEM and the 486 feet FEM.

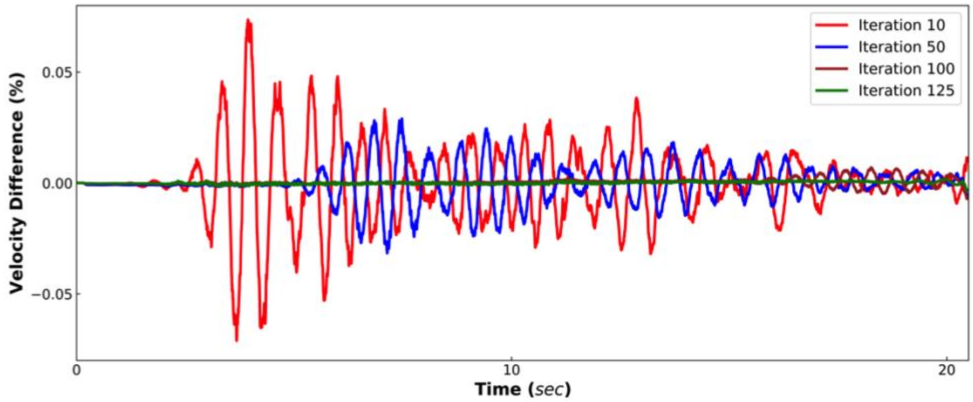


Figure 10. Percentile difference of the FEM and desired inlayer velocity history curves at specific iterations for the