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International benchmark on numerical simulations for 1D, non-linear site response (PRENOLIN): verification phase based on canonical cases

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Abstract

PRENOLIN is an international benchmark presently underway to test multiple numerical simulation codes capable of predicting non-linear seismic site response with various constitutive models. One of the objectives of this project is the assessment of the uncertainties associated with non-linear simulation of one-dimensional (1D) site effects. A first verification phase (i.e. comparison between numerical codes on simple, idealistic cases) will be followed by a validation phase, comparing the predictions of such numerical estimations with actual strong motion recordings obtained at well-known sites. The benchmark presently involves 19 different prediction teams and 23 different non-linear computational codes.

We present here the main results of the verification phase dealing with simple cases. Three different idealized soil profiles were tested over a wide range of shear strains with different input motions and different boundary conditions at the sediment/bedrock interface. A first iteration focusing on the elastic and visco-elastic cases proved to be useful to ensure a common understanding and to identify numerical issues before pursuing the non-linear modeling. Besides minor (but always possible…) mistakes in the implementation of input parameters and output units, the initial discrepancies between the numerical results can be attributed to (1) different understanding of the expression "input motion" in different communities, and (2) different implementations of material damping and possible numerical dispersion. The second round of computations thus allowed a convergence of all teams to the Haskell-Thomson analytical solution in elastic and visco-elastic cases. For non-linear computations we investigate the epistemic uncertainties related only to wave propagation modeling using different non-linear constitutive models. Such epistemic uncertainties are shown to increase with the strain level and to reach values around 0.2 (natural log scale) for a

5m/s^2 reference motion, which may be reduced by almost 50% when the various constitutive models do use the same shear strength and damping implementation.

Introduction

Including site effects in seismic hazard assessments requires the consideration, at some stage, of non-linear behavior of soils, which may greatly affect their dynamic responses to strong motion and significantly modify their amplifications behavior compared to weak motion (computed or measured). Even in areas of moderate seismicity, the hazard level at long to very long return periods (i.e., several thousands to tens of thousands years) may be large enough to generate significant strains in shallow, soft soil layers, which in turn leads to a degradation of their mechanical properties such as hysteretic behavior with loss of shear stiffness and increased energy dissipation (Bonilla et al., 2005; Iai et al., 1995; Ishibashi and Zhang, 1993; Seed, 1969; Vucetic and Dobry, 1991; Yu et al., 1993; Zeghal et al., 1995).

Such dependence of the dynamic soil response on the level of seismic loading, conventionally denoted as "non-linear effects" (Beresnev et al., 1995), involves rather complex mechanical processes, which may be grouped roughly in two main classes. The first is the degradation of the mechanical properties of the material, which is often characterized by a decrease in the shear modulus coupled with an increase in energy dissipation; while the second is related to pore pressure changes in water-saturated granular soils, linked with volumetric changes of the soil skeleton under shear stress, and may generate liquefaction in loose sandy soils. Our interest here focused on the first type of non-linearity, without any consideration of pore water pressure generation or liquefaction.

The first type of non-linear effect (i.e. without liquefaction) was identified by geotechnical earthquake engineering studies following the 1967 Caracas earthquake, and was later confirmed both by laboratory tests and recordings obtained on "vertical arrays" with two or more accelerometric sensors at different depths within the same borehole. For instance, a statistical analysis of the numerous recordings of the Japanese KiKnet network (Régnier et al., 2013) concluded that, for Peak Ground Acceleration (PGA) levels exceeding 0.75 m/s^2 (a rather moderate level) at an outcrop, there is a 40 % chance of observing a non-linear soil response, leading to significant modifications with respect to the linear, low-strain response. These changes generally imply a reduction of the response amplification of the signal's high-frequency content and often a slight-to-significant increase of its low frequency content. Therefore, linear soil response estimates cannot be considered as being systematically on the safe side, and on the other hand, the high frequency reductions may significantly contribute to the safety margins. As a consequence, the accuracy, robustness and reliability of non-linear site effects prediction directly impacts the estimation of seismic hazard and associated risks, especially at long return periods.

While a consensus has undoubtedly been reached on the existence of non-linear effects, their quantification and modeling remains a challenge. Indeed, numerous techniques have been proposed for the assessment of site effects in the linear domain using empirical and/or modeling approaches on generic or site-specific basis. Conversely, empirical estimation of non-linear site effects is more limited, especially in moderate seismicity areas where the on-site instrumental approach can only be a long (to very long)-term investment. Aside from a generic approach based on existing recordings (Derras et al., 2012; Sandıkkaya et al., 2013), the only presently possible way for site-specific estimates is thus numerical simulation. Obviously, such analysis must be preceded by a precise geotechnical and geophysical characterization of the underground structure, and the choice of a suitable non-linear constitutive model.

Given the complexity of non-linear behavior of soils, many constitutive models and codes have been developed for such simulations. When the deformation remains moderate (i.e., smaller than about 0.1-0.3 %), the so-called "equivalent linear model", which is a linear approach with an iterative adjustment of visco-elastic properties (shear modulus and damping) to the local strain level, is often used and accepted in practice. However, when the strain level exceeds these values (i.e., above 0.2-0.5 %), which can occur in very soft soils and/or with very strong input motions, a complete non-linear modeling, with an appropriate constitutive law fed by the correct soil parameters is required. These models fall into two categories: relatively simple constitutive laws with few parameters, that cannot reproduce a wide range of loading/unloading paths; and more complex models with many parameters (sometimes exceeding 10), which can succeed in describing all possible behaviors, but with parameters that can be difficult to determine or calibrate.

The ability to accurately predict non-linear site responses has indeed already been the subject of two recent comparative tests. It was one of the targets of the pioneering blind tests initiated in the late 80's/early 90's, on 2 sites of Ashigara Valley (Japan) and Turkey Flat (California); however, those sites lacked strong motion records until the 2004 Parkfield earthquake during which the Turkey Flat site experienced a 0.3 g motion. Since the soils were fairly stiff, the nonlinearity was not very strong. A new benchmarking of 1D non-linear codes was thus carried out in the last decade, based on the Turkey Flat site and a few other sites with vertical array data (La Cienega, Caifornia; the KGWH02 KiK-net site in Japan, and Lotung in Taiwan). Its main findings, reported by Kwok et al. (2008) and in (Stewart and Kwok, 2009) emphasized the key importance of the way these codes are used and of the required in-situ measurements. Significant differences between records and predictions have been postulated as being due to an incorrect velocity profile (although it was derived from redundant borehole measurements), a non-1D soil geometry (non-horizontal layers), and imperfections / deficiencies in the constitutive models, which were unable to represent the actual curves for stiffness reduction and damping increase. Another test was undertaken on the Euroseis European test site (Mygdonian graben near Thessaloniki, Greece) as part of the

Cashima/E2VP project, which included two separate exercises on two-dimensional (2D) non-linear numerical simulation codes and tree-dimensional (3D), linear simulation codes. The 2D non-linear (NL) benchmark proved inconclusive, as major differences were found between the few considered codes, with multiple possible causes (2D numerical scheme, damping implementation, and NL constitutive laws (see Foerster et al., 2015). Given the fact that the codes used for these tests are routinely used in engineering practice for predictions of non-linear site responses, especially for moderate seismicity countries lacking strong motion recordings, there is a clear need to conduct further tests in better controlled conditions, in particular with in situ and laboratory measurements for an optimal tuning of the non-linear parameters used in each code.

For this reason, the PRENOLIN project considers only 1D soil columns to test the non-linear codes in the simplest possible, though realistic, geometries. It is organized in two phases: (1) a verification phase aiming at a cross-code comparison on very simple (and "idealistic") 1D soil columns with prescribed linear and non-linear parameters; (2) a validation phase for comparison between numerical predictions and actual observations, for sites as close as possible to a 1D geometry (horizontal stratification), without liquefaction evidence and with already available sets of downhole and surface recordings for weak to very strong motions and later complemented by careful in-situ and laboratory measurements designed as close as possible to the participants requirements. The sites were selected within the Japanese KiK-net and PARI (Port and Airport Research Institute) accelerometric networks.

The purpose of this article is to present and discuss the results of the verification phase, with a special focus on the epistemic uncertainties associated with the constitutive laws and numerical schemes of the simulation codes. The first section describes the 3 idealized soil columns and the requested computations, considering different boundary conditions (rigid / elastic base, associated respectively with within / outcropping reference motion). The next section lists the numerous teams that volunteered to participate in this exercise and the main characteristics of their codes. The simulation results are then presented and compared, first in the linear case (with and without attenuation), and then in the non-linear case for various input signals and levels, with a discussion in each case on the amount and origins of uncertainty.

The canonical cases

The verification phase of this project aims at establishing the similarity between the computed wave motions at the surface of a soil column affected by amplification using different numerical codes, quantifying the amount of code-to-code differences and, as much as possible, understanding them. The computed responses were compared with analytical solutions when available. Figure 1 summarizes the calculations performed during the verification phase, for the linear (elastic and visco-elastic), and non-linear cases. In the elastic and visco-elastic cases, for which analytical results are available and provided that all participants/users share a common understanding of the physical soil parameters to be used, no differences (or minor) in the results are expected. These first calculations are needed in order to ensure a proper predictability of the induced deformation (shear strain) for all soil and seismic wavefield properties. On the other hand, for non-linear cases, discrepancies between the different computations are expected: the goal is to identify their origins in relation to the constitutive models and/or the numerical schemes (or other possible issues), to quantify the associated epistemic uncertainty, and to reduce it to its minimum level as much as possible.

The experiment was designed around three 1D canonical cases, chosen to represent simple and idealistic soil conditions overlying stiff bedrock substrata:

1) Profile 1 (P1) is a shallow (20 m thick), homogeneous soil layer presenting a significant velocity impedance ratio at rock, with amplification in the intermediate frequency range [2-10 Hz].

2) Profile 2 (P2) is a thick (100 m) soil layer with S-wave velocity gradually increasing with depth, overlying a very stiff bedrock, with a low fundamental frequency (below 1 Hz).

3) Profile 3 (P3) consists of two homogeneous layers with moderate velocity contrasts, overlying a very stiff bedrock, with expected amplification effects in the intermediate frequency range (2-10 Hz). The goal is to investigate non-linearity effects within both layers, since significant strains can develop at or near each interface.

Various reference motions are considered for each profile, from very simple signals intended to capture the basic physics of NL behavior (pulse like and cyclic, quasi-monochromatic signals with increasing amplitude), to realistic accelerograms. For the later, two strong motions were selected with very different spectral content (high and low frequency contents), and scaled to three PGA levels, in order to generate a wide range of shear strain levels in the soil column.

These reference motions were applied at the bedrock level, with two boundary conditions representative of the actual case studies: in one case, the reference motion was considered to mimic the outcropping motion at the surface of the underlying bedrock ("elastic" condition), while in the other it was considered to mimic the "within" motion recorded by a virtual sensor at the sediment-bedrock interface ("rigid" condition).

Figure 1 : ***The three simple idealized profile cases studied here (P1-3), for the elastic and non-elastic domains, and for a rigid and elastic soil-bedrock base, using a Ricker pulse and 3 accelerations of different PGA and frequency contents.***

Soil properties

The properties describing the (1D) linear and non-linear soil behavior for each profile include elastic, visco-elastic and non-linear soil properties. They are displayed in Figure 1 and Figure 2, and summarized in Table 1.

The basic characteristics of soil profiles (i.e., thickness, density and seismic waves velocities) were chosen in order to be representative of typical soil profiles. Values of P-waves velocity (V_P) are derived from the profiles of S-waves velocity (V_S) shown in Figure 2, using assumed values of Poisson ratio (0.4 for soil and 0.3 for bedrock). Profiles P1 and P3 exhibit constant seismic velocities in each layer, while P2 includes a velocity gradient with a regular increase from Vs = 150 m/s at the surface to Vs = 500 m/s at the soil-bedrock interface, according to the equation:

$$Vs(z) = Vs_1 + \left(\frac{(Vs_2 - Vs_1)(z - Z_1)}{Z_2 - Z_1} \right)^{\alpha}$$

Eq 1

where V_{S1} = 150 m/s and V_{S2} = 500 m/s are the shear-wave velocities at depths Z_1 = 0 m and Z_2 = 100m, respectively, and $V_S(z)$ is the shear wave velocity at depth z; α is taken equal to 0.25.

Visco-elastic properties

We only consider intrinsic material damping (Biot, 1956; Johnston et al., 1979; Leurer, 1997), without any additional component from scattering. Intrinsic attenuation can be quantified by the quality factor Q (more commonly used in seismology), or the damping ratio ξ (used in engineering seismology). Here Q and ξ are the quality factor and the damping ratio of the S-waves. They are linked by the formula Q = 1/(2ξ), and can be determined by the loss of energy over one wavelength. Pure elastic materials totally restore the seismic energy after deformation, and should therefore have infinite Q values; as the numerical codes used here require a finite value as input, the "elastic" case was computed with very high values of Q (very low ξ) for both soil and bedrock (Q = 5000). For visco-elastic and non-linear (soft) materials, the energy dissipation at low strain was constrained to vary according to Vs, through the classical – never appropriately justified by measurements - relationship Q = $V_S/10$, or equivalently ξ = $5/V_S$ (Vs in m/s) (Olsen et al., 2003).

Non-linear soil properties

The non-linear properties of each layer were characterized using classical $G/G_{max}(\gamma)$ and $\xi(\gamma)$ curves, relating the decay of shear modulus (G) normalized by the elastic shear modulus (G_{max}) and increase of damping ξ with the shear strain γ. The $G/G_{max}(\gamma)$ and $D(\gamma)$ curves were constructed following a simple hyperbolic model based on the following equations:

$$K_0 = (1 - \sin(\phi)).OCR^{\sin(\phi)}$$

Eq 2

$$\sigma'_m = \sigma'_v (1 + 2K_0)/3$$

Eq 3

$$\tau_{\max} = \sigma'_m \sin(\phi)$$

Eq 4

$$\gamma_{ref} = \tau_{\max} / G_{\max}$$

Eq 5

$$G/G_{max} = 1/(1+\gamma/\gamma_{ref})$$

Eq 6

$$\xi = \xi_{min} + (\xi_{max} - \xi_{min})(\gamma/\gamma_{ref})/(1+\gamma/\gamma_{ref})$$

Eq 7

where the control parameters are the friction angle $\Phi = 30°$, the over-consolidation ratio OCR = 1 and the gravitational acceleration g = 9.81 m/s^2. Only cohesionless material was considered here, so that the shear strength τ_{max} is computed using the vertical stress and the friction angle. Both, $\sigma m'$ and $\sigma' v$ are the effective mean and vertical stresses; γ is the shear strain. The reference shear strain γ_{ref} corresponds to the strain for which G = 0.5G_{max} (in the hyperbolic model as describe above it is given by Eq 5), K_0 is the coefficient of earth pressure at rest, and ξ_{min} and ξ_{max} are the minimum damping values at very low strain (= intrinsic material damping considered above for the visco-elastic behavior), and the maximum at very high strain, respectively.

Only one $G/G_{max}(\gamma)$ and $\xi(\gamma)$ curves were provided for P1, five for P2 (increasing for each 20 m depth interval), and two for P3 (one for each homogeneous layer). We assume a constant strength per soil layer for all soil models. They are illustrated in Figure 2. For P1 and P2, they are fitting a hyperbolic curve defined by the low strain shear modulus $G_{max} = \rho V_S^2$ and the shear strength τ_{max} at the center of each layer or sublayer. For P3 the $G/G_{max}(\gamma)$ and $\xi(\gamma)$ chosen models were very similar to one another using the previous hyperbolic model. For P3, the set of Darendeli models (Darendeli, 2001) was used and adjusted to a simple hyperbolic model as for P1 and P2; as Darendeli's models are defined only up to a maximum shear strain of 1 %, the P3 curves were defined by multiplying the shear strength τ by factors 1.1 and 2 at depths of 10 m and 35 m, respectively, and the final curves were then computed based on the hyperbolic models associated to these values.

Some numerical codes include sophisticated constitutive models for NL soil behavior, which require very specific additional parameters, which should be consistent with the $G/G_{max}(\gamma)$ and $\xi(\gamma)$ curves supplied for the other codes. The definition of these additional parameters was done individually by each team, with the following simple assumptions: the soil is cohesionless (i.e. c' = 0 and Plasticity Index PI = 0), and the water table is located at 100m depth, the soil particle size distribution is defined with D_{10} = 0.2 mm and D_{50} = 0.35 mm, and a uniformity coefficient = D_{60}/D_{10} = 1.8.

TABLE 1

Figure 2 : Vs profiles, G/G_{max} and damping curves for the 3 idealized profiles.

Reference rock motion

In the first phase of the project, each participant was provided (i) a simple Ricker pulse input motion derived analytically, and (ii) two real acceleration time histories scaled to three different PGA levels (0.5, 1 and 5 m/s^2) to observe the evolution from linear to non-linear soil behavior. The two accelerograms were selected to be representative of very different frequency contents, in order to analyze its influence in the non-linear computations. Each accelerogram was pre-processed in the same way as explained further below. The Fourier transform of the three normalized input motions are illustrated in Figure 3.

The pulse-like input motion

The Ricker pulse input motion corresponds to acceleration, velocity and displacement time-histories defined by equations (8) to (10). A central frequency of 4 Hz was chosen to produce sufficient energy at the fundamental frequency of each of the three profiles, while having a broad band energy in the main bandwidth of earthquake geotechnical engineering, i.e. 1 -10 Hz.

$$a(t) = \left[1 - 2(\pi t f_c)^2\right]\exp(-(\pi t f_c)^2)$$

Eq 8

$$v(t) = t\exp(-(\pi t f_c)^2)$$

Eq 9

$$d(t) = \frac{1}{-2(\pi f_c)^2}\exp(-(\pi t f_c)^2)$$

Eq 10

where f_c is the central frequency and a(t), v(t) and d(t) are the acceleration, velocity and displacement time histories, respectively. The acceleration time histories and the normalized Fourier Transform spectra for the three input motions are illustrated in Figure 3.

Real reference input motions

To investigate the effect of frequency content on the computation of non-linear soil behavior, we used two real input motions with different frequency contents recorded at rock outcrop sites. One has a predominant frequency of 11.4 Hz, and the other of 4.8 Hz: they are labeled hereafter HF and LF, respectively. The metadata of these two recordings are described in Table 2 and their acceleration, velocity and displacement time histories are illustrated in Figure 3. We can observe that the spectral shape are quite different, the main energy of the signal for the LF motion lies between 0.5 to 10 Hz and for the HF motion between 5 to 20 Hz. The duration of the HF event is about 80 s while it is shorter for the LF motion around 15 s. In this work, we considered only the horizontal EW component of each recording.

TABLE 2

The velocity and displacement time histories of these two recordings were calculated from the original raw acceleration data, following this procedure: (1) removal of the mean, (2) zero padding of the signal by applying Boore's approach (Boore and Bommer, 2005) over a specific time duration corresponding to 20 s before the first, and after the last, zero-crossing of the original acceleration time series, (3) high-pass filtering of the signal, and (4) integrating twice to obtain consistent velocity and displacement time histories.

Figure 3: Normalized reference motion used for the verification phase of this project PRENOLIN.

Participants and tested numerical codes

We compared 23 different numerical codes used by 21 participating teams, as listed in Table 3. As some teams use several codes, each computational case/team is annotated by a letter and a number. Two or more teams used the same code, including Deepsoil (4 teams for the verification and 5 for the validation), FLAC (2 teams) and OpenSees (3 teams). Others used the same constitutive model, notably Iai's (1990) model (2 teams), Iwan's model (Ishihara, 1996; Iwan, 1967) (4 teams) and the Hujeux model (Aubry et al., 1982) (2 teams). The participants teams were composed by people having different background and expertise which can relevant for analyzing the site response variability. Firstly, two disciplines are represented in this benchmark, seismology and geotechnical earthquake engineering and secondly, the participants are either developers or users.

We identified three different, non-exclusive code groups, according to three main characteristics: (1) the type of numerical scheme, (2) the way to implement the attenuation, either in the low strain range or in the large strain range, and (3) the type of non-linear constitutive models. Each of these three groups is detailed in the next sections.

TABLE 3

The numerical scheme

The 20 codes that solve the problem in time domain are split in two main categories: two types of spatial approximations are considered:

(a) The Finite Element Method (FEM) is by far the most common, used by 18 teams and implemented in three different ways:

  i) Standard method (ST.FEM), used by 12 teams: B-0, D-0, H-0, L-1, M-1, N-0, R-0, S-0, T-0, U-0, W-0 and Z-1.

  ii) Spectral method (SP.FEM), used by 1 team: Q-0

  iii) Discontinuous Galerkin method (DG.FEM), used by 1 team: Y-0.

(b) The Finite Difference Method (FDM) is used by 10 teams: A-0, C-0, E-0, F-0, G-0, J-0, K-0, L-2, M-0 and M-2;

The last remaining teams (J-1, T-1 and Z-0) consider the problem in the frequency domain and use a linear equivalent method involving linear, visco-elastic material with several iterations to tune the visco-elastic properties in each layer to the shear strain and modulus reduction and damping curves (Schnabel et al., 1972).

Implementation of attenuation

Low strain attenuation: At low strain levels (less than 10^{-4}-10^{-2} %), elasto-plastic constitutive models and most of the non-linear models have damping values close to zero, which is physically unrealistic, since all soils and rocks exhibit a hysteretic behavior in the stress-strain plane even for weak deformations, indicating dissipation of energy.

In the frequency domain, implementation of a prescribed attenuation factor is relatively straightforward. In theory, fulfillment of the causality principle leads to a (slight) frequency dependence of the shear wave velocity, which should be specified (together with the damping value) at a specific frequency f_0 (Aki and Richards, 2002) . However, this is not implemented in all codes: some consider a truly frequency-independent attenuation with a defined reference frequency for the velocity, while some dropped the causality principle and have frequency independent velocities.

In the time domain, attenuation can be approximated by implementation of a set of relaxation functions using rheological models such as the generalized Maxwell model (Blanch et al., 1995; Day and Bradley, 2001; Day and Minster, 1984, 1984; Graves and Day, 2003) or modeled by a Rayleigh damping formulation. Both methods present pros and cons. The usage of rheological models to approximate attenuation is physical; however, adds memory constraints to the computations. The greater the number of relaxation functions used, the better the attenuation factor will be approximated (although one should not use too many (see for example Peyrusse et al., 2014). On the contrary, the Rayleigh damping method is much easier to be implemented numerically; nevertheless, the parameters are not easily determined, and automatically involve a significant frequency dependence of Q. For low attenuation (below a damping ratio of 20%) it has been shown that Rayleigh damping and the generalized Maxwell model become equivalent (Semblat, 1997).

For the entire set of codes tested here, four kinds of attenuation implementations were used:

(1) Frequency-independent attenuation (frIA): Some model considered frequency independent attenuation instead of the use of the frequency dependent Rayleigh Damping/attenuation in the time domain analysis. Models A-0, E-0, K-0, Q-0, T-1 and Z-0 use series of Maxwell/Zener elements (Blanch et al., 1995; Day and Bradley, 2001; Day and Minster, 1984, 1984; Graves and Day, 2003), which imply an almost constant attenuation over a specific, broad enough frequency range. Models F-0, J-0 and M-2 used the frequency independent attenuation as proposed in (Phillips and Hashash, 2009a).

(2) Frequency-dependent attenuation (frDA), such as the Rayleigh damping (simplified or full). It was used by 10 teams: B-0, G-0, H-0, L-1, M-0, M-1, R-0, S-0, T-0, W-0, Y-0 and Z-1.

(3) Low strain frequency independent hysteretic damping (LSHD). It was used by 4 teams: C-0, N-0, D-0 and R-0.

(4) Numerical damping (ND). 3 teams (U-0, N-0 and D-0) use variant of the Newmark integration scheme to simulate attenuation effects with purely numerical damping tools, while another team (L-1) used it to filter out numerical noise (NDfilt).

High strain attenuation: High strain attenuation can be computed directly from the hysteretic behavior of the soil subjected to strong ground motion (loading / unloading cycles). However, it was demonstrated it is difficult to reproduce simultaneously the specified decrease of G/G_{max} with increasing shear strain, and the increasing of damping. For this reason, a few teams (A-0, B-0, E-0 and T-0) chose to use a “damping control” (which implies a modification of the "Masing rules, and is thus labeled as ‘no-Masing rules’): it is based on a mapping that converts a hysteresis loop in such a way that it will satisfy the hysteretic damping at the current strain level (IAI et al., 1992). Other teams (J-0) used the method as proposed in Phillips and Hashash (2009a) which modifies the unload and reload rules of the extended Masing rules.

Non-linear constitutive models

In geotechnical earthquake engineering, non-linear soil behavior is a well-established concept. In laboratory experiments, such as cyclic tri-axial tests, the non-linear soil behavior is expressed by hysteresis loops in axial stress-strain plots, which can be linked to shear stress-strain plots. The soil response under cyclic loading (representing seismic loading) depends on the properties of the cyclic loading (e.g. time history, peak amplitude) and on the soil properties (e.g. strength, relative density).

In non-linear models, the true hysteresis soil behavior is simulated by the use of constitutive models which mimic the experimental hysteresis curves, or the shear modulus decay ($G/G_{max}(\gamma)$) and attenuation ($\xi(\gamma)$) curves.

According to information gathered from each participant, the codes tested here are implemented with various non-linear models, including:

- ⇒ Iai model (Iai et al., 2011; Iai and Ozutsumi, 2005): B-0, E-0,
- ⇒ MKZ modified hyperbolic model (Matasovic and vucetic, 1995, 1993): A-0
- ⇒ Cundall’s model (Cundall, 2006): M-0
- ⇒ Iwan’s model (Ishihara, 1996; Iwan, 1967): K-0, Q-0 , U-0, Y-0
- ⇒ Logarithmic function model (Puzrin and Shiran, 2000) : L-1
- ⇒ Modified Hujeux model (Aubry et al., 1982): D-0, N-0, S-0
- ⇒ Multiyield model (Elgamal et al., 2003; Yang et al., 2003): H-0
- ⇒ Extended Hyperbolic model (Phillips and Hashash, 2009b) : F-0, H-0, J-0, M-2, T-0
- ⇒ HSsmall (Isotropic hardening elasto-plastic soil model) (Schanz et al., 1999): Z-1
- ⇒ Pisanò 3D Elastic-plastic model (Pisanò and Jeremić, 2014): R-0;
- ⇒ BWGG: Extented Bouc Wen model (Gerolymos and Gazetas, 2005): G-0
- ⇒ Modified extended hyperbolic model: C-0
- ⇒ Manzari-Dafalias model: W-0 (Dafalias and Manzari, 2004)

In order to compare the different constitutive models, stress/strain controlled tests could have been conducted. However, some of the teams were not able to perform it. To overcome this difficulty, we asked the teams to compute nonlinear simulations with their codes on one of the idealized soil profile (P1) with a sinus input motion of increasing amplitude and with a rigid substratum base (Figure 4). The frequency of the input motion being low enough to avoid any issues with wave propagation. Moreover, the result of such simulation was asked at the node before the soil/bedrock interface, having a strength of 65 KPa.

The resulting plots are illustrated in Figure 5 for the total length of motion and in Figure 6 for a specific zoom on the first two cycles (blue for the first and red for the second).

The full duration of motion leads to very high strain levels (5%), and the stress-strain curves are highly variable from one computation to another. Even for a similar constitutive model, the curves can differ. For Iwan's model U-0 and Y-0 results are close one to another while different from K-0 and Q-0. The shape of the curves depends also on the use or not of damping control. For instance, teams A-0, B-0, D-0, E-0, J-0, T-0 and F-0 used damping control and all curves exhibit stress-strain curves with secant modulus degrading with strain. Note that J-1 is an equivalent linear method and the stress-strain curves do not exhibit hysteresis.

Some teams could not follow the prescribed shear strength values (M-0, M-1, M-2, R-0, S-0, W-0) mainly because of depth dependency of the shear strength implemented in the code. They used very different values; the comparison of the corresponding stress-strain curve is thus irrelevant. Therefore, we looked at the first two cycles of motion that involve much lower strain (not exceeding 0.5%): the stress-strain curves are closer to each another although some indicate larger hysteresis loop (B-0) or lower maximal shear strain (Z-1). This comparison helped to draw the attention on the lack of versatility of some of the used NL codes, because of some built-in features based on empirical correlations or geotechnical relations (between the shear strength and the confining pressure, for instance), which prevent from considering fully arbitrary sets of NL parameters.

Figure 4 : *Acceleration time history of the sinus motion with central 1s period*

Figure 5 : *Stress-strain curve for a soil element of shear strength 65kPa subjected to a sinusoidal input seismic motion of 10s**.***

Figure 6 : *Stress-strain curve for a soil element of shear strength 65kPa subjected to the first two cycles of a sinusoidal input seismic motion.*

Code usage protocols

Reference frequency for visco-elastic damping (Maxwell/Zener Model)

Relatively little is known about low-strain, intrinsic attenuation in real soils. Its traditional implementation supposes frequency independent damping values. This is readily achieved using the Kelvin-Voigt model when solving the wave propagation in the frequency domain (Ishihara, 1996). Conversely, the Maxwell/Zener generalized body better describes inelastic material properties in both the time and frequency domain solution of wave propagation (Moczo et al., 2004). However, the use of this rheology implies a slight velocity dispersion to fulfill the causality principle. It is therefore needed to carefully define a reference frequency for the reference velocity value, especially when different numerical methods are compared with one another (Peyrusse et al., 2014). *[This reference frequency must not be confused with the frequency bandwidth definition of the quasi-constant Q value used in the frequency independent attenuation method aforementioned, it should simply be within this frequency bandwidth.]*

A reference frequency was thus defined for each profile, at which common velocity and attenuation values were fixed. As indicated by some authors (Liu and Archuleta, 2006; Moczo et al., 2004) the values of reference frequency used in most cases is close to 1 Hz (as many 3D computations including shallow, soft material, have rather low upper bound maximum frequencies). On the other hand, it is often suggested to select a frequency close to the frequency of interest. In our case, given the definition of the pulse-like motion, we chose a reference frequency of 4 Hz, i.e. the central value of the input wavelet.

Definition and implementation of the reference motion

We tested two base conditions at the sediment-substratum interface: (i) an elastic base, and (ii) a rigid base. The first condition corresponds to the usual hazard assessment studies, where the rock ground motion is derived from deterministic or probabilistic analysis, and corresponds to the design motion at the surface of an outcropping rock. The second one corresponds to the case where a recording is obtained at depth within a down-hole array, and is used to derive the motion at surface or shallower depths. Depending on the communities or point of views, the implementation of input (or reference) motions into algorithms can be quite different, indicating that the terms "input motion" or "reference motion" are not understood in the same way by all the participants. For the seismological community, the input motion is often seen as the seismic signal carried by the up-going incident wave, while for the geotechnical community, it is often understood as the motion at a given reference rock site, resulting from the total-wavefield (up-going and down-going waves): this reference site may be either at rock surface (it then includes the free-surface effect), or at depth (for instance the downhole sensor of a vertical array, which includes the interferences between the up-going and down-going waves).

For the case of a perfectly rigid substratum, the reference input motion is the signal imposed at the soil-bedrock interface. This definition was clear among all teams. It was not so clear for the elastic substratum condition, whereby a more precise definition was required, since the greatest differences in the first round results came from different understandings of the term "input motion" by the various teams. The terminology must therefore be clearly stated:

- Outcrop motion: Seismic motion recorded at the surface and corresponding to free surface conditions in the outcropping rock. For 1-D cases, with vertically propagating seismic waves and homogeneous rock, this free-surface effect is simply a frequency-independent factor of 2, with respect to the up-going wave signal.

- Surface motion: Seismic motion recorded at the free surface of a sedimentary site and subjected to amplification effects.

- Within motion: Seismic motion recorded at depth, usually at a downhole site: in our case, this location corresponds to the interface between sediment and rock substratum (i.e., z = 20, 100 and 50 m, for profiles P1, P2 and P3, respectively). This motion contains the total wave-field composed of the incident up-going and reflected down-going waves.

- Incident motion: Seismic motion that is carried by the incoming waves just before they enter the sedimentary filling. In our case, it is the seismic motion carried by the vertically incident plane wave, and it cannot be measured directly.

Considering the confusion among the participants linked with different working traditions in different communities, we decided to use the concepts of “outcrop” and “within” input motions to define the "reference motion" at the downhole sensor, as recommended by Kwok et al. (2008) and Stewart and Kwok (2009). In linear/equivalent linear/non-linear site response analyses, two cases can be distinguished:

(1) if the reference motion is an outcrop recording, then one should use an elastic base condition with an up-going wave carrying a signal equal to exactly half the outcropping motion;

(2) if the reference motion is a within motion recorded by a downhole sensor, then one should use a rigid base condition without modifying the input motion.

In order to avoid any ambiguity, we will systematically use the expression "reference motion" which should be understood as detailed above for the elastic and rigid base conditions

Comparison of predictions

Methodology of comparison

The participants were asked to compute the acceleration and stress-strain time histories at virtual sensors located at different depths within the soil profile. A total of ten virtual sensors were selected for each profile, with a depth interval equal to 1/10th of the total soil thickness: every 2 m for P1, every 10 m for P2 and every 5 m for P3. Acceleration and stress-strain values should be computed at staggered points: from the very surface for acceleration, and from half the depth interval for stress-strain values.

From the "raw" results provided by each participant, a comparative analysis was performed on the computed acceleration time histories, transfer function, 5% pseudo-response spectra, the depth distribution of peak shear strain and PGA, and the stress-strain plots at different depths. Such comparisons were done for each profile, for each computational case (linear vs. non-linear, elastic vs. visco-elastic soil behavior, and rigid vs. elastic substratum conditions) and for the different input motions.

For the sake of simplicity and conciseness, the main section of the present article presents results for only the P1 case. The P2 and P3 profiles are compared to P1 results in terms of variability of the surface motion only, but the conclusions are based on the results from all three profiles.

Visco-elastic computations

Figure 7 displays the comparison for the P1 profile of the surface acceleration for the pulse-like motion under an elastic substratum condition, for the linear elastic computation for a short window (3 s) of signal. All results converged towards the analytical solution calculated with the Haskell-Thomson method (Haskell, 1953; Thomson, 1950), but this was achieved only after the second iteration. There were indeed unexpected and significant discrepancies in amplitude at the end of the first iteration, and that came from: (1) inconsistent implementation and understanding of the term "input motion" (clarified as mentioned in the code usage protocols), (2) problems with units, or (3) representations of soil properties. During the first iteration, som phase discrepancies could be also identified, associated either to the assignment of the "input motion" at different depths some distance below the sediment/rock interface (which caused a constant time delay), or to increasing time delays for the late cycles, that were associated to numerical dispersion.

Figure 7*: Comparison of the acceleration at the surface of P1 profile, for the pulse-like input motion, for the linear elastic computation and for the elastic substratum case.*

Figure 8 shows the results of visco-elastic computations of the acceleration at the surface of the pulse-like motion with a rigid substratum condition. The convergence was also obtained after the second iteration, with minor corrections (similar to the ones observed for the elastic case) and after having specified the reference frequency to be considered for the implementation of damping. We chose a reference frequency of 4 Hz, which is exactly the central frequency of the pulse-like motion (Figure 8).

Figure 8 *Comparison of the acceleration at the surface of P1 profile, for the pulse-like input motion, for the linear visco-elastic computation and for the rigid substratum case.*

These unexpected issues were corrected after the first iteration to ensure a satisfactory convergence. This should however raise our awareness on the possibility of such misunderstandings and resulting errors, when site response computations are asked without clear enough specifications about the definition of the reference motion.

Non-linear computations

Once agreement between the model predictions was reached for simple, linear cases for which analytical solutions are available, the variability of the results of non-linear calculations can be fully associated with differences in implementation of non-linear soil behavior.

Figure 9 compares the Fourier transfer functions (surface over reference bedrock motion) and Figure 10 compares pseudo-response spectra at the surface for the P1 profile, with a rigid substratum case. The subplots of these two figures illustrate the results for the high frequency (HF) waveform scaled to the lowest (0.5 m/s^2) and largest PGA (5 m/s^2) (a and c, respectively), and for the low frequency (LF) waveform scaled to the lowest and largest PGA (0.5 m/s^2 – b, and 5 m/s^2, d, respectively). The frequency content of the input motion and the scaling of the input motion prove to have a large influence on the non-linear soil behavior in the numerical simulations, and consequently on the variability of the results.

While the results from all teams exhibit a very satisfactory similarity (though larger than for the visco-elastic case) for the HF waveform scaled to the lowest PGA (a), differences between the model predictions are much greater for the highest PGA (c). This observation is more pronounced when looking at the LF input motion. Even for the lowest PGA (b), the variability increases significantly compared to the HF input motion, and it becomes very large for the large amplitude LF motion (scaled to 5 m/s^2, d).

The amount of variability between the results has been quantified through the calculation of the standard deviation (in log10 unit) for each frequency value and is illustrated in Figure 11. The variability is greater for the low frequency content input motion scaled to the highest PGA except close to the first frequency peak of the linear transfer function. As expected, strong non-linear soil behavior during this solicitation shifts the first frequency peak of the transfer function to the low frequencies. The variability of the transfer function is similarly shifted.

Such variability is strongly linked to the peak shear strain reached in the soil column. For the LF input motion scaled to the highest PGA, the threshold shear strain above which the numerical simulations can no longer be considered as reliable (according to their authors), was reached by some codes. Indeed, some teams (L-1 and Z-0) consider a maximal reliable deformation between 1 to 2%; while others consider their code to work well over a wide range of deformation and are limited by the dynamic soil properties resolution only. For the computations using the HF and LF motions scaled to the highest PGA, we observe that the two equivalent linear methods (J-1 and Z-0) exhibit a very high de-amplification beyond 7 Hz, compared to the other simulations, which shows the classical over-damping limitation of that method. For the last two cases (HF and LF accelerograms scaled to 5 m/s^2), the peak shear strain values are illustrated in Figure 12. It was calculated for each code/team couple, and for all the 10 sensor depths of the P1 profile. The largest peak strain values, largely exceeding 1%, are reached at the deepest points for the LF input motion, while it remains about 10 times smaller (max 0.3%) for the HF motion, despite the identical PGA values on the input motion. Besides, given the shape of the G/G_{max} and $\xi(\gamma)$ curves, one may notice that the frequency-content of the input motion induced variability in the peak shear strain results which correspond to an even larger variability in the G/G_{max} and $\xi(\gamma)$ values. For instance, at 7m depth, the peak shear strain for the LF motion is between 0.02 to 1% while it is between

0.03 to 0.1% for the HF motion. This makes the G/G_{max} varies from 0.28 for the LF motion to 0.8 for the HF motion. Thus, one may understand that the results will be very sensitive to the details of the constitutive model and the way that G/G_{max} and ξ (γ) curves are approximated.

Incidentally, one may also notice that for P1, the peak shear strain occurs at the deepest point, close to the sediment/bedrock interface. Indeed, wave propagation in nonlinear media is the cumulative effect of impedance contrast at the soil-bedrock interface, material strength, and intensity of the input motion. These combined effects make it difficult to analyze these results even when they are numerical and consider simple soil geometry.

Figure 9 : Comparison of the surface to reference Fourier spectra ratio, for the non-linear comparison using for the left sub-graphs the high frequency input motion and for the right sub-graphs the low-frequency input motion and with for the first line the weakest input motion PGA and the second line the highest input motion PGA.

Figure 10 : Comparison of the acceleration pseudo-response spectra at the ground surface, for the non-linear computation using for the left sub-graphs the high frequency input motion and for the right sub-graphs the low-frequency input motion and with for the first line the weakest input motion PGA and the second line the highest input motion PGA

Figure 11 : Standard deviation (in log unit) of the transfer function (left panel) and response spectra (right panel) depending of the input motion used.

Figure 12 : Peak shear strain profiles reached at each depth by each team for the high and low frequency reference motion scaled at the highest PGA level (5 m/s^2), for the profile 1 and for rigid substratum conditions

Epistemic uncertainty

Quantification of the variability of the results

We quantified the variability between the simulations by the standard deviations (log10 units) of several ground motion intensity parameters, starting with PGA values [σ_{PGA}], and then considering pseudo-response spectrum ordinates at different periods [$\sigma_{PSA(T)}$], peak strains [$\sigma_{\gamma max}$], and a few energy related quantities.

The PGA values at the surface are first compared with the empirical variability (i.e. single station, within-event variability "Φ_{SS}"). Figure 13 illustrates the evolution of σ_{PGA} for the surface site of P1 for the 5 different computational cases and the different reference motion and boundary conditions. These are the linear–elastic, the linear-visco-elastic, and the non-linear computations with the input motions scaled to the lowest (0.5 m/s^2), intermediate (1 m/s^2) and highest (5 m/s^2) PGA. The σ_{PGA} is calculated for the pulse-like, the HF and the LF motions. The left subplot displays the results for the rigid substratum case (reference motion = within motion at sediment-basement interface), while the right subplot stands for the elastic substratum case (reference motion = outcropping rock motion). The most striking features of these plots can be summarized as follows:

a) the (almost) systematic increase of σ_{PGA} with increasing PGA level, whatever the input signal and the type of boundary conditions
b) the (almost) systematically larger values of σ_{PGA} for the LF input motion compared to the HF input motion case (around twice greater for the three PGA values) : this corresponds to the higher strains generated by the LF motion. A similar plot as a function of peak strain instead of peak ground acceleration would exhibit a larger continuity between results of both input waveforms
c) the larger σ_{PGA} values for non-linear computations compared to the linear case (except for the very specific case of linear-elastic response with rigid boundary conditions, discussed later)
d) the maximum obtained σ_{PGA} value (0.15) remains below the specific single-station, within-event variability $\Phi_{SS,PGA}$ value for a site with a V_{S30} equivalent to P1 (Rodriguez-Marek et al., 2011), which is around 0.2. The uncertainties linked with the

> NL simulations remain below the “natural” single site response variability. The latter one however includes the sensitivity to the characteristics of the incident wavefield, which is not accounted for here as only vertically incident plane waves are considered. Nonetheless, the use PGA as a main metric is not enough. It is helpful to use spectral accelerations at other periods as well.

Our results indicate an exceptionally high σ_{PGA} value for one linear computation, the linear-elastic one with the HF reference motion and rigid boundary conditions. This computational case is the simplest but also the most demanding for a propagating seismic wave. Considering that no seismic attenuation (damping) is considered for this specific computation (in the material or in the substratum), some codes usually use numerical attenuation to control real motion amplitudes. Thus, the high uncertainty observed here reflects variability in the implementation of the numerical damping for each code/team couple, together with the high sensitivity to the configuration, with a non-zero Fourier content of the reference motion at depth, at a frequency where destructive interferences between up-going and down-going waves should result in a null motion.

Figure 13 : Standard deviation (in log10 unit) of the PGA at the surface of the P1 profile, for the 5 different computational cases (linear –elastic, linear visco-elastic, non-linear with input motion scaled to the lowest (0.5m/s^2), medium (1m/s^2) and highest (5m/s^2) PGA, for the pulse-like, the high frequency and the low frequency content motions. The left sub-plot shows the results for the rigid substratum case and the right sub-plot for the elastic substratum.

We then explored the variability of various seismic intensity measures: (i) the response spectra at the surface (SA) at three different periods (0.1, 1 and 3 s), (ii) the peak shear strain at the bottom of the sediment layer (ε), (iii) the Cumulative Absolute Velocity (CAV), (iv) the Arias Intensity (IA), (v) the root mean square acceleration (Arms), and (vi) the 5%-95% Trifunac-Brady duration (DT). The tendencies are quite similar for the HF and LF motions, but are sensitive to the sediment/substratum limit condition (elastic vs. rigid). Considering that σ_{PGA} is greater for the LF motion, we choose that motion to illustrate the results in Figure 14.

For the rigid substratum case (left subplot), three groups of intensity parameters can be identified. The first group is composed of duration-dependent intensity parameters, i.e., CAV, IA and DT, which exhibit the largest σ values. The second group is composed of acceleration parameters (PGA, SA(T), Arms) and characterized by a lower σ, especially for long period [SA (T = 1 s)]. The third group consists only of the peak strain, with generally intermediate σ values, which however exhibit the largest variability form one case to another. These three groups can also be distinguished in the elastic substratum case (right subplot), for which the largest case-to-case variability is also observed for the peak strain, exhibiting the highest σ for the highest PGA values. The duration-dependent parameters of the first group are less variable under elastic boundary conditions especially at low to intermediate PGA levels and in the linear domain: rigid base conditions are very demanding for low damping materials, which maps much more on duration than on peak values.

Figure 14 : Standard deviation (in log unit) of the different intensity parameters for the P1 profile, for the 5 different computational cases (linear –elastic, linear visco-elastic, non-linear with input motion scaled to the lowest (0.5m/s^2), medium (1m/s^2) and highest (5m/s^2) PGA, for the low frequency content motion. The left sub-graph shows the results for the rigid substratum case and the right sub-graph for the elastic substratum.

The other profiles provided similar results as to the variability of predictions. As an example, Figure 15 compares the PGA variability, for the LF motion and a rigid substratum case, for the three profiles. The trends are similar for the three profiles: similar σ values, same tendency to increase with PGA. These results also stand for the elastic substratum case, as well as the fact that the variability σ is lower for the HF motion for the three profiles, by about a factor of two compared to the LF motion.

Figure 15 : Standard deviation (in log unit) of the PGA for the profile 1 2 and 3, for the 5 different computational cases (linear –elastic, linear visco-elastic, non-linear with input motion scaled to the lowest (0.5m/s^2), medium (1m/s^2) and highest (5m/s^2) PGA, for the low frequency content motion and for the rigid substratum case.

Origins of the variability: Can it be reduced?

Definition of Groups and Sub-groups

We considered four a priori ways to group the results according to some characteristics of the numerical codes: (G1) implemented attenuation method, (G2) numerical scheme, (G3) constitutive model, (G4) shape of the hysteretic curve according to (1) the ability to represent the actual shear strength value (here at the bottom of P1), and (2) the use or not of Masing rules for the loading/unloading path (damping control or not). Each group is further sorted into several sub-groups as follows.

Case G1 concerns the implementation of linear, intrinsic damping, as defined in the first part of this article. It is sub-divided into 3 sub-groups: (i) G1a: frequency-independent attenuation (A-0, E-0, F-0, J-0, J-1, K-0, M-0, Q-0 and Z-0), (ii) G1b: Rayleigh damping (B-0, G-0, H-0, L-1, M-1, R-0, S-0, T-0, W-0, Y-0 and Z-1), and (iii) G1c: low strain hysteretic damping (C-0, N-0, D-0 and R-0).

Case G2 is based on the numerical discretization scheme, which is sub-divided into 2 subgroups: (i) G2a: finite-element (B-0, D-0, H-0, L-1, M-0, N-0, Q-0, R-0, S-0, T-0, U-0, W-0, Y-0 and Z-1), and (ii) G2b: finite-difference (A-0, C-0, E-0, F-0, G-0, J-0, K-0, L-2, M-2). A third sub-group could be considered G2c: consisting of equivalent linear codes working in the frequency domain (J-1 and Z-0).

Case G3 is based on the constitutive model. To ensure sufficient teams within each group, we split the code/team couple into 4 sub-groups according to the main constitutive model used:

(i) G3a: IaI's model (B-0, E-0, Q-0), (ii) G3b: Iwan's model (K-0, L-1, U-0, Y-0), (iii) G3c: Philips and Hashash's model (F-0, J-0, L-2, M-2, T-0), and (iv) G3d: all other models.

Case G4 is based on the shape of the hysteresis loop according to (1) the shear strength used by each code/team couple and (2) the use of Masing rules or not for the loading/unloading path.

In the "canonical" models initially designed by the organizing team, the soil shear strength profile was assumed to be constant with depth in each soil layer, and had prescribed modulus reduction and damping curves. However, in most real situations, the shear strength should increase with depth. Even though these profiles were considered as "idealized" and simply intended to perform these verification tests, some teams felt very uncomfortable with this unrealistic assumption and decided to change the shear strength profile, by introducing a more realistic increase in shear strength with depth, having nevertheless, the imposed strength values at the center of each layer. Consequently, the actual non-linear soil parameters considered by each team were not identical, which is certainly responsible for part of the final variability observed, especially for large ground motions, for which the actual strain and damping are more sensitive to the shear strength than to the shear velocity, particularly at or close to major interfaces. For this reason, we further sorted each code/team couple into 2 sub-groups, by analyzing the stress-strain plots for the LF motion and the highest PGA at the bottom of P1 (illustrated in Figure 16). We choose this computational case because it is the most challenging in term of maximal shear strain reach in the soil column and therefore can highlight the differences between the computations. We found the following sub-groups: (i) shear strength is equal to 65kPa, as stated by the organizing team (A-0, B-0, C-0, E-0, F-0, G-0, H-0, K-0, Q-0, U-0, T-0, Y-0), and (ii) all others that exceeded this value (D-0, J-0, J-1, L-1, N-0, M-0, M-1, M-2, R-0, S-0, W-0, Z-0, Z-1).

In addition, we also consider the damping control implementation, (or in other words the use or not of the Masing loading/unloading rules). It has a major influence on the hysteresis curves and hence on the non-linear soil behavior, also illustrated in Figure 16. It is split into 2 sub-groups: (i) damping control is used, i.e. the Masing rules are not applied (A-0, B-0, E-0, F-0, J-0, M-2), and (ii) no damping control used (all other teams).

Combining these two last parameters we end-up for G4 with three subgroups as follow: (i) G4-a: Specified shear strength and use of damping control (A-0, B-0, E-0, F-0, T-0), (ii) G4-b: Specified shear strength and no use of damping control and (C-0, G-0, H-0, K-0, Q-0, U-0, Y-0) (iii) G4-c: Different shear strength (D-0, J-0, J-1, L-1, N-0, M-0, M-1, M-2, R-0, S-0, W-0, Z-0, Z-1).

Figure 16 : Stress-strain curves at the bottom of P1 Profile for the Rigid substratum case subjected to the low frequency motion (in color and the high frequency motion in black scaled to the highest PGA (5m/s2). The grey curves are for code/team couples that exceed the specified shear strength of 65 KPa, whereas the coloured curves represent the code/team couples that use 65 KPa. The red curves are for codes using damping control and the blue curves the others.

Variability within the sub-groups

Considering the level of code-to-code variability, and its increase with PGA or strain level, a major issue regarding non-linear computations is whether such variability, i.e. the uncertainty in the predicted motion, is intrinsic to these kinds of calculations, or can be reduced, and in the latter case, how? We thus looked at the variability within each subgroup of the four main grouping, in order to identify those, which are associated to a significantly reduced scatter.

The standard deviations (σ_{log}, calculated in $\log_{10}$ units) of three parameters describing the computed surface accelerations and the strain levels at the bottom of P1, were used as a metrics to validate the ability of a given grouping item to reduce the scatter of results. These parameters are the surface PGA and the acceleration response spectra (RS) at periods 0.3 s and 0.09 s (corresponding to P1's first and second resonance frequencies, respectively). For each, the variability was measured within each subgroup of the 4 groups. If the groupings are physically relevant, the within-subgroup variability should be significantly reduced.

Figure 17 shows the standard deviation values for each sub-group in each group (G1, G2, G3 and G4) relative to the general standard deviation (all unsorted code/team couples) illustrated by the dotted gray line. The standard deviation of the PGA, response spectra at two periods and maximal deformation are calculated on the results for the profile P1, with the rigid substratum case and using the low frequency input motion scaled to the highest PGA (i.e. the motion that induces the strongest deformation in the soil column).

G1 and G2 (i.e. low strain attenuation and numerical scheme implementation, respectively) do not exhibit much lower σ_{log} values compared to the general σ_{log}, except for the lowest PGA input motion. Conversely, G3 to G4 (i.e. constitutive model, shear strength and damping control groups) do show reduced σ_{log} relative to the general σ_{log}, with G4 demonstrating the strongest reductions (by at least a factor of 2).

We can therefore conclude that (i) the shear strength is a key parameter for non-linear computations, and (ii) the constitutive model has a large influence; however (iii) the use (or not) of Masing rules appears to have an even greater influence for strong input motion.

Figure 17 : Standard deviation values (σ_{log}, in log_{10} units) of four parameters for the non-linear computation using the low-frequency content input motion scaled to the highest PGA : PGA (upper left), Response spectra at 0.27 s (upper right), Response spectra at 0.09 s (lower left) all three at the surface of P1 and the maximal shear deformation at the bottom of the P1 profile (lower right). The standard deviation are given for each group of the four groupings: depending on their low strain attenuation implementation (G-1) their numerical scheme (G-2) their constitutive models (G-3) and their values of shear strength at the bottom of P1 and use of damping control or not (G-5). The grey area illustrates the standard deviation for all code/team couples.

Figure 18 compares the pseudo-acceleration response spectra at the surface of the P1 profile with a rigid substratum condition subjected to LF and HF input motions scaled at the medium (1m/s^2) and highest (5 m/s^2) PGA levels. The response spectra are sorted according to the G4 sub-grouping, and the associated σ_{log} is represented by the thin lines on top of each subplot (the numbers on the right side indicate the number of code/team pairs in each sub-group). G4 enables a clear distinction of the response spectra; particularly for the most demanding LF input motion. The σ_{log} values from the two sub-groups with identical τ_{max} (G4a and G4b) are considerably reduced below 2 s, compared to the rest of the computations (G4c). This period bandwidth is relative to the PGA of this LF input motion. Similarly, for the HF input motion, the σ is reduced below 1 s.

Besides, the response spectra computed for the strongest input motions (HF and LF) at the surface of groups G4a and G4b are significantly different one to another which show the large impact of using damping control or not. The response spectra computed with damping control are more damped at intermediate frequencies (period between [0.2 to 0.7] s and [0.2 to 1] s for the HF and LF motion respectively) and less attenuated at low frequencies (periods greater than 0.7 and 1 s for the HF and LF motion respectively).

Figure 18 : Comparison of the pseudo- acceleration response spectra at the ground surface of P1 with rigid substratum condition, for the non-linear computation using for the left sub-graphs the high frequency input motion and for the right sub-graphs the low-frequency input motion and with for the first line the middle input motion PGA and the second line the highest input motion PGA. The response spectra were sorted according to three groups: group 1 is composed of the code/team couples using similar τ_{max} and damping control constitutive model. Group 2 use similar τ_{max} and no damping control and Group 3 are the other code team couples.

Conclusions

In the PRENOLIN's verification phase, the linear computation involving a simple pulse-like (Ricker) input motion proved to be very useful in understanding and eliminating some of the discrepancies between the different numerical codes that were compared. It was found that code-to-code differences can be attributed to three different sources: (1) minor mistakes in input parameter implementation or output units, (2) different understanding of the expression "input motion" within different communities, and (3) different intrinsic attenuation and numerical integration implementations. This benchmark showed that any nonlinear code should be tested with simple linear cases before going into nonlinear computations to ensure the proper implementation of the elastic soil parameters.

Most of the codes tested in this verification benchmark were designed mainly for non-linear computations. Therefore, although the codes should well reproduce the soil behavior at low strains, their actual performance are mainly tested for their soil behavior predictions during strong shaking in real cases.

The results obtained so far indicate a code-to-code variability, which increases with the shear strain level (which in turn depends on both the PGA level, stiffness of the soil and the frequency content of the reference input motion). We also found that, whatever the soil profiles used (among the 3 soil profiles considered), the overall code-to-code variability in the worst case (with strain levels exceeding 1%) remained lower than the random variability of GMPE single-station σ values for PGA. Nevertheless, an important conclusion is that given the scatter in the nonlinear results, a realistic analysis should use more than one code to perform a site response computation.

The effect of different non-linear soil model implementations was explored in this study and our main observations indicate that the epistemic uncertainty (i.e. the code-to-code variability) can be significantly reduced by describing more precisely some specific input parameters, especially the soil shear strength profile, which is found to be a key specification in addition to the degradation curves. In addition, for one particular non-linear soil model implemented in different codes (Iai's model), the variability of the stress-strain curves were found to be large, and mainly caused by the damping control parameter, depending on whether it was used to simultaneously fit the strain-dependence of both shear modulus and damping, or not, in order to follow the Masing loading/unloading rules. All these features and conclusions need to be checked against actual data to provide support for defining best practice for modeling out of the many available: vertical arrays with multiple down-hole sensors are the best available in-situ instrumentations to go forward. The benchmark undoubtedly benefits a lot from the various expertise fields of the participants ranging from geotechnical earthquake engineering to engineering seismology.

Data and ressources

Time histories used in this study were collected from the KiK-net web site www.kik.bosai.go.jp and http://www.kik.bosai.go.jp/kik/ (last accessed November 2011) and from University of Iceland, Engineering Research Institute, Applied Mechanics Laboratory, Reykjavik, Iceland.

Acknowledgements

Such an exercise was made possible by the interest and funding of the French and Italian nuclear industry under the project "SIGMA". It was made successful thanks to the dedicated and proactive participation of many teams from all over the world: A large number of teams reacted very positively to our invitation to take part in this (risky) benchmarking exercise,

among them the developers of a wide variety of internationally used constitutive laws and/or codes. Such a broad participation witnesses the actual need for such a carefully controlled comparison, and also brought an invaluable enrichment to the project, which undoubtedly benefitted greatly from the deep expertise of the participants. PRENOLIN is part of two larger projects: SINAPS@, funded by the ANR (the French National Research Agency), and SIGMA, funded by a consortium of nuclear operators (EDF, CEA, AREVA, ENL).

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TABLES

Table 1: Soil properties for all three simple profile cases studied here (P1-3), for the elastic and non-elastic domains.

Profile	LINEAR									NL
	Z [m]	**Vs** [m/s]	**Vp** [m/s]	**ρ** [kg/m^3]	**Q Elastic**	**ξ_{min} Elastic**	**Q Visco-Elastic**	**ξ_{min} Visco-Elastic**	**F_0 Linear Elastic** [Hz]	
P1	0-20	300	700	2000	5000	10^{-4}	30	0.0166	3.75	N°1-P1
	-	1000	1900	2500			200	0.0025		-
P2 *Mono-layer + $V_{gradient}$*	0-20	150-500	360-1220	2000			34	0.01547	1.16	N°1-P2
	20-40						40	0.0250		N°2-P2
	40-60						44	0.0113		N°3-P2
	60-80						47	0.0106		N°4-P2
	80-100						49	0.0102		N°5-P2
	-	2000	3700	2500			200	0.0025		-
P3 *Bi-layer*	0-20	300	700	2000			30	0.0166	1.48	N°1-P3
	20-100	600	1500	2000			60	0.0083		N°2-P3
	-	2000	3700	2500			200	0.0025		-

Table 2 Seismic metadata of the two real input motions used in the verification phase of the Prenolin project.

Event Freq. Content	**Event ID**	**Mw**	**Z** [km]	**Epi. Dist.** [km]	**Station ID**	**Station Geology**	**Seismo. Comp.**	**Vs30** [m/s] Mean harmonic S-waves velocity over the first 30m depth
HF	IWTH-170112022202	6.4	122	39	IWTH17 (Kik-net, Japan)	Rock	EW	>1200
LF	06756. 20000617	6.6	15	5	Flagbjarnarholt (Iceland)	A	H1	Unknown

Table 3: Participants to the PRENOLIN project Verification phase.

Team Name	Affiliation	Team Index		Code Name	*Code Reference*
D. Assimaki & J. Shi	*Georgia tech, US*	A	0	GEORGIA-NL-FDM	(Matasovic and Kavazanjian Jr, 2006; Matasovic and vucetic, 1993)
S. Iai	*DPRI, Japan*	B	0	FLIP	(Susumu Iai, 1990)
S. Kramer	*Univ. Washington, US*	C	0	PSNL	(In development)
E. Foerster	*CEA, France*	D	0	CYBERQUAKE	(Modaressi and Foerster, 2000)
C. Gelis	*IRSN, France*	E	0	NOAH-2D	(Susumu Iai, 1990)
A. Giannakou	*Fugro, France*	F	0	DEEPSOIL 5.1	(Hashash et al., 2012)
G. Gazetas E. Garini & N. Gerolymos	*NTUA, Greece*	G	0	NL-DYAS	(Gerolymos and Gazetas, 2006, 2005)
J. Gingery	*UCSD, US*	H	0	OPENSEES-UCSD-SOIL-MODEL	(http://opensees.berkeley.edu/)
Y. Hashash & J. Harmon	*Univ, Illinois,US*	J	0	DEEPSOIL-NL 5.1	(Hashash et al., 2012)
		J	1	DEEPSOIL-EL 5.1	(Hashash et al., 2012)
P. Moczo, J. Kristek & A. Richterova	*CUB*	K	0	1DFD-NL-IM	…
S. Foti & S. Kontoe	*Politecnico di Torino & Imperial College, Italy*	L	1	ICFEP	(Kontoe, 2006; Potts and Zdravkovic, 1999; Taborda et al., 2010)
		L	2	DEEPSOIL-NL 5.1	(Hashash et al., 2012)
G. Lanzo, S. Suwal, A. Pagliaroli & L. Verrucci	*Univ. Rome La Sapienza, Italy*	M	0	FLAC_7,00	(ITASCA, 2011)
		M	1	DMOD2000	(Matasović and Ordóñez, 2007)
		M	2	DEEPSOIL 5.1	(Hashash et al., 2012)
F. Lopez-Caballero & S. Montoya-Noguera	*ECP, France*	N	0	GEFDyn	(Aubry and Modaressi, 1996)
F. De-Martin	*BRGM, France*	Q	0	EPISPEC1D	(Iai, 1990) http://efispec.free.fr

B .Jeremić , F. Pisanò & K. Watanabe	***UCD, LBLN, TU Delft & Shimizu Corp***	R	0	real ESSI Simulator	http://sokocalo.engr.ucdavis.edu/~Jeremić/Real_ESSI_Simulator/
A. Nieto-Ferro	***EDF, France***	S	0	ASTER	http://www.code-aster.org
A. Chiaradonna, F. Silvestri & G. Tropeano	***UNICA and Univ. Naples, Italy***	T	0	SCOSSA_1,2	(Tropeano et al., 2015)
		T	1	STRATA	
M.P. Santisi d'Avila	***Univ. Nice Sophia-Antipolis, France***	U	0	SWAP_3C	(Santisi d'Avila et al., 2012, 2013; Santisi d'Avila and Semblat, 2014)
D. Mercerat and N. Glinsky	***CEREMA, France***	Y	0	DGNL	(Mercerat and Glinsky, 2015)
D. Boldini, A. Amorosi, A. di Lernia & G. Falcone	***Univ. Bologna and Sapienza University of Rome, Italy***	Z	0	EERA	(Bardet et al., 2000)
		Z	1	PLAXIS	(Benz, 2006; Benz et al., 2009)
M. Taiebat & P. Arduino	***Univ. Vancouver, Canda***	W	0	Opensees	(http://opensees.berkeley.edu/)

List of Figures

Figure 1

Click here to download Figure fig_1_Orga_verification.png

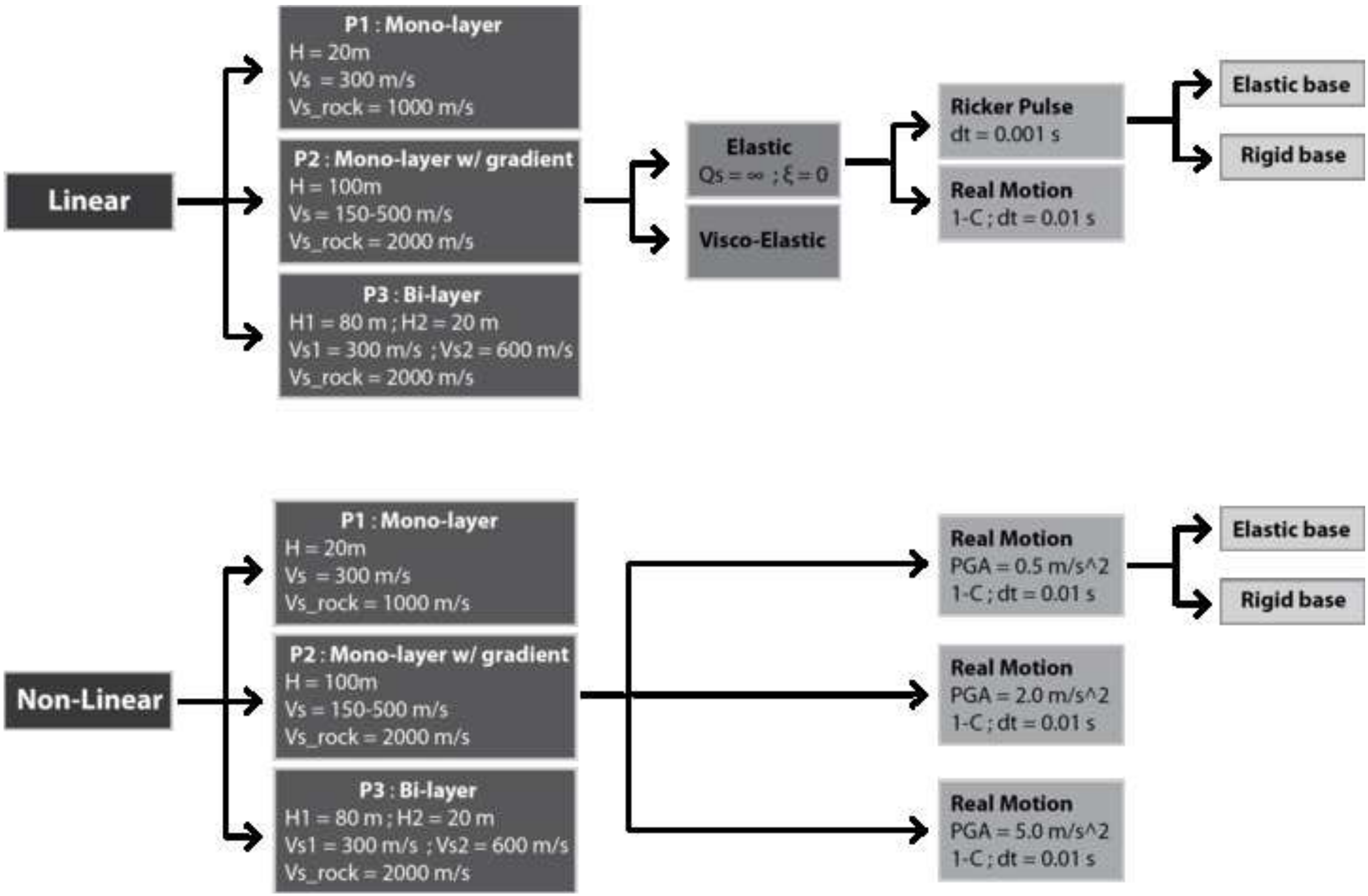
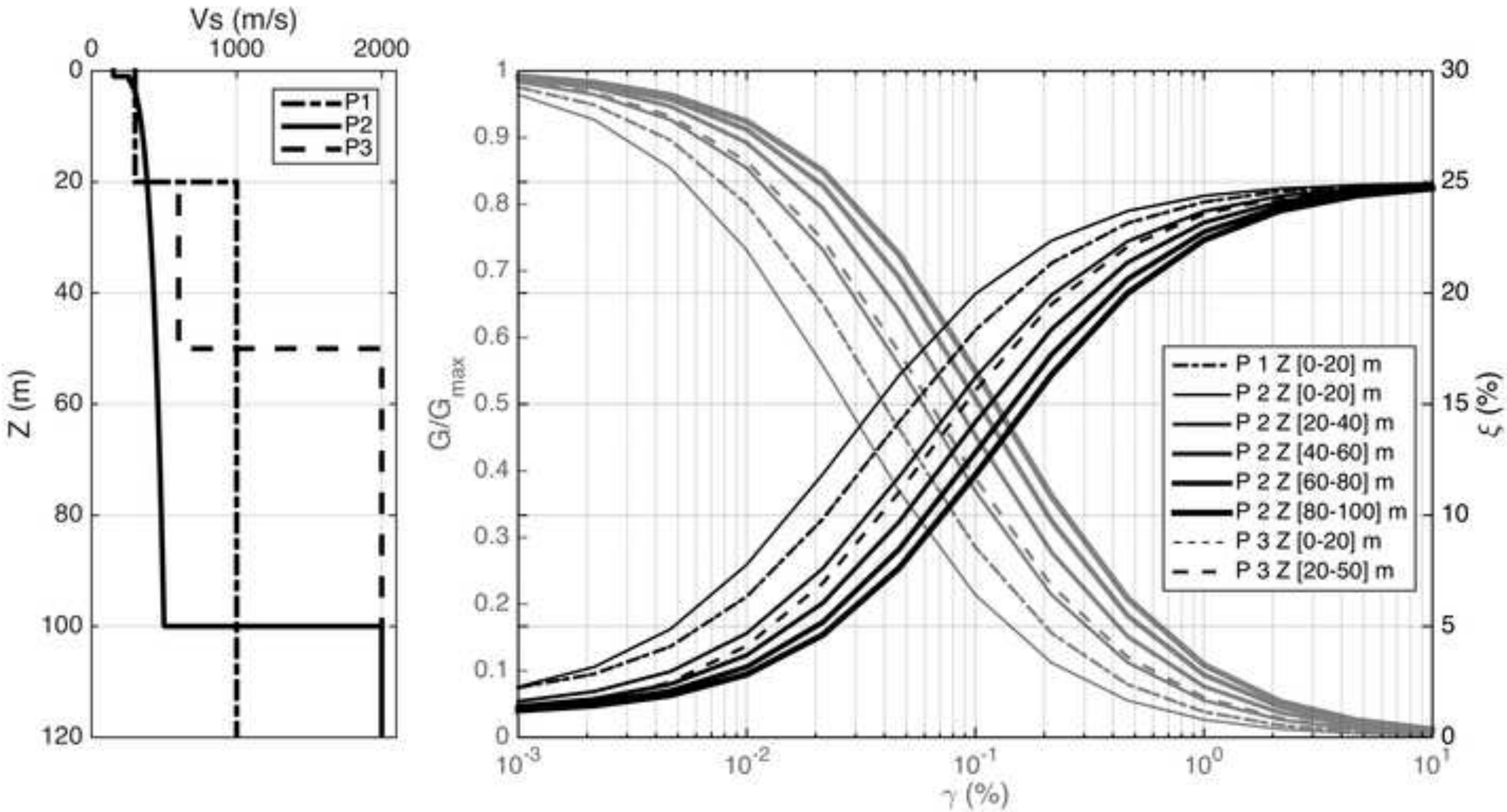


Figure 2



Vs (m/s)
0
1000
2000
Z (m)
0
20
40
60
80
100
120
P1
P2
P3
G/G$_{max}$
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
10^{-3}
10^{-2}
10^{-1}
10^{0}
10^{1}
γ (%)
ξ (%)
30
25
20
15
10
5
0
P 1 Z [0-20] m
P 2 Z [0-20] m
P 2 Z [20-40] m
P 2 Z [40-60] m
P 2 Z [60-80] m
P 2 Z [80-100] m
P 3 Z [0-20] m
P 3 Z [20-50] m

Figure 3

Click here to download Figure Fig_3_input_normalised.png

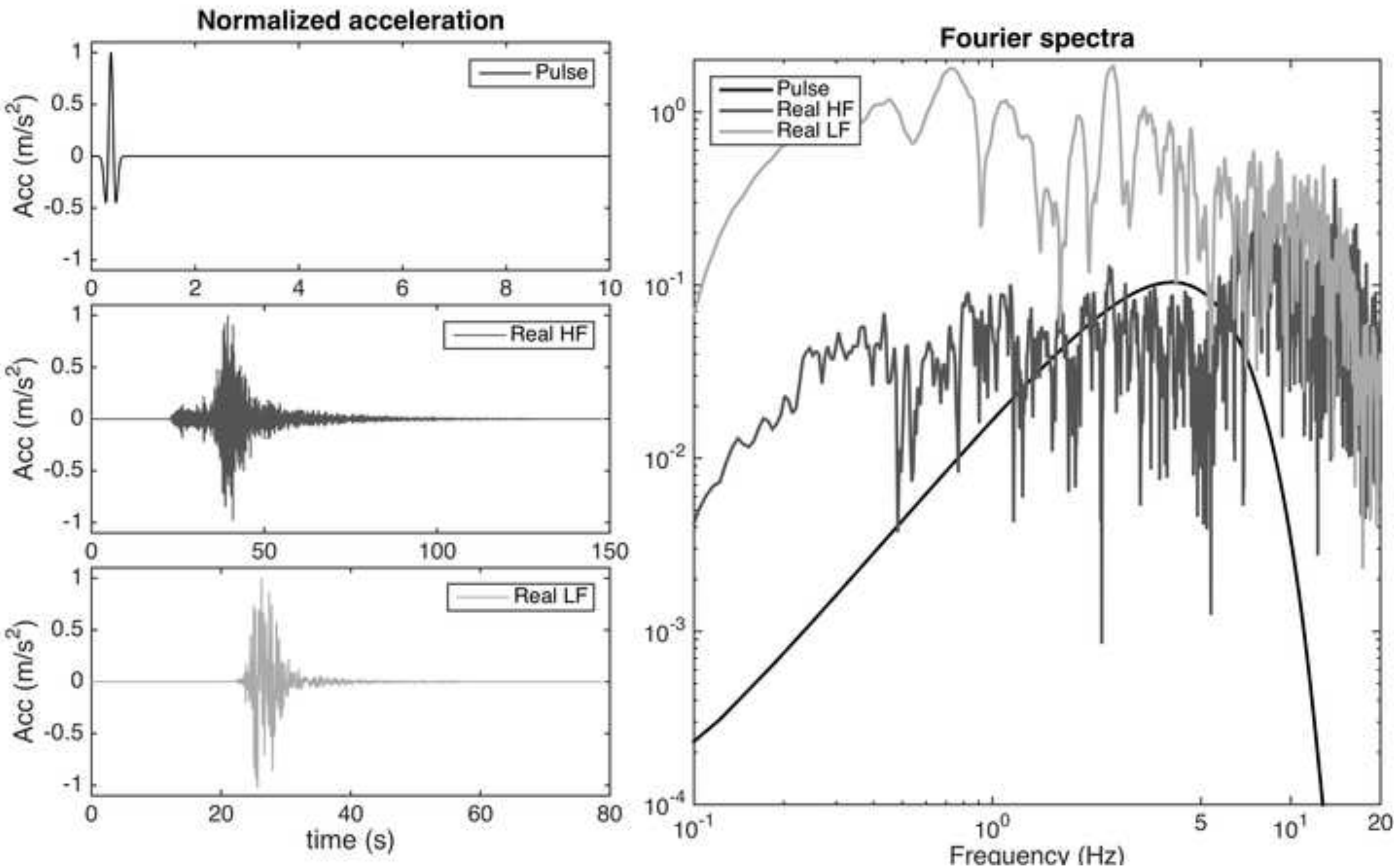


Figure 4

Click here to download Figure fig_4_TS_sinus_1_2_Cycles.png

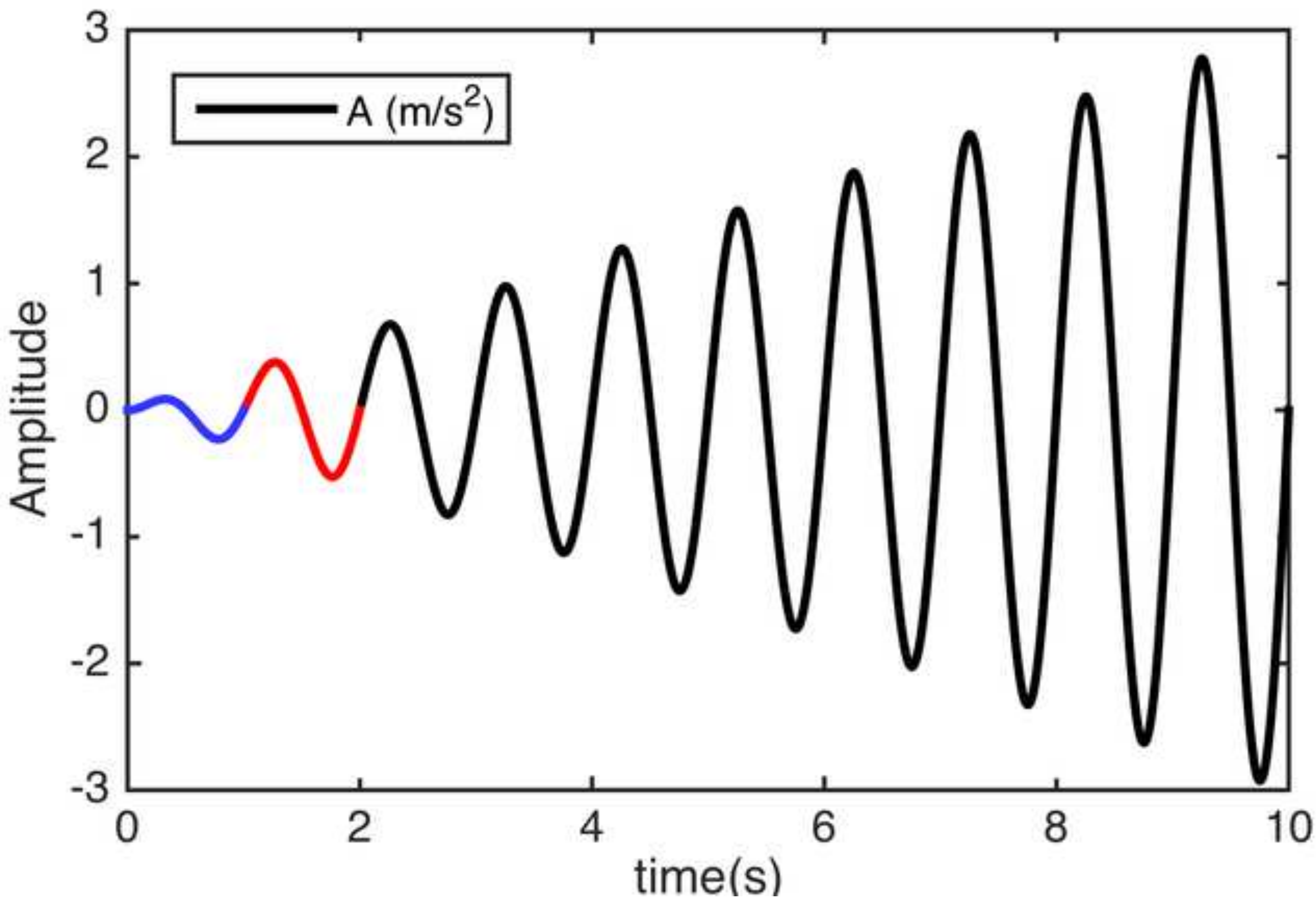


Figure 5

Click here to download Figure fig_5_ss_temps_P1_NL_sinus_1_R_Z10_all.png

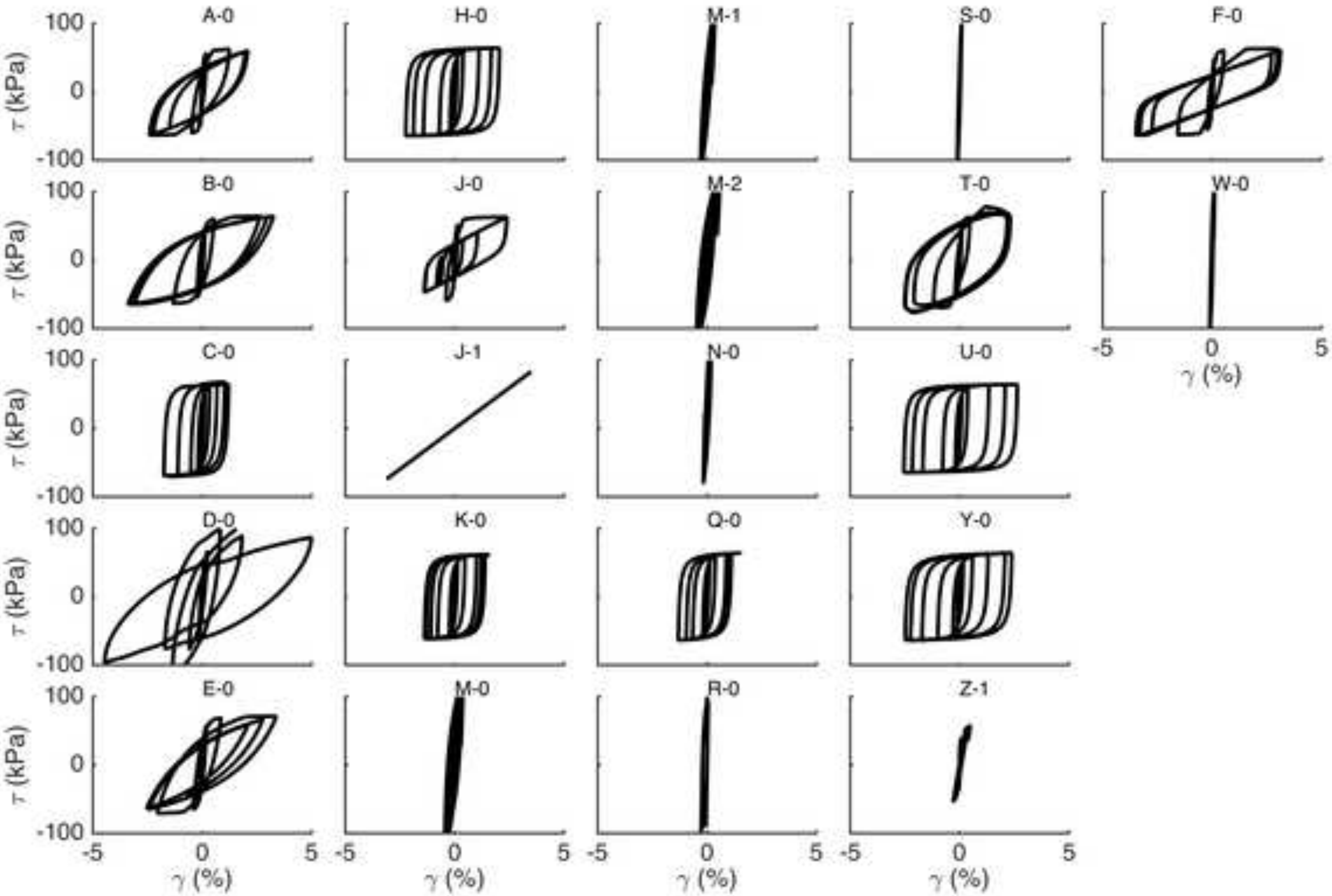


Figure 6

Click here to download Figure fig_6_ss_temps_P1_NL_sinus_1_R_Z10_1_2_cycles_2.png

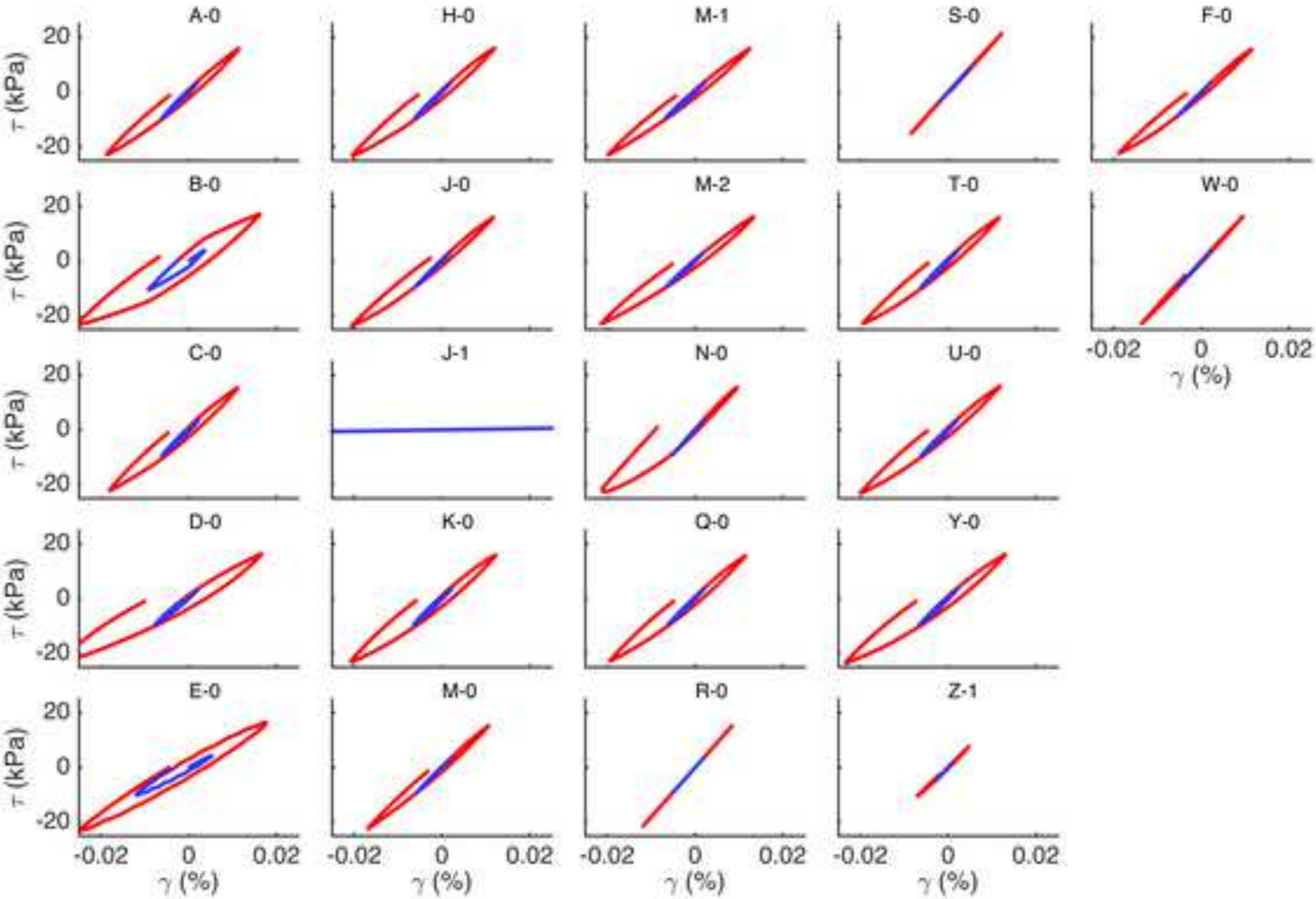


Figure 7

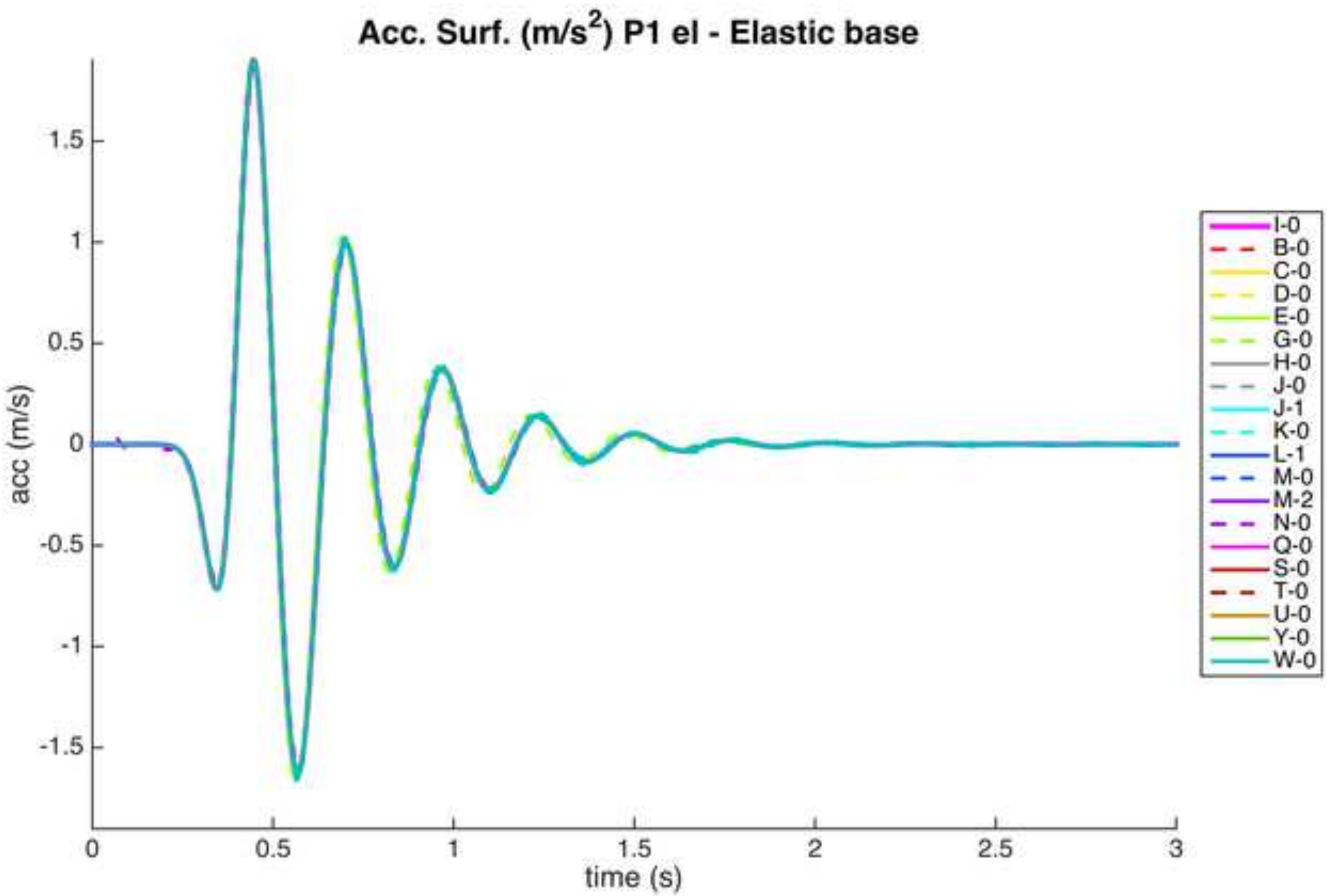

Acc. Surf. (m/s^2) P1 el - Elastic base
acc (m/s)
time (s)
1.5
1
0.5
0
-0.5
-1
-1.5
0
0.5
1
1.5
2
2.5
3
I-0
B-0
C-0
D-0
E-0
G-0
H-0
J-0
J-1
K-0
L-1
M-0
M-2
N-0
Q-0
S-0
T-0
U-0
Y-0
W-0

Figure 8

Click here to download Figure fig_8_comp_AC_temps_all_together_Z_0_P1_ve_non_disp_puls_R.png

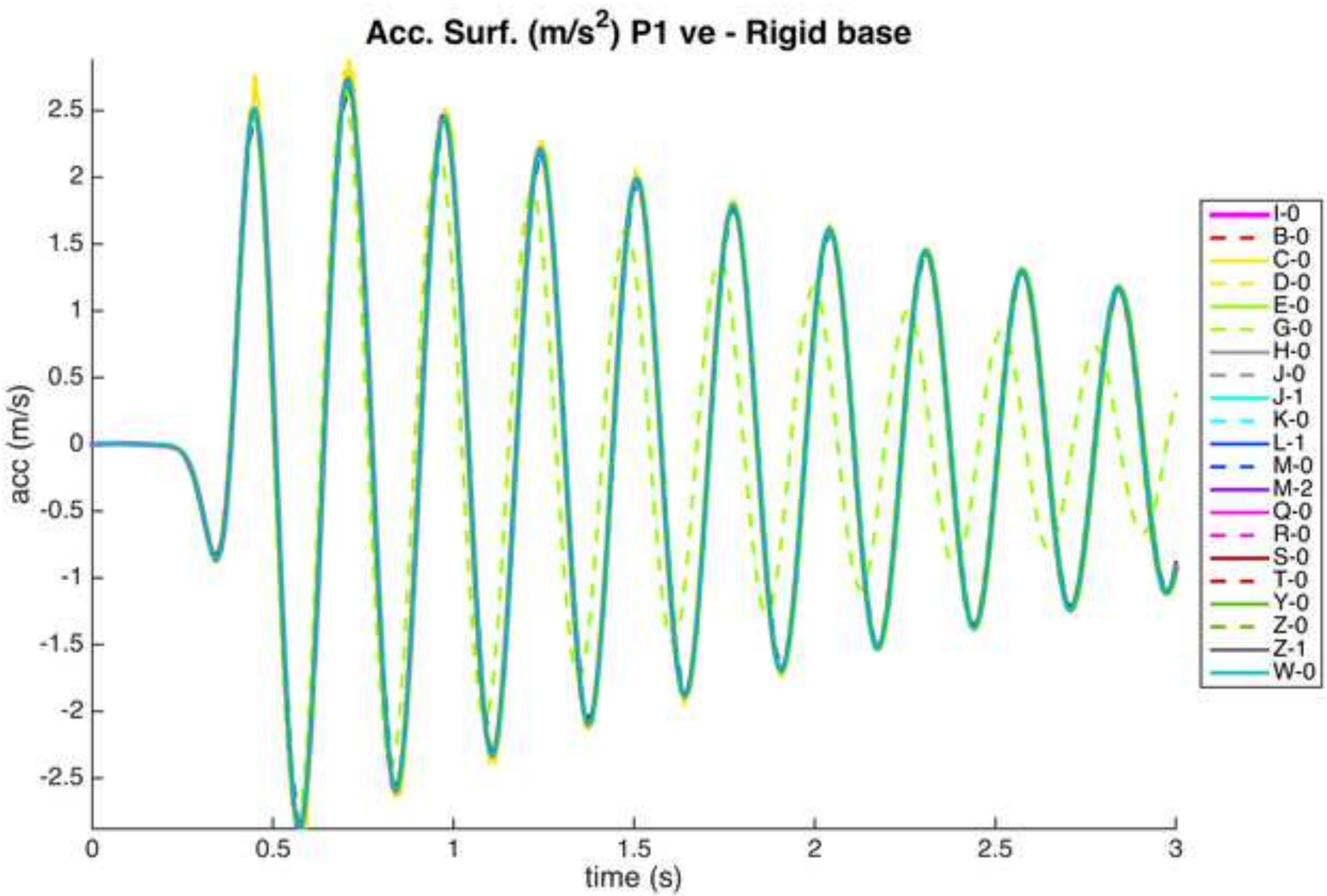


Figure 9

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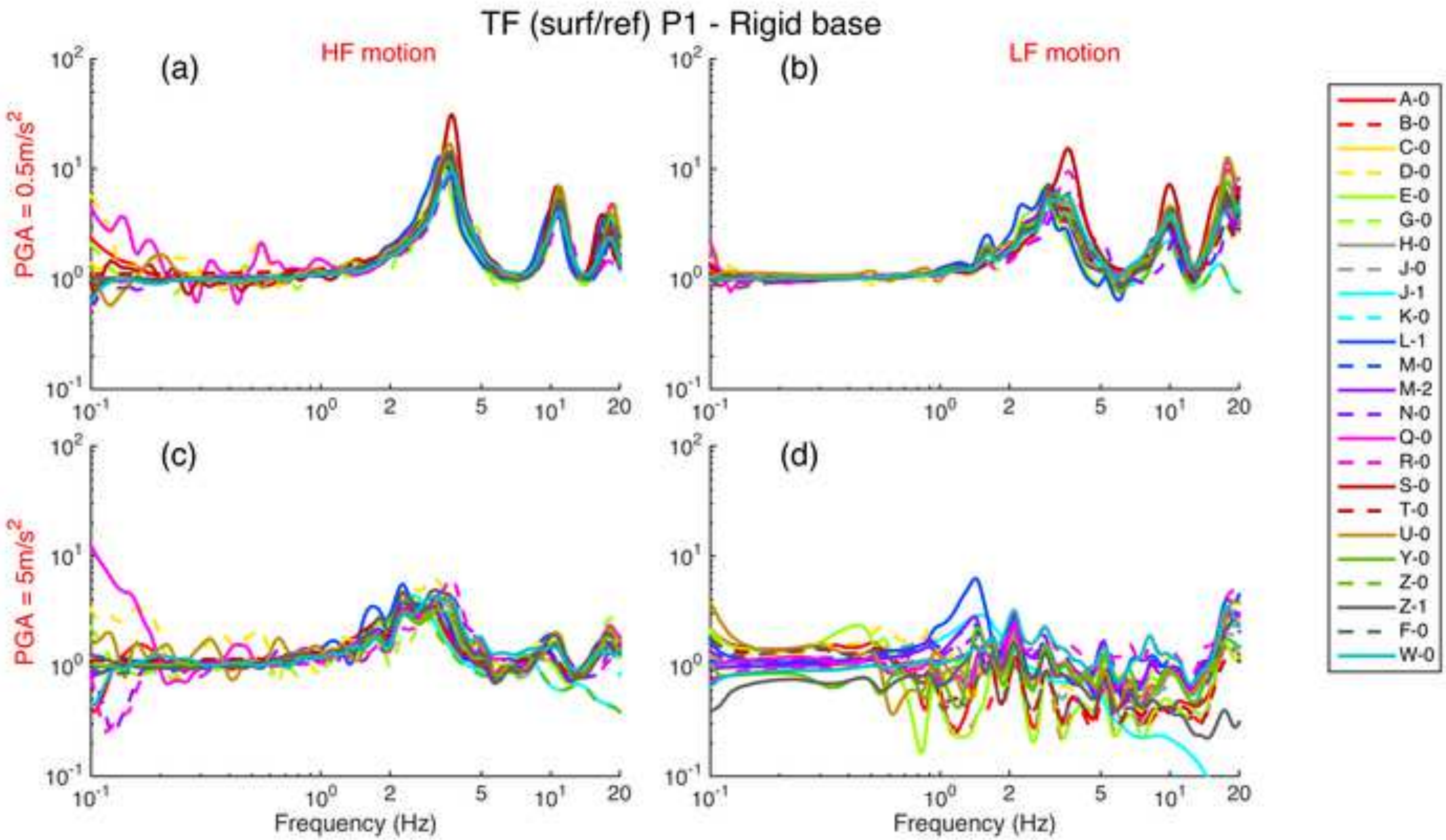


Figure 10

Click here to download Figure fig_10_lin_log_comp_SA_P1_R.png

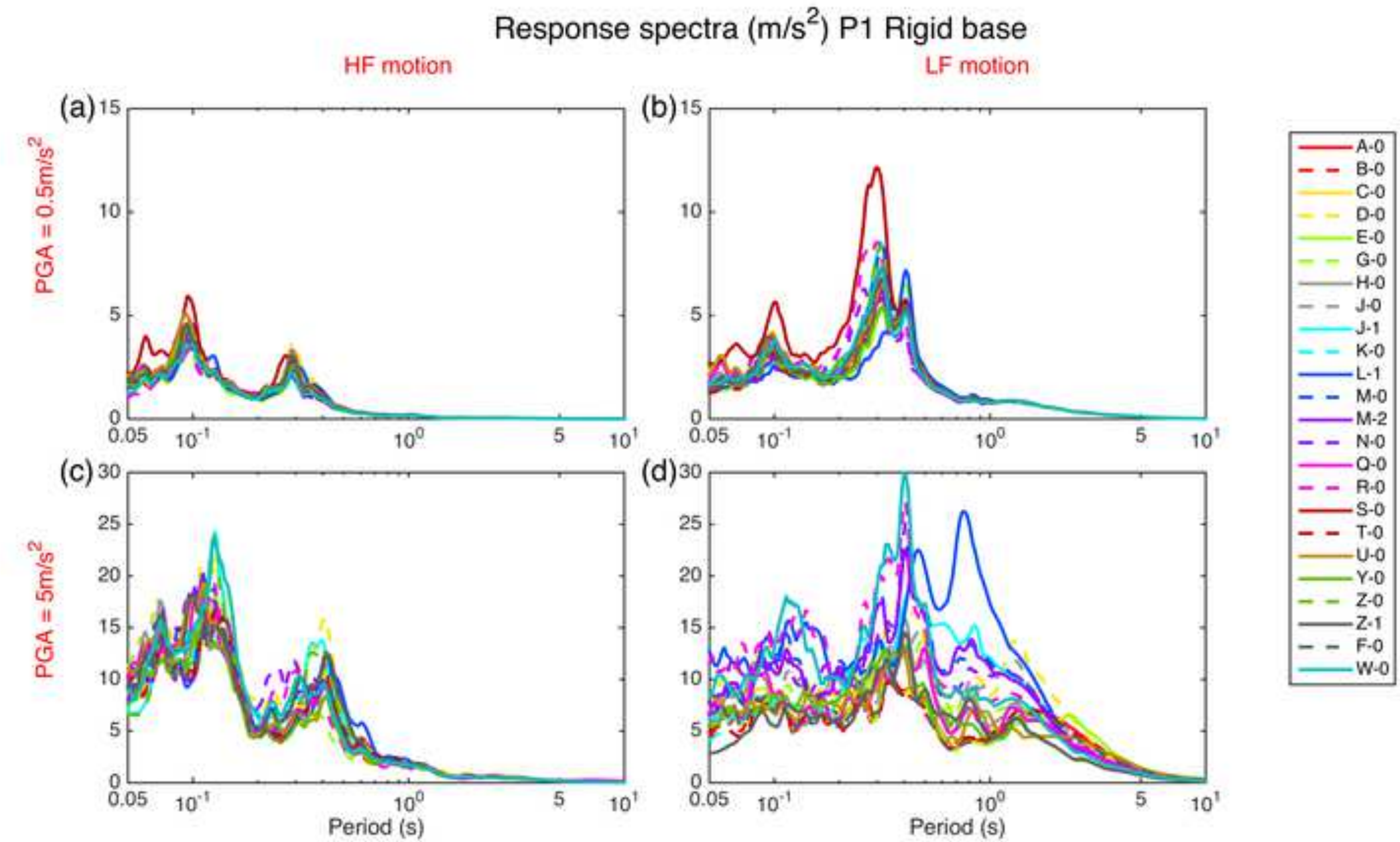


Figure 11 Click here to download Figure fig_11_comp_TF_RS_P1_R_var.png

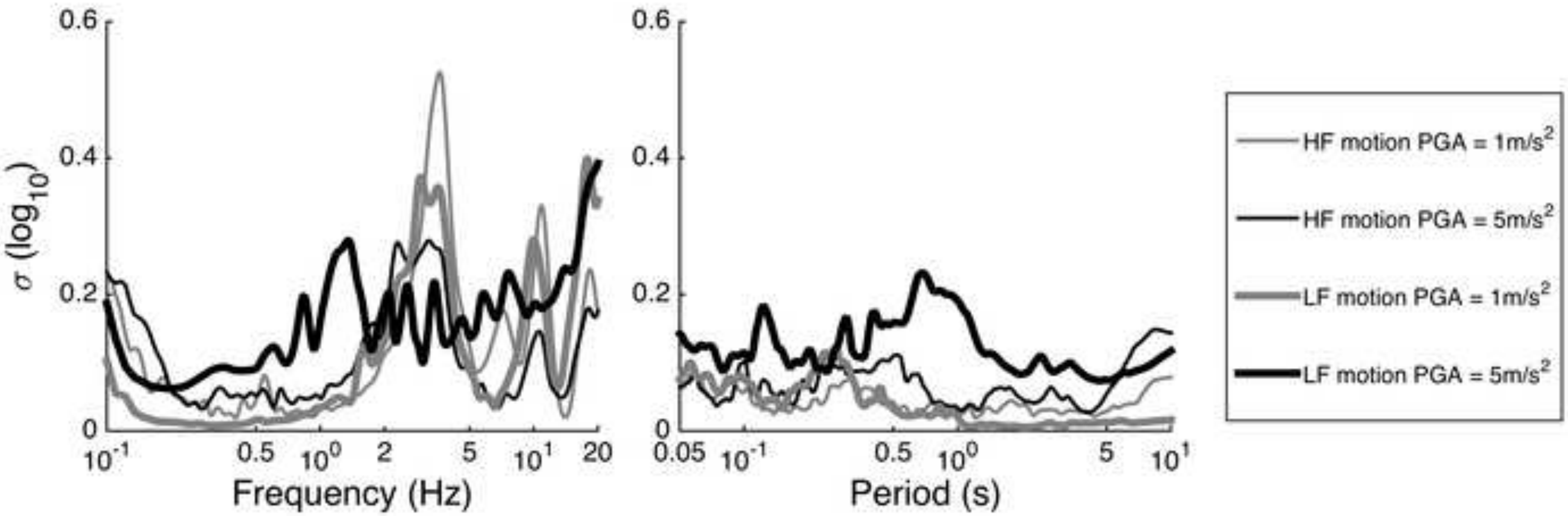


Figure 12

Click here to download Figure fig_12_GG_D_def_max_prof_P1_R.png

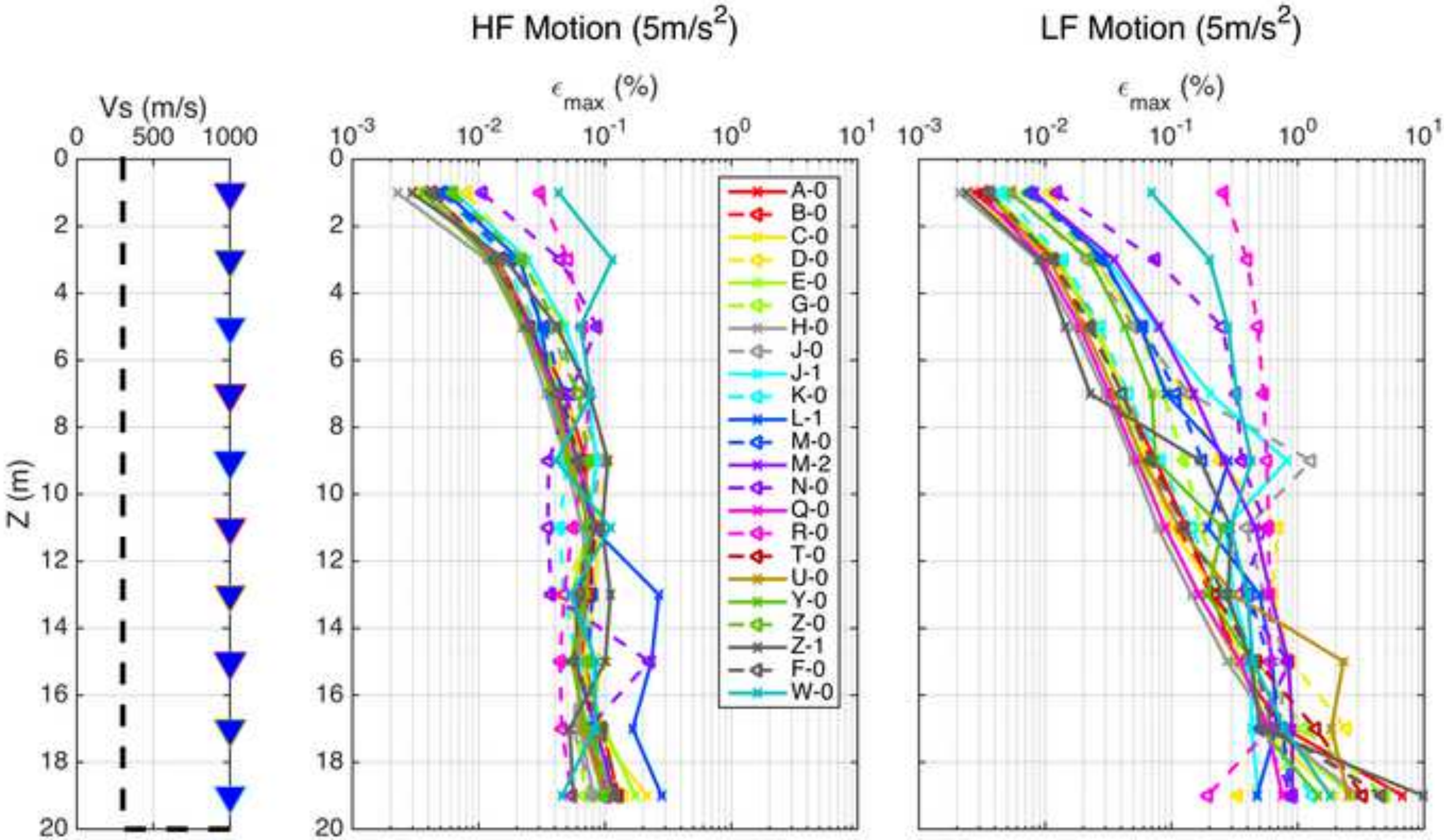


Figure 13

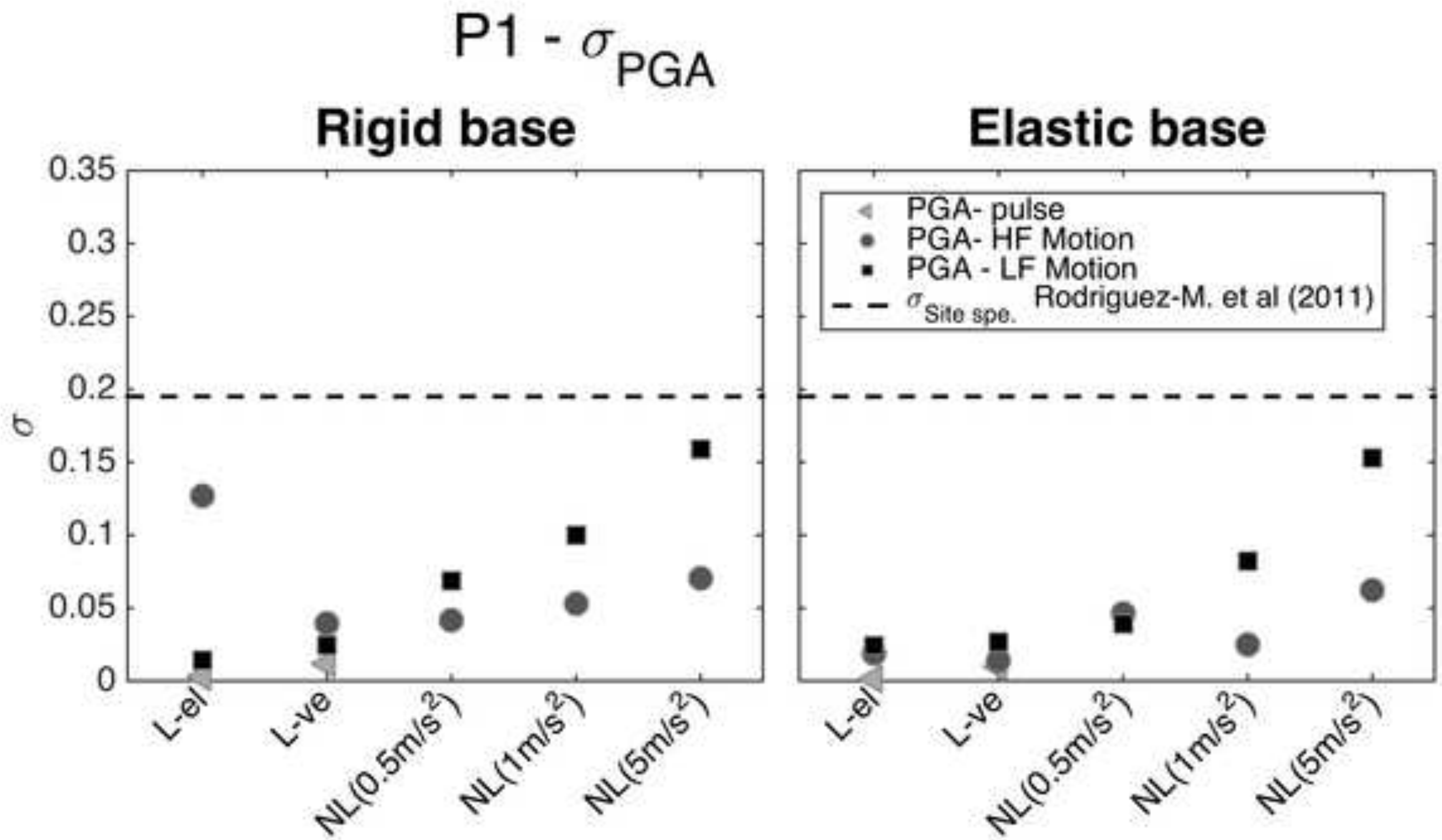

P1 - σ_{PGA}
Rigid base
Elastic base
PGA- pulse
PGA- HF Motion
PGA - LF Motion
$\sigma_{Site\ spe.}$ Rodriguez-M. et al (2011)
σ
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
L-el
L-ve
NL(0.5m/s^2)
NL(1m/s^2)
NL(5m/s^2)

Figure 14 Click here to download Figure fig_14_comp_var_LF_P1_intensity_param.png

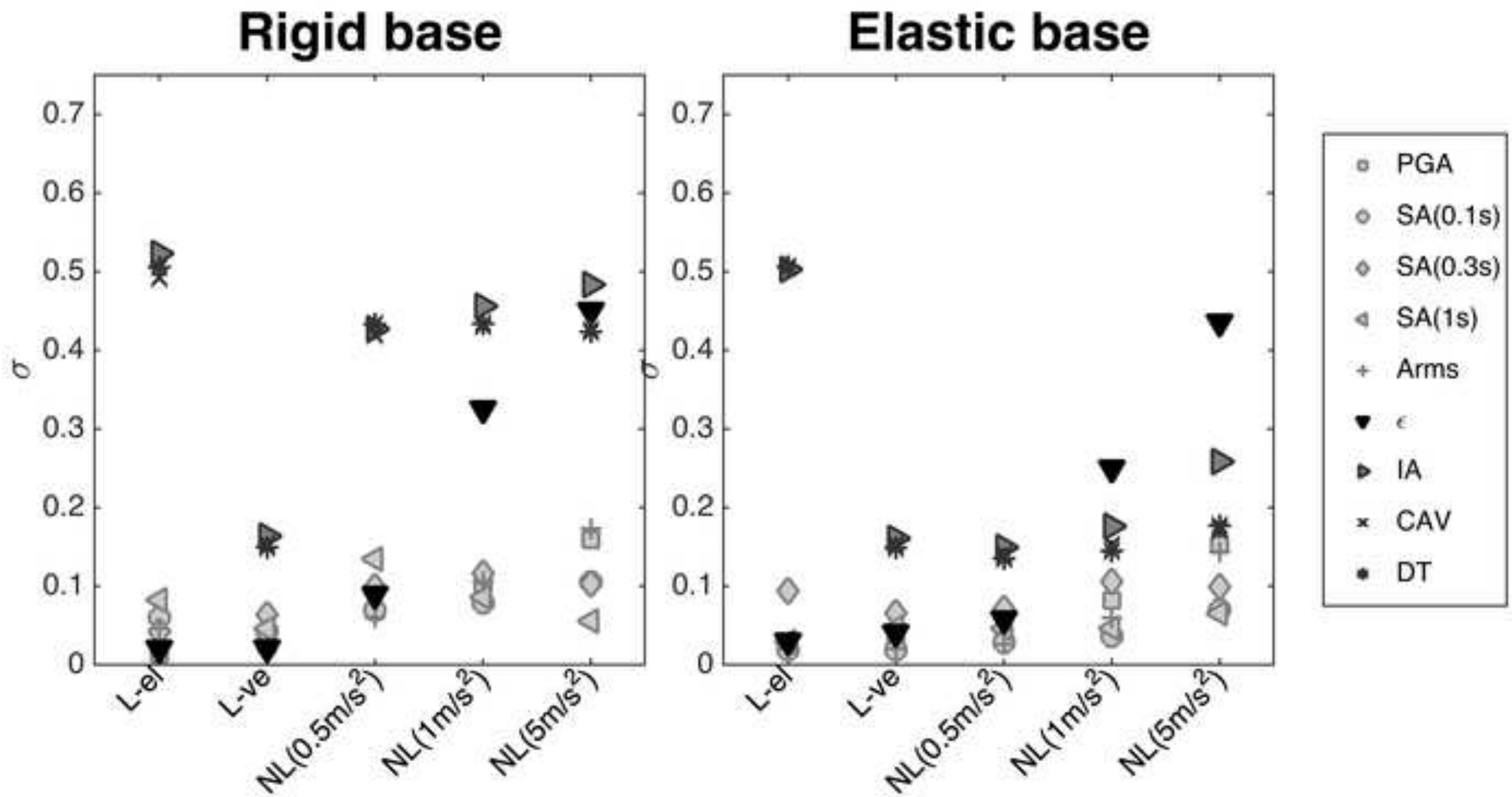


Figure 15

Click here to download Figure fig_15_comp_var_sigma_R_PGA_LF_3_profiles.png

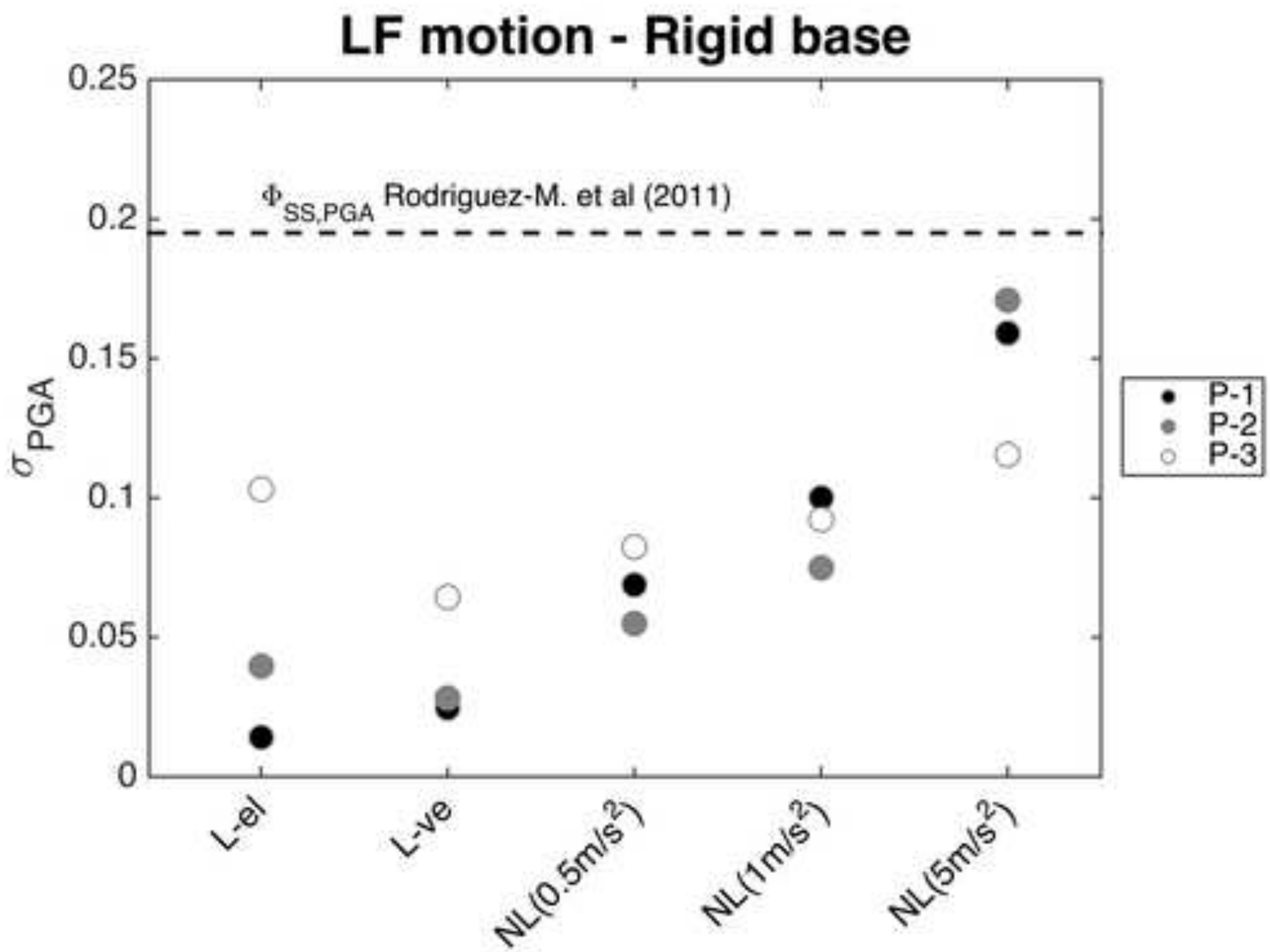


Figure 16

Click here to download Figure fig_16_ss_temps_P1_NL_real_3_LF_R_Z10Group_sim_shera_strength_damp_control.png

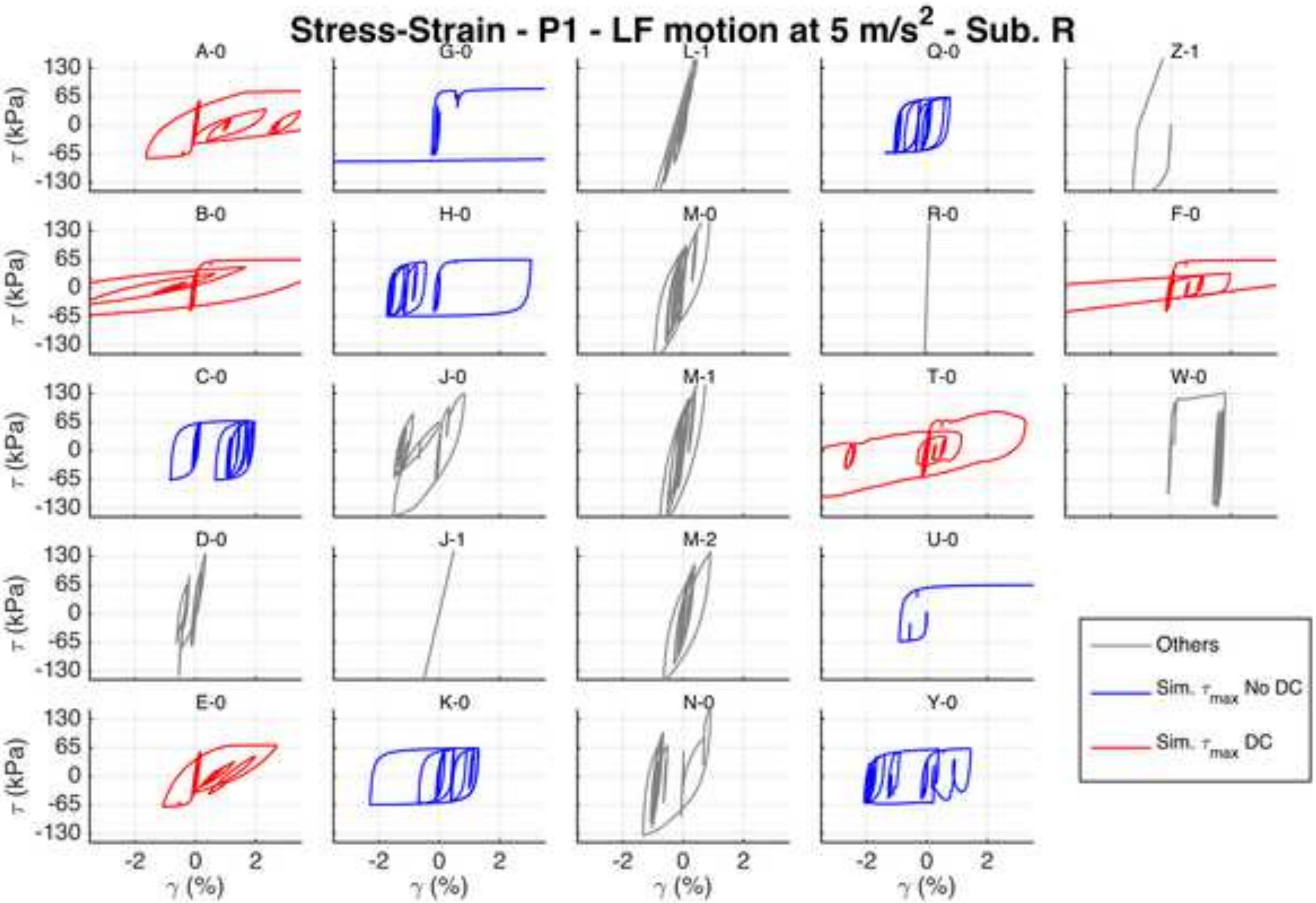
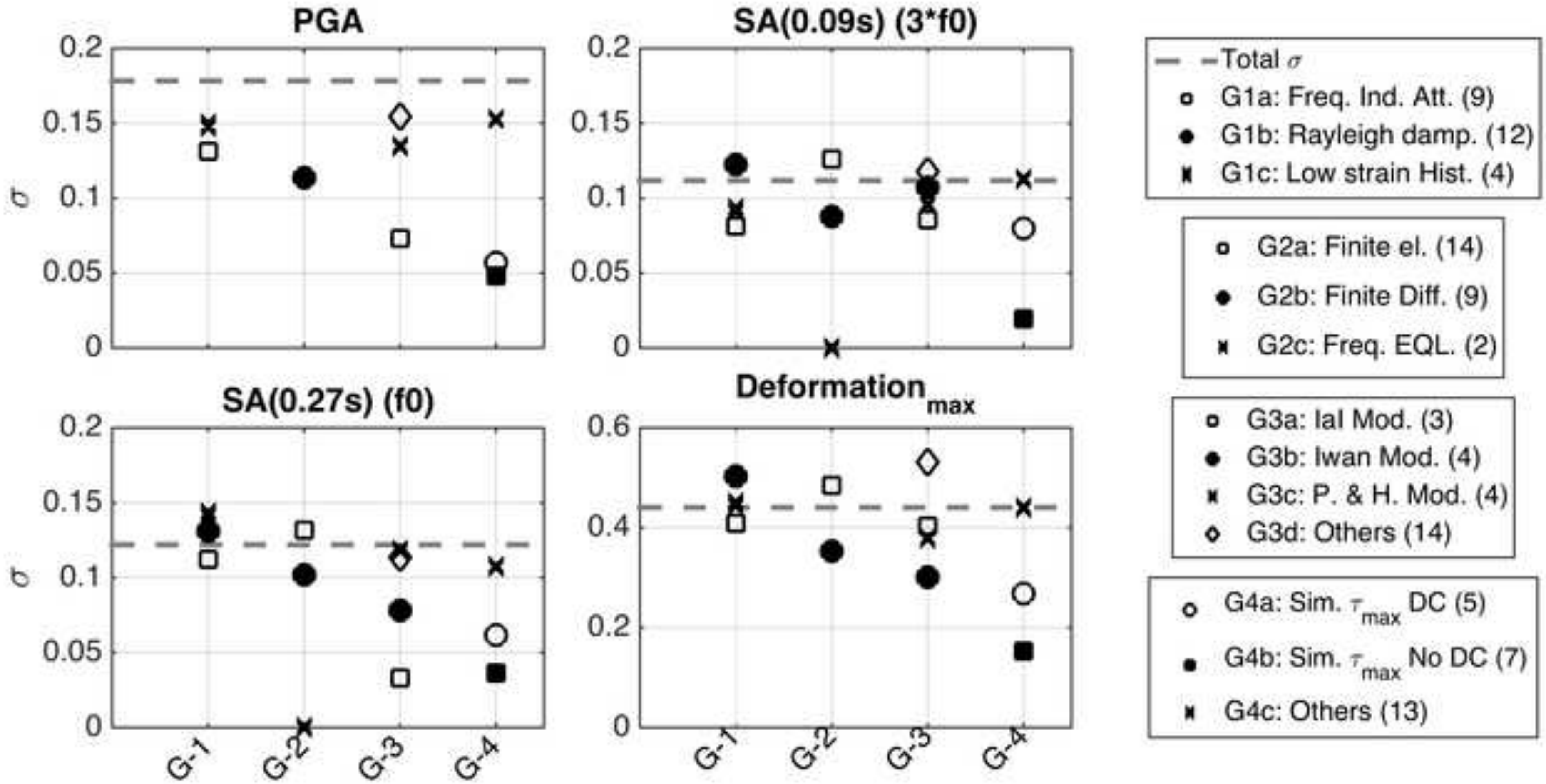


Figure 17



PGA
SA(0.09s) (3*f0)
SA(0.27s) (f0)
Deformation max
σ
0.2
0.15
0.1
0.05
0
0.6
0.4
0.2
G-1
G-2
G-3
G-4
Total σ
G1a: Freq. Ind. Att. (9)
G1b: Rayleigh damp. (12)
G1c: Low strain Hist. (4)
G2a: Finite el. (14)
G2b: Finite Diff. (9)
G2c: Freq. EQL. (2)
G3a: IaI Mod. (3)
G3b: Iwan Mod. (4)
G3c: P. & H. Mod. (4)
G3d: Others (14)
G4a: Sim. τ max DC (5)
G4b: Sim. τ max No DC (7)
G4c: Others (13)

Figure 18 Click here to download Figure fig_18_def_SA_P1_NL_real_3_LF_R_Z1Group_damping_control.png