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PRENOLIN: International Benchmark on 1D Nonlinear SiteResponse Analysis—Validation Phase Exercise

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Abstract:	This article presents the main results of the validation phase of the PRENOLIN project. PRENOLIN is an international benchmark on 1-D non-linear site response analysis. This project involved 19 teams with 23 different codes tested. It was divided into two phases; with the first phase verifying the numerical solution of these codes on idealized soil profiles using simple signals and real seismic records. The second phase described in this paper referred to code validation for the analysis of real instrumented sites. This validation phase was performed on two sites (KSRH10 and Sendai) of the Japanese strong-motion networks KiK-net and PARI, respectively, with a pair of accelerometers at surface and depth. Extensive additional site characterizations were performed at both sites involving in-situ and laboratory measurements of the soil properties. At each site, sets of input motions were selected to represent different PGA and frequency content. It was found that the code-to-code variability given by the standard deviation of the computed surface response spectra is around 0.1 (in log10 scale) regardless of the site and input motions. This indicates a quite large influence of the numerical methods on site effect assessment and more generally on seismic hazard. Besides, it was observed that site-specific measurements are of primary importance and their interpretation needs careful attention when compared with reference results given in the literature. Finally, the lessons learned from this exercise are synthetized, resulting also in a few recommendations for future benchmarking studies, and the use of 1D NL, total stress site response analysis.
Author Comments:	Dear Editor,

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	This is the second paper we are publishing on this benchnmark exercice called PRENOLIN. The last paper was published last year in BSSA. As you will see, the co-author list is long and we understand it can come with difficulties for editorial subject but it reflects the number of researchers that participate to this project either as organization team or players of this benchmark. All of the co-authors provided corrections to the manuscript presented here. We were very pleased with the decision of BSSA to agree on that list for the previous paper and we hope that you will provide as well a positive answer for this article. Very kind regards
	Julie Régnier on the behalf of the PRENOLIN organization team (7 first co-authors)
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¹ **PRENOLIN: international benchmark on**

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Validation phase exercise

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57 Abstract

This article presents the main results of the validation phase of the PRENOLIN project. PRENOLIN is an international benchmark on 1-D non-linear site response analysis. This project involved 19 teams with 23 different codes tested. It was divided into two phases; with the first phase verifying the numerical solution of these codes on idealized soil profiles using simple signals and real seismic records. The second phase described in this paper referred to code validation for the analysis of real instrumented sites.

64 This validation phase was performed on two sites (KSRH10 and Sendai) of the Japanese 65 strong-motion networks KiK-net and PARI, respectively, with a pair of accelerometers at 66 surface and depth. Extensive additional site characterizations were performed at both 67 sites involving in-situ and laboratory measurements of the soil properties. At each site, 68 sets of input motions were selected to represent different PGA and frequency content. It 69 was found that the code-to-code variability given by the standard deviation of the 70 computed surface response spectra is around 0.1 (in log10 scale) regardless of the site 71 and input motions. This indicates a quite large influence of the numerical methods on 72 site effect assessment and more generally on seismic hazard. Besides, it was observed 73 that site-specific measurements are of primary importance for defining the input data in 74 site response analysis. The non-linear parameters obtain from the laboratory 75 measurements should be compared with curves coming from the literature. Finally, the 76 lessons learned from this exercise are synthetized, resulting also in a few 77 recommendations for future benchmarking studies, and the use of 1D NL, total stress 78 site response analysis.

79 Introduction

In seismology and earthquake engineering, site effects are widely recognized as an important factor for (mainly) amplifying the resulting surface ground motion. Those site effects are spatially variable depending on the local geomorphology and mechanical properties of the soil; they may vary from one event to the other as the site response to seismic loading is non-linear during strong ground motion (e.g. Amorosi et al., 2016; Bonilla et al., 2005; Gunturi et al., 1998; Zeghal et al., 1995; Ishibashi and Zhang, 1993; Yu et al., 1993; Elgamal et al., 1995; Vucetic and Dobry, 1991; Seed, 1969).

Site-specific analysis of the site response involving its non-linear soil behavior is still very challenging. In low seismicity areas, the lack of strong ground motion recordings limits empirical evaluations. To overcome this limitation, numerical simulations are of primary interest since they allow for simulating strong ground motions beyond the available recordings. In engineering practice, those analyses would, in most of the cases, be limited to methods involving the use of linear or equivalent linear methods in a 1-D site configuration.

94 As mentioned in the preceding companion paper (Régnier et al., 2016a), previous 95 comparative tests were performed to study 1-D non-linear site response. The first blind 96 tests performed in the late 80's/early 90's, on the Ashigara Valley (Japan) and Turkey 97 Flat (California) sites were very instructive in the linear domain, because of the lack of 98 strong motion recordings at that time. The 2004 Parkfield earthquake, which produced 99 0.3 g at Turkey Flat site, was the opportunity to launch a new benchmarking exercise for 100 1D non-linear codes. This also considered a few other sites with vertical array data and 101 large enough ground motion (La Cienega, California; the KGWH02 KiK-net site, Japan; 102 and a site in Lotung, Taiwan). The results reported in Stewart et al. (2008), Kwok et al. 103 (2008), and Stewart and Kwok (2008, 2009) emphasized the importance of the needed 104 in-situ measurements and of the actual way these codes are used. The origin of the 105 significant mismatch between records and predictions has been attributed to incorrect 106 velocity profiles (despite the redundant borehole measurements), to deviations from 1D 107 geometry (non-horizontal soil layers), and deficiencies in the constitutive models 108 (unsatisfactory match to the actual degradation curves). The 1D codes used for these 109 tests remain, however, extensively used in routine engineering practice for site response 110 estimates, and various developments have been carried out to implement new, or 111 updated, constitutive models, It is therefore needed to repeat such benchmarking 112 activities, notably in other parts of the world which may have different engineering 113 practice, and which were not involved in the previous comparison exercises (which are 114 always good also for young scientists and engineers who never had such a benchmarking 115 experience).

The objective of this two-phase (verification and validation) PRENOLIN exercise is to understand the variability associated to the implementation of the non-linear soil behavior in numerical simulations, and to assess the resulting uncertainties. It was decided to devote the calculations on simple cases focusing on the numerical implementation of the non-linear soil behavior (rheology and soil parameters) to be as close as possible to the standard engineering practice.

122 In this work we evaluate 1-D wave propagation of SH waves (only one component of 123 motion) having vertical incidence and assuming no pore pressure effects (total stress 124 analysis). These three assumptions mentioned above are not a sensitive issue when 125 dealing with the verification case (although realistic cases were selected to be close to the true physical processes). However, when dealing with validation and comparison with real data, they may have very strong impacts and for most sites and input motions they might be violated to various degrees.

129 We choose our sites for the validation to minimize the impact of those assumptions.

130 The verification phase helped to create a synergy between the participants and the 131 organizing teams. We defined a common vocabulary for the implementation of the 132 calculations (as the Non-Linear (NL) communities from different areas of the world may 133 have different practice and a different understanding of the same words). By analyzing 134 the whole set of NL simulations, we found that the code-to-code variability increases with the shear strain level. However, even in the worst case corresponding to large 135 136 loading and strain levels exceeding 1%, it remained lower than the single-station, random variability of GMPE σ values for PGA. Given the scatter in the nonlinear results, 137 138 we thus concluded that a realistic analysis should use more than one constitutive model 139 to incorporate at least partially the epistemic uncertainty in site response computations. 140 It was also found that, in order to reduce the epistemic uncertainty, which is partially 141 accounted for by the code-to-code variability, one may need to precisely describe specific 142 input parameters, especially the soil shear strength profile. In addition, the variability 143 between code is considerably reduced when they all used the same loading and 144 unloading process (Masing rules or non-Masing rules, referred to as damping control 145 models) (Régnier et al., 2016a)

To keep this exercise relatively simple, it was decided from the beginning not to deal
with the problem of pore pressure effects (e.g. Elgamal et al., 1996; Zeghal et al., 1995).
We acknowledge that this hypothesis might not be realistic in saturated soils subjected

149 to strong-motion cyclic loads. One of the chosen PARI sites (Onahama) may have 150 experienced cyclic mobility during the Tohoku-Oki, Mw 9 March 11th, 2011 earthquake 151 (Roten et al., 2013). However, dealing with nonlinear site response increases the number of soil parameters to consider, not only with respect to traditional linear estimates, but 152 153 also in relation to the complexity of the constitutive model used to describe the soil non-154 linear behavior. Since soil nonlinear behavior, even in the absence of pore pressure 155 effects, remains a challenge, the main effort of the validation exercise was focused on 156 total stress analysis only, disregarding the simulation of excess pore pressure generation. 157 To verify effects of water pressure build-up on the recordings acceleration time history, 158 some teams used site response analyses with excess pore water pressure generation to 159 compare with the total stress cases and are shown in the electronic supplement (Figure 160 S6 and S7). The recommendations of this report are only for total stress site response 161 analyses.

A detailed presentation of the organization of the project and the participant teams was done in the previous paper related to the verification phase (Régnier et al., 2016a). The table of the participants with description of the methods is thus provided here only in the electronic supplement (Table S1).

The objectives of this paper are to share the experiences on the validation exercise concerning (i) the analysis of data from laboratory and in-situ tests data to define the input soil parameters for the simulations, (ii) the processing and selection of the seismic input motions, (iii) the calculations of the non-linear site response and finally (iv) to quantify the epistemic uncertainty for 1-D nonlinear site response analysis on real sites, both in terms of code-to-code variability and code-to-data distance.

PRENOLIN : Validation results.

172 **Target sites**

173 Selection criteria

The selection of sites was performed on strong-motion databases involving vertical arrays so that the empirical soil column response (often called transfer function) can be calculated. Considering the hypothesis of the numerical methods and the objective to implement non-linear soil behavior, the sites were selected on the basis of the following requirements:

179 (1) Availability of both strong and weak motion recordings,

180 (2) Plausibility of 1-D geometrical soil configuration, i.e. satisfactory agreement between

181 numerical and empirical site responses in the linear / weak-motion range,

182 (3) The depth of downhole sensor must be less than 300 m.

(4) the possibility to perform complementary investigations in the immediate proximityof the site

To fulfill the first and second criteria, sites that recorded at least two earthquakes with PGAs higher than 50 cm/s² at the downhole sensor were selected. Only the KiK-net site configurations identified as fulfilling the 1D criteria proposed by Thompson et al. (2012) and Régnier et al. (2014) were considered. In addition, a visual comparison between the numerical and empirical site response curves was performed and a special attention was given to the matching of fundamental resonance frequency.

191 The fourth criterion also constrained the site selection, as the nearby urbanization may 192 prevent the drilling of new boreholes or the ability to perform new surface 193 measurements.

195 Dataset

196 Presentation

197 The sites selection was done on the KiK-net and PARI (Port and Airport Research 198 Institute) networks. KiK-net is composed of 688 stations, with high-quality surface and 199 downhole digital 3-component accelerometers. Among the KiK-net sites, 668 are 200 characterized with shear and compressive wave-velocity profiles. These velocity profiles 201 were obtained from downhole or PS logging measurements (depending on the site). 202 Most of the borehole sensors are located between 100 m and 200 m depth. Two thirds of 203 the sites have a Vs30<550 m/s. In the NEHRP and Eurocode 8 regulation, sites with 204 Vs30< 800 m/s are classified as sites prone to site effects, which confirms that the KiK-205 net database is very interesting for the analysis of these phenomena. The PARI sites are 206 much shallower than the KiK-net sites: the down-hole sensor depth is around 10 m and 207 the corresponding Vs profile is also available.

208 Data processing

More than 46,000 (six component) recordings were analyzed beforehand (Régnier et al., 2013) to derive the empirical site response (i.e., the transfer function from downhole to surface) at the 688 sites. In addition, on 668 sites with available Vs profiles, the numerical linear site response was also calculated on the basis of the velocity profile provided in the KiK-net data base (see Data and resource section). On the PARI network, only two sites were analyzed: Sendai (30 earthquake recordings), and Onahama (42 earthquake recordings).

The empirical site response is usually evaluated by using a spectral ratio between simultaneous recordings on sediments and on a nearby rock site (the so-called reference site). When this technique is applied, the main issue to overcome is the selection of a reliable reference site. The reference site should not amplify seismic waves, and should be close enough to the studied site so that the travel path from the seismic source remains equivalent for both sites.

222 Vertical arrays of accelerometers overcome the reference-site issue. Indeed, the down-223 hole station located at depth represents the reference station. Thus, for each KiK-net 224 (and PARI) sites and each earthquake recording, the Borehole Fourier Spectral Ratio 225 (BFSR) were calculated. BFSR is the ratio between the Fourier spectra of the horizontal 226 components recorded at the surface and the corresponding ones at depth. Yet we 227 acknowledge that the use of downhole records introduces an additional difficulty in 228 numerical modeling due to the contamination of the control motion by the downgoing 229 wavefield, which is sensitive both to the details of velocity and damping soil profile, and 230 to the complexity of the incoming wavefield (various types of body waves with multiple 231 incidence angles, together with possibility of surface waves, see Bonilla et al., 2002; 232 Régnier et al., 2014).

Before evaluating the *BFSR* a specific data processing procedure was applied that consisted in removing the mean, applying a tapering Hanning window on 2% of the signal, non-causal filtering between 0.1 and 40 Hz, FFT calculation and a Konno-Omachi smoothing (with b = 40) before performing the surface to down-hole spectral ratio. The linear site response was obtained by calculating the geometric average of all recordings with a PGA at the surface below 25 cm/s².

The empirical site response curves were compared to the equivalent numerical ones
(*BFSR_{num}*). The numerical site responses (*BFSR_{num}* and *OFSR_{num}*, indicating respectively

the transfer functions having the reference at the down-hole and at the rock outcrop, respectively) were computed using a Haskell-Thomson 1-D linear method (Haskell, 1953; Thomson, 1950). For the calculation of the transfer function, we also added a Konno-Ohmachi smoothing (b = 40) that was applied directly to the transfer function curve (since it is the direct result from the Haskell-Thomson method) and the same frequency sampling as the one dealing with the analysis of earthquake recordings was used.

248 The soil parameters that are required to compute the numerical transfer functions are 249 the shear wave velocity profile *Vs*, the density profile and the quality factor profile. In the 250 KiK-net (and PARI) database, only the Vs profiles are available. For the density, a 251 constant value along the profile equal to 2000 Kg/m³ was used. The quality factor (Q) 252 was directly derived from the Vs value following the rule of thumb scaling : Q = Vs/10253 (Olsen et al., 2003) used by many authors when no measurement is available from 254 laboratory data measurements (see section "From in-situ and lab data to input 255 *parameters*"). Others models for the low strain attenuation could have been used as well 256 such as proposed in (Darendeli, 2001; Menq, 2003).

257 Selected sites

5 KiK-net sites (FKSH14, IBRH13, IWTH04, KSRH10 and NIGH13) passed the selection
criteria, together with the 2 PARI sites that were initially chosen. Four KiK-net sites were
removed for various reasons: liquefaction susceptibility (FKSH14), rocky geology
(IBRH13), mountainous environment (IWTH04) and insufficient nonlinearity (NIGH13).
A detailed study on the effects of the topography and non-horizontal layering on
waveforms and transfer functions of KiK-net sites can be found in De Martin et al., (
2013).

The three remaining sites, i.e., KSRH10, Onahama and Sendai sites were therefore selected for further in-situ investigations for the purpose of the validation phase. Later on, for the Onahama site, it was found that the soil was susceptible to liquefaction (Roten et al., 2013) and clearly show 2D/3D site configuration. Consequently, the calculations performed on this site will not be presented in the present paper, although they were part of the validation phase.

The locations of KSRH10 (Hokkaido region) and Sendai (Tohoku region) are illustratedin Figure 1.

273

Figure 1

274 According to the initial available geotechnical data, KSRH10 is mainly composed of 275 clayey soil while Sendai site is composed of sandy soil. KSRH10 site is a deep 276 sedimentary site with 40m of low velocity soil layers; the down-hole sensor is located at 277 a depth of 255m (plot (a) of Figure 2). The site is located on the lower plateaus with 278 about 30m in elevation along the right bank (southern) side of the upper Anebetsu River. 279 The soil column consists of recent Younger Volcanic Ash deposits until 5m at depth, 280 followed by volcanic and Tuffaceous sand until 40m and underlie by an alternation of 281 sandstone and shale.

The Sendai site is a shallow site with 7 m of soft soil deposits and with the down-hole sensor located at 10.4m at depth. According to the Shogama 1:50 000 geological map, the site is in a large flat valley covered by beach ridge deposits (Holocene) consisting of gravel and sand. These surface deposits are underlain by the Pliocene, Geba Formation, forming the northern and eastern hills and consist of gravel stone, sandstone, tuff, tuffaceous siltstone, and lignite.

288 As illustrated in Figure 2 (plots b and e), empirical weak motion BFSR (surface PGA 289 lower than 25 cm/s²) and the linear numerical 1-D site response (dotted black line) 290 exhibit a satisfactory similarity, especially for Sendai site. The shallow Sendai site is 291 characterized by a high resonance frequency around 8.2 Hz, while the thicker KSRH10 292 site is characterized by a lower fundamental resonance frequency of 1.7 Hz; slight 293 differences can be seen however between observations and simulations as to the 294 frequencies of the first two peaks. The numerical simulations (dotted lines) provide site 295 response amplitude much higher at the first two frequency peaks and above 15Hz while 296 it is lower for the third and fourth peaks. This first comparison shows that the sites are 297 close to a 1-D site configuration, but exhibit a more complex behavior than those 298 predicted for a simple soil column subjected to pure vertically incident plane S waves.

299 When the sites were selected for the validation, one of the requirements was that the 300 sites exhibit some non-linear soil behavior for one or several recordings. In Figure 2 301 (plots c and f) the BFSR for weak motion is compared to the BFSR calculated from 302 motions with large PGA at the surface. For KSRH10, non-linear soil behavior is 303 significant for the input motions with PGA greater than 47 cm/s² at the down-hole 304 station (outside the average ± standard deviation area) and it is even greater for the 305 strongest events (KSRH100411290332 with PGA equal to 81 cm/s2 and 306 KSRH100309260450 with PGA equal to 110 cm/s² – table 1). For Sendai, non-linear soil 307 behavior is significant when the downhole PGA exceeds 46 cm/s² (outside the average \pm 308 standard deviation area) and it is even greater for the strongest event (F-2958 with PGA 309 equal to 252 cm/s2).

Figure 2

311 Selection of the input motions

312 A selection of 10 and 9 input motions for KSRH10 and Sendai, respectively was 313 performed among the available earthquake recordings. Their epicenters, magnitudes 314 and peak accelerations at the surface are illustrated in Figure 1 and provided in table 1. 315 The site response computations were performed on 5 input motions at KSRH10 (TS-0-K, 316 TS-1-K, TS-2-K, TS-4-K and TS-9-K) and 4 at Sendai (TS-1-S, TS-2-S, TS-5-S and TS-8-S), 317 numbered from strongest to weakest. Only the results of these input motions are shown 318 in this article. Nevertheless, we provide the information for all available input motions as 319 they may be used for future validation exercises.

320 The PGA and the frequency content of a recording are two relevant parameters of the 321 input motion for describing the expected degree of non-linear soil behavior (Assimaki 322 and Li, 2012; Régnier et al., 2016b). The input motions for KSRH10 and Sendai sites 323 were selected with 3 different PGA levels (at the downhole sensor), respectively. The 324 PGA was calculated on the acceleration time histories of the geometrical mean of the EW 325 and NS components, filtered between 0.1 and 40 Hz. In each group of PGA level, we 326 quantified the frequency content using the central frequency following Eq 1 (statistical 327 moments order 2 and 0, Sato et al., 1996) but the values were not significantly variable 328 from one event to another. We therefore also considered several magnitudes and 329 epicentral distances couples.

330
$$Fc = \sqrt{\frac{\int f^2 A^2(f) df}{\int A^2(f) df}}$$

331

Eq 1

where *Fc* is the central frequency, *f* the frequency, A(f) the amplitude of the Fourier Spectrum of the accelerogram. The resulting values exhibit a significant but rather erratic variability, without obvious link to magnitude, epicentral distance or depth. We therefore also considered several magnitudes and epicentral distances couples.

The selected events for the KSRH10 and Sendai sites are listed in the Table 1 along with their main characteristics (Mw, depth, epicentral distance, PGA at the down-hole and at the surface, and central frequency at the down-hole recording). The frequency sampling at KiK-net is between 100 and 200 Hz depending on the event and at Sendai it is 100 Hz. For KSRH10, four input motions with PGA at the down-hole sensor higher than 50 cm/s² were available and selected, whereas only 3 were available in Sendai site.

342

Table 1

343 Signal to noise ratio

We checked the quality of all events by assuring that their signal-to-noise ratios were high enough (S/N>3) over a broad frequency spectrum of 0.1 to 50 Hz, for all three components.

Orientation of the surface to down-hole sensors

We checked that both surface and down-hole sensors are oriented in a similar way. We rotated anti-clockwise the surface horizontal components with a 1° azimuth increment, starting from the original EW orientation, and we calculated the correlation coefficient 351 with the down-hole EW component. Both signals were filtered between 0.1 and 1 Hz. 352 The angle characterized by the maximum correlation would approximately correspond 353 to the angle between the two EW components of the surface and the down-hole sensors. 354 Correlations are maximum without rotation angles at Sendai and at KSRH10. It is close 355 to the calculations performed by D. Kosaka (PARI) who found a deviation of 7.2° 356 counterclockwise at Sendai also using long period motions correlations. Results are 357 approximately similar to the results of Maeda et al. (2005) who found a deviation of 2.2° 358 clockwise at KSRH10. These values suggest that both surface and down-hole EW-359 components are mostly oriented parallel one to each other, and even if slight deviations 360 of the order of 7° may occur, it would not impact significantly the soil response functions.

361 Verification of the verticality of the incident waves

362 Basic assumptions are made in most 1D simulation codes when propagating a wave 363 through a soil column. One of them is that the wavefield consists of vertically 364 propagating plane S waves; hence the input wave motion at the bottom of the soil 365 column is fully represented by the horizontal components and all vertically propagated 366 towards the surface. However, except for teleseismic (long-distance) events, the seismic 367 waves from local or regional events are very likely to have not only non-vertical 368 incidence (unless located directly underneath the sensor), but also multiple incidence 369 because of crustal scattering

Recording a non-vertical incident wavefield implies that the total seismic energy is distributed all over the three components. Therefore, if only one component of the 3D wavefield is used to represent one type of seismic phase (in our case, the shear wave), then it is highly likely that the wave energy, PGA and strains are underestimated at the down-hole sensor. However, depending on the thickness of the soil column and its

¹⁸

375 characteristics, the upward propagating body waves become increasingly vertical
376 towards the surface and therefore the surface sensor will give a more complete
377 representation of the entire shear wave energy on the horizontal components.
378 Therefore, two consequences can result from this observation:

(1) Although both the numerical and empirical input motions are the same, their
transfer functions would be different, since the energy on the horizontal components
at the surface will be greater in the empirical observation at KSRH10 than the
numerical simulations.

383 (2) The actual seismic loading at the base of the soil column is underestimated
384 compared to what the soil experiences, and therefore its possible non-linear
385 behavior can also be underestimated.

To determine the direction of propagation of an input motion, we used a polarization analysis based on the 3-component covariance matrix. In the electronic supplement (figure S1) we can see that at KSRH10, the polarization analysis indicates that the waves mostly propagate with a vertical incidence (low incidence angle). At Sendai, except for two recordings, the waves are not linearly polarized; therefore, the calculations of the direction of propagations are not relevant.

392 From in-situ and laboratory data to input parameters

393 Identifying the most relevant parameters to be used for simulating the non-linear wave 394 propagation process in a soil deposit was one of the main challenges tackled during the 395 verification exercise. For the elastic and visco-elastic properties, *Vs, Vp*, density and low 396 strain attenuation profiles were used. For the non-linear soil properties, the modulus

PRENOLIN : Validation results.

397 reduction and damping with shear strain curves, with the soil shear strength profile 398 were found to be the key parameters to significantly reduce the code to code site 399 response variability (Régnier et al., 2016). For more complicated non-linear models such 400 as Hujeux (Aubry et al., 1982), more more laboratory measurements are required to 401 define its model parameters, however some of this parameters could be defined using 402 well known soil mechanics correlations.

The challenge for the validation phase was to determine the value of those parameters for a real site. The specifications for the laboratory and in-situ measurements were defined in accordance with the prescriptions coming from the organization team with a few associated geotechnical experts and the participating teams, and bounded by the available budget and the measurement capacity of the local company performing the measurements, together with a few logistical issues linked with the exact location of the vertical array and the surrounding environment.

410 Site investigation

411 *Measurements performed*

To obtain the linear and non-linear soil parameters, in-situ measurements and multiplelaboratory measurements were conducted on disturbed and undisturbed soil samples.

The in-situ measurements were subcontracted to Oyo company and consisted in: (1) boring investigation to determine soil stratigraphy and to perform the soil sampling. The diameter of the borehole was 116 mm up to a depth where tripled-tube samplings were used (for sandy soil or relatively stiff clayey soil) then 86 mm; (2) Undisturbed soil samples (80 cm long) were collected using the thin-wall sampler for the soft clay soil and using the tripled-tube samplers for the sand and stiffer clayey soil; (3) Standard 420 Penetration Tests (SPT); (4) PS logging by suspension method for KSRH10 and down-421 hole method for Sendai, and (5) multiple MASW (Multichannel Analysis of surface 422 Waves) at the investigated sites to characterize the spatial variability of the underground structure at shallow depth, together with single point ambient vibrations recordings. 423 424 The laboratory soil tests were conducted on disturbed and undisturbed soil samples. 425 The tests on disturbed samples enable to determine physical characteristics such as 426 particle size distribution, liquid and Atterberg limits. The tests on undisturbed soil 427 samples aim at defining the density and to perform a wide range of laboratory tests such 428 as Undrained and drained tri-axial compressional test, oedometer tests by incremental 429 loading, cyclic undrained and drained tri-axial compression test (undrained for 430 investigating the liquefaction potential) and, for rock samples, unconfined 431 compressional tests. The methods used to perform the laboratory tests are defined by 432 Japanese normative specifications.

For each borehole, the number of undisturbed soil samples was defined according to the expected soil stratigraphy (on the basis of pre-existing KiK-net or PARI information), to ensure at least one sample in each homogeneous soil unit.

The number and location of soil samples are specified in Table 2 together with the down-hole sensor depth and the maximum depth of the complementary drillings. The details of the locations of the laboratory measurements are available in the electronic supplement (Figure S2 to S5). Figure 3 shows the locations of the boreholes having the accelerometers with respect to the boreholes performed for the laboratory measurements and the MASW lines.

PRENOLIN : Validation results.

442

Figure 3

443

Table 2

444 Uncertainties of soil parameter measurements

Because of the inherent variability of the soil and the systems errors in the measurements and sampling methods, a non-negligible level of uncertainty remains in the soil parameters measured through the laboratory tests. Repeatability of the soil samples and laboratory measurements are a possible way to ensure a reliable definition of the soil parameters. This approach was not applied in this exercise due to budget constraints. We therefore carefully analyzed the data and compared with literature data for similar types of soil.

To minimize the impact of soil spatial variability, the new boreholes were performed as close as possible to the instrumented ones. MASW lines performed between the two boreholes indicate a low spatial variability of the soil parameters for KSRH10 and Sendai (while they did indicate a significant variability at shallow depth for the third site in Onahama, which was also one of the reasons to drop this site for the validation exercise).

457 Interpretation of the laboratory and in-situ data

458 Elastic and visco-elastic properties

For the elastic properties, several methods were used to determine the soil parameters. We have preferred the methods that provide the direct in-situ evaluation of the soil properties, yet we did compare the results with alternative techniques characterized by indirect measurements. We used the PS logging to obtain the *Vs* profile and then we use the earthquake recordings to adjust it. As shown in Figure 4, the *Vs* profile was adjusted to improve the fit between the fundamental resonance frequency recorded andpredicted for the KSRH10 and Sendai sites.

For KSRH10, the initial *Vs* profile was based on the PS logging investigation down to 50 m depth; beyond this depth, we considered the values of the Vs coming from the KiK-net database, where PS-logging method was used as well. In this project, it was decided to adjust the linear transfer function from Thomson-Haskell predictions to the instrumental observations of surface-borehole spectral ratios, to ensure that the discrepancies between the prediction and the observations during the benchmark were associated to non-linear soil behavior, and not to other causes.

To adjust the numerical linear transfer function to the observation, we modify mostly the Vs profile coming from KiK-net for which no information was available on the measurement.

For Sendai, to improve the fit between the weak motion site response calculated with
linear site response analysis and computed from weak motions, a gradient type *Vs*profile (Eq 2) was chosen.

479
$$Vs = Vs_1 + (Vs_2 - Vs) \left[\frac{z - Z_1}{Z_2 - Z_1}\right]^{\alpha}$$

480

Eq 2

481 Where, $Vs_1 = 140$ m/s, $Vs_2 = 460$ m/s and $\alpha = 0.7$, Z_1 is the depth at which begin the 482 gradient (0 m) and Z_2 (7 m) the depth where it finishes.

483 The Poisson coefficient (ν) was computed using the PS logging and rounded. To ensure 484 consistency between the values of *Vs*, *Vp* and ν , the *Vp* parameter was obtained from *Vs* 485 and the rounded ν . The density was obtained from the undisturbed soil sample and the 23

PRENOLIN : Validation results.

486 low strain attenuation was deduced from the un-drained cyclic tri-axial test, and when 487 not available by using the rule-of-thumb (Qs = Vs/10) (Olsen et al., 2003).

488

Figure 4

489 Non-linear soil properties

490 The initial plan was to use only the measured non-linear parameters, i.e., the 491 degradation curves measured in the lab. It had however to be modified to ensure a 492 better fit to the strong motion data: the non-linear soil properties were actually updated 493 during the iterations of calculations, so that three sets of non-linear soil parameters 494 were used. The first one (called SC1) came simply from the use of non-linear 495 degradation parameters defined in the literature, and anchored to elastic soil properties. 496 Here the Darendeli formulation was adopted (Darendeli, 2001). The second (SC2) and 497 third (SC3) soil parameter sets are directly based on interpretations of the laboratory 498 data. One objective of the benchmark was to focus on routine practice with relatively 499 simple models. Furthermore, the participants were also free to build/use their own soil 500 model based on the raw experimental laboratory test. Yet, additional non-linear soil 501 parameters could have been tested as well, such as models that could handle both low 502 and high strain as detailed in Groholski et al. (2016) and Yee et al. (2013).

503 Darendeli formulation (Darendeli, 2001) was used to define the G/G_{max} and damping 504 ratio curves as a function of shear strain for SC1 soil column. To compute such values, 505 the knowledge of the confining effective stress (σ '), the over consolidation ratio (*OCR*), 506 the plasticity index (*PI*) and the damping ratio at low strains (*D_{min}*) was required.

507 The SC2 non-linear curves were constructed from the cyclic tri-axial compression test 508 results. We normalized the Young's modulus decay curves (from the lab 5th cycle of 509 loading) by the low strain Young's modulus (E_0). E_0 is the value of the hyperbolic model (Hardin and Drnevich, 1972) that mimics the lab results at 0.0001%. We assimilate this 510 511 E/E_0 decay curve to the shear modulus G/G_{max} decay curve, with G_{max} associated to the 512 in-situ velocity measurements, i.e., $G_{max} = \rho Vs^2$. The shear strain used, γ , was considered 513 equal to 3/2 of the axial strain directly measured during the triaxial test. Indeed, the 514 shear strain is the difference between the axial and radial strain $\gamma = \varepsilon_a - \varepsilon_r$. During the 515 cyclic triaxial test under undrained conditions volumetric changes are zero: therefore, 516 the volumetric strain is null $\varepsilon_v = \varepsilon_a + 2\varepsilon_r = 0$. From the previous two equations we can 517 deduce that: $\gamma = 3/2\varepsilon_a$ (Vucetic and Dobry, 1988).

518 The elastic shear modulus values from the laboratory tests (*G_{max}^{lab}*) are generally under-519 estimated compared to the in-situ measurements (*G_{max}^{insitu}*), especially for cyclic tri-axial 520 tests (indeed cyclic tri-axial tests are not reliable at low strain, below 10⁻⁴ %). Tatsuoka 521 et al. (1995) showed that this could be due to sample disturbance where stronger 522 differences are observed depending on the type of shear strain measurement. When 523 local measurements of shear strain are performed using internal gauges (inside the soil 524 sample) compared to external measurements (usual measurement), the discrepancies 525 are much smaller.

When normalizing the shear modulus curve to obtain the G/G_{max}^{lab} curve, the G_{max}^{lab} should be corrected. The coefficient of correction to be applied is not well defined but lies between 1.2 and 4 (Lopez-Caballero personal communication, 2015). A correction procedure was set up in this study to partially correct this value (Noguera, 2016). The procedure accounts for the measurements errors when using external measurements instead of local ones but does not consider error due to soil sample disturbance. Considering that this procedure was defined during the project (after the calculations on 533 Sendai site) it was only applied to the laboratory data for KSRH10 site.

The above-mentioned procedure consists in three steps: (1) to find the maximum shear modulus (G_{max}^{lab}) by fitting the logarithmic equation (Eq 3) proposed by (Nakagawa and Soga, 1995) as measured for intermediate strain values; (2) to compare the G_{max}^{lab} values found with those from other tests or in the literature. Note that this value should not be more than twice the G_{max}^{lab} as it only accounts for the external-to-local strain measurement error; (3) to normalize the fitted decay curve and multiply it by G_{max}^{insitu} value.

541
$$G/G_{\max}(\gamma) = \frac{1}{1 + \alpha |\gamma|^{\beta}}$$

542

543 The SC3 model was built using the hyperbolic model (Eq 4 and Eq 5) constrained by the G_{max} and the shear strength (τ_{max}). The latter was estimated from the depth and the 544 545 cohesion and friction properties according to Eq 6. This formula was derived from the 546 Mohr circle for simple shear test (Hardin and Drnevich, 1972). The shear strength used 547 here is the maximum shear stress not the maximum lateral shear stress, with the 548 effective cohesion (c') and the friction angle (φ') coming from the monotonic 549 compressional test and the coefficient of soil at rest (Ko) coming from Jaky's formula (1-550 *sin(φ'), Jaky, 1944).*

Eq 3

551
$$G/G_{\max}(\gamma) = \frac{1}{1 + \gamma/\gamma_{ref}}$$

Eq 4

PRENOLIN : Validation results.

552
$$\gamma_{ref} = \frac{\tau_{max}}{G_{max}}$$

Eq 5

Eq 6

553
$$\tau_{max} = \frac{(1+K_0)\sigma'_0 \sin\varphi'}{2} + c'\cos\varphi'$$

554

The comparison of the non-linear curves is illustrated in Figure 5 and Figure 6 for KSRH10 and Sendai, respectively. For both sites, the SC1 non-linear curves have generally more shear modulus reduction for the same shear strain than those coming from the laboratory data, even after the correction procedure was applied.

559 The mechanical properties of the KSRH10 and Sendai sites are synthesized in *Table 3*

560

Table 4

and Table 4, respectively. For KSRH10, three sets of non-linear parameters were tested,
whereas only the first two were tested on Sendai site. From in-situ surveys, the water
table is located respectively at 2.4 m and 1.45 m below the ground surface for KSRH10
and Sendai site.

565

Table 3

Table 4

567

Figure 5

568

Figure 6

569 Validation results

570 Calculations performed

571 Two iterations of calculations were performed at KSRH10 and Sendai sites. The first 572 iteration was completely blind (i.e. only input motions were given to the participants) 573 while during the second one the surface motions were also provided. Three soil column 574 models were tested for KSRH10 and two for Sendai. Both EW and NS components were 575 used. In addition, the rotated horizontal motion corresponding to the maximum peak 576 acceleration was considered; simulations using this rotated horizontal component were 577 performed. However, the results were not significantly different from those obtained 578 using the EW or NS components. Thus, these computations are not shown here.

579 All the participating teams were asked to provide the acceleration time histories and the 580 stress-strain curves at different depths in the soil columns. For KSRH10, the acceleration 581 time histories were computed at various ground levels from the surface (GL): 0, -6, -11, 582 -15, -20, -24, -28, -25, -39, -44, -84 and -255 m depth, corresponding to the main soil 583 layers interfaces. The stress-strain curves were computed at GL-3, -8.5, -13, -17.5, -22, -584 26, -31.5, -37, -41.5, -64 and -169.5 m, which correspond to the middle of the soil layers. 585 For Sendai, the acceleration time history was provided from GL-0 to -8 m every 1 m, 586 while the stress-strain curves from GL-0.5 to -7.5 m also every 1 m.

587 Analysis of results

588 This article focuses on the analysis of the whole dataset returned from each team to 589 estimate the level of uncertainties associated to NL modelling.

590 Several sources of uncertainties are involved in 1-D non-linear site response analyses. 591 On one hand, there are epistemic uncertainties coming from the main assumption of the 592 method (1-D, vertical propagation of SH waves), soil parameters measurements (which 593 should however impact all the predictions in the same way), the numerical model and 594 the users. On the other hand, there are random uncertainties coming from the input 595 motions, which are influenced by both the seismic sources and soil heterogeneities. We 596 did use several input motions; however, the number of them is not sufficient to take into 597 consideration all the random uncertainty ranges. The previous verification phase 598 (Régnier et al., 2016) provided an estimation of the code-to-code variability linked to the 599 numerical method such as numerical integration schemes, implementations of damping, 600 constitutive models and users practice. Conversely, the validation phase involves 601 comparison with observations and therefore calculations of residuals. We assume that 602 the residual can be described as a random variable with normal distribution center 603 around a non-necessarily zero mean (models may over or under-predict), with 604 associated standard deviation.

We calculated and compared the misfit between the observations and the computations with the code-to-code variability of the surface response spectra averaged over a period band-width close to the resonance period of the site, namely [0.7 f₀, 1.3 f₀]. The misfit reflects the total uncertainty (epistemic and random) of the results; whereas the codeto-code variability illustrates the part of the epistemic uncertainty associated with the

choice of a numerical model. Then, this code-to-code variability is compared to theprevious results obtained in the verification exercise done on simplified soil profiles.

An additional analysis was performed to consider the soil column choice in the whole
uncertainty assessment. The misfit was calculated for each input motion and all soil
columns (SC1, SC2 and SC3) together.

Finally, the results at each site were analyzed, through the distribution of the computed transfer functions and response spectra, using the 25th and 75th percentiles of all computations, and comparing them to the observations. Concomitantly, we also computed and analyzed the distribution of residuals on response spectra to quantify the discrepancy and identify when the observations were under and over-estimated. This operation was carried out at each oscillator period.

621 Code-to-code variability versus misfit

622 A first analysis of the code-to-code variability (called hereafter σ_{c2c}) relative to the 623 variability of the residuals between the recording and the simulations (called hereafter 624 *Misfit*) is provided.

To quantify the *Misfit*, we calculate the root mean square distance between each prediction with the observation of the response spectra as proposed in equation Eq 7 and averaging it (geometric mean) over the periods of interest of the site.

628
$$Misfit = \left(\prod_{i=1}^{n} RMSD_{obs-num}(T_i)\right)^{\frac{1}{n}}$$

629

where

630
$$RMSD_{obs-num}(T_i) = \sqrt{\frac{1}{Nc - 1} \sum_{j=1}^{Nc} \left[LSA_{obs}(T_i) - LSA_{num,j}(T_i) \right]^2}$$

Eq 7

631 Where n is the number period sample between $T_1=1/(1.3f_0)$ and $T_2=1/(0.7f_0)$ of the discrete

632 response spectra SA; $L\overline{SA_{obs}}$ is the logarithmic (to base 10) transformation of the observed

633 SA; LSA_{num,j} is the logarithmic transformation of jth surface predicted SA.

634 The code-to-code variability (σ_{c2c}) is the standard deviation of the predictions as defined 635 in equation Eq 8, averaged over the same period range as before.

636

637
$$\sigma_{c2c} = \left(\prod_{i=1}^{n} RMSD_{num}(T_i)\right)^{\frac{1}{n}}$$

638

639
$$RMSD_{num}(T_i) = \sqrt{\frac{1}{Nc - 1} \sum_{j=1}^{Nc} \left[L\overline{SA_{num}}(T_i) - LSA_{num,j}(T_i) \right]^2}$$

640

Eq 8

641 Where n is the number period sample between $T_1=1/(1.3f_0)$ and $T_2=1/(0.7f_0)$ of the discrete 642 response spectra SA; $L\overline{SA_{num}}$ is the logarithmic (to base 10) transformation of the mean 643 predicted SA; $LSA_{num,j}$ is the logarithmic transformation of j^{th} surface predicted SA.

The considered period range spans an interval of $\pm 30\%$ around the fundamental resonance frequency of the sites (*f*₀) (8.2 and 1.7 Hz for Sendai and KSRH10, respectively). For Sendai it corresponds to the frequency range from 5.47 to 10.66 Hz,
equivalent to periods between [0.09-0.18] s. For KSRH10, the adopted frequency interval
is [1.19-2.21] Hz, corresponding to periods between [0.45-0.84] s. The periods are logscale sampled.

Regarding the soil column, the SC1 soil model provided closer results to the observation
compared to the SC2 at Sendai. On the opposite, for KSRH10, SC2 and SC3 models led to
lower misfit values.

Figure 7 compares the *Misfit* on the East-West component for Sendai and North-South component for KSRH10 (filled markers), with the code-to-code distance σ_{c2c} (empty markers). The simulated component (EW or NS) with less discrepancy with the observations was chosen.

657 For Sendai, we compared the results for four input motions (1, 2, 5 and 8, for which all 658 calculations were performed) and for the two soil models SC1 and SC2. For KSRH10, it is 659 illustrated for 5 input motions (0,1,2, 4 and 9) and for the 3 soil models (SC1, SC2 and 660 SC3). As expected, the misfit is systematically higher than the code-to-code variability 661 regardless the input motion or site considered: actually they could be equal only if the predictions are unbiased in average, i.e., if $L\overline{SA_{num}}(T_i) = LSA_{obs}(T_i)$ for every period 662 663 T_i. – which actually never happens... It is also worth observing that the code-to-code 664 variability is quite the same regardless the input motion or the site or the soil columns 665 considered and has a minimum value of 0.06 and a maximum value of 0.15. This 666 suggests that for 1-D seismic response analyses, the epistemic uncertainty related to the 667 choice of the numerical method and soil constitutive model should be considered 668 between 0.06 and 0.15 (in log10 scale). Those values, when compared to well-known
669 uncertainty estimation such as in GMPEs (Ground Motion Prediction Equations), lying 670 between 0.15 to 0.35 (Strasser et al., 2009), are significant and should be taken into 671 account for seismic hazard assessment. One must also keep in mind that such values 672 correspond to a relatively narrow frequency range, and may not be representative of the 673 variabilities in other frequency ranges (as may be seen on Figure 8).

674 The misfits are generally lower for the weakest input motions regardless the site 675 considered, although slightly more pronounced for the KSHR10 site. This is expected 676 because the predictions are closer to one another and also closer to the observations 677 when the soil response in mainly in the linear range. An exception is observed at Sendai 678 for TS-5-S, for which the misfit is larger for a moderate PGA. The misfits between 679 observations and simulations are found significantly lower at Sendai than at KSRH10. 680 They lie between 0.1 and 0.25 at Sendai, while they are in the interval between 0.08 and 681 0.35 for KSRH10.

Regarding the soil column, the SC1 soil model provided closer results to the observation
compared to the SC2 at Sendai. On the opposite, for KSRH10, SC2 and SC3 models led to
lower misfit values.

685

Figure 7

686 **Comparison between verification and validation epistemic variability**

In this section, the variability of the predictions performed during (1) the verification phase on canonical cases (3 profiles P1, P2 and P3 with fundamental resonance frequencies of 3.75, 1.16, 1.58 Hz respectively) and (2) the validation phase on real sites are compared in terms of standard deviation (log10 unit) of the PGA and spectral accelerations at 0.1, 0.3 and 1s at the surface. 692 We provide the results in Figure 8. For the validation phase, the standard deviation is 693 calculated for KSRH10 for the input motions: TS-9-K, TS-4-K, TS2-K, TS-1-K and TS-0-K. 694 For Sendai it is calculated for the input motions TS-8-S, TS-5-S, TS-2-S and TS-1-S. As far 695 as the verification phase is concerned, we considered only the results for the first profile 696 P1, the rigid substratum case and the non-linear computations. The numerical results 697 depend on the input motion level and on the frequency content (HF stand for High 698 Frequency input motion and LF for Low frequency, see Régnier et al., 2016): the 699 variability increases with the strain level developed in the soil column. It is therefore 700 higher at high PGA and for the low frequency content input motion. The Low Frequency 701 (LF) waveform generates higher strains compared to the High Frequency (HF) 702 waveform, for the same PGA level (black empty triangles). This is because the frequency 703 content of the input motion is close to the resonance frequency of the canonical site. 704 Therefore, strong resonance effects are expected. In the validation phase, the variability 705 is generally larger for the stronger input motions except at KSRH10 site for periods 706 above 0.3s.

707

Figure 8.

708 **Propagation of the epistemic uncertainty**

We built a logic tree similarly to what is done in Probabilistic Seismic Hazard Assessment, to propagate the uncertainty of the numerical simulation and interpretation of the soil data from in-situ and laboratory measurements to the site response and surface response spectra assessment. For each site (Sendai or KSRH10) and each input motion this logic tree is composed of two nodes (as shown in Figure 9). The first node is the soil column (SC1 and SC2 for Sendai, SC1, SC2 and SC3 for KSRH10) and the second node is relative to the team and code couple (from EA-0 - team A with his first code - to EZ-1 - team Z with his second code). All branches of the tree have the same weight. The
uncertainty is quantified by the standard deviation of residuals of the logarithm (log10
unit) of the results (here PGA and response spectra at the surface at 3 periods 0.1, 0.3
and 1s) as defined in Eq 7.

Table 5 synthesizes the standard deviation of the results obtained in the present
PENOLIN exercise. It might be noted that the root mean square of residuals (RMSD) are
in most cases lower for Sendai compared to KSRH10, except for longer periods (1s).
(period close to the KSRH10 fundamental resonance frequency where the fit is good).
The fit is generally better for weak input motions, except at Sendai for the response
spectra above 0.1s.

Figure 9

726

Table 5

727

728 Comparison of transfer function and response spectra between soil columns

Let us analyze more precisely for each period (frequency) the differences between predictions and observations. The 25% and 75% percentiles of the surface response spectra and the borehole transfer functions are compared to the observations, for a strong and a weak input motion. We selected TS-1-S and TS-8-S for Sendai and TS-1-K and TS-9-K for KSRH10.

Besides, to quantify the discrepancy between observations and predictions the averageresiduals (R) per period was calculated according to as shown in Eq 9.

736
$$R(T) = \frac{\sum_{j=1}^{Nc} \left[LSA_{obs}(T) - LSA_{num,j}(T) \right]}{Nc}$$

Eq 9

Where Nc is the number of computations; LSA_{obs} is the logarithmic (base 10) of the
observed SA; LSAnum,j is the logarithmic of the jth surface predicted SA.

739 Sendai

740 In Sendai, the results of the computations are closer to the observations when using soil 741 model 1 (SC1), which was defined using literature parameters. In Figure 10, the transfer 742 functions are compared, the fundamental resonance frequency (*fo*) of the observations 743 for a weak input motion (TS-8-S) is equal to 8.5 Hz, which is well reproduced by the 744 numerical computations using either SC1 or SC2 soil parameters. For the strongest input 745 motion (TS-1-S) fo is equal to 7.3Hz in the observations. In the computations, fo is similar 746 when using SC1 soil parameters while for SC2, fo is slightly above (7.8 Hz), indicating a 747 lower level of non-linearity when using SC2 soil parameters compared to SC1. Similarly, 748 as illustrated in Figure 11, the surface response spectrum is well reproduced by the 749 computations using the two soil columns for weak motion (TS-8-S) while for the strong 750 motion (TS-1-S) the prediction using SC1 is closer to the observation as compared to the 751 SC2 soil column. Those observations are also highlighted in Figure 12, the residuals are 752 close to 0 for the weakest input motion (TS-8-S) for which the two soil columns provide 753 similar estimations. For the strongest input motion we observe an over-estimation 754 below 0.2 s. For TS-5, the under-estimation is observed for both soil columns while for 755 TS-1-S and TS-2-S it is mainly observed for SC2. It shows that the discrepancy between 756 the observations and the predictions for TS-5-S does not depend on the soil column 757 characteristics and may be associated with the input motion specificity.

758 These first results were somehow disappointing, since site-specific measurements failed 759 in predicting the observations where generic parameters succeeded. We investigate the 760 source of this discrepancy that could come either from a measurement error or a mis-761 interpretation of the laboratory tests. For Sendai site, there is a large variability between the measured laboratory shear modulus and the in-situ measurement. The shear 762 763 modulus from the laboratory measurement is equal to 25 MPa at 3.3 m depth, compared 764 to 100 MPa from the in situ measurement of Vs (230 m/s) and density values (1890 765 kg/m^3). This observation suggest that the correction of the laboratory data should have 766 been apply to Sendai site as well.

The procedure to correct the G/G_{max} curves, as indicated previously, has not been applied to the laboratory at Sendai data before the calculations, but it was performed a posteriori.

The comparison of the G/G_{max} curves of SC2 model with G/G_{max} curves from laboratory data did not indicate modifications that could explain the large misfit for SC2 model. We recall that this procedure is supposed to correct only for measurement errors between external and local shear strain devices. Considering the large uncertainty that can lie in the value of the G_{max}^{lab} it is highly recommended that low strain measurements, such as resonant column or bender element should be used in addition to cyclic tri-axial test to define these parameters.

777	Figure 10
778	Figure 11
779	Figure 12

780 KSRH10

KSRH10 is a deep sedimentary site with down-hole station at GL-255 m. We can observe
that the site response (Fourier transfer function) is variable depending on the
component of motion and hard to reproduce above the fundamental resonance peak.

784 In Figure 13, the recorded transfer function is variable from one component of motion to 785 another especially for TS-1-K, where the amplitude of the fundamental resonance peak 786 is equal to 25 for the NS component and only 16 for the EW component. The frequency 787 peaks between 2 to 5 Hz are variable from component to component and event to event 788 and could not be predicted by the 1D assumptions made in this study. For TS-1-K we 789 observe a second and third peaks at 2.3 and 3.3 Hz for the EW component and at 2.7 and 790 3.2 Hz for the NS component while only one peak is predicted by the numerical 791 simulation with a very high amplitude at 2.7 Hz (high amplitude in the surface Fourier 792 spectrum). For TS-9-K, we observe 3 peaks at 2.3, 3.2 and 4 Hz and only one is predicted 793 by numerical simulations at 2.6 Hz. The fourth peak, close to 7 Hz, is more stable from 794 one component to another, but the frequency is slightly over-estimated and the 795 corresponding amplitude is under-estimated when using the SC1 column.

At high frequencies (above 12 Hz), a de-amplification is observed in the empirical borehole transfer function that is not reproduced by the simulations. One possible explanation is the existence of a noticeable soil-structure interaction at the accelerometer sites: this was proposed by DPRI based on their own experience at several

sites and many events. This may also add to some side effects of the low-pass filtering of
all KiK-net recordings below 25 Hz. Other possible contributions to this high-frequency
reduction may be larger damping (especially for low strains) or scattering from shallow,
small size heterogeneities.

For the fundamental resonance peak, we found that SC2 and SC3 soil columns coming from in-situ measurements provided closer results to the observations than the SC1 defined using literature G/G_{max} curves (Darendeli, 2001). As shown in Figure 13, for TS-1-K input motion, the amplitude of the observed resonance frequency peak at 1.4 Hz is around 25 for the NS component, while SC1 column predicted amplitudes (in the 25-75 percentiles envelope) between 12 and 14 only.

810 As seen in Figure 14, the amplitude of the surface response spectra for the NS 811 component is relatively well reproduced for the weak input motion TS-9-K, but the 812 periods of maximal amplitudes are shifted, creating an over-estimation at 0.13 s and 813 0.34 s and an under-estimation at 0.18 and 0.25 s. These discrepancies are related to the 814 differences between the observed and computed frequency peaks in the transfer 815 function. For TS-1-K, the amplitude is well reproduced except for SC1 soil column for 816 which the amplitude is significantly under-estimated between 0.1 and 0.34 s. The 817 surface response spectra from the computations using SC2 and SC3 soil columns are 818 closer to the observations.

In Figure 15 we observe the residuals of the response spectra; the recordings are under and over-estimated especially for periods close to 0.35 s for TS-1-K to TS-9-K input motions. The under-estimation amplitude increases with the input motion intensity and is more important for the soil column SC1.

823 At KSRH10, the site-specific measurements on non-linear properties provide more 824 satisfactory results than the generic curves. The type of soil of KSRH10 was analyzed in detail by one of the participant (Lanzo, personal communication). The non-linear 825 826 parameters (normalized shear modulus reduction and damping curves) defined in SC2 827 and SC3 are less non-linear compare to classical literature curves even for similar type of 828 soil (i.e SC1). This observation is consistent with the fact that KSRH10 is composed of 829 volcanic sand, as indicated by site geology. Several authors have shown that volcanic 830 sand exhibits a lower non-linear behavior as compared to classical sand. For example, 831 laboratory experimental tests conducted by Senetakis et al., (2013) show that pumice 832 sands exhibit a slower normalized stiffness decay and a lower dissipative behaviour than 833 classic gravel curves (Rollins et al., 1998). Similar experimental results have been 834 obtained in Italy on volcanic materials such as Colle Palatino tuff in Rome (Pagliaroli et 835 al., 2014), Naples Pozzolana (Papa et al., 1988) and Orvieto (Central Italy) pyroclastic 836 materials (Verrucci et al., 2015). Thus, Darendeli's curves built on "classical" sand data 837 could not necessarily reproduce the non-linear soil behavior at this site. This exercise 838 suggests that one should be careful when using generic soil curves, such as Darendeli 839 (2001) or others: the soil nature is important to evaluate their relevancy in site response 840 analyses.

841	Figure 13
842	Figure 14
843	Figure 15

844 **Discussion and conclusions**

845 Applicability of the calculations

The computations performed in this benchmark are limited by three main assumptions: (1) 1-D wave propagation in horizontally layered media (2) SH waves (only one component of motion) with vertical incidence and (3) total stress analysis.

849 1-D structure

850 We succeed to reproduce at both sites the fundamental resonance frequency (for weak 851 and strong motions) but the higher modes remain difficult to reproduce. The sites were 852 chosen (over 688 sites in KiK-net) to fulfill specific criteria warranting limited deviations 853 from a 1-D site configuration. We observe at KSRH10 site that 1-D numerical simulation 854 could not reproduce the observed site response over the whole frequency range, even 855 for weak motions: this is likely to indicate that the site has more complex geometry. The 856 1-D structure assumption is a very strong one which may not be realistic. However, 857 moving forward for more complex geometries requires more detailed site 858 characterization over a broader area, adequate interpretation of the data and application 859 of 2-D and 3-D numerical simulations (Amorosi et al., 2016; Dupros et al., 2010; Taborda 860 et al., 2010). Benchmarking non-linear numerical codes for 2D or 3D geometries is a real 861 challenge, which should start with a carefully designed verification exercise.

862 Vertically incident SH waves

863 Vertically incident SH waves loading implies two distinct assumptions: a) vertically864 incident plane waves, and b) a uni-directional motion in the whole soil column.

The former has been partially tested through a polarization analysis and has been found only partially fulfilled. In any case, there is not enough information from the two-sensor recordings to constrain the complexity of the incident wavefield, which is an unknown mixture of body waves with varying incidence angles and backazimuths, and surface waves with varying back-azimuths.

870 Concerning the second assumption, one participant tested a code with an 871 implementation of a 3 Components (3C) non-linear constitutive relation. Using a 3C non-872 linear constitutive model has been shown to be relevant for strong ground motion 873 prediction with 1D wave propagation models for a large event such as Tohoku 2011 874 (Santisi d'Avila and Semblat, 2014). The 3D loading path due to the 3C-polarization leads 875 to multiaxial stress interaction that reduces soil strength and increases nonlinear effects.

Therefore, for Sendai site, results of 1D analysis performed using 3 component codes (1D, 3-components) were compared to those obtained by 1D analysis with only one component of motion (1D, 1-component, vertical incidence) see figure S6 in the electronic supplement. Simulations were carried out with reference to the SC2 soil column, which is less non-linear than the SC1, and considering the strongest input motion (TS-1-S).

As illustrated in the electronic supplement figure S6 no significant differences between the two results were observed and cannot explain the discrepancy between the predictions and the observation. Additional research is needed to further explore theimpact of 3C motions versus 1C assumption.

886 Total stress versus effective stress analysis

887 Even though the exercise was limited to a total stress analysis, and designed accordingly, 888 we thought useful to benefit from the willingness of some volunteer participants to 889 investigate whether the type of analysis (effective or total stress) or the input soil model 890 used could improve the fit. Two teams (W-0 and H-0) performed those calculations at 891 Sendai site. In the electronic supplement, figure S7 presents the borehole transfer 892 function computed for soil columns SC1 and SC2 sorted according to the type of analysis 893 (Total stress analysis, Equivalent linear or effective stress analysis), as compared to the 894 observations for two different input motions (TS-1-S and TS-8-S). We observe that the 895 weak motion effective stress analysis provides results close to total stress analysis, as 896 expected. This is also the case for the strongest input motion when dealing with SC2 soil 897 parameters. However, one team (W-0) succeeds to reproduce the observations even with 898 SC2 soil parameters, when using an effective stress-analysis. That team indicated that 899 the non-linear input data used in this analysis were calibrated directly from the 900 laboratory tests and adjusted to the in-situ measurements of elastic properties. The 901 *G/G_{max}* curves obtained were closer to the curves used in SC1 rather than SC2. Therefore, 902 no evidence of efficiency of effective stress analysis compared to total stress analysis is 903 available for Sendai site. Additional research is required to further evaluate the 904 conditions under which an effective stress analysis is needed.

905 Equivalent linear method

906 In addition to the time domain nonlinear site response analyses limited by these three907 assumptions, the equivalent linear method (EQL) was also tested. This approach

908 involves a linear computation, coupled to an iterative process that adjusts at each 909 iteration the value of the shear modulus and damping, according to the maximum shear strain calculated at the middle of each soil layer. This method is largely used in 910 911 earthquake engineering practice since the pioneering work of Schnabel et al., 1972. 912 Three teams used an equivalent linear method (J-1 and Z-0). Team J-1 performed all the 913 calculations, whereas Z-0 provided the results of the equivalent linear method only for 914 the weakest input motions. In the electronic supplement, the figure S7 depicts the 915 results obtained from the equivalent linear method. The EQL results are close to the total 916 stress analysis for the weakest input motion and for the strongest input motion when 917 SC2 soil parameters are considered. However, for the SC1 soil column it should be noted 918 that the shift of the fundamental resonance frequency towards lower values is much 919 higher for the EQL methods and that the high frequencies are largely de-amplified. The 920 strain levels for TS-1-S using SC1 reach 0.3 % and up to 0.7 % for some computations, 921 while for SC2 maximum shear strain values are below 0.2%. For SC1 such high shear 922 strain level implies a decay of shear modulus to 0.1 times the maximum shear modulus. 923 Such results confirm that the EQL approach should not be used beyond strain levels 924 around 0.2 %, consistently with the results presented in Kim et al., (2016) after a 925 comprehensive set of numerical simulations for many different sites, and those also 926 obtained earlier by Ishihara (1996) and Yoshida and Iai (1998).

927 Main outcomes on NL prediction uncertainties

The present benchmarking exercise allowed to provide some quantitative estimates on the epistemic uncertainty associated to 1D non-linear modeling, which should be considered as lower bound estimates as it is rather rare for practical engineering studies to have as many information as in the present case. Figures 7, 8 11 and 15 indicate that: 932 • The code-to-code variability is generally in the range 0.05-0.25 (log10 scale), with
933 a slight trend to decrease with increasing period.

The smaller code-to-code variabilities are found to correspond to the "SC1 case",
i.e. here to the Darendeli model, while higher variabilities are found for NL
models based on in-situ sampling and dedicated laboratory characterization. We
interpret this finding as related to the higher non-linearity level implied by the
Darendeli model, resulting in generally weaker motion. The decrease of the
uncertainties due to an increase of non-linear soil behavior has been notify in
previous studies (Bazzurro and Cornell, 2004; Stewart et al., 2017).

The misfit (i.e. root mean square average distance to the actual motion) is larger
than the code-to-code variability because of model errors (soil parameters,
improper 1D assumption, total stress, etc.), and misfit increases with increasing
loading level (Figure 7). It may reach values from 0.25 to 0.35 (log10 scale)
around the site fundamental frequency where the variability is the highest

This misfit exhibits a strong frequency dependence, with the lowest values below
 the fundamental frequency, and largest around f0 and above. Models may over predict the site response at some frequency, and under-predict it at other
 frequencies

The prediction of non-linear site response seems easier for shallow soil deposits
 than for deeper deposits: the first obvious reason is that the code-to-code
 variability is mainly visible beyond the site fundamental frequency, which is
 higher for shallow deposits. In addition, deep deposits not only imply a larger
 number of sample measurements corresponding to varying depths, but the

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955 wavefield is likely to be more complex, as well as the perturbations due to non 1D956 layering.

The widely used, average models such as the Darendeli's ones (SC1), are found to
perform rather well for one site (better than the models based on in-situ
sampling and laboratory measurements (SC2 and SC3), and less well for the other
site. It is impossible to generalize from only two sites, but it is worth mentioning
that the first case corresponds to very shallow (depth smaller than 10 m), mainly
sandy materials, while the second corresponds to deeper, more clayey material,
exhibiting less non-linearity than predicted by Darendeli's model

964 Finally, one should keep in mind that the results were obtained for rigid base conditions 965 only since they correspond to an input motion recorded at a down-hole sensor. In most 966 practical engineering studies, the "reference motion" corresponds to outcropping rock 967 conditions. The epistemic uncertainty and misfit are then likely to increase especially 968 when the base of the soil column corresponds to much harder bedrock than the 969 "standard" conditions corresponding to a shear wave velocity around 800 m/s: the need 970 for host-to-target adjustments (Campbell, 2003; Van Houtte et al., 2011; Al Atik et al., 971 2014) then results in increased epistemic uncertainty as recently emphasized by 972 Ktenidou and Abrahamson (2016), Aristizabal et al. (2017) and Laurendeau et al. (2017), 973

974 From lessons learnt to tentative recommendations for further benchmarking exercises and 975 NL modelling

976 This last section intends to provide advices for users of 1-D non-linear site response977 codes and for next benchmarking exercises, since there is still need for further works,

978 regarding effective stress analysis, NL effect for 2D or 3D media and for more complex979 incident wavefield as well.

We try to provide an overview of the issues one can encounter when applying those methods and the adapted solutions we found. They are built only partly on the results of this benchmark and refers to several other studies, among which previous recent benchmarks on similar methods (Stewart and Kwok, 2009; Stewart, 2008).

We formulate the following recommendations for applying 1-D nonlinear site responsein absence of pore pressure effects.

986 *Preliminary checks*

987 Whatever the numerical method, it is necessary to verify and, if possible, to validate the 988 code used. In particular, if the method used or developed has not been already verified 989 or validated, canonical cases have been uploaded on the Internet for online 990 verification/validation (See Data Resources section).

A verification and validation study, coupled with a documentation of the theory and
implementation of a site response method or software, is highly desirable prior to any
analyses.

994 The decision of applying a non-linear analysis rather than a linear or equivalent linear 995 method can follow recommendations for a priori evaluation of differences between EQL 996 and nonlinear site response simulations such as those presented in Kim et al. (2016). 997 EQL results are considered unreliable when the peak strains – or some associated 998 proxies such as PGV/VS30 - exceed some thresholds – which may be frequency 999 dependent (Assimaki and Li, 2012, Kim et al., 2016).

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1000 If only one numerical method is used, consider that the variability on the results 1001 (standard deviation on pseudo-response spectra) due to the choice of the numerical 1002 method is around 0.1 (in log10 scale unit) in average, but may reach values up to 0.2 at 1003 short periods or around the site fundamental frequency (Régnier et al., 2016a).

1004 Input data

1005 Input motion

1006 The definition and the processing of the input motion coming from recorded motions1007 (outcrop or within) requires careful attention.

1008 In this study, the input waveforms were processed according to the procedures proposed 1009 by Boore and Bommer, (2005), which include the following steps: (1) removing the 1010 mean, (2) finding the first and last zero-crossing and then adding zeroes before and after 1011 these points over a specific time as a function of the number of poles of the high pass 1012 filter to be used (here we added 20 s before and after) and (3) applying a Butterworth 1013 high pass filter at 0.1 Hz. This kind of pre-processing is very important when using codes 1014 that integrate different input motions (for example, from acceleration, from velocity or 1015 from displacement, respectively), in order to ensure having compatible acceleration, 1016 velocity and displacement time histories depending of the code's input. In addition, the 1017 so prepared input motion has no energy below and above the frequency resolution of 1018 the numerical method, which avoids a possible overestimation of permanent surface 1019 displacements.

1020 As recommended by Kwok et al., (2007), in linear/equivalent and linear/non-linear site 1021 response analyses, two cases can be distinguished: (1) if the reference motion is an 1022 outcrop recording, then one should use an elastic base condition with an up-going wave 1023 carrying a signal equal to exactly half the outcropping motion; (2) if the reference 1024 motion is a within motion recorded by a down-hole sensor, then one should use a rigid 1025 base condition without modifying the reference motion or should deconvolve the down-1026 going wave from the within motion and input the up-going wave with elastic base 1027 condition.

1028 Soil characterization

1029 It is recommended that a linear analysis be conducted prior to any nonlinear simulations
1030 to check that the elastic and visco-elastic properties have been well defined and
1031 implemented (check of the expected fundamental resonance frequency if available).

1032 If the site is suspected to have significant lateral variability (Matsushima et al., 2014), 1033 then the characterization should involve measurements of the spatial variability of the 1034 soil layer (depth and soil properties) and 2D or 3D site response may be needed to 1035 capture site effects.

1036 Non-linear parameters, should be defined as a function of depth. The shear modulus 1037 reduction and damping curves as functions of shear strain should be associated and 1038 compatible with the shear strength and V_S profiles. To find the values of the non-linear 1039 parameters, it is recommended to use site specific measurements (e.g. drilling, sampling 1040 and laboratory measurements) with comparisons to literature data and relationships for 1041 similar materials (e.g. Darendeli, 2001; Ishibashi and Zhang, 1993; Menq, 2003; Roblee 1042 and Chiou, 2004; Zhang, 2006).

1043 In this study, it was found that the SC1 model provided good results for one site, but not 1044 the other, which was better captured with the SC2 and SC3 soil curves. This was 1045 attributed to the unique nature of the geology of the second site. Pre-defined literature

1046 curves can produce acceptable estimations of site response, but need to be evaluated 1047 based on the site geology. In this study, cyclic-triaxial tests were found to be not always reliable at low strains. Resonant column or blender element tests are useful and can 1048 provide complementary measurements to constrain the normalized stiffness decay 1049 1050 curves at low strains. Further investigation in future studies, for a larger set of sites and 1051 simulations, would greatly help in establishing consensual procedures for bridging in-1052 situ, low-strain and laboratory high-strain measurements. Elastic properties measured 1053 in the laboratory should be compared with in-situ measurements; the soil sample size, 1054 the soil disturbance and the measurement errors can lead to discrepancies between the 1055 measurement in the laboratory and in-situ. The way to adapt the nonlinear curves and elastic properties measured in the lab to elastic properties measured in situ should be 1056 1057 detailed and uncertainties on these parameters should be accounted for: the lack of a 1058 common, widely accepted procedure is a source of large epistemic uncertainty in the 1059 assessment of NL soil properties, and thus in the prediction of site response under 1060 strong motion.

1061 **Conclusion**

1062 This benchmark was limited to 1-D non-linear total stress analysis. This simple case was 1063 chosen to ensure an, as clear as possible, identification of the impact of various 1064 approaches to implement the non-linearity and the associated parameters. We 1065 calculated the variability between predictions and the misfit with observations. The 1066 variability between codes indicate that the choice of a non-linear model must be coupled 1067 with an uncertainty from 0.05-0.25 (in log10 scale) to reflect the variability from the code, the numerical method, the constitutive model and the user. This uncertainty is 1068 generally not considered in any site-specific response analysis. The misfit is even greater 1069

1070 than the variability between codes, and is associated to the definition of the soil 1071 parameters and intrinsic assumptions of the method (1-D site and vertical propagation 1072 of SH waves without pore water pressure effects). The misfit increases with increasing 1073 loading level and may reach values from 0.25 to 0.35 (log10 scale) around the site 1074 fundamental frequency. It is frequency dependent and can be an over-prediction and an 1075 under-prediction depending of the frequency bandwidth.

Further investigations are needed to propose recommendations as for the method to
obtain non-linear parameters. Indeed, at Sendai site pre-defined literature curves
provided better results, whereas for KSRH10, site specific curves from laboratory tests
were closer to the observations.

1080 The experience gained from this thorough benchmarking exercise allows to propose 1081 some recommendations for either operational studies or future, more advanced 1082 benchmarks. The latter are definitely needed as some issues, in relation to the main 1083 assumptions behind the widely used 1D approach, were clearly identified, as potentially 1084 impacting the misfit between numerical predictions and instrumental recordings: 1085 complexity of the geometry, dimensionality of the input motion and complexity of the 1086 wavefield, or constitutive model with or without water pressure. Addressing those 1087 issues was much beyond the scope of the present project, but each of them would 1088 deserve a dedicated benchmark. We hope that sharing the PRENOLIN experience will 1089 contribute to the design of such future studies.

PRENOLIN : Validation results.

1090 Data and Resources

1091 **Time histories used in this study** were collected from the KiK-net website 1092 www.kik.bosai.go.jp and http://www.kik. bosai.go.jp/kik/ (last accessed November 1093 2011) and from PARI, Port and Airport Institute in Japan .

- 1094 **Some codes used in this work have the following URL links**:
- 1095 ASTER, http://www.code-aster.org (last accessed October 2015);
- 1096 EPISPEC1D, http://efispec.free.fr (last accessed October 2015);
- 1097 Real ESSI simulator, http://sokocalo.engr.
- 1098 ucdavis.edu/~jeremic/Real_ESSI_Simulator/ (last accessed October 2015);
- 1099 OpenSees, http://opensees.berkeley.edu/ (last accessed October 2015);
- 1100 DEEPSOIL, http://deepsoil.cee. illinois.edu/ (last accessed October 2015);
- 1101 SeismoSoil, http://asimaki.caltech.edu/resources/index.html#software (last accessed
- 1102 October 2015). The unpublished manuscript by
- 1103 Verification and validation exercises :
- For 2D/3D linear methods: <u>http://www.sismowine.org</u> (last accessed July 2017)
- 1105 for1-D non-linear (PRENOLIN): <u>http://prenolin.org</u> (last accessed July 2017)

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1288 Figure 12 Comparison of the residuals and associated standard deviation of the surface

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1290 and TS-8-S recorded at Sendai with the envelope represented by the 25 and 75 $\,$

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1292 Figure 12: Idem Figure 10. For KSRH10, for the input motions TS-1-K and TS-9-K and

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1298 and SC3.

1300 **TABLES**

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Table 1: Selected event characteristics (PGA for EW components)

			Fs*				PGA _{borhole}	PGA _{surface}	
		CodeN	(Hz)		Depth [‡]	Depi§	**	++	Fc ^{‡‡}
	EQ	ame		Mw [†]	(km)	(km)	(cm/s²)	(cm/s ²)	(Hz)
KSRH10	KSRH100309260450	TS-0-K	200	8	42	180	110	558	6,6
	KSRH100411290332	TS-1-K	200	7,1	48	32	81	319	4,8
	KSRH100412062315	TS-2-K	200	6,9	46	44	69	386	4,2
	KSRH100411290336	TS-3-K	200	6	46	37	64	199	6,0
	KSRH100404120306	TS-4-K	200	5,8	47	43	27	162	5,3
	KSRH100904282021	TS-5-K	100	5,4	38	69	25	163	4,0
	KSRH100501182309	TS-6-K	200	6,4	50	38	25	125	6,7
	KSRH100912280913	TS-7-K	100	5	85	39	9	58	6,5
	KSRH100805110324	TS-8-K	100	5,1	88	63	8	46	6,2
	KSRH100309291137	TS-9-K	200	6,5	43	105	7	54	4,6
Sendai	F-2958	TS-1-S	100	9	23,7	163	252	481	6,9
	F-1889	TS-2-S	100	7,1	72	81	62	244	9,0
	F-1932	TS-3-S	100	6,4	11,9	19	61	208	10,3
	F-2691	TS-4-S	100	6,8	108,1	169	25	89	7,1
	F-3012	TS-5-S	100	5.9	30.7	96	25	72	7.5
	F-2659	TS-6-S	100	7.2	7.8	83	35	82	7.7
	F-1856	TS-7-S	100	5.9	41.2	95	12	32	7.8
	F-2862	TS-8-S	100	6.4	34.5	208	5	7	3.4
	F-2730	TS-9-S	100	5.8	47	176	3	12	6.2

1302

* Fs : Sampling frequency

[†] Moment Magnitude

[‡] Hypocentral depth

[§] Epicentral distance

** Peak Ground Acceleration at the down-hole station

^{††} Peak Ground Acceleration at surface station

[#] Central Frequency

Table 2: Geological characteristics of the 2 selected sites with locations of the undisturbed soil samples.

Site	Down- hole sensor depth (m)	Max. complementary drilling depth (m)	Type of soil	Number of samples (location)
Sendai	8	10	Sand	2 (3.3 & 5.4 m)
KSRH10	250	50	Sand /clay	6 (3.5, 7.5, 14.5, 22.5, 29,7 & 34 m)

Z* (m)	Vs† (m/s)	Vp (m/s)	ρ^{\ddagger} (kg/m ³)	Qs§	ξ**	Set of G/G_{max} and damping curves	$ au_{max}^{\dagger\dagger}$ (kPa)
6	140	1520	1800	25	0.02	SC1-1,SC2-1,SC3-1	
11	180	1650	1800	25	0.02	SC1-2,SC2-2,SC3-2	
15	230	1650	1500	25	0.02	SC1-3,SC2-3,SC3-3	6 II.
20	300	1650	1500	25	0.02	SC1-4,SC2-3,SC3-3	Eq
24	250	1650	1600	25	0.02	SC1-5,SC2-4,SC3-4	to]
28	370	1650	1600	25	0.02	SC1-6,SC2-5,SC3-5	ng e
35	270	1650	1800	35	0.0142	SC1-7,SC2-5,SC3-5	ate
39	460	1650	1800	25	0.02	SC1-8,SC2-6,SC3-6	cul
44	750	1800	2500	75	0.0066	Linear	a Cal
84	1400	3400	2500	140	0.0035	Linear	
255	2400	5900	2500	240	0.0020	Linear	

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* Depth of the soil layer

 † Shear wave velocity of the soil layer

[‡] Density of the soil layer

[§] Elastic Damping ratio

** Elastic attenuation

^{††} Shear strength

Table 4: Soil	properties.	from the	Sendai site
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Z (m)	Vs (m/s)	Vp (m/s)	ρ (kg/m ³)	Qs	ξ	Set of G/G_{max} and damping curves
1	120	610	1850	25	0.02	SC1-1, SC2-1
2	170	870	1850	25	0.02	SC1-2, SC2-1
3	200	1040	1850	7.14	0.07	SC1-3, SC2-1
4	230	1180	1890	7.14	0.07	SC1-4, SC2-2
5	260	1300	1890	7.14	0.07	SC1-5, SC2-2
6	280	1420	1890	7.14	0.07	SC1-6, SC2-2
7	300	1530	1890	7.14	0.07	SC1-7, SC2-2
10.4	550	2800	2480	50	0.01	Linear

1315 Table 5: Standard deviation values of the residuals of the logarithm results for all teams and soil columns (PGA,

spectra acceleration at 3 periods).

		Sor	dai				KCDH10		
	TS-1-S	TS-2-S	TS-5-S	TS-8-S	TS-0-K	TS-1-K	TS-2-K	TS-4-K	TS-9
PGA	0.16	0.11	0.11	0.09	0.20	0.17	0.17	0.17	0.10
SA(0.1s)	0.10	0.10	0.12	0.10	0.23	0.22	0.23	0.20	0.15
SA(0.3s)	0.11	0.09	0.09	0.09	0.17	0.14	0.15	0.09	0.07
SA(1s)	0.10	0.08	0.09	0.09	0.11	0.06	0.07	0.07	0.11

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Figure 1

































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PRENOLIN: international benchmark on 1D nonlinear site response analysis – Validation phase exercise

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Electronic supplement

The electronic supplement contains elements that provide details on: (1) the codes used during the benchmark with the table of the participants and codes used (Table S1) (2) the analysis performed on the accelerometric data to determine the angle of incidence of seismic waves at KSRH10 and Sendai site (Figure S1). (3) A figure that indicate the location of the soil sampling performed to characterize Sendai and KSRH10 sites (Figure S2 from S5) and (4) Figures on applicability of the numerical methods (Figure S6 and Figure S7).

A detailed presentation of the Polarization analysis for verification of the verticality of the incident propagating input motions is given below:

Since P- and S-waves show a high degree of linear polarization (contrary to Rayleigh waves which are generally elliptically polarized (i.e. Montalbetti and Kanasewich, 1970), the direction of propagation for P-waves is parallel to the particle motion trajectory (hodograph) and perpendicular to the direction of propagation for S-waves. Rayleigh wave particle motion is within the vertical-radial plane, and Love waves are polarized in a horizontal plane perpendicular to the propagation direction. Due to reflections and scattering, a rather a complex particle motion trajectory is observed in real seismograms instead of pure polarization states. The hodograph can therefore be fitted to an ellipsoid in a least-squares sense by means of a covariance analysis (Flinn, 1965; Jurkevics, 1988).

The 3x3 covariance matrix is symmetric, has real non-negative eigenvalues, and its eigenvectors are the principal axes of an ellipsoid that represents the best fit to the data in a least-squares sense. The eigenvector associated with the largest eigenvalue points into the main polarization direction, i.e. the long axis of the ellipsoid. The direction of polarization is calculated from the components (direction cosines) of this eigenvector. The direction can be described by a horizontal azimuth angle ϕ and by the deviation from the vertical direction or apparent incidence angle θ as (Maercklin, 1999):

$$\theta = \arctan\left(\frac{\sqrt{x^2 + y^2}}{z}\right)$$

Eq SI

$$\varphi = \arctan\left(\frac{x}{y}\right)$$

Eq S2

Here, x and y denote the two horizontal components of the principal eigenvector V and z its vertical component.

The choice of the time window length and the frequency bandwidth are subject to tradeoffs between resolution and variance (e.g. Wang & Teng, 1997). A short time window and a narrow bandwidth avoid averaging over different phases allow for the resolution of frequency-dependent polarization, whereas a longer window and a wider frequency band yield more stable polarization estimates. Besides that, any filtering before the polarization analysis has to be applied in the same way to all three components and should not distort the signal significantly. In our analysis we whose a time window of 1s and we filtered the data unsing a low-pass filter with a cut-off frequency of 2Hz.

After the main polarization direction is determined, the degree of linear or planar polarization is evaluated. The rectilinearity RL is a measure of the degree of linear polarization of an event (Flinn, 1965). Montalbetti & Kanasewich (1970) and Kanasewich (1981) define RL as:

$$RL = 1 - \left(\frac{\lambda_2}{\lambda_1}\right)$$

Eq S3

with the two largest eigenvalues λ_1 and λ_2 . With this definition the range of values is between RL = 0 for elliptical or undetermined polarization, and RL = 1 for exactly linear polarization. In **Error! Reference source not found.**, we potted the incidence in stereographic projection for the two sites (left KSRH10 and right Sendai) of the wave at the borehole station (bottom stereonet) and at the surface (top stereonet).

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1. List of table caption

Table S1: List of the teams that participate to the PRENOLIN benchmark with references on the code used.

2. List of Figure caption

Figure S1: Stereonet of the incident waves recorded at KSRH10 (left graphs) and Sendai (right graphs) at the surface sensor (upper graphs) and at the borehole station (bottom graphs).

Figure S2: Localization of the soil sampling at Sendai site 1/2

Figure S 3: Localization of the soil sampling at Sendai site 2/2

Figure S4: Localization of the soil sampling at KSRH10 site 1/2

Figure S5: Localization of the soil sampling at KSRH10 site 2/2

Figure S6: Comparison of the surface response spectra of the EW component of the input motion TS-1-S recorded at Sendai with the computed ones using SC2. The results of the computations using 1 component of motion and using the three components of motions simultaneously are individually plotted. The rest of the simulations are plotted using the average ± 1 standard deviation.

Figure S7: Comparison of the empirical surface to within spectral ratio of the EW component of the input motions TS-1-S and TS-8-S at Sendai with the computed ones. The Equivalent linear methods and the effective stress analysis are individually plotted while the rest of the results are plotted using the average ± 1 standard deviation.





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triaxial cyclique test 0 (rock) rock unconfined compression 0 test triaxial cyclique test 0 (clay) triaxial cyclique test -. (sand) liquefaction test (sand) . -201 compression triaxial test 0 CU Test Labo compression triaxial test . -CD consolidation properties 0 bulk density . liquid limit and test 0 plastic limit particle size distribution Physical water conten . density . -Method for obtaining soil samples core pack sampler 0 using rotary triple-tube sampler . using thin-walled tube 0 sampler with fixed piston PSSL (Downhaul) 0 Tests in-site PSSL (Suspension) 0 k0 0 (Self Drilling LLT) SPT (Method for stan-dard penetration test) 0 Drilling Core(0.5m) 0 Drilling size(mm) -99 0--\$8¢ 통 ore ÷ 2 Assumption stratum Vs(km/s) 9 3 合計 Soil property Gravel clay Sandy soil classification Т Τ Т D Depth (m 2 5 10

Measurements quantities (SENDAI(2))

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Figure S3

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Measurements quantities (Kushiro)

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Measurements quantities (Kushiro(2))




Table S1: List of the teams that participate to the PRENOLIN benchmark with references on the code used.

Team Name	Affiliation	Te In	am dex	Code Name	Code Reference
D. Asimaki & J. Shi	Caltech, Pasadena, California	Α	0	SeismoSoil	(Li and Assimaki, 2010; Matasovic and vucetic, 1993; Shi and Asimaki, 2017)
S. lai	DPRI, Kyoto University, Kyoto, Japan	В	0	FLIP	(lai, 1990)
S. Kramer	University of Washington, Seattle, Washington	с	0	PSNL	(In development)
E. Foerster	CEA, France	D	0	CYBERQUAKE	(Modaressi and Foerster, 2000)
C. Gelis	IRSN, France	Ε	0	NOAH-2D	(lai, 1990)
A. Giannakou	Fugro, Nanterre Cedex, 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)5)
J. Gingery & A. Elgamal	UCSD, La Jolla, California	н	0	OPENSEES-UCSD- SOIL-MODEL	See Data and Resources
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, Comenius University, Bratislava, Slovakia	к	0	1DFD-NL-IM	
S. Foti & S. Kontoe	Politecnico di Torino, Torino, Italy	L	1	ICFEP	(Kontoe, 2006; Potts and Zdravkovic, 1999; Taborda et al., 2010)
	and Imperial College, United Kingdom	L	2	DEEPSOIL-NL 5.1	(Hashash et al., 2012)
G. Lanzo, S. Suwal, A. Pagliaroli & L. Verrucci	University of Rome La Sapienza	м	0	FLAC_7,00	(ITASCA, 2011)
	University of Chieti-Pescara, Italy	М	1	DMOD2000	(Matasović and Ordóñez, 2007)
		М	2	DEEPSOIL 5.1	(Hashash et al., 2012)
F. Lopez-Caballero & S. Montoya-Noguera	CentraleSupélec, Paris-Saclay University, Châtenay-Malabry, France	N	0	GEFDyn	(Aubry and Modaressi, 1996)
F. De-Martin	BRGM, France	Q	0	EPISPEC1D	(Iai, 1990) See Data and Resources
B .Jeremić , F. Pisanò & K. Watanabe	UCD, LBLN, TU Delft & Shimizu Corp	R	0	real ESSI Simulator	See Data and Resources
A. Nieto-Ferro, D. Vandeputte	EDF, Paris & Aixen-Provence, France	s	0	ASTER	See Data and Resources
A. Chiaradonna, F. Silvestri & G. Tropeano	UNICA and University of Naples, Naples, Italy	т	0	SCOSSA_1,2	(Tropeano et al., 2016)
		т	1	STRATA	
M.P. Santisi d'Avila	University of Nice Sophia Antipolis, Nice, 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,	Unversity of Bologna, Sapienza	z	0	EERA	(Bardet et al., 2000)
Falcone	Politecnico di Bari, Italy	z	1	PLAXIS	(Benz, 2006; Benz et al., 2009)
M. Taiebat & P. Arduino	UBC, British Columbia, Canada and University of Washington, Seattle, Washington	w	0	Opensees	See Data and Resources

DPRI, Disaster Prevention Research Institute; CEA, Commissariat à l'Energie Atomique; IRSN, Institut de Radioprotection et de Sûreté Nucléaire; NTUA, National Technical University of Athens; UCSD, University of California, San Diego; ECP, Eclode Centrale Paris-Supelec; BRGM, Bureau De Recherches Géologiques et Minières; UCD, University of California, Davis; LBLN, Lawrence Berkeley National Laboratory; EDF, Electricité de France; UNICA, University of Calgari; CEREMA, centre d'études et d'expertise sur les risques, l'environnement, la mobilité et l'aménagement; UBC, University of British Columbia.