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International Benchmark on Numerical Simulations for 1D, Nonlinear Site Response (PRENOLIN): Verification Phase Based on Canonical Cases

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

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58

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59 **Abstract**

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

68 We present here the main results of the verification phase dealing with simple cases. Three
69 different idealized soil profiles were tested over a wide range of shear strains with different
70 input motions and different boundary conditions at the sediment/bedrock interface. A first
71 iteration focusing on the elastic and visco-elastic cases proved to be useful to ensure a
72 common understanding and to identify numerical issues before pursuing the non-linear
73 modeling. Besides minor mistakes in the implementation of input parameters and output
74 units, the initial discrepancies between the numerical results can be attributed to (1) different
75 understanding of the expression "input motion" in different communities, and (2) different
76 implementations of material damping and possible numerical energy dissipation. The second
77 round of computations thus allowed a convergence of all teams to the Haskell-Thomson
78 analytical solution in elastic and visco-elastic cases. For non-linear computations we
79 investigate the epistemic uncertainties related only to wave propagation modeling using
80 different non-linear constitutive models. Such epistemic uncertainties are shown to increase
81 with the strain level and to reach values around 0.2 (log₁₀ scale) for a PGA of 5m/s² at the

82 base of the soil column, which may be reduced by almost 50% when the various constitutive
83 models used the same shear strength and damping implementation.

84

85 **Introduction**

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

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

105 The first type of non-linear effect (i.e. without liquefaction) was identified by geotechnical
106 earthquake engineering studies following the 1967 Caracas earthquake and further
107 corroborated by observations after the 1989 Loma Prieta (Chin and Aki, 1991) and the 1994

108 Northridge (Field et al., 1998) earthquakes. Moreover, this was later confirmed both by
109 laboratory tests and recordings obtained on "vertical arrays" with two or more accelerometers
110 at different depths within the same borehole. For instance, a statistical analysis of the
111 numerous recordings of the Japanese KiK-net network (Régnier et al., 2013) concluded that,
112 for Peak Ground Acceleration (PGA) levels exceeding 0.75 m/s^2 (a rather moderate level) at
113 an outcrop, there is a 40 % chance of observing a non-linear soil response, leading to
114 significant modifications with respect to the linear, low-strain response. These changes
115 generally imply a reduction of the response amplification of the signal's high-frequency
116 content and often a slight-to-significant increase of its low frequency content. Therefore,
117 linear soil response estimates cannot be considered as being systematically on the safe side,
118 and on the other hand, the high frequency reductions may significantly contribute to the safety
119 margins. As a consequence, the accuracy, robustness and reliability of non-linear site effects
120 prediction directly impacts the estimation of seismic hazard and associated risks, especially at
121 long return periods.

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

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

145 The ability to accurately predict non-linear site responses has indeed already been the subject
146 of two recent comparative tests. It was one of the targets of the pioneering blind tests initiated
147 in the late 80's/early 90's, on 2 sites of Ashigara Valley (Japan) and Turkey Flat (California);
148 however, those sites lacked strong motion records until the 2004 Parkfield earthquake during
149 which the Turkey Flat site experienced a 0.3 g motion. Since the soils were fairly stiff, the
150 nonlinearity was not very strong. A new benchmarking of 1D non-linear codes was thus
151 carried out in the last decade, based on the Turkey Flat site and a few other sites with vertical
152 array data (La Cienega, California; the KGWH02 KiK-net site in Japan, and Lotung in
153 Taiwan). Its main findings, reported by Kwok et al. (2008) and in Stewart and Kwok (2009)
154 emphasized the key importance of the way these codes are used and of the required in-situ
155 measurements. Significant differences between records and predictions have been postulated
156 as being due to an incorrect velocity profile (although it was derived from redundant borehole
157 measurements), a non-1D soil geometry (non-horizontal layers), and imperfections /

158 deficiencies in the constitutive models, which were unable to represent the actual curves for
159 stiffness reduction and damping increase. Another test was undertaken on the Euroseis
160 European test site (Mygdonian graben near Thessaloniki, Greece) as part of the
161 Cashima/E2VP project, which included two separate exercises on two-dimensional (2D) non-
162 linear numerical simulation codes and three-dimensional (3D), linear simulation codes. The
163 2D non-linear (NL) benchmark proved inconclusive, as major differences were found
164 between the few considered codes, with multiple possible causes, i.e. 2D numerical scheme,
165 damping implementation, and NL constitutive laws (see Foerster et al., 2015). Given the fact
166 that the codes used for these tests are routinely used in engineering practice for predictions of
167 non-linear site responses, especially for moderate seismicity countries lacking strong motion
168 recordings, there is a clear need to conduct further tests in better controlled conditions, in
169 particular with in situ and laboratory measurements for an optimal tuning of the non-linear
170 parameters used in each code.

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

181 The purpose of this article is to present and discuss the results of the verification phase, with a
182 special focus on the epistemic uncertainties associated with the constitutive laws and

183 numerical schemes of the simulation codes. The first section describes the 3 idealized soil
184 columns and the requested computations, considering different boundary conditions (rigid /
185 elastic base, associated respectively with within / outcropping reference motion). The next
186 section lists the numerous teams that volunteered to participate in this exercise and the main
187 characteristics of their codes. The simulation results are then presented and compared, first in
188 the linear case (with and without attenuation), and then in the non-linear case for various input
189 signals and levels, with a discussion in each case on the amount and origins of uncertainty.

190 **The canonical cases**

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

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

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

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

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

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

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

229

FIGURE 1

230 *Soil properties*

231 The properties describing the (1D) linear and non-linear soil behavior for each profile include
232 elastic, visco-elastic and non-linear soil properties. They are displayed in Figure 1 and Figure
233 2, and summarized in Table 1.

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

$$241 \quad V_S(z) = V_{S1} + (V_{S2} - V_{S1}) \left(\frac{z - Z_1}{Z_2 - Z_1} \right)^\alpha$$

242 *Eq 1*

243 where $V_{S1} = 150$ m/s and $V_{S2} = 500$ m/s are the shear-wave velocities at depths $Z_1 = 0$ m and
244 $Z_2 = 100$ m, respectively, and $V_S(z)$ is the shear wave velocity at depth z ; α is taken equal to
245 0.25.

246 *Visco-elastic properties*

247 We only consider intrinsic material damping (Biot, 1956; Johnston et al., 1979; Leurser, 1997),
248 without any additional component from scattering. Intrinsic attenuation can be quantified by
249 the quality factor Q (more commonly used in seismology), or the damping ratio ξ (used in
250 engineering seismology). Here Q and ξ are the quality factor and the damping ratio of the S-

251 waves. They are linked by the formula $Q = 1/(2\xi)$, and can be determined by the loss of
252 energy over one wavelength. Pure elastic materials totally restore the seismic energy after
253 deformation, and should therefore have infinite Q values; as the numerical codes used here
254 require a finite value as input, the "elastic" case was computed with very high values of Q
255 (very low ξ) for both soil and bedrock ($Q = 5000$). For visco-elastic and non-linear (soft)
256 materials, the energy dissipation at low strain was constrained to vary according to V_s ,
257 through the classical – never appropriately justified by measurements - relationship $Q =$
258 $V_s/10$, or equivalently $\xi = 5/V_s$ (V_s in m/s) (Olsen et al., 2003).

259 **Non-linear soil properties**

260 The non-linear properties of each layer were characterized using classical $G/G_{\max}(\gamma)$ and $\xi(\gamma)$
261 curves, relating the decay of shear modulus (G) normalized by the elastic shear modulus
262 (G_{\max}) and increase of damping ξ with the shear strain γ . Normally, we would expect to have
263 available measurements from laboratory or field experiments to derive the necessary
264 parameters for a given rheology model. However, in order to verify the ability of different
265 models to take into account the same shear modulus reduction and damping ratio data and to
266 simplify the canonical soil description, and also to start with an analysis of only the “code-to-
267 code variability”, $G/G_{\max}(\gamma)$ and $\xi(\gamma)$ curves were constructed following a simple hyperbolic
268 model based on the following equations:

$$269 \quad K_0 = (1 - \sin(\phi))OCR^{\sin(\phi)}$$

270 *Eq 2*

$$271 \quad \sigma'_m = \sigma'_v(1 + 2K_0)/3$$

272 *Eq 3*

273 $\tau_{\max} = \sigma'_m \sin(\phi)$

274 **Eq 4**

275 $\gamma_{ref} = \tau_{\max} / G_{\max}$

276 **Eq 5**

277 $G / G_{\max} = 1 / (1 + \gamma / \gamma_{ref})$

278 **Eq 6**

279 $\xi = \xi_{\min} + (\xi_{\max} - \xi_{\min})(\gamma / \gamma_{ref}) / (1 + \gamma / \gamma_{ref})$

280 **Eq 7**

281 where the control parameters are the friction angle $\Phi = 30^\circ$, the over-consolidation ratio OCR
 282 $= 1$ and the gravitational acceleration $g = 9.81 \text{ m/s}^2$. Only cohesionless material was
 283 considered here, so that the shear strength τ_{\max} is computed using the vertical stress and the
 284 friction angle. Both, σ'_m and σ'_v are the effective mean and vertical stresses; γ is the shear
 285 strain. The reference shear strain γ_{ref} corresponds to the strain for which $G = 0.5G_{\max}$ (in the
 286 hyperbolic model as describe above it is given by Eq 5), K_θ is the coefficient of earth pressure
 287 at rest, and ξ_{\min} and ξ_{\max} are the minimum damping values at very low strain (intrinsic material
 288 damping considered above for the visco-elastic behavior), and the maximum at very high
 289 strain, respectively.

290 Only one $G/G_{\max}(\gamma)$ and $\xi(\gamma)$ curves were provided for P1, five for P2 (increasing for each 20
 291 m depth interval), and two for P3 (one for each homogeneous layer). We assume a constant
 292 strength per soil layer for all soil models. They are illustrated in Figure 2. For P1 and P2, they
 293 are fitting a hyperbolic curve defined by the low strain shear modulus $G_{\max} = \rho V_s^2$ and the
 294 shear strength τ_{\max} at the center of each layer or sublayer. For P3 the $G/G_{\max}(\gamma)$ and $\xi(\gamma)$
 295 chosen models were very similar to one another using the previous hyperbolic model. For P3,

296 the set of Darendeli models (Darendeli, 2001) was used and adjusted to a simple hyperbolic
297 model as for P1 and P2; as Darendeli's models are defined only up to a maximum shear strain
298 of 1 %, the P3 curves were defined by multiplying the shear strength τ by factors 1.1 and 2 at
299 depths of 10 m and 35 m, respectively, and the final curves were then computed based on the
300 hyperbolic models associated to these values.

301 Some numerical codes include sophisticated constitutive models for NL soil behavior, which
302 require very specific additional parameters, which should be consistent with the $G/G_{max}(\gamma)$ and
303 $\xi(\gamma)$ curves supplied for the other codes. The definition of these additional parameters was
304 done individually by each team, with the following simple assumptions: the soil is
305 cohesionless (i.e. $c' = 0$ and Plasticity Index $PI = 0$), the soil particle size distribution is
306 defined with $D_{10} = 0.2$ mm and $D_{50} = 0.35$ mm, and a uniformity coefficient $D_{60}/D_{10} = 1.8$.

307 **TABLE 1**

308 **FIGURE 2**

309 ***Reference rock motion***

310 In the first phase of the project, each participant was provided (i) a simple Ricker pulse input
311 motion derived analytically, and (ii) two real acceleration time histories scaled to three
312 different PGA levels (0.5, 1 and 5 m/s²) to observe the evolution from linear to non-linear soil
313 behavior. The two accelerograms were selected to be representative of very different
314 frequency contents, in order to analyze its influence in the non-linear computations. Each
315 accelerogram was pre-processed in the same way as explained further below. The Fourier
316 transforms of the three normalized input motions are illustrated in Figure 3.

317 **The pulse-like input motion**

318 The Ricker pulse input motion corresponds to acceleration, velocity and displacement time-
319 histories defined by equations (8) to (10). A central frequency of 4 Hz was chosen to produce

320 sufficient energy at the fundamental frequency of each of the three profiles, while having a
321 broad band energy in the main bandwidth of earthquake geotechnical engineering, i.e. 1-10
322 Hz.

$$323 \quad a(t) = [1 - 2(\pi t f_c)^2] \exp(-(\pi t f_c)^2)$$

324 *Eq 8*

$$325 \quad v(t) = t \exp(-(\pi t f_c)^2)$$

326 *Eq 9*

$$327 \quad d(t) = \frac{1}{-2(\pi f_c)^2} \exp(-(\pi t f_c)^2)$$

328 *Eq 10*

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

332 **Real reference input motions**

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

342

TABLE 2

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

349

FIGURE 3

350

351 **Participants and tested numerical codes**

352 We compared 23 different numerical codes used by 21 participating teams, as listed in Table
353 3. As some teams use several codes, each computational case/team is annotated by a letter and
354 a number. Two or more teams used the same code, including Deepsoil (4 teams for the
355 verification and 5 for the validation), FLAC (2 teams) and OpenSees (3 teams). Others used
356 the same constitutive model, notably Iai's (1990) model (2 teams), Iwan's model (Ishihara,
357 1996; Iwan, 1967) (4 teams) and the Hujoux model (Aubry et al., 1982) (2 teams). The
358 participant teams were composed of people having different background and expertise, which
359 can be relevant for analyzing the site response variability. Firstly, two disciplines are
360 represented in this benchmark, seismology and geotechnical earthquake engineering and
361 secondly, the participants are either developers or users.

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

366
367
368

TABLE 3

369 ***The numerical scheme***

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

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

- 374 i) Standard method (ST.FEM), used by 12 teams: B-0, D-0, H-0, L-1, M-1, N-0, R-
375 0, S-0, T-0, U-0, W-0 and Z-1.
- 376 ii) Spectral method (SP.FEM), used by 1 team: Q-0
- 377 iii) Discontinuous Galerkin method (DG.FEM), used by 1 team: Y-0.
- 378 (b) The Finite Difference Method (FDM) is used by 10 teams: A-0, C-0, E-0, F-0, G-0, J-
379 0, K-0, L-2, M-0 and M-2;

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

384 ***Implementation of attenuation***

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

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

396 In the time domain, attenuation can be approximated by implementation of a set of relaxation
397 functions using rheological models such as the generalized Maxwell model (Blanch et al.,

398 1995; Day and Bradley, 2001; Day and Minster, 1984, 1984; Graves and Day, 2003) or
399 modeled by a Rayleigh damping formulation. Both methods present pros and cons. The usage
400 of rheological models to approximate attenuation is physical; however, it adds memory
401 constraints to the computations. The greater the number of relaxation functions used, the
402 better the attenuation factor will be approximated, although one should not use too many (see
403 for example Peyrusse et al., 2014). On the contrary, the Rayleigh damping method is much
404 easier to be implemented numerically; nevertheless, the parameters are not easily determined,
405 and automatically involve a significant frequency dependence of Q . For low attenuation
406 (below a damping ratio of 20%) it has been shown that Rayleigh damping and the generalized
407 Maxwell model become equivalent (Semblat, 1997).

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

409 (1) Frequency-independent attenuation (frIA): Some model considered frequency
410 independent attenuation instead of the use of the frequency dependent Rayleigh
411 Damping/attenuation in the time domain analysis. Models A-0, E-0, K-0, Q-0, T-1 and
412 Z-0 use series of Maxwell/Zener elements (Blanch et al., 1995; Day and Bradley,
413 2001; Day and Minster, 1984, 1984; Graves and Day, 2003), which imply an almost
414 constant attenuation over a specific, broad enough frequency range. Models F-0, J-0
415 and M-2 used the frequency independent attenuation as proposed in (Phillips and
416 Hashash, 2009a).

417 (2) Frequency-dependent attenuation (frDA), such as the Rayleigh damping (simplified or
418 full), 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
419 and Z-1.

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

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

425

426 **High strain attenuation:** High strain attenuation can be computed directly from the
427 hysteretic behavior of the soil subjected to strong ground motion (loading / unloading cycles).
428 However, it was demonstrated it is difficult to reproduce simultaneously the specified
429 decrease of G/G_{max} with increasing shear strain, and the increasing of damping. For this
430 reason, a few teams (A-0, B-0, E-0 and T-0) chose to use a “damping control” (which implies
431 a modification of the “Masing rules”, and is thus labeled as “no-Masing rules”): it is based on
432 a mapping that converts a hysteresis loop in such a way that it will satisfy the hysteretic
433 damping at the current strain level (Iai et al., 1992). Other teams (J-0) used the method
434 proposed in Phillips and Hashash (2009a), which modifies the unload and reload paths of the
435 extended Masing rules.

436 *Non-linear constitutive models*

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

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

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

448 ⇒ Iai's model (Iai et al., 2011; Iai and Ozutsumi, 2005): B-0, E-0,

449 ⇒ MKZ modified hyperbolic model (Matasovic and Vucetic, 1993): A-0

450 ⇒ Cundall's model (Cundall, 2006): M-0

451 ⇒ Iwan's model (Ishihara, 1996; Iwan, 1967): K-0, Q-0, U-0, Y-0

452 ⇒ Logarithmic function model (Puzrin and Shiran, 2000) : L-1

453 ⇒ Modified Hujieux model (Aubry et al., 1982): D-0, N-0, S-0

454 ⇒ Multiyield model (Elgamal et al., 2003; Yang et al., 2003): H-0

455 ⇒ Extended Hyperbolic model (Phillips and Hashash, 2009) : F-0, H-0, J-0, M-2, T-0

456 ⇒ HSsmall (Isotropic hardening elasto-plastic soil model) (Schanz et al., 1999): Z-1

457 ⇒ Pisanò 3D Elastic-plastic model (Pisanò and Jeremić, 2014): R-0;

458 ⇒ BWGG: Extended Bouc Wen model (Gerolymos and Gazetas, 2005): G-0

459 ⇒ Modified extended hyperbolic model: C-0

460 ⇒ Manzari-Dafalias model: W-0 (Dafalias and Manzari, 2004)

461 In order to compare the different constitutive models, stress/strain controlled tests could have
462 been conducted. However, some of the teams were not able to perform it. To overcome this
463 difficulty, we asked the teams to compute nonlinear simulations with their codes on one of the
464 idealized soil profiles (P1) with a sinusoidal input motion of increasing amplitude and with a
465 rigid substratum base (Figure 4). The frequency of the input motion was low enough to avoid

466 any issues with wave propagation. Moreover, the result of this simulation was recorded at the
467 node above the soil/bedrock interface, having a strength of 65 KPa.

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

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

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

485 **FIGURE 4**

486 **FIGURE 5**

487 **FIGURE 6**

488 ***Code usage protocols***

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

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

502 A reference frequency was thus defined for each profile, at which common velocity and
503 attenuation values were fixed. As indicated by some authors (Liu and Archuleta, 2006; Moczo
504 et al., 2004) the values of reference frequency used in most cases is close to 1 Hz (as many
505 3D computations including shallow, soft material, have rather low upper bound maximum
506 frequencies). On the other hand, it is often suggested to select a frequency close to the
507 frequency of interest. In our case, given the definition of the pulse-like motion, we chose a
508 reference frequency of 4 Hz, i.e. the central value of the input wavelet.

509 **Definition and implementation of the reference motion**

510 We tested two base conditions at the sediment-substratum interface: (i) an elastic base, and
511 (ii) a rigid base. The first condition corresponds to the usual hazard assessment studies, where

512 the rock ground motion is derived from deterministic or probabilistic analysis, and
513 corresponds to the design motion at the surface of an outcropping rock. The second one
514 corresponds to the case where a recording is obtained at depth within a down-hole array, and
515 is used to derive the motion at surface or shallower depths. Depending on the communities or
516 points of view, the implementation of input (or reference) motions into algorithms can be
517 quite different, indicating that the terms "input motion" or "reference motion" are not
518 understood in the same way by all the participants. For the seismological community, the
519 input motion is often seen as the seismic signal carried by the up-going incident wave, while
520 for the geotechnical community, it is often understood as the motion at a given reference rock
521 site, resulting from the total-wavefield (up-going and down-going waves). This reference site
522 may be either at the rock surface (it then includes the free-surface effect), or at depth (for
523 instance the downhole sensor of a vertical array, which includes the interferences between the
524 up-going and down-going waves) (Bonilla et al., 2002).

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

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

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

536 - Within motion: Seismic motion recorded at depth, usually at a downhole site: in our case,
537 this location corresponds to the interface between sediment and rock substratum (i.e., $z = 20$,
538 100 and 50 m, for profiles P1, P2 and P3, respectively). This motion contains the total wave-
539 field composed of the incident up-going and reflected down-going waves.

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

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

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

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

553 In order to avoid any ambiguity, we will systematically use the expression "reference
554 motion", which should be understood as detailed above for the elastic and rigid base
555 conditions. Furthermore, this reference motion implicitly considers vertically incident plane S
556 waves only. Finally, each team was free to choose the time step needed to fulfill the stability
557 conditions of the used numerical scheme.

558 **Comparison of predictions**

559 *Methodology of comparison*

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

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

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

576 *Visco-elastic computations*

577 Figure 7 displays the comparison for the P1 profile of the surface acceleration for the pulse-
578 like motion under an elastic substratum condition, for the linear elastic computation for a
579 short window (3 s) of signal. All results converged towards the analytical solution calculated
580 with the Haskell-Thomson method (Haskell, 1953; Thomson, 1950), but this was achieved

581 only after the second iteration. There were indeed unexpected and significant discrepancies in
582 amplitude at the end of the first iteration, caused by: (1) inconsistent implementation and
583 understanding of the term "input motion" (clarified as mentioned in the code usage protocols),
584 (2) problems with units, or (3) representations of soil properties. During the first iteration,
585 some phase discrepancies could be also identified, associated either to the assignment of the
586 "input motion" at different depths some distance below the sediment/rock interface (which
587 caused a constant time delay), or to increasing time delays at the end of the computed cycles,
588 that were associated to numerical dispersion.

589 **FIGURE 7**

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

596 **FIGURE 8**

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

601 *Non-linear computations*

602 Once agreement between the model predictions was reached for simple, linear cases for which
603 analytical solutions are available, the variability of the results of non-linear calculations can

604 be fully associated with differences in implementation of non-linear soil behavior. Each team
605 chose its appropriate time step to fulfill the stability conditions of their numerical scheme. We
606 acknowledge that this might be another cause for some discrepancies in the results, especially
607 for strongly nonlinear cases, but such numerical issues are also one component of the code-to-
608 code variability that is under investigation in this verification exercise.

609 Figure 9 compares the Fourier transfer functions (surface over reference bedrock motion) and
610 Figure 10 compares pseudo-response spectra at the surface for the P1 profile, with a rigid
611 substratum case. The subplots of these two figures illustrate the results for the high frequency
612 (HF) waveform scaled to the lowest (0.5 m/s^2) and largest PGA (5 m/s^2) (a and c,
613 respectively), and for the low frequency (LF) waveform scaled to the lowest and largest PGA
614 (0.5 m/s^2 – b, and 5 m/s^2 , d, respectively). The frequency content of the input motion and the
615 scaling of the input motion prove to have a large influence on the non-linear soil behavior in
616 the numerical simulations, and consequently on the variability of the results.

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

623 The amount of variability between the results has been quantified through the calculation of
624 the standard deviation (in \log_{10} units) for each frequency value and is illustrated in Figure 11.
625 The variability is greater for the low frequency content input motion scaled to the highest
626 PGA except close to the first frequency peak of the linear transfer function. As expected,
627 strong non-linear soil behavior during this solicitation shifts the first frequency peak of the

628 transfer function to the low frequencies. The variability of the transfer function is similarly
629 shifted.

630 Such variability is strongly linked to the peak shear strain reached in the soil column. For the
631 LF input motion scaled to the highest PGA, the threshold shear strain above which the
632 numerical simulations can no longer be considered as reliable (according to their authors),
633 was reached by some codes. Indeed, some teams (L-1 and Z-0) consider a maximal reliable
634 deformation between 1 to 2%; while others consider their code to work well over a wide
635 range of deformation and are limited by the dynamic soil properties resolution only. For the
636 computations using the HF and LF motions scaled to the highest PGA, we observe that the
637 two equivalent linear methods (J-1 and Z-0) exhibit a very high de-amplification beyond 7
638 Hz, compared to other simulations, which shows the classical over-damping limitation of that
639 method. For the last two cases (HF and LF accelerograms scaled to 5 m/s²), the peak shear
640 strain values are illustrated in Figure 12. It was calculated for each code/team couple, and for
641 all the 10 sensor depths of the P1 profile. The largest peak strain values, largely exceeding
642 1%, are reached at the deepest points for the LF input motion, while it remains about 10 times
643 smaller (max 0.3%) for the HF motion, despite the identical PGA values on the input motion.
644 Besides, given the shape of the G/G_{\max} and $\xi(\gamma)$ curves, one may notice that the frequency-
645 content of the input motion induced variability in the peak shear strain results which
646 corresponds to an even larger variability in the G/G_{\max} and $\xi(\gamma)$ values. For instance, at 7m
647 depth, the peak shear strain for the LF motion is between 0.02 to 1% while it is between 0.03
648 to 0.1% for the HF motion. This means the G/G_{\max} varies from 0.28 for the LF motion to 0.8
649 for the HF motion. Thus, one may understand that the results will be very sensitive to the
650 details of the constitutive model and the way that G/G_{\max} and $\xi(\gamma)$ curves are approximated.

651 Incidentally, one may also notice that for P1, the peak shear strain occurs at the deepest point,
652 close to the sediment/bedrock interface. Indeed, wave propagation in nonlinear media is the

653 cumulative effect of impedance contrast at the soil-bedrock interface, material strength, and
654 intensity of the input motion. These combined effects make it difficult to analyze these results
655 even when they are numerical and consider simple soil geometry.

656 **FIGURE 9**

657 **FIGURE 10**

658 **FIGURE 11**

659 **FIGURE 12**

660 **Epistemic uncertainty**

661 *Quantification of the variability of the results*

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

666 The PGA values at the surface are first compared with the empirical variability (i.e. single
667 station, within-event variability " Φ_{SS} "). Figure 13 illustrates the evolution of σ_{PGA} for the
668 surface site of P1 for the 5 different computational cases and the different reference motion
669 and boundary conditions. These are the linear-elastic, the linear-visco-elastic, and the non-
670 linear computations with the input motions scaled to the lowest (0.5 m/s^2), intermediate (1
671 m/s^2) and highest (5 m/s^2) PGA. The σ_{PGA} is calculated for the pulse-like, the HF and the LF
672 motions. The left subplot displays the results for the rigid substratum case (reference motion =
673 within motion at sediment-basement interface), while the right subplot stands for the elastic

674 substratum case (reference motion = outcropping rock motion). The most striking features of
675 these plots can be summarized as follows:

- 676 a) the (almost) systematic increase of σ_{PGA} with increasing PGA level, whatever the input
677 signal and the type of boundary conditions
- 678 b) the (almost) systematically larger values of σ_{PGA} for the LF input motion compared to
679 the HF input motion case (around twice as great for the three PGA values) : this
680 corresponds to the higher strains generated by the LF motion. A similar plot as a
681 function of peak strain instead of peak ground acceleration would exhibit a larger
682 continuity between results of both input waveforms
- 683 c) the larger σ_{PGA} values for non-linear computations compared to the linear case (except
684 for the very specific case of linear-elastic response with rigid boundary conditions,
685 discussed later)
- 686 d) the maximum obtained σ_{PGA} value (0.15) remains below the specific single-station,
687 within-event variability $\Phi_{\text{SS,PGA}}$ value for a site with a V_{S30} equivalent to P1
688 (Rodriguez-Marek et al., 2011), which is around 0.2. The uncertainties linked with the
689 NL simulations remain below the “natural” single site response variability (at least for
690 PGA values not exceeding 5 m/s^2 and the very peculiar and simplified input wavefield
691 considered here, consisting of pure vertically incident plane S waves. The latter one
692 however includes the sensitivity to the characteristics of the incident wavefield, which
693 is not accounted for here as only vertically incident plane waves are considered.
694 Nonetheless, the use of PGA as a main metric is not enough. It is helpful to use
695 spectral accelerations at other periods as well.

696 Our results indicate an exceptionally high σ_{PGA} value for one linear computation, the linear-
697 elastic one with the HF reference motion and rigid boundary conditions. This computational
698 case is the simplest but also the most demanding for a propagating seismic wave. Considering

699 that no seismic attenuation (damping) is considered for this specific computation (in the
700 material or in the substratum), some codes usually use numerical attenuation to control real
701 motion amplitudes. Thus, the high uncertainty observed here reflects variability in the
702 implementation of the numerical damping for each code/team couple, together with the high
703 sensitivity to the configuration, with a non-zero Fourier content of the reference motion at
704 depth, at a frequency where destructive interferences between up-going and down-going
705 waves should result in a null motion.

706 **FIGURE 13**

707 We then explored the variability of various seismic intensity measures: (i) the response
708 spectra at the surface (SA) at three different periods (0.1, 1 and 3 s), (ii) the peak shear strain
709 at the bottom of the sediment layer (ε), (iii) the Cumulative Absolute Velocity (CAV), (iv) the
710 Arias Intensity (IA), (v) the root mean square acceleration (Arms), and (vi) the 5%-95%
711 Trifunac-Brady duration (DT). The tendencies are quite similar for the HF and LF motions,
712 but are sensitive to the sediment/substratum limit condition (elastic vs. rigid). Considering
713 that σ_{PGA} is greater for the LF motion, we choose that motion to illustrate the results in Figure
714 14. For the rigid substratum case (left subplot), three groups of intensity parameters can be
715 identified. The first group is composed of duration-dependent intensity parameters, i.e., CAV,
716 IA and DT, which exhibit the largest σ values. The second group is composed of acceleration
717 parameters (PGA, SA(T), Arms) and characterized by a lower σ , especially for long period
718 [SA (T = 1 s)]. The third group consists only of the peak strain, with generally intermediate σ
719 values, which however exhibit the largest variability from one case to another. These three
720 groups can also be distinguished in the elastic substratum case (right subplot), for which the
721 largest case-to-case variability is also observed for the peak strain, exhibiting the highest σ for
722 the highest PGA values. The duration-dependent parameters of the first group are less
723 variable under elastic boundary conditions especially at low to intermediate PGA levels and

724 in the linear domain: rigid base conditions are very demanding for low damping materials,
725 which maps much more on duration than on peak values.

726 **FIGURE 14**

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

733 **FIGURE 15**

734 ***Origins of the variability: Can it be reduced?***

735 **Definition of Groups and Sub-groups**

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

742 Case G1 concerns the implementation of linear, intrinsic damping, as defined in the first part
743 of this article. It is sub-divided into 3 sub-groups: (i) G1a: frequency-independent attenuation
744 (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,
745 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-
746 0, N-0, D-0 and R-0).

747 Case G2 is based on the numerical discretization scheme, which is sub-divided into 2
748 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,
749 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-
750 2). A third sub-group could be considered G2c: consisting of equivalent linear codes working
751 in the frequency domain (J-1 and Z-0).

752 Case G3 is based on the constitutive model. To ensure sufficient teams within each group, we
753 split the code/team couple into 4 sub-groups according to the main constitutive model used:
754 (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:
755 Philips and Hashash's model (F-0, J-0, L-2, M-2, T-0), and (iv) G3d: all other models.

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

759 In the "canonical" models initially designed by the organizing team, the soil shear strength
760 profile was assumed to be constant with depth in each soil layer, and had prescribed modulus
761 reduction and damping curves. However, in most real situations, the shear strength should
762 increase with depth. Even though these profiles were considered as "idealized" and simply
763 intended to perform these verification tests, some teams felt very uncomfortable with this
764 unrealistic assumption and decided to change the shear strength profile, by introducing a more
765 realistic increase in shear strength with depth, having nevertheless, the imposed strength
766 values at the center of each layer. Consequently, the actual non-linear soil parameters
767 considered by each team were not identical, which is certainly responsible for part of the final
768 variability observed, especially for large ground motions, for which the actual strain and
769 damping are more sensitive to the shear strength than to the shear velocity, particularly at or
770 close to major interfaces. For this reason, we further sorted each code/team couple into 2 sub-

771 groups, by analyzing the stress-strain plots for the LF motion and the highest PGA at the
772 bottom of P1 (illustrated in Figure 16). We choose this computational case because it is the
773 most challenging in terms of maximal shear strain reached in the soil column and therefore
774 can highlight the differences between the computations. We found the following sub-groups:
775 (i) shear strength is equal to 65kPa, as stated by the organizing team (A-0, B-0, C-0, E-0, F-0,
776 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,
777 L-1, N-0, M-0, M-1, M-2, R-0, S-0, W-0, Z-0, Z-1).

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

783 Combining these two last parameters we end-up for G4 with three subgroups as follow: (i)
784 G4-a: Specified shear strength and use of damping control (A-0, B-0, E-0, F-0, T-0), (ii) G4-
785 b: Specified shear strength and no use of damping control and (C-0, G-0, H-0, K-0, Q-0, U-0,
786 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,
787 W-0, Z-0, Z-1).

788 **FIGURE 16**

789 **Variability within the sub-groups**

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

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

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

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

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

816

FIGURE 17

817 Figure 18 compares the pseudo-acceleration response spectra at the surface of the P1 profile
818 with a rigid substratum condition subjected to LF and HF input motions scaled at the medium

819 (1m/s²) and highest (5 m/s²) PGA levels. The response spectra are sorted according to the G4
820 sub-grouping, and the associated σ_{log} is represented by the thin lines on top of each subplot
821 (the numbers on the right side indicate the number of code/team pairs in each sub-group). G4
822 enables a clear distinction of the response spectra; particularly for the most demanding LF
823 input motion. The σ_{log} values from the two sub-groups with identical τ_{max} (G4a and G4b) are
824 considerably reduced below 2 s, compared to the rest of the computations (G4c). This period
825 bandwidth is relative to the PGA of this LF input motion. Similarly, for the HF input motion,
826 the σ is reduced below 1 s.

827 Besides, the response spectra computed for the strongest input motions (HF and LF) at the
828 surface of groups G4a and G4b are significantly different one to another which shows the
829 large impact of using damping control or not. The response spectra computed with damping
830 control are more damped at intermediate frequencies (period between [0.2 to 0.7] s and [0.2 to
831 1] s for the HF and LF motion respectively) and less attenuated at low frequencies (periods
832 greater than 0.7 and 1 s for the HF and LF motion respectively).

833 **FIGURE 18**

834 **Conclusions**

835 In the PRENOLIN's verification phase, the linear computation involving a simple pulse-like
836 (Ricker) input motion proved to be very useful in understanding and eliminating some of the
837 discrepancies between the different numerical codes that were compared. It was found that
838 code-to-code differences can be attributed to three different sources: (1) minor mistakes in
839 input parameter implementation or output units, (2) different understanding of the expression
840 "input motion" within different communities, and (3) different intrinsic attenuation and
841 numerical integration implementations. This benchmark showed that any nonlinear code

842 should be tested with simple linear cases before going into nonlinear computations to ensure
843 the proper implementation of the elastic soil parameters.

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

848 The results obtained so far indicate a code-to-code variability, which increases with the shear
849 strain level (which in turn depends on both the PGA level, stiffness of the soil and the
850 frequency content of the reference input motion). For example, for the low frequency input
851 motion, at low PGA values (0.5 m/s^2) at the base of the soil column, the standard deviation is
852 0.065 in logarithmic units, whereas for a PGA of 5 m/s^2 , the standard deviation is 0.17. We
853 also found that, whatever the soil profiles used (among the 3 soil profiles considered), the
854 overall code-to-code variability in the worst case (with strain levels exceeding 1%) remained
855 lower than the random variability of GMPE single-station σ values for PGA. Nevertheless, an
856 important conclusion is that given the scatter in the nonlinear results, a realistic analysis
857 should use more than one code to perform a site response computation.

858 The effect of different non-linear soil model implementations was explored in this study and
859 our main observations indicate that the epistemic uncertainty (i.e. the code-to-code
860 variability) can be significantly reduced by describing more precisely some specific input
861 parameters, especially the soil shear strength profile, which is found to be a key specification
862 in addition to the degradation curves. In addition, when damping control is used, the
863 variability of the stress-strain curves was found to be large. This point deserves further
864 research in future studies. All these features and conclusions need to be checked against
865 actual data to provide support for defining best practice for modeling non-linearities in soft

866 soils. In that aim, vertical arrays having multiple down-hole sensors constitute indeed the best
867 available in-situ instrumentation to go forward with this kind of study. The benchmark
868 undoubtedly benefits a lot from the various expertise fields of the participants ranging from
869 geotechnical earthquake engineering to engineering seismology.

870 **Data and resources**

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

875 Some codes used in this work have URL links:

876 ASTER: <http://www.code-aster.org>;

877 EPISPEC1D: <http://efispec.free.fr>;

878 real ESSI Simulator : http://sokocalo.engr.ucdavis.edu/~Jeremić/Real_ESSI_Simulator/;

879 Opensees: <http://opensees.berkeley.edu/>

880 DEEPSOIL: <http://deepsoil.cce.illinois.edu/>

881 SeismoSoil: <http://asimaki.caltech.edu/resources/index.html#software>

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895

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1155 **TABLES**

1156 *Table 1: Soil properties for all three simple profile cases studied here (P1-3), for the elastic and non-elastic*
 1157 *domains.*

Profile	LINEAR									NL ⁱ
	Z ⁱⁱ [m]	V _s ⁱⁱⁱ [m/s]	V _p ^{iv} [m/s]	ρ ^v [kg/m ³]	Q ^{vi} Elastic	ξ _{min} ^{vii} Elastic	Q Visco- Elastic	ξ _{min} Visco- Elastic	f ₀ ^{viii} 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 <i>Mono- layer + V_{gradient}</i>	0-20	150- 500	360- 1220	2000			34	0.0154 7	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 <i>Bi-layer</i>	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	-			

ⁱ Describe the G/G_{max} and damping curves used for each soil layer

ⁱⁱ Depth of the soil layer

ⁱⁱⁱ Shear wave velocity

^{iv} Compressional wave velocity

^v Density

^{vi} Quality factor

^{vii} Elastic damping

^{viii} Fundamental resonance frequency of the soil

1158

1159 *Table 2 Seismic metadata of the two real input motions used in the verification phase of the Prenolin*
 1160 *project.*

Event Freq. Content	Event ID	Mw _{ix}	Z ^x [km]	Epi. Dist. [km]	Station ID	Station Geology	Seismo Comp.	Vs30 ^{xi} [m/s]
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

1161

^{ix} Moment magnitude

^x Depth of epicenter

^{xi} Mean harmonic S-wave velocity over the first 30m depth

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1163

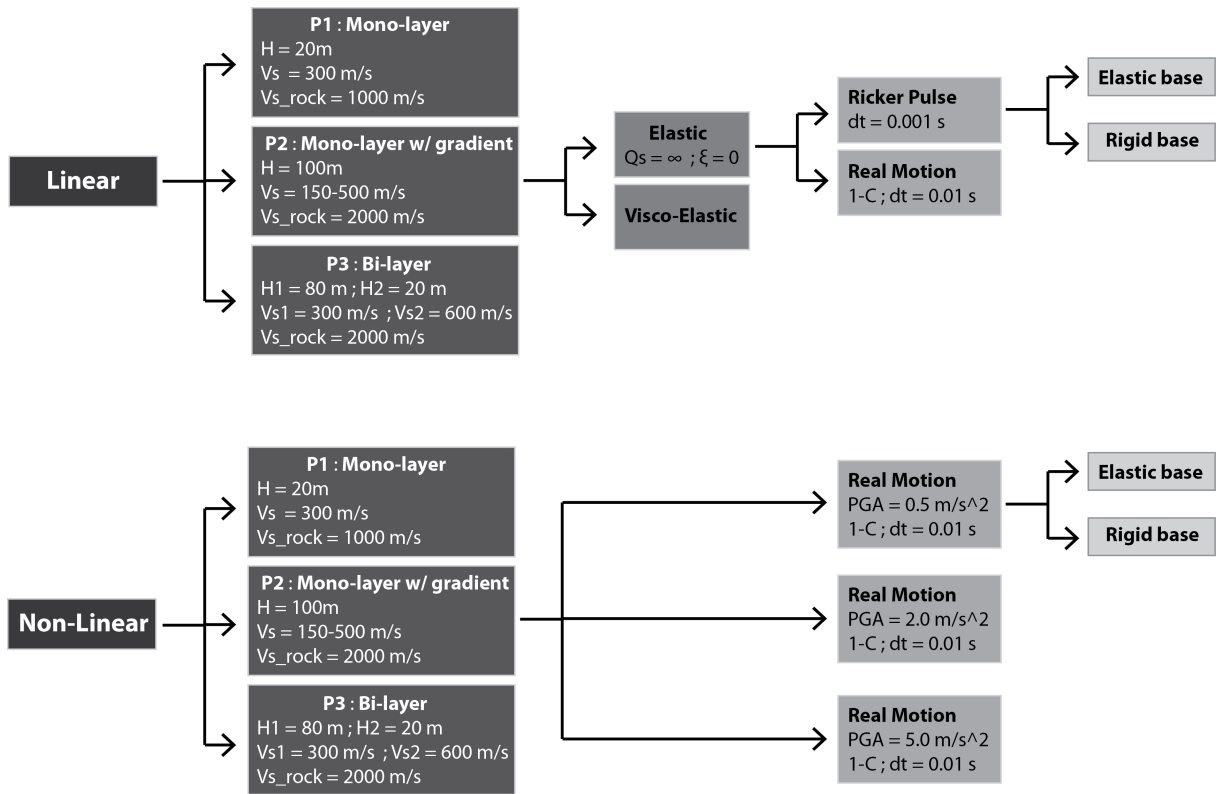
Table 3: Participants to the PRENOLIN project Verification phase.

Team Name	Affiliation	Team Index		Code Name	Code Reference
<i>D. Asimaki & J. Shi</i>	<i>Caltech, US</i>	A	0	SeismoSoil	(Shi and Asimaki, 2016; Li and Assimaki, 2010; Matasovic and vucetic, 1993)
<i>S. Iai</i>	<i>DPRI, Kyoto University, Japan</i>	B	0	FLIP	(Iai, 1990)
<i>S. Kramer</i>	<i>Univ. Washington, US</i>	C	0	PSNL	(In development)
<i>E. Foerster</i>	<i>CEA, France</i>	D	0	CYBERQUAKE	(Modaressi and Foerster, 2000)
<i>C. Gelis</i>	<i>IRSN, France</i>	E	0	NOAH-2D	(Iai, 1990)
<i>A. Giannakou</i>	<i>Fugro, France</i>	F	0	DEEPSOIL 5.1	(Hashash et al., 2012)
<i>G. Gazetas, E. Garini & N. Gerolymos</i>	<i>NTUA, Greece</i>	G	0	NL-DYAS	(Gerolymos and Gazetas, 2006, 2005)
<i>J. Gingery & A. Elgamal</i>	<i>UCSD, US</i>	H	0	OPENSEES-UCSD-SOIL-MODEL	See Data and Resources

<i>Y. Hashash & J. Harmon</i>	<i>Univ, Illinois, US</i>	J	0	DEEPSOIL-NL 5.1	(Hashash et al., 2012)
		J	1	DEEPSOIL-EL 5.1	(Hashash et al., 2012)
<i>P. Moczo, J. Kristek & A. Richterova</i>	<i>CUB, Comenius University, Bratislava, Slovakia</i>	K	0	1DFD-NL-IM	
<i>S. Foti & S. Kontoe</i>	<i>Politecnico di Torino & Imperial College, Italy</i>	L	1	ICFEP	(Kontoe, 2006; Potts and Zdravkovic, 1999; Taborda et al., 2010)
		L	2	DEEPSOIL-NL 5.1	(Hashash et al., 2012)
<i>G. Lanzo, S. Suwal, A. Pagliaroli & L. Verrucci</i>	<i>Univ. Rome La Sapienza & University of Chieti-Pescara, Italy</i>	M	0	FLAC_7,00	(ITASCA, 2011)
		M	1	DMOD2000	(Matasović and Ordóñez, 2007)
		M	2	DEEPSOIL 5.1	(Hashash et al., 2012)
<i>F. Lopez-Caballero & S. Montoya-Noguera</i>	<i>ECP, France</i>	N	0	GEFDyn	(Aubry and Modaressi, 1996)
<i>F. De-Martin</i>	<i>BRGM, France</i>	Q	0	EPISPEC1D	(Iai, 1990) See Data and Resources
<i>B. Jeremić, F. Pisanò & K. Watanabe</i>	<i>UCD, LBLN, TU Delft & Shimizu Corp</i>	R	0	real ESSI Simulator	See Data and Resources
<i>A. Nieto-Ferro</i>	<i>EDF, France</i>	S	0	ASTER	See Data and Resources
<i>A. Chiaradonna, F. Silvestri & G. Tropeano</i>	<i>UNICA and Univ. Naples, Italy</i>	T	0	SCOSSA_1,2	(Tropeano et al., 2015)
		T	1	STRATA	
<i>M.P. Santisi d'Avila</i>	<i>Univ. Nice Sophia-Antipolis, France</i>	U	0	SWAP_3C	(Santisi d'Avila et al., 2012, 2013; Santisi d'Avila and Semblat, 2014)
<i>D. Mercerat and N. Glinsky</i>	<i>CEREMA, France</i>	Y	0	DGNL	(Mercerat and Glinsky, 2015)
<i>D. Boldini, A. Amorosi, A. di Lernia & G. Falcone</i>	<i>Univ. Bologna and Sapienza University of Rome, Italy</i>	Z	0	EERA	(Bardet et al., 2000)
		Z	1	PLAXIS	(Benz, 2006; Benz et al., 2009)
<i>M. Taiebat & P. Arduino</i>	<i>UBC, Canada & UW, USA</i>	W	0	Opensees	See Data and Resources

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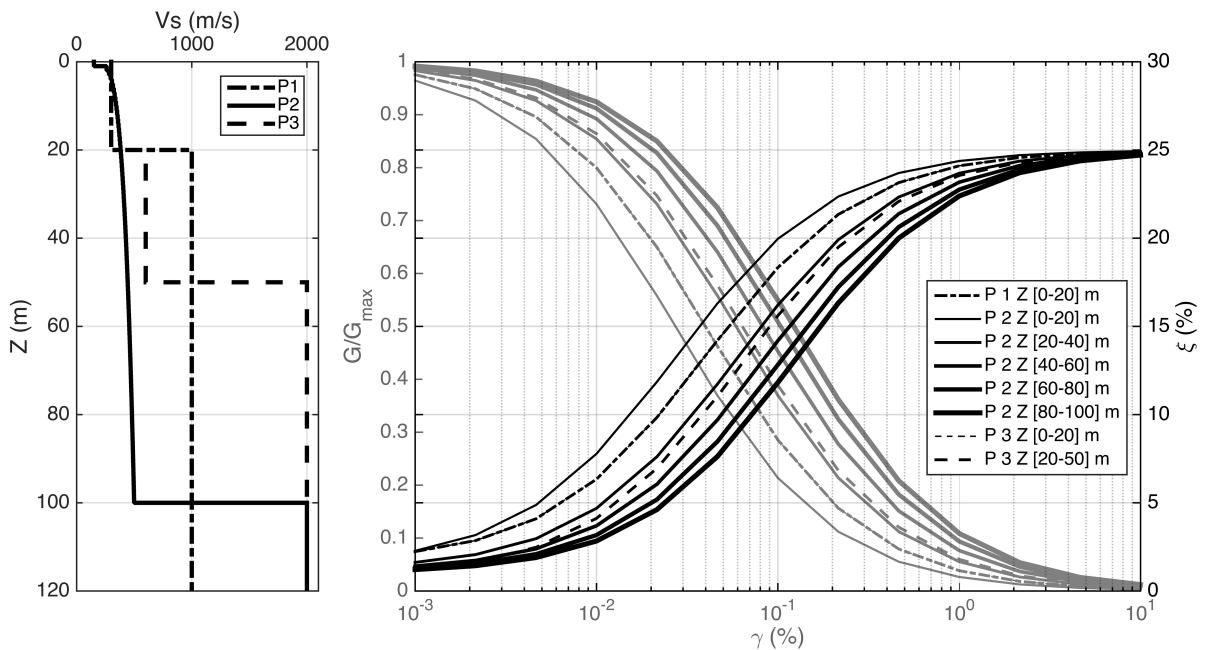


1167

1168 **Figure 1 :** The three simple idealized profile cases studied here (P1-3), for the elastic and non-elastic domains,

1169 and for a rigid and elastic soil-bedrock base, using a Ricker pulse and 3 accelerations of different PGA and

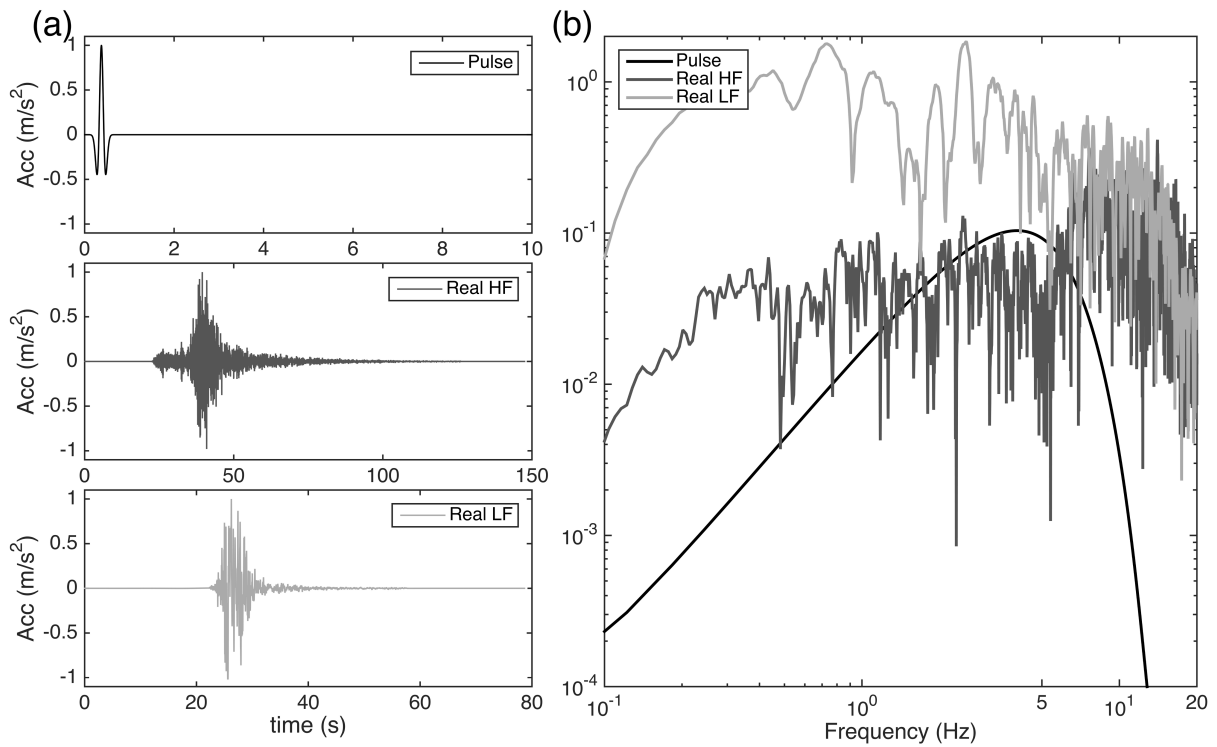
1170 frequency contents.



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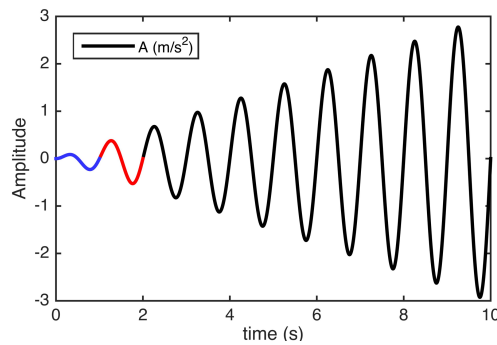
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Figure 2: V_s profiles, G/G_{max} and damping curves for the 3 idealized profiles.



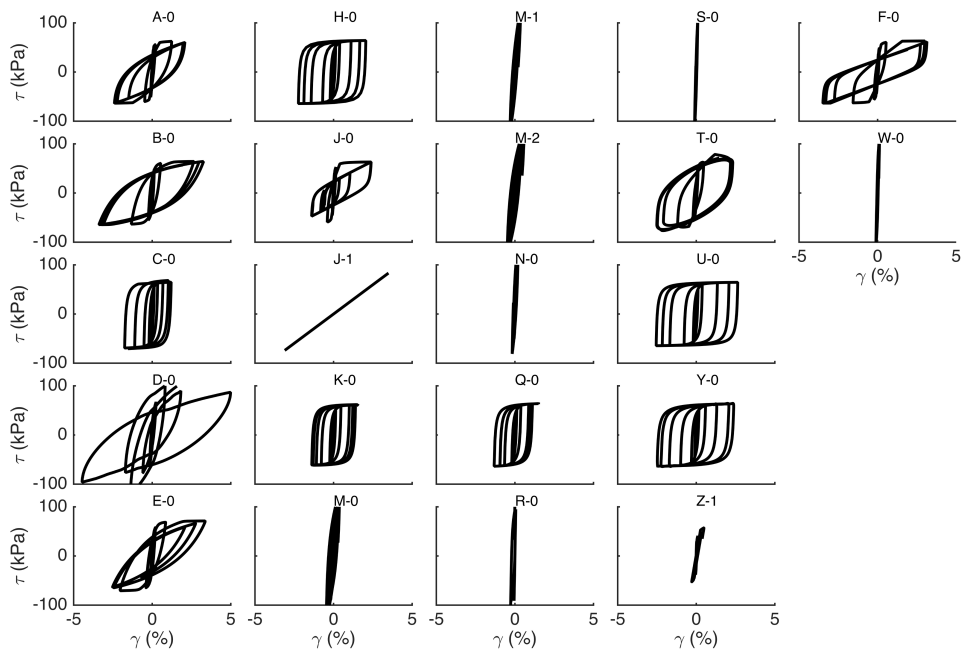
1173

1174 **Figure 3:** Normalized acceleration of the reference motions used for the verification phase (Pulse, real motions
1175 : High Frequency motion and Low Frequency motion) of this project PRENOLIN (panel a) with the associated
1176 Fourier spectrum(panel b)



1177

1178 **Figure 4 :** Acceleration time history of the sinusoidal motion with central 1s period



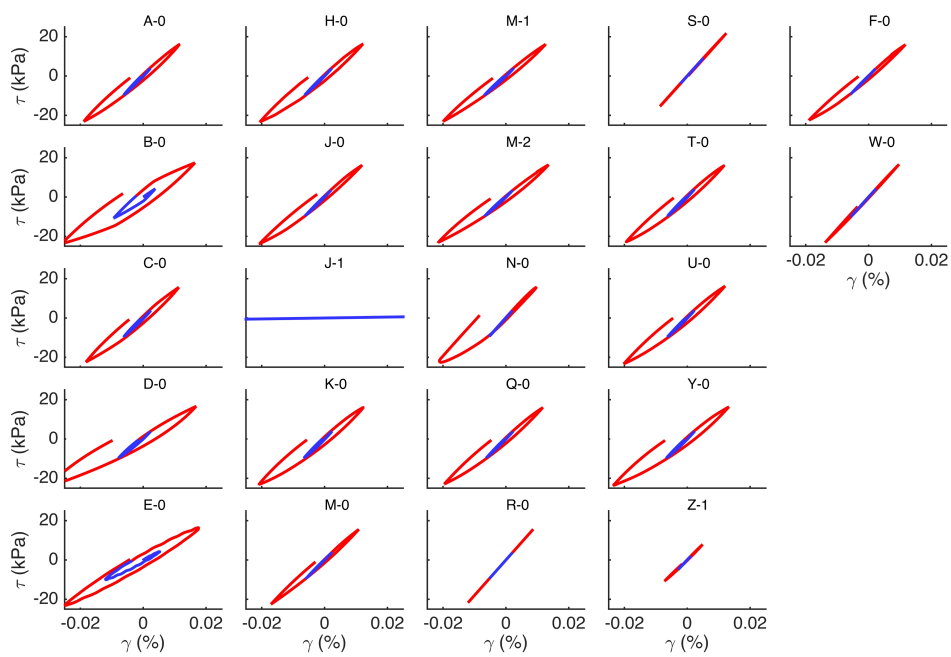
1179

1180 **Figure 5 :** Stress-strain curve for a soil element of shear strength 65kPa subjected to a sinusoidal input seismic

1181

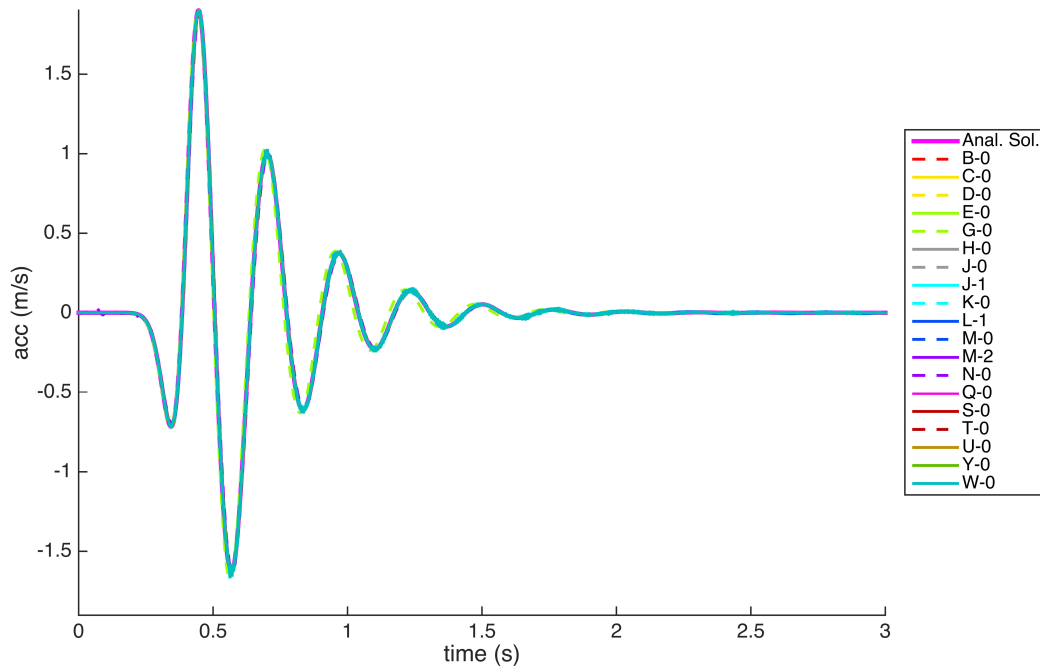
motion of 10s.

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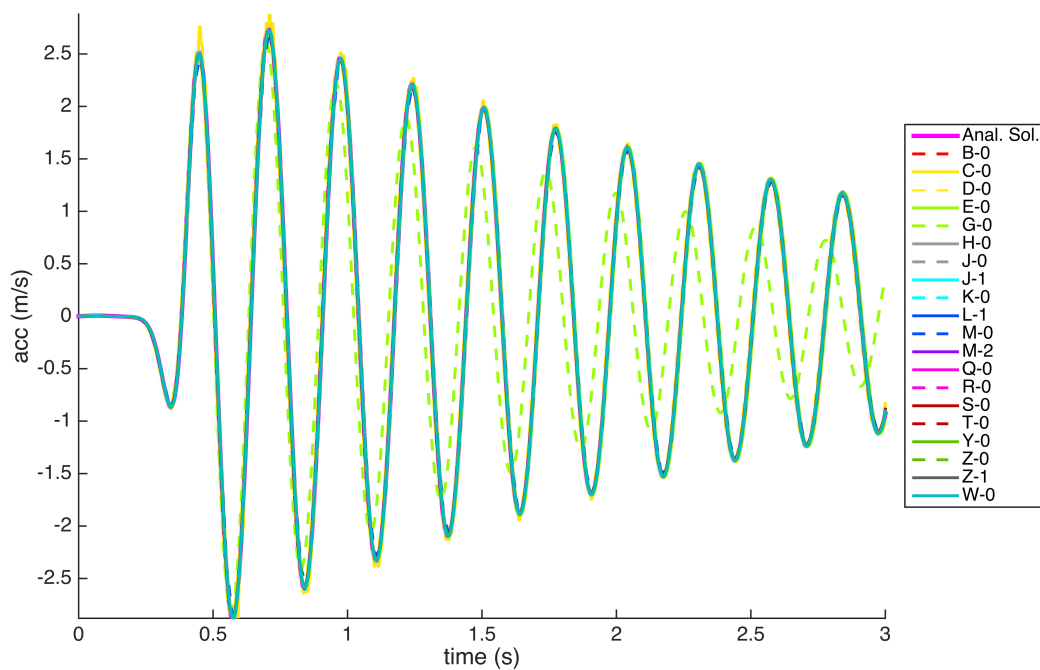


1183

1184 **Figure 6** : Stress-strain curve for a soil element of shear strength 65kPa subjected to the first two cycles of a
1185 sinusoidal input seismic motion.

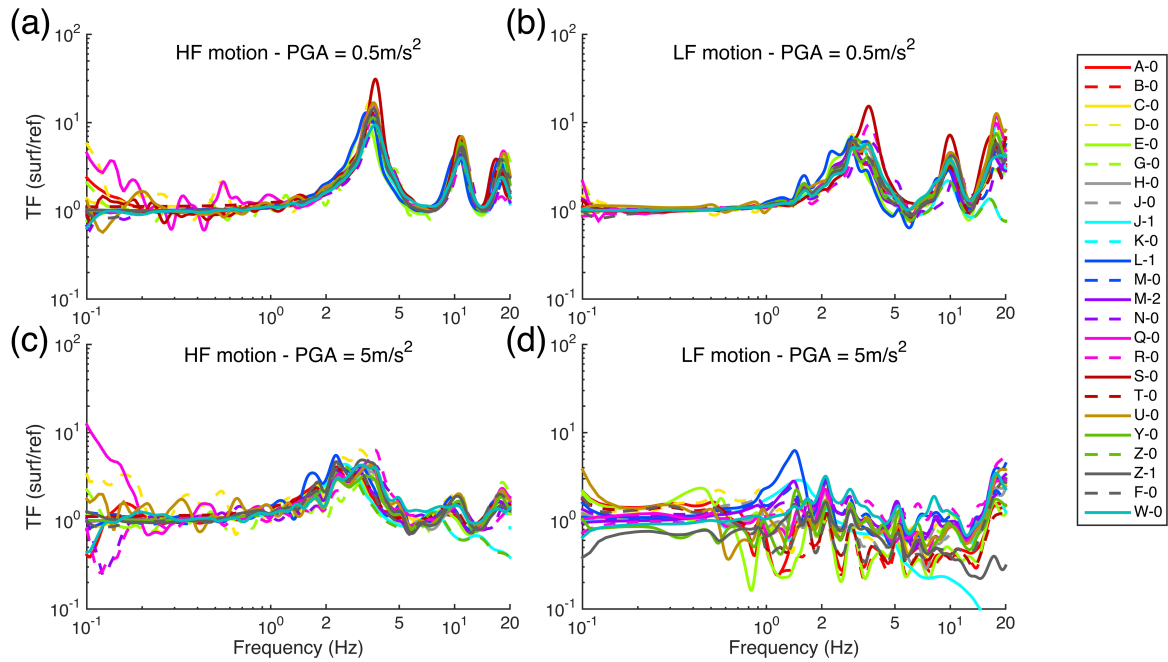


1186
1187 **Figure 7**: Comparison of the acceleration at the surface of P1 profile, for the pulse-like input motion,
1188 for the linear elastic computation and for the elastic substratum case.

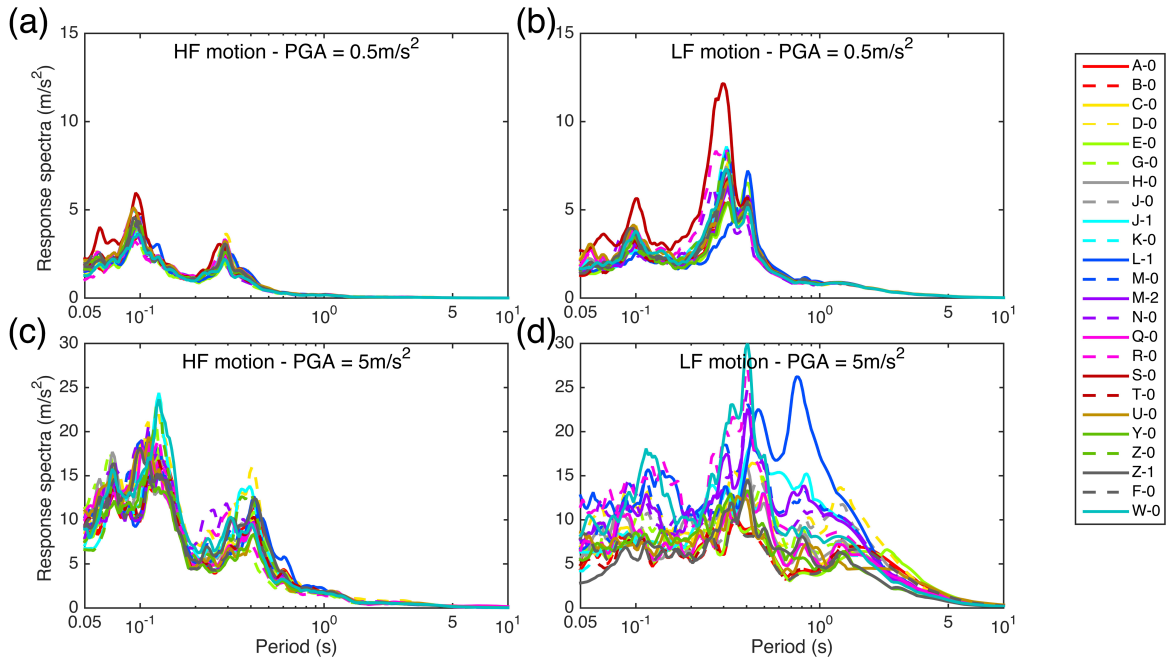


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1190 **Figure 8** Comparison of the acceleration at the surface of P1 profile, for the pulse-like input motion,
 1191 for the linear visco-elastic computation and for the rigid substratum case.



1192
 1193 **Figure 9** : Comparison of the surface to reference Fourier spectra ratio, for the non-linear computations using
 1194 the high frequency input motion scaled to 0.5 m/s² (graph a) and scaled at 5m/s² (graph c) and using the low-
 1195 frequency input motion scaled to 0.5 m/s² (graph b) and scaled at 5m/s² (graph d).



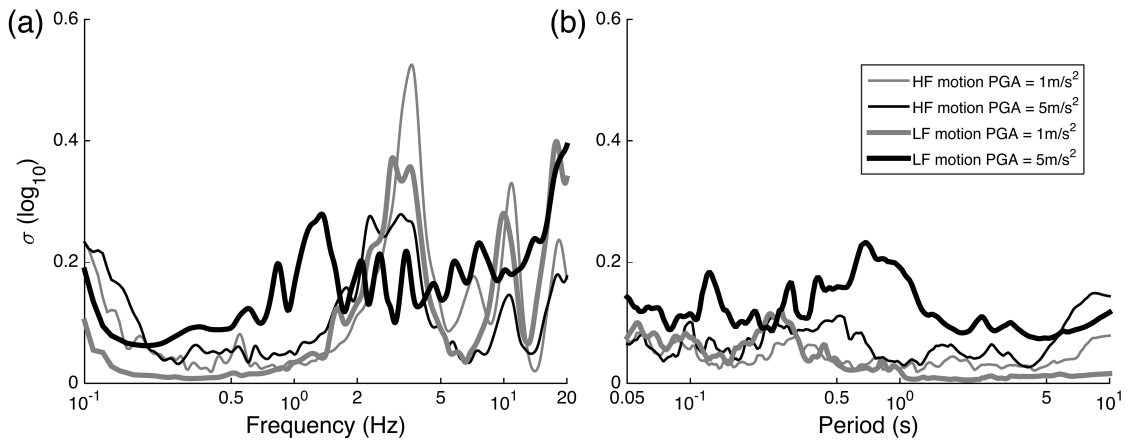
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Figure 10 : Comparison of the acceleration pseudo-response spectra at the ground surface, for the non-linear computations using the high frequency input motion scaled to 0.5 m/s^2 (graph a) and scaled at 5 m/s^2 (graph c) and using the low-frequency input motion scaled to 0.5 m/s^2 (graph b) and scaled at 5 m/s^2 (graph d).

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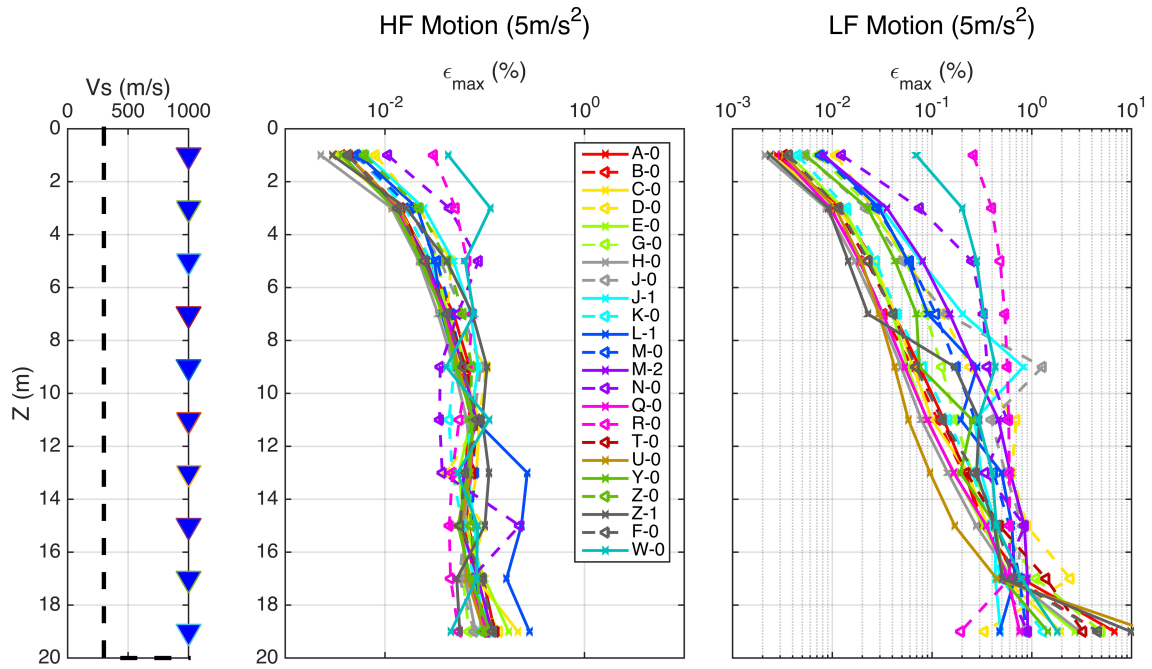


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Figure 11 : Standard deviation (in log unit) of the transfer function (left) and response spectra (right) depending of the input motion used.

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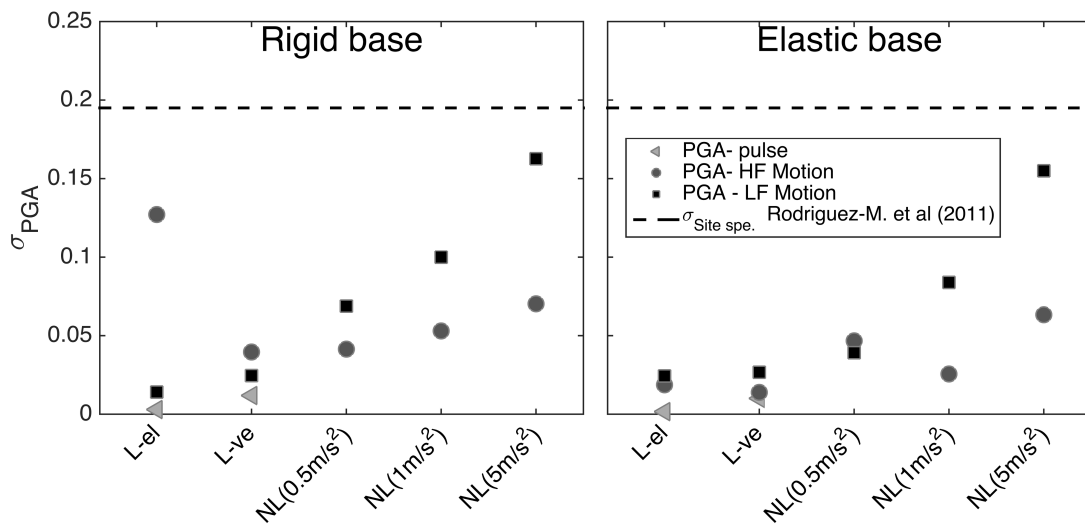
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1204

Figure 12 : Peak shear strain profiles reached at each depth by each team for the high and low frequency

1205

reference motion scaled at the highest PGA level (5 m/s^2), for the profile 1 and for rigid substratum conditions.



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1207

Figure 13 : Standard deviation (in log10 unit) of the PGA at the surface of the P1 profile, for the 5 different

1208

computational cases (linear –elastic, linear visco-elastic, non-linear with input motion scaled to the lowest

1209

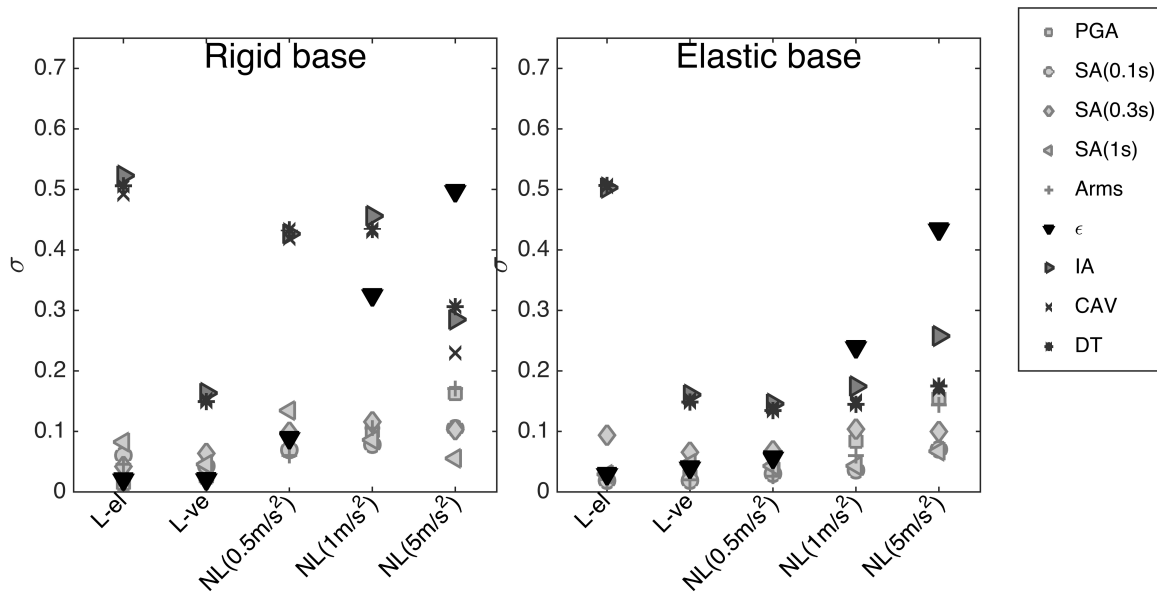
(0.5 m/s^2), medium (1 m/s^2) and highest (5 m/s^2) PGA, for the pulse-like, the high frequency and the low frequency

1210

content motions. The left sub-plot shows the results for the rigid substratum case and the right sub-plot for the

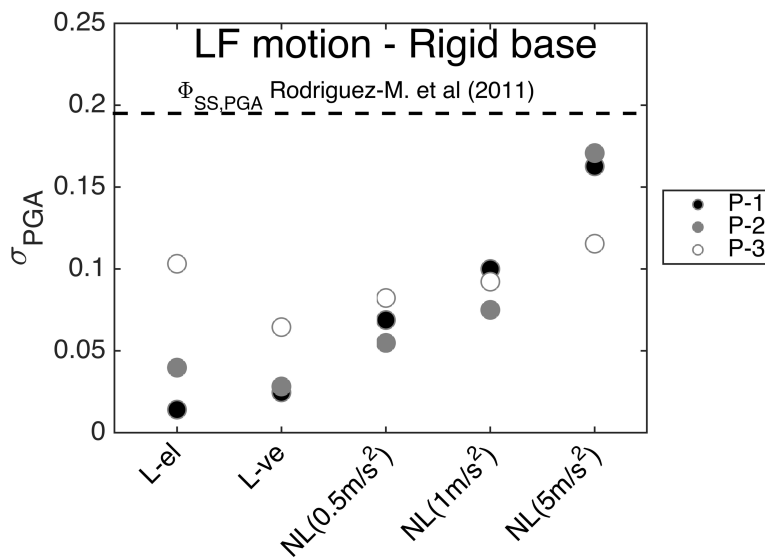
1211

elastic substratum.



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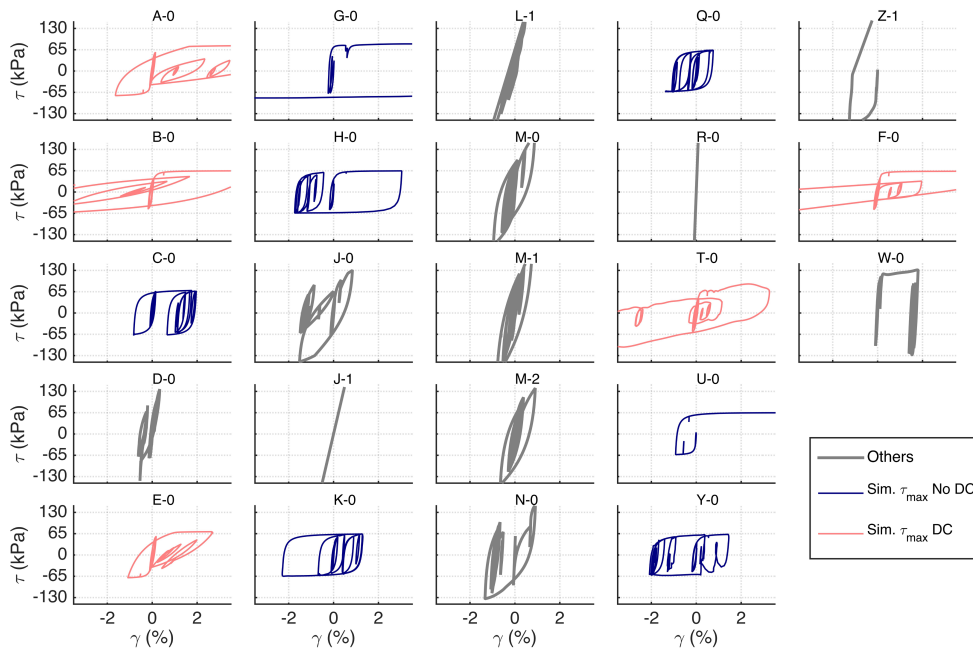
1213 **Figure 14** : Standard deviation (in log unit) of the different intensity parameters for the P1 profile, for the 5
 1214 different computational cases (linear –elastic, linear visco-elastic, non-linear with input motion scaled to the
 1215 lowest (0.5m/s²), medium (1m/s²) and highest (5m/s²) PGA, for the low frequency content motion. The left sub-
 1216 graph shows the results for the rigid substratum case and the right sub-graph for the elastic substratum.



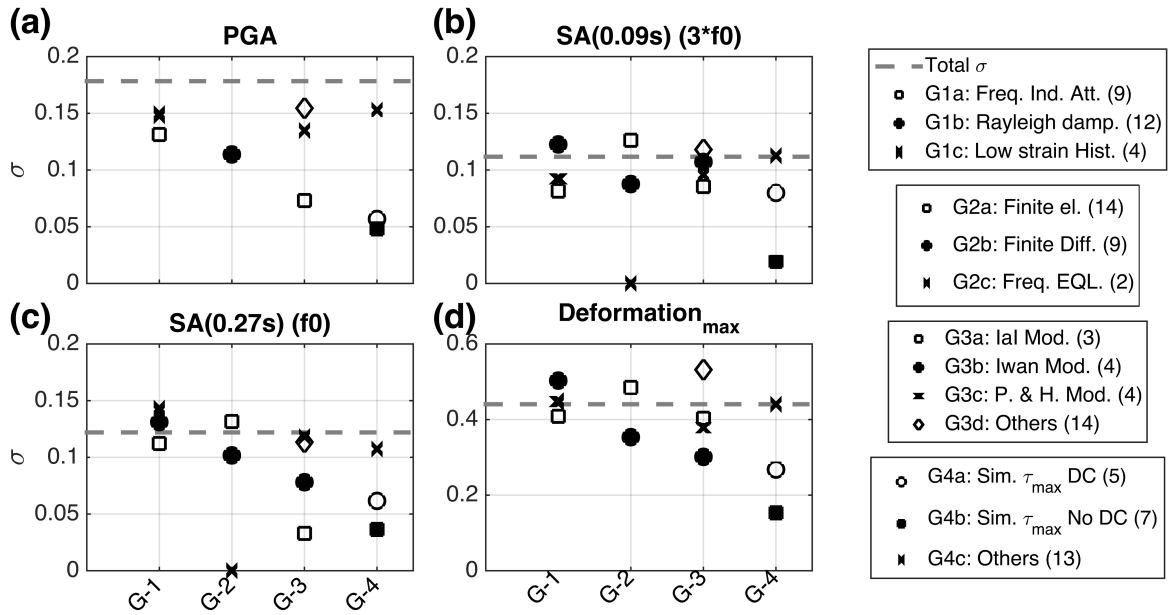
1217

1218 **Figure 15** : Standard deviation (in log unit) of the PGA for the profile 1 2 and 3, for the 5 different
 1219 computational cases (linear –elastic, linear visco-elastic, non-linear with input motion scaled to the lowest

1220 (0.5m/s²), medium (1m/s²) and highest (5m/s²) PGA, for the low frequency content motion and for the rigid
 1221 substratum case.



1222
 1223 **Figure 16** : Stress-strain curves at the bottom of P1 Profile for the Rigid substratum case subjected to the low
 1224 frequency motion (in color and the high frequency motion in black scaled to the highest PGA (5m/s²)). The
 1225 thicker curves are for code/team couples that exceed the specified shear strength of 65 KPa, whereas the thin
 1226 curves represent the code/team couples that use 65 KPa. The lighter curves are for codes using damping control
 1227 and the darker curves the others.



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1229

Figure 17 : Standard deviation values (σ_{log} , in \log_{10} units) of four parameters for the non-linear computation

1230

using the low-frequency content input motion scaled to the highest PGA : PGA (a), Response spectra at 0.27 s

1231

(b), Response spectra at 0.09 s (c) all three at the surface of P1 and the maximal shear deformation at the

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bottom of the P1 profile (d). The standard deviation are given for each group of the four groupings: depending

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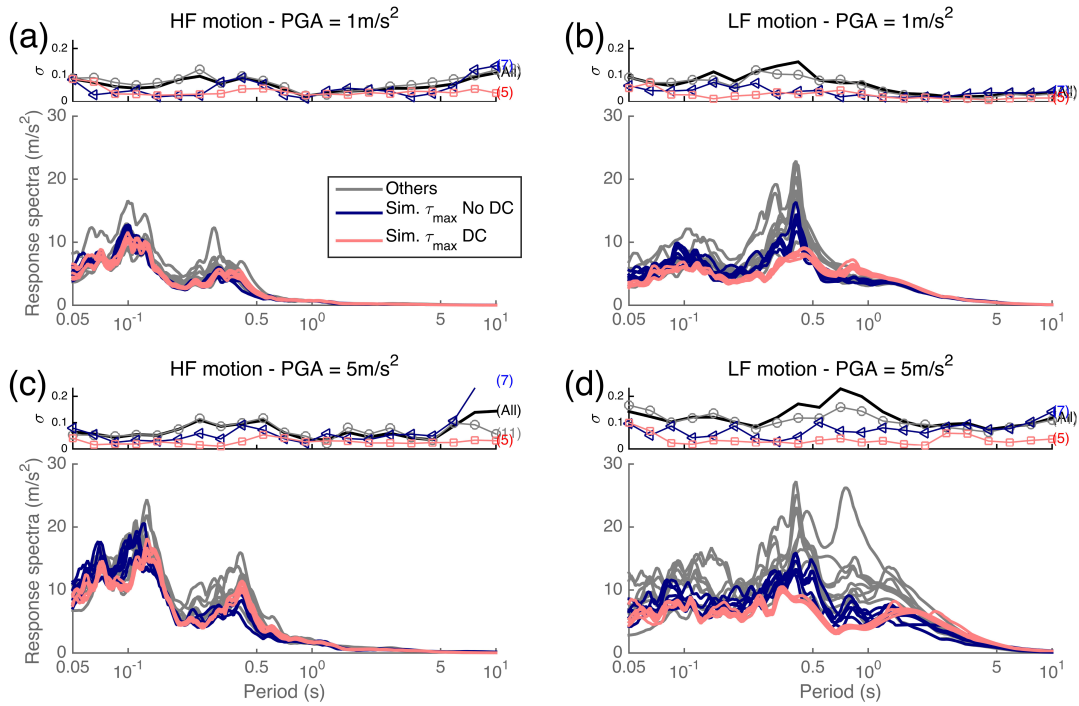
on their low strain attenuation implementation (G-1) their numerical scheme (G-2) their constitutive models (G-

1234

3) and their values of shear strength at the bottom of P1 and use of damping control or not (G-5). The dotted

1235

grey line illustrates the standard deviation for all code/team couples.



1236

1237

Figure 18 : Comparison of the pseudo- acceleration response spectra at the ground surface of P1 with rigid

1238

substratum condition, for the non-linear computation using for (a) and (c) graphs the high frequency input

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motion and for (b) and (d) graphs the low-frequency input motion and with for the (a) and (b) graphs the middle

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input motion PGA and the (c) and (d) graphs the highest input motion PGA. The response spectra were sorted

1241

according to three groups: group 1 is composed of the code/team couples using similar τ_{max} and damping control

1242

constitutive model. Group 2 use similar τ_{max} and no damping control and Group 3 are the other code team

1243

couples. The standard deviation for each sub-group is compared to the standard deviation using all results in

1244

the upper graphs, the figures indicate the number of simulations that are in each group.

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1246

1247 **List of Figures**

1248 **Figure 1 :** The three simple idealized profile cases studied here (P1-3), for the elastic and non-elastic
1249 domains, and for a rigid and elastic soil-bedrock base, using a Ricker pulse and 3 accelerations of
1250 different PGA and frequency contents.

1251 **Figure 2:** Vs profiles, G/G_{\max} and damping curves for the 3 idealized profiles.

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1253 motions : High Frequency motion and Low Frequency motion) of this project PRENOLIN (panel a)
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1261 the linear elastic computation and for the elastic substratum case.

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1263 the linear visco-elastic computation and for the rigid substratum case.

1264 **Figure 9 :** Comparison of the surface to reference Fourier spectra ratio, for the non-linear computations
1265 using the high frequency input motion scaled to 0.5 m/s^2 (graph a) and scaled at 5m/s^2 (graph c)
1266 and using the low-frequency input motion scaled to 0.5 m/s^2 (graph b) and scaled at 5m/s^2 (graph
1267 d).

1268 **Figure 10 :** Comparison of the acceleration pseudo-response spectra at the ground surface, for the non-
1269 linear computations using the high frequency input motion scaled to 0.5 m/s^2 (graph a) and scaled
1270 at 5m/s^2 (graph c) and using the low-frequency input motion scaled to 0.5 m/s^2 (graph b) and scaled
1271 at 5m/s^2 (graph d).

1272 **Figure 11 :** Standard deviation (in log unit) of the transfer function (left) and response spectra (right)
1273 depending of the input motion used.

1274 **Figure 12** : Peak shear strain profiles reached at each depth by each team for the high and low frequency
1275 reference motion scaled at the highest PGA level (5 m/s^2), for the profile 1 and for rigid substratum
1276 conditions.

1277 **Figure 13** : Standard deviation (in \log_{10} unit) of the PGA at the surface of the P1 profile, for the 5 different
1278 computational cases (linear –elastic, linear visco-elastic, non-linear with input motion scaled to the
1279 lowest (0.5 m/s^2), medium (1 m/s^2) and highest (5 m/s^2) PGA, for the pulse-like, the high frequency
1280 and the low frequency content motions. The left sub-plot shows the results for the rigid substratum
1281 case and the right sub-plot for the elastic substratum.

1282 **Figure 14** : Standard deviation (in log unit) of the different intensity parameters for the P1 profile, for the
1283 5 different computational cases (linear –elastic, linear visco-elastic, non-linear with input motion
1284 scaled to the lowest (0.5 m/s^2), medium (1 m/s^2) and highest (5 m/s^2) PGA, for the low frequency
1285 content motion. The left sub-graph shows the results for the rigid substratum case and the right sub-
1286 graph for the elastic substratum.

1287 **Figure 15** : Standard deviation (in log unit) of the PGA for the profile 1 2 and 3, for the 5 different
1288 computational cases (linear –elastic, linear visco-elastic, non-linear with input motion scaled to the
1289 lowest

1290 **Figure 16** : Stress-strain curves at the bottom of P1 Profile for the Rigid substratum case subjected to the
1291 low frequency motion (in color and the high frequency motion in black scaled to the highest PGA
1292 (5 m/s^2). The thicker curves are for code/team couples that exceed the specified shear strength of
1293 65 KPa , whereas the thin curves represent the code/team couples that use 65 KPa . The lighter
1294 curves are for codes using damping control and the darker curves the others.

1295 **Figure 17** : Standard deviation values (σ_{\log} , in \log_{10} units) of four parameters for the non-linear
1296 computation using the low-frequency content input motion scaled to the highest PGA : PGA (a),
1297 Response spectra at 0.27 s (b), Response spectra at 0.09 s (c) all three at the surface of P1 and the
1298 maximal shear deformation at the bottom of the P1 profile (d). The standard deviation are given for
1299 each group of the four groupings: depending on their low strain attenuation implementation (G-1)
1300 their numerical scheme (G-2) their constitutive models (G-3) and their values of shear strength at
1301 the bottom of P1 and use of damping control or not (G-5). The dotted grey line illustrates the
1302 standard deviation for all code/team couples.

1303 **Figure 18** : Comparison of the pseudo- acceleration response spectra at the ground surface of P1 with
1304 rigid substratum condition, for the non-linear computation using for (a) and (c) graphs the high
1305 frequency input motion and for (b) and (d) graphs the low-frequency input motion and with for the
1306 (a) and (b) graphs the middle input motion PGA and the (c) and (d) graphs the highest input motion
1307 PGA. The response spectra were sorted according to three groups: group 1 is composed of the
1308 code/team couples using similar τ_{max} and damping control constitutive model. Group 2 use similar
1309 τ_{max} and no damping control and Group 3 are the other code team couples. The standard deviation
1310 for each sub-group is compared to the standard deviation using all results in the upper graphs, the
1311 figures indicate the number of simulations that are in each group.

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1313