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PRENOLIN PROJECT: RESULTS OF THE VALIDATION PHASE AT SENDAI SITE.

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Abstract

One of the objectives of the PRENOLIN project is the assessment of uncertainties associated with non-linear simulation of 1D site effects. An international benchmark is underway to test several numerical codes, including various non-linear soil constitutive models, to compute the non-linear seismic site response. The preliminary verification phase (i.e. comparison between numerical codes on simple, idealistic cases) is now followed by the validation phase, which compares predictions of such numerical estimations with actual strong motion data recorded from well-known sites. The benchmark presently involves 21 teams and 21 different non-linear computations. Extensive site characterization was performed at three sites of the Japanese KiK-net and PARI networks. This paper focuses on SENDAI site. The first results indicate that a careful analysis of the data for the lab measurement is required. The linear site response is overestimated while the non-linear effects are underestimated in the first iteration. According to these observations, a first set of recommendations for defining the non-linear soil parameters from lab measurements is proposed. PRENOLIN is part of two larger projects: SINAPS@, funded by the ANR (French National Research Agency) and SIGMA, funded by a consortium of nuclear operators (EDF, CEA, AREVA, ENL).

Keywords: Non-linear site response analysis, validation, Sendai site.

1. Introduction

While a consensus has undoubtedly been reached on the existence of non-linear effects, their quantification and modeling remains a challenge, despite the existence of a commonly accepted practice. The ability to accurately predict non-linear site responses has indeed already been the subject of two recent comparative tests. It was one of the targets of the pioneering blind tests initiated in the late 80's/early 90's on 2 sites of Ashigara Valley (Japan) and Turkey Flat (California); however, those sites lacked strong motion records until the 2004 Parkfield earthquake during which the Turkey Flat site experienced a 0.3g motion. A new benchmarking of 1D non-linear codes was thus carried out in the last decade. Its main findings were reported by Kwok et al., (2008) and Stewart and Kwok, 2009, who emphasized the key importance of the way these codes are used and of the required in-situ measurements. Tests on 2D NL (Non-Linear) modeling were also attempted within the framework of the Cashima/E2VP project (Bard et al 2011), but the coupling of geometrical complexity and non-linearity proved to be premature to perform such kind of computations.

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

In this article, we present the results of the validation phase at Sendai PARI site. The first iteration consisted in forward computations without knowledge of the true surface soil response, while in the following iterations, this information was made available to the participating teams and the soil column parameters (elastic and non-linear) were modified.

2. The codes tested

We compared 21 different numerical codes used by 21 participating teams; some teams tested several codes and some codes were tested by different teams: SeismoSoil (A-0), FLIP (B-0), PSNL (C-0), CYBERQUAKE (D-0), NOAH-2D (E-0), DEEPSOIL (J-0 equivalent linear method and J-1, F-0 and M-2, for the non-linear method) NL-DYAS (G-0), OPENSEES (H-0), 1DFD-NL-IM (K-0), ICFEP (L-1), FLAC.7.00 (M-0), DMOD2000 (M-1), GEFDYN (N-0), EPISPEC1D (Q-0), real ESSI (R-0), ASTER (S-0), SCOSSA-1,2 (T-0), SWAP-3C (U-0), GDNL (Y-0), SANISAND (W-0), EERA (Z-0) and PLAXIS (Z-1).

3. Site selection

Sites were selected from the KiK-net and PARI networks. The vertical accelerometric array configuration allowed the calculation of borehole site responses. The PARI sites are much more superficial than the KiK-net sites, the downhole sensor is only ~10 to 15 m deep, and a Vs profile is therefore available down to that depth.

More than 46,000 (six-component) recordings from KiK-net were analyzed, to derive a) the empirical site response at the chosen site and b) the numerical linear site response from the available Vs profile. Two additional sites from the PARI network were analyzed, Sendai and Onahama (30 and 80 earthquake recordings, respectively).

The site selection was performed on the basis of the following requirements: (1) availability of both strong and weak events recordings, (2) plausibility of a 1D geometrical soil configuration, i.e., satisfactory agreement between numerical and empirical site responses in the linear / weak motion range, and (3) the downhole sensor must not be too deep (depth < 250 m).

To fulfill the first and second criteria, we selected sites that recorded at least two earthquakes with PGAs higher than 50 cm/s^2 at the downhole sensor and we selected 1D KiK-net site configurations identified and reported by Thompson et al., (2012) in addition to visual inspections of the comparison between the numerical and empirical site response curves. Initially, 5 KiK-net sites (FKSH14, IBRH13, IWTH04, KSRH10 and NIGH13) and 2 PARI sites were selected, and finally 4 KiK-net sites were removed due to liquefaction susceptibility (FKSH14), rocky geology (IBRH13), mountainous environment (IWTH04) and insufficient nonlinearity (NIGH13). We selected 3 sites among the remaining ones -KSRH10, Onahama and Sendai - to be fully characterized for the purpose of the validation phase.

4. Site characterization and soil column definition

An extensive measurement campaign was carried out at each of these 3 sites, to obtain the in-situ V_S, V_P and density profiles (using suspension logging for KSRH10 and downhole PS logging for Onahama and Sendai). Additional MASW measurements were performed to check the spatial variability of the soil properties. To constrain the non-linear soil parameters, multiple laboratory measurements were conducted on (1) Disturbed soil samples: Moisture content, soil particle density, particle size distribution, liquid and plastic limits and (2) Undisturbed soil samples: Wet density, tri-axial compression test (either drained for sandy soil or un-drained for clayey soil), consolidation tests, cyclic undrained tri-axial test for sandy samples and cyclic tri-axial test to obtain the non-linear soil properties. The number and location of the undisturbed soil samples is specified in Table 1, along with the downhole sensor depth, the maximal depth of impedance contrast and the type of soil.

Table 1: Geological characteristics of the 3 selected sites with locations of the undisturbed soil samples.

Site	**Downhole sensor depth (m)**	**Max. impedance contrast depth (m)**	**Type of soil**	**Number of cyclic tri-axial test (location)**
Sendai	10.4	7	Sand	2 (3.3 & 5.4 m)
Onahama	11	17	Sand	3 (4.5, 7.5 &11.4 m)
KSRH10	250	44	Sand /clay	6 (3.5, 7.5, 14.5, 22.5, 29,7 & 34 m)

For Sendai site, 3 iterations were performed:

In the first iteration, the organizing team estimated the velocity profile of the soil column using these data together with the observed linear empirical site response. The soil column is described in Table 2, and the shear modulus degradation and damping curves with respect to deformation, constructed from the cyclic tri-axial compression test results, are illustrated in Figure 2.

Equation 1 describes the Vs profile at Sendai and Vp is deduced from the Poison ratio (ν) and Vs values.

$$Vs = Vs_1 + (Vs_2 - Vs_1)\left[Z - Z_1 / Z_2 - Z_1\right]^{\alpha}, where Vs_1 = 140 m/s, Vs_2 = 460 m/s, \alpha = 0.7 \quad (1)$$

The non-linear curves were derived from cyclic tri-axial compression test results and are illustrated in Figure 1. We normalized the young modulus decay curves (from the lab 5th cycle of loading) by the low strain young modulus (E0). E0 is the value of the Hardin-Drnevich model [4] that mimics the lab results at 0.0001%. We assimilate this E/E0 decay curve to the shear modulus G/Gmax decay curve, with Gmax associated to the in-situ velocity measurements, i.e., Gmax = ρV_S^2. The shear strain used was considered equal to 3/2 of the axial strain directly measured during the triaxial test. Indeed, the shear strain is the difference between the axial and radial stress $\gamma = \varepsilon_a - \varepsilon_r$. During the cyclic triaxial test under undrained conditions volumetric changes are zero. Then the volumetric strain is null $\varepsilon_v = \varepsilon_a + 2\varepsilon_r = 0$. From the previous two equation we can deduce that: $\gamma = 3/2\varepsilon_a$

In the second iteration, two improved soil columns were proposed by the organizing team (SC1 and SC2) and only by the participants (SCE) with modified non-linear soil parameters as described in Table 2 and described in Figure 2:

- **SC1:** Considering both the differences between strong-motion predictions and observations, the significant deviations of the NL curves proposed for iteration 1, and the usual degradation curves provided in the scientific literature for similar soils, the working group decided to propose the [5] curves to replace the lab measurements curves. The former curves are more non-linear than the actual lab measurements. They were in addition tuned to a larger low strain damping value to fit the observed weak motion amplification. It is worth noting that, at this stage, the benchmark was no longer blind. This soil column with these non-linear properties was called SC.
- **SC2:** The organizing team insisted on keeping NL degradation curves corresponding to the initial interpretation of lab measurements, and thus proposed another set of soil column properties, simply modifying the low strain damping values according to the weak motion amplification observations. This second set of soil column properties was called SC2.
- **SCE:** An optional calculation was also proposed to the participants. It consisted in allowing each team to define his/her own preferred soil model on the basis of his/her interpretation of in-situ and lab measurements, and allowing them in addition to use either a total stress analysis or an effective stress analysis. One team proposed a specific shear modulus decay curve (SCE-W), based on the lab measurements as SC2 but adjusted to the in-situ measurements (i.e. shear modulus), since the shear modulus from the lab test is four times lower than the in-situ shear modulus.

In the third iteration, the same soil columns as for iteration 2 were used except for the location of the down-hole sensor that was changed and corrected to its actual location that is 10.4 m instead of 8 m in the first two iterations

The location of the down-hole sensor has, in this specific case, a very small impact on the results. Figure 1 illustrates the comparison between the linear transfer functions computed with different sensor locations for the Sendai site. We can observe that for 8 to 10 m the location of the down-hole sensor has a very limited impact of the frequency of the first peak from (8.8 to 8.3 Hz for 10.4m and 8m depth respectively). **For clarity purposes and to be consistent, the next figures will show only the results of iteration 1 and 2.**

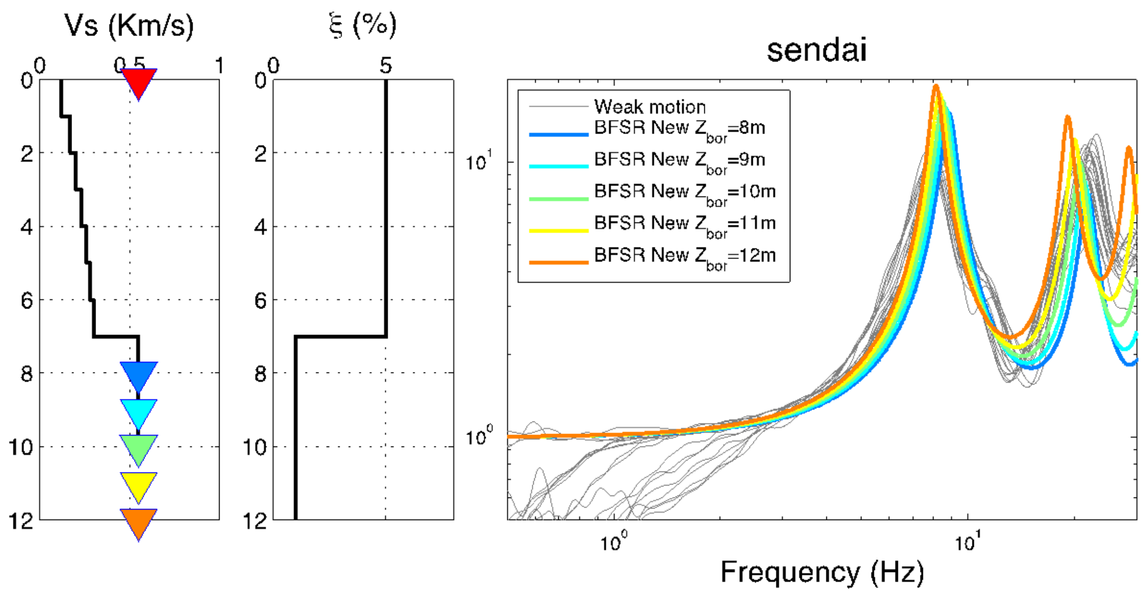

Figure 1: Impact of the down-hole sensor location at Sendai site on linear transfer function.

Table 2: Soil properties from the Sendai site. The last three columns decribe the G/G_{max} and damping curves used for the various soil columns: iteration1 and iteration 2 & 3 (SC1 and SC2).

Z(m)	Vs(m/s)	Vp(m/s)	ρ(kg/m3)	ξ It-1	ξ It-2 SC1	ξ It-2 SC2	NL It1	NL SC1	NL SC2
1	120	610	1850	0.025	0.02	0.05	SC2-1	SC1-1	SC2-1
2	170	870	1850	0.025	0.02	0.05	SC2-1	SC1-2	SC2-1
3	200	1040	1850	0.025	0.07	0.05	SC2-1	SC1-3	SC2-1
4	230	1180	1890	0.02	0.07	0.05	SC2-2	SC1-4	SC2-2
5	260	1300	1890	0.02	0.07	0.05	SC2-2	SC1-5	SC2-2
6	280	1420	1890	0.02	0.07	0.05	SC2-2	SC1-6	SC2-2
7	300	1530	1890	0.02	0.07	0.05	SC2-2	SC1-7	SC2-2
8	550	2800	2480	0.01	0.01	0.01	NaN	NaN	NaN

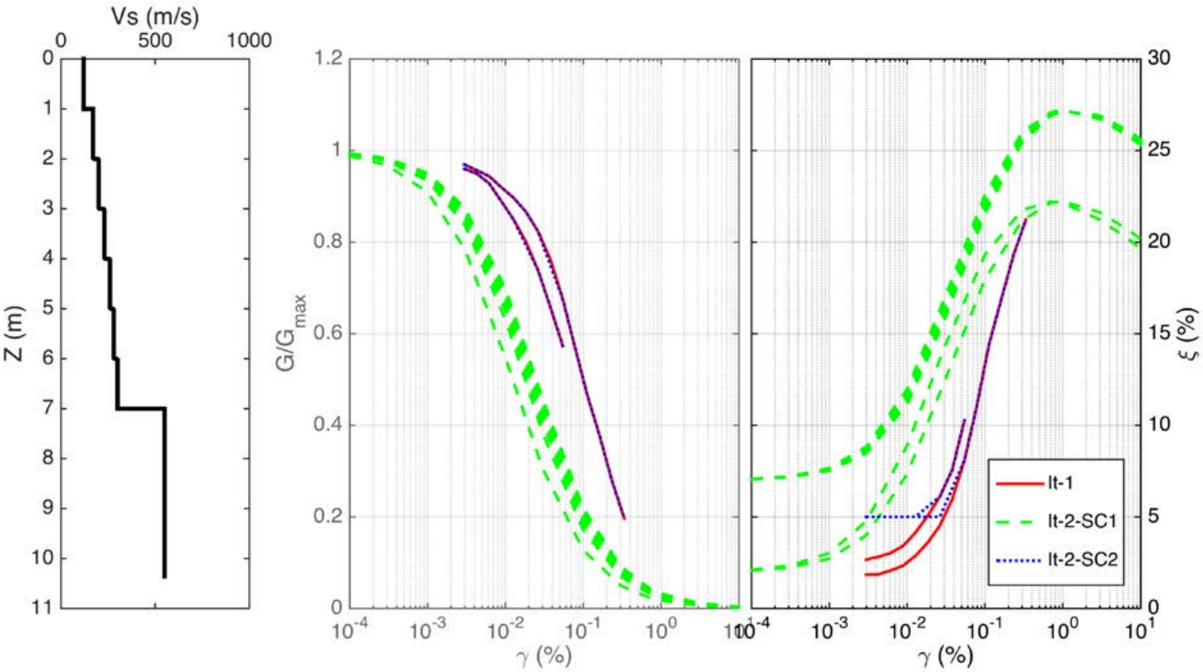

Figure 2: Vs profiles, G/Gmax and damping curves relative to shear strain, at Sendai

5. Input motion selection

The PGA (Peak Ground Acceleration) and the frequency content of the considered signal are two relevant parameters of the input motion for describing the expected degree of non-linear soil behavior [6]. Nine input motions at Sendai site were selected, representing 3 different PGA levels (≥ 0.6, 0.2-0.3 m/s2 and ≤ 0.1 m/s2 at the downhole sensor) and approximately 3 distinct frequency contents. PGA was calculated on the acceleration time histories as the quadratic mean of the EW and NS components, filtered between 0.1 and 40 Hz. The numbering of input motion corresponds to decreasing PGA level from #1 to #9. The time history of the input motions are illustrated in the figure 3. We recall that the input motions are down-hole recordings which are directly used in the simulation along with a rigid substratum hypothesis (here we imoose the motion at the base of the soil column). These are recommendations coming from previous benchmark excercices including the verification phase of PRENOLIN [7].

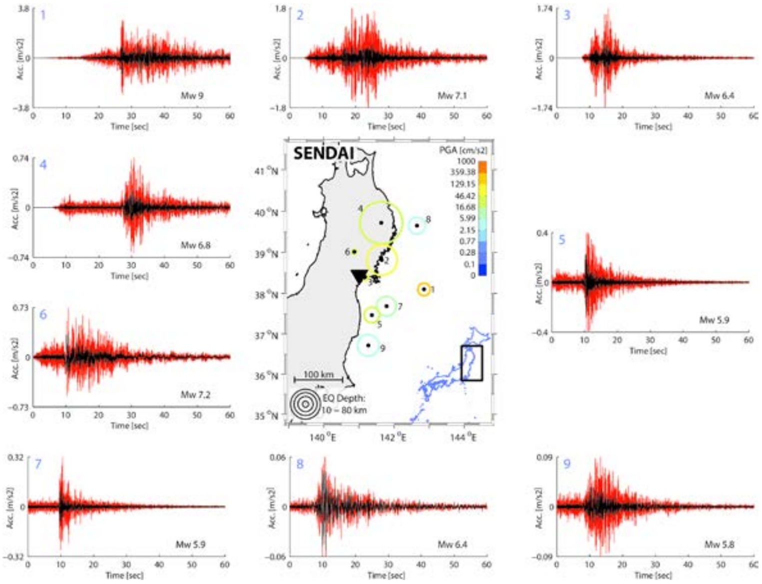

Figure 3: Time history of the selected input motions at Sendai site. The black lines represent the down-hole recording and the red line the surface recording.

6. Method of analyses of the computations

The particpants were asked to calculate the propagation of 9 input motions. They had to provide, the accelerations for 8 virtual receiver locations (at the surface and interfaces) and the stress-strain histories at 7 locations (at the middle of each soil layer). From these results, we then performed a comparative analysis for a number of parameters:

- A few engineering parameters as selected by [8], i.e., PGA, response spectra at different period ranges, CAV, duration, and cross-correlation.
- Surface / downhole sensor amplification for Fourier (BFSRs) and response spectra
- Depth dependence of peak shear strain, shear strength and PGA,
- G/G_{max} curves, stress-strain curves at selected receivers
- Additional time-frequency analyses (ratio between surface and downhole Stockwell-transforms).

In this paper only the Surface / Downhole sensor amplification for Fourier spectra will be shown.

7. Validation results on Sendai

To compare the results of the 2 iterations and the different soil columns used, we calculated the average ± one standard deviation (assuming a log normal distribution) of all predictions. Figure 4 represents the comparison of the empirical surface / downhole transfer function (black line) with the predictions, for the iteration 1 (grey area), the iteration 2 with the SC2 soil column (green lines), the SC1 soil column (red lines) and the preferred soil columns SCE (blue lines) the subplot (a) is for the strongest input motion (TS-1), (b) for the second strongest (TS-2), (c) is for a moderate motion (TS-5) and (d) is for a weak motion (TS-8). Our main observations are :

- The variability of the computations decreases from iteration 1 to iteration 2, for all the ground motion intensity measures that are considered here.

- For the input motion 8, the empirical solution is in the prediction envelope whatever the iteration or soil column. Nevertheless, the computations are getting closer to the real data at the second iteration especially for the preferred ("SCE") soil models.

- For the input motion 5, all iterations and soil columns fail to predict the real data,
- For input motions 1 and 2, the results are getting closer to the empirical transfer function from iteration 1 to 2. The fit is also significantly improved with the soil column SC1 and the preferred soil column SCE compared to the soil column SC2.

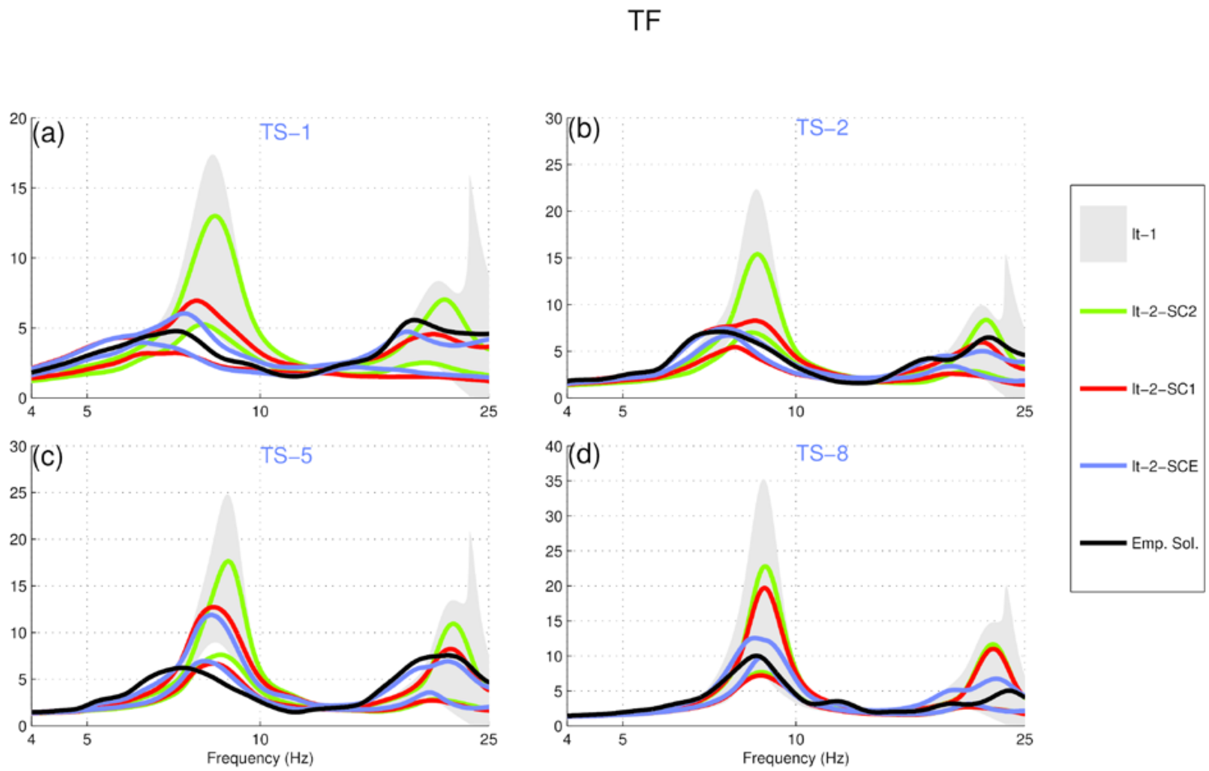

Figure 4: Comparison of the **empirical transfer function** (black line) with the computations, for the iteration 1 (grey area) for the iteration 2 with the SC2 soil column (green lines), the SC1 soil column (red lines) and the preferred soil columns SCE.

8. Discussion: From lab measurements to soil model

One of the main scopes of the second iteration was to understand the discrepancy between the observed non-linear soil behavior and the simulated one based on the initial interpretation of lab measurements data.

We have seen that the results were better using soil column 1 (SC1) defined with literature parameters. The soil column defined with non-linear parameters directly from the laboratory measurements (SC2) fail to predict strong motion behavior. Either the lab data are not describindg well the non-linear soil behaviour of the soil or it has been mis-interpreted.

A working group in charge of proposing a consensual procedure to translate the lab tests into soil model parameters was set up:

- It has been found that there is no one consensus to go from lab data to input data for the G/G_{max} curves especially using only cyclic- tri-axial test. Considering the difficulties to calibrate the values of G_{max}^{lab} with only cyclic-tri-axial test it has been unanimously admitted that “a low strain type” dynamic test such as resonant column or blender element should be perform in priority. The cyclic tri-axial test should be done, in priority, when suspecting liquefaction.
- The elastic shear modulus values from the lab tests (G_{max}^{lab}) are generally under-estimated compare to the in-situ measurements (G_{max}^{insitu}), especially for cyclic tri-axial tests (indeed cyclic tri-axial tests are not

reliable at low strain, below 10-4 %).Tatsuoka et al., (1995) showed that this could be due to sample disturbance where stronger differences are observed depending on the type of shear strain measurement. When local measurements of shear strain are performed using internal gauges (inside the soil sample) compare to external measurements (classic measurement), the discrepancies 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 (Caballero personal communication, 2015).

- A correction procedure was set up to partially correct this value. The procedure corrects for external to local measurement errors and not for soil sample disturbance.
- For Sendai site, the shear modulus from the lab measurement is equal to 25 Mpa against 100 Mpa from the *in situ* measurement of Vs (230m/s) and density values (1890 kg/m^3). The correction procedure define during the project could not be applied in this case considering the large discrepancy between lab and in-situ measurement.

9. Recommendations for definig non-linear soil parameters

Our recommendations (based on the analysis of the results at KSRH10 site as well) are the following:

- The description of NL soil characteristics and their variations with depth should also be performed very carefully: it should include, in addition to the density and velocity profiles, the strain dependency of G/Gmax and damping, ς, for each layer, and the shear strength profile.
- The defined shear modulus and damping curves for each soil layer should be systematically compared with the values and correlations available in literature for similar soils, and thoroughly discussed in case of large deviations. Examples of correlations for nonlinear curves that exist in literature (e.g. [5], [9], [10])
- The selected nonlinear soil models should properly replicate the prescribed modulus reduction and damping curves for each soil layer, and any deviations in behavior need to be documented.
- The implied shear strength of the strain-dependent curves should match the shear strength of the in-situ soil.
- Resonant Column/Torsional Shear (RC/TS) tests are the preferred test for obtaining small-strain modulus reduction and damping behaviour. These tests can be complimented with cyclic triaxial tests with attached bender elements to constrain large-strain soil properties. Bender elements are the best way to obtain G_{max} from cyclic triaxial tests, but may give quite different G_{max} values than those calculated from the field-measured V_S profile due to sampling disturbance and specimen size.

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