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NONLINEAR SOIL PROPERTIES ESTIMATED FROM DOWNHOLE ARRAY RECORDINGS AT KASHIWAZAKI-KARIWA NUCLEAR POWER PLANT IN THE NIIGATA-KEN CHUETSU-OKI EARTHQUAKES

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ABSTRACT:

Nonlinear soil properties are back-calculated using strong motion downhole array recordings at the Service Hall of the Kashiwazaki-Kariwa nuclear power plant during the 2007 Niigata-ken Chuetsu-oki earthquakes. A technique used for this inversion is genetic algorithm combined with a one dimensional equivalent linear response analysis in which soil damping ratios vary with Fourier amplitude of shear strain in the frequency domain. The Holocene and Pleistocene dune sands with V_s = 200-350 m/s at depths smaller than about 70 m showed strong nonlinear behavior with shear modulus ratios of about 0.01-0.8 and damping ratios of about 20-35 % at maximum shear stains of about $2x10^{-3}$ - $3x10^{-2}$ during the main event. The reduction in shear modulus ratio is consistent with previous laboratory test results for sand consolidated under similar confining pressures but the back-calculated damping ratio seems to be higher than that of the previous studies. The mudstone with V_s =500 m/s located at depths greater than about 70 m did not show any strong nonlinearity during the main shock, with shear modulus ratios of about 1.0 and damping ratios of up to 5 % at maximum shear strains up to about $5x10^{-4}$ - $2x10^{-3}$.

KEYWORDS: Niigata-ken Chuetsu earthquake, nonlinear site response, shear modulus, damping ratio, back-analysis, genetic algorithm

1. INTRODUCTION

The Niigata-ken Chuetsu-oki earthquake (M_j=6.8) that occurred on July 16, 2007, with an epicenter off the Niigata Prefecture, affected the coastal areas of the southwestern Niigata prefecture including Kashiwazaki city and Kariwa village as well as the Kashiwazaki-Kariwa nuclear power plant of Tokyo Electric Power Company (TEPCO). The earthquake claimed a death toll of fifteen, with more than two thousand injuries. About forty thousand residential houses were either totally collapsed or were partially damaged. Both ground shaking and ground failure problems affected residential houses in the area. Most of the houses affected by ground shaking were very old, built with traditional Japanese construction technique and with large opening along the street, and thus constituted a soft first story prone to deformation parallel to the street. Many houses on reclaimed alluvial sandy deposits and slopes of sand dunes suffered from ground failure problems involving soil liquefaction. Ground failure problems also affected many infrastructures such as railways, highways, roads, bridges, embankments and lifelines (Kayen et al, 2007).

A matter of utmost concern is the performance of the Kashiwazaki-Kariwa nuclear power plant during and after the earthquake. A fire broke out around a house electric transformer and many less critical structures as well as incidental components such as pipes and ducts were damaged; however, three reactors in full operation as well as one in a start-up state during the quake were automatically shutdown and all the critical structures including reactor and turbine buildings seemed to be in good condition.

A total of 97 accelerometers of old and new systems were installed at the site (TEPCO, 2007a). The old system includes 36 stations in the reactor and turbine buildings, 2 for the main exhaust stacks, and 29 on and in the ground, while the new system includes 28 in the reactor and turbine buildings, and 2 on the ground surface at Units 1 and 5. The strong motions at 33 locations were recorded for the main shock, but unfortunately, the recordings obtained at the other 63 locations from the old system as well as one from the new system, including three free-field downhole arrays close to the reactor buildings, were lost, with the exception of the peak values of the old system. The recovered 33 recordings included all of the new system except one in the Unit 3 turbine building, and one 4-depth free-field downhole array at the Service Hall of the old system (TEPCO, 2007a, 2007b).

The downhole array records at the Service Hall, the only ones that included “within” ground motions at the site, seem to be particularly important not only to determine the input rock outcrop motions for analytically reviewing dynamic behavior of the critical buildings but also to estimate nonlinear dynamic soil properties at the site under high confining pressures that are difficult to observe in the conventional laboratory test but definitely required for the analysis. The objective of this paper is to estimate nonlinear soil properties and rock outcrop motion of the site based on an inverse analysis of the downhole array records using genetic algorithms combined with a one-dimensional equivalent linear response analysis.

2. SITE CONDITIONS AND OBSERVED RECORDS AT SERVICE HALL DOWNHOLE ARRAY STATION

The Kashiwazaki-Kariwa nuclear power plant is located along the coast on the north of Kashiwazaki city, about 16 km from the epicenter. Figure 1 shows a map of the site. The plant is not only one of the largest nuclear power plants in the world, having seven generators (Units 1 - 7) with a total output capacity of 8.2 GW, but also the first one that experienced strong ground shaking. The Service Hall downhole array and its observation station are located on the east of the main gate, as indicted in Fig. 1.

Figure 2 shows the geological and geophysical logs along with the location of the downhole accelerometers. The elevation of the site is 67.5 m, which is 62.2 m or 55.2 m higher than those (5.3m or 12.3 m) of Units 1-4 or 5-7. The Holocene and Pleistocene sand dune deposits overlie the Pleistocene Yasuda Formation that in turn overlies the Pliocene Nishiyama Formation. The shear wave velocities, V_{so} of the Holocene sand dune (New sand dune), Pleistocene sand dune (Banjin Formation), Yasuda Formation, and Nishiyama Formation are 310, 310-350, 350, and 500-640 m/s, respectively. Their mass densities are estimated to be 1.65, 1.65-1.80, 1.80, and 1.65-1.75 Mg/m^3, respectively.

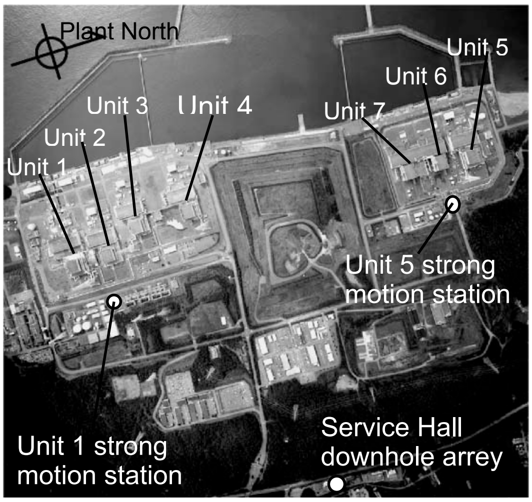

Figure 1. Aerial photograph of Kashiwazaki-Kariwa nuclear power plant (GSI, 2007)

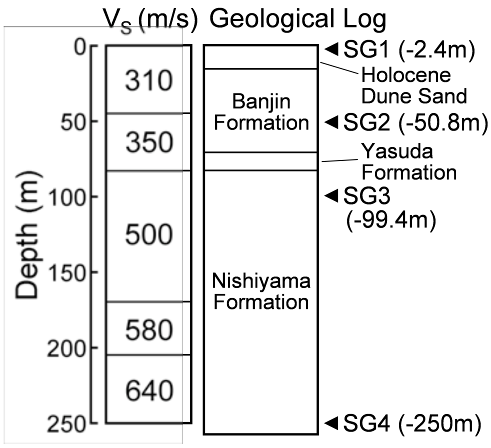

Figure 2. Geological and geophysical logs with accelerometer locations

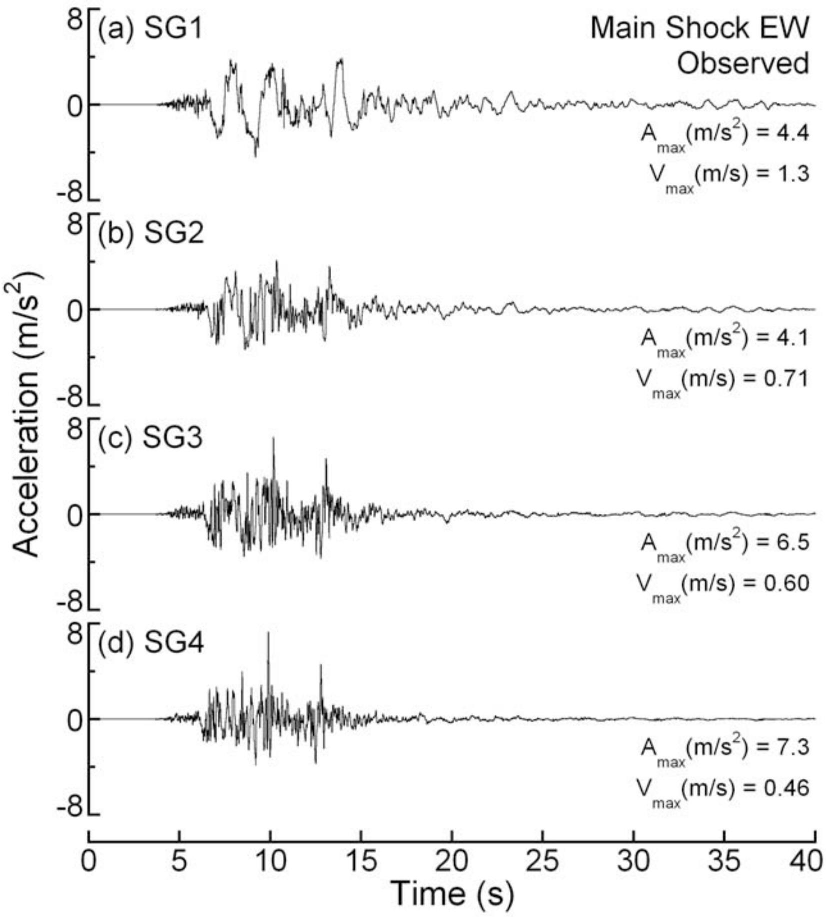

Figure 3. EW acceleration time histories observed by the downhole array during the main shock

The Service Hall downhole array includes four three-component accelerometers installed at depths of 2.4 m, 50.8 m, 99.4m, and 250 m, as shown in Figure 2. The NS and EW directions of the accelerometers were set to the two principal axes of the plant buildings and thus were rotated clockwise 18.9 degrees from the true ones. The ground water table was located at a depth of about 35 m. The 2007 Niigata-ken Chuetsu-oki earthquake caused a ground settlement of about 15 cm around the observation station at the Service Hall. No apparent sign of soil liquefaction was identified nearby.

Downhole array recordings for the main shock as well as six aftershocks are available (TEPCO, 2007a-2007c). The ground motions during the main shock dominate in the E-W direction, which is nearly perpendicular to the coastline. Figure 3 shows the EW acceleration time histories observed by the downhole array during the main shock. The peak accelerations were de-amplified from 7.3 m/s^2 at a depth of 250 m to 4.4 m/s^2 near the ground surface, losing their short period component, whereas the peak velocities were amplified from 0.46 m/s to 1.3 m/s. Figure 4 shows the distributions of peak horizontal ground acceleration with depth for the main shock together with those for the two aftershocks that occurred at 15:37 hrs and 21:08 hrs on the same day, herein called aftershocks L and S. The de-amplification of acceleration towards the ground surface occurred only during the main shock. Figure 5 compares the amplitude ratios between 2.4 m and 250 m depths for the main shock and two aftershocks. The amplitude ratios at periods less than 1 s are significantly lower in the main shock than in either of the two aftershocks. The above findings in Figures 3 to 5 suggest that degradation of stiffness and/or increase in damping of the near-surface soil might have occurred and lowered the amplitude ratios in the short period range during the main shock, compared with those of the aftershocks.

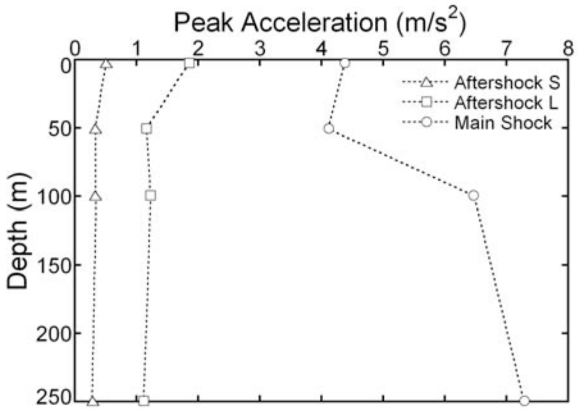

Figure 4. Distribution of peak acceleration with depth for three earthquakes

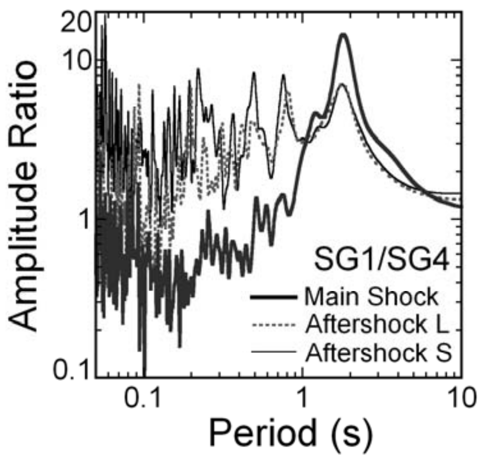

Figure 5. Comparison of amplitude ratios from different earthquakes

3. INVERSE ANALYSIS OF STRONG MOTION ARRAY RECORDS USING GENETIC ALGORITHMS

Nonlinear soil characteristics are back-calculated for the deposit of the Service Hall at the Kashiwazaki-Kariwa nuclear power plant based on its strong motion downhole array records during the 2007 Niigata-ken Chuetsu-oki earthquakes. The goal of this inversion is to find a soil layer model that minimizes the misfit of observed and computed Fourier amplitude ratios between any of the two depths in the array (e. g., Kobayashi et al, 1999), defined as:

$$F = \sum_{i=1}^{I-1} \sum_{j=i+1}^{I} \sum_{k=k_{\min}}^{k_{\max}} w_k^2 (\log_{10} A_{\mathrm{m},ij}(f_k) - \log_{10} A_{\mathrm{C},ij}(f_k))^2 \tag{1}$$

in which A_m and A_C are the observed and computed Fourier amplitude ratios between the i-th and j-th accelerometers in the array, I is the number of accelerometers, k_{min} and k_{max} are integers defined as f_{min}/T and f_{max}/T in which f_{min} and f_{max} are the minimum and maximum frequencies to be considered, T is the duration of the earthquake and w_k is a weighting factor defined as $1/f_k$.

It is assumed that A_C in the above equation be determined with a one-dimensional equivalent-linear response analysis of a deposit in which damping ratios are dependent on Fourier amplitude of shear strain in the frequency domain (e.g., Sugito et al, 1994), which is an extended version of SHAKE (Schnabel et al, 1972) to improve its deficit in over-damping in the short period range during strong shaking. It is also assumed that the target soil deposit consists of N sub-layers including the bottom half space, each characterized by the mass density, thickness, equivalent shear wave velocity, and damping ratio in the frequency domain defined as:

$$h(f) = h_{\min} + (h_{\max} - h_{\min})(\gamma_{\mathrm{eff}}(f)/\gamma_{\mathrm{ref}}) / (1 + \gamma_{\mathrm{eff}}(f)/\gamma_{\mathrm{ref}}) \tag{2}$$

$$\gamma_{\mathrm{eff}}(f) = 0.8\,\gamma_{\max} \quad \Gamma(f)/\Gamma_{\max}(f) \tag{3}$$

in which h_{min} and h_{max} are the minimum and maximum damping ratios; γ_{ref}, γ_{max}, $\gamma_{eff}(f)$, $\Gamma(f)$ and $\Gamma_{max}(f)$ are the reference shear strain, maximum shear strain in the time domain, effective shear strain for a given frequency f, Fourier amplitude of shear strain at a given f, and the maximum Fourier amplitude of shear strain in the frequency domain, respectively. Thus, once knowing all the soil properties in the deposit, A_C in Eqn. (1) can be determined by iterative procedure until $h(f)$ becomes compatible with Fourier amplitude of shear strain.

Adopted in the optimization using Eqn. (1) are genetic algorithms (GA; Goldberg, 1989, Kobayashi et al., 1999) in which four parameters including the equivalent shear wave velocity, minimum and maximum damping ratios and reference shear strain of each sub-layer are sought with other parameters such as the thickness and mass density being predetermined and with N=15, I=4, T=81.92 s, f_{max}=25 Hz, and f_{min}=0.2 Hz.

In the GA space, an 8-bit Gray coded integer is used for each of the unknown parameters. This leads to a 480 (8x15x4)-bit integer (chromosome) for an individual soil layer model consisting of 15 sub-layers with four unknown parameters each. An initial population of 200 soil layer models is generated randomly, covering the range of possible solutions, and the succeeding generation of the same population is reproduced until the 500th generation. The parameter search ranges are 0-5% for h_{min}, 15-40% for h_{max}, 10^{-4}-10^{-2} for γ_{ref} and $(0.05\text{-}0.5)V_{so}$-$(0.7\text{-}1.2)V_{so}$ for V_s. Roulette wheelselection is used to choose and mate a pair for the new generation based on the fitness of each individual soil layer model defined by 1/F, with a crossover rate of 0.7 and a mutation rate of 0.02. The soil layer model having the best fitness in the final generation is assumed to be the solution for one trial. A total of ten trials are made for each set of the array data observed during the main shock and the two aftershocks L and S.

4. NONLINEAR SOIL PROPERTIES AND ROCK OUTCROP MOTION ESTIMATED FROM INVERSE ANALYSIS

Figures 6 to 8 compare the Fourier amplitude ratios computed for the back-calculated soil layer models having the best fitness with those of the observed records for the aftershocks S and L and the main shock. A good agreement exists between the observed and computed amplitude ratios for the three events, indicating that the back-calculated soil profiles are reasonably reliable.

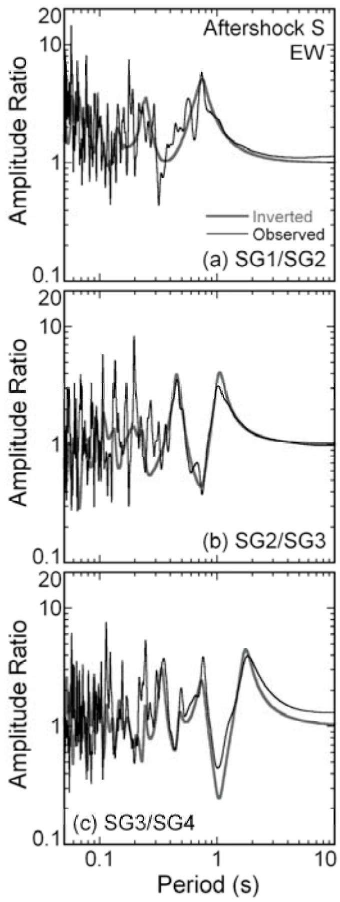

Figure 6. Observed and computed amplitude ratios for aftershock S

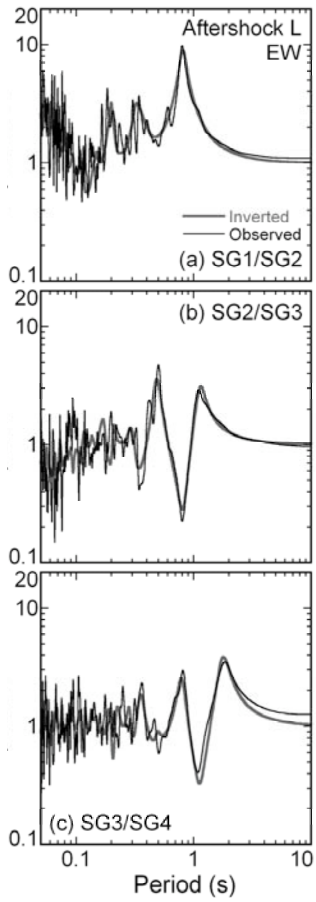

Figure 7. Observed and computed amplitude ratios for aftershock L

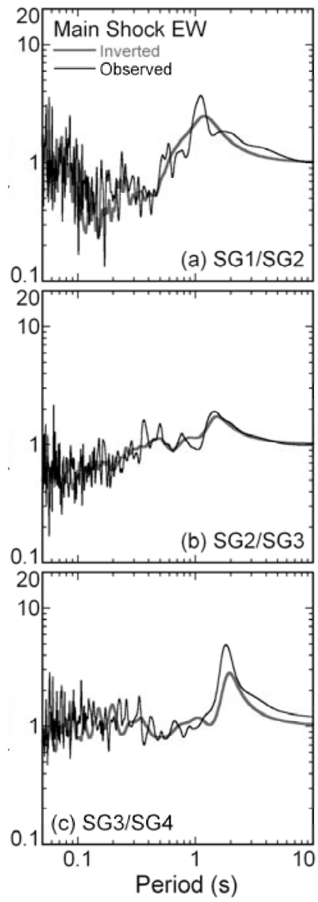

Figure 8. Observed and computed amplitude ratio for main shock

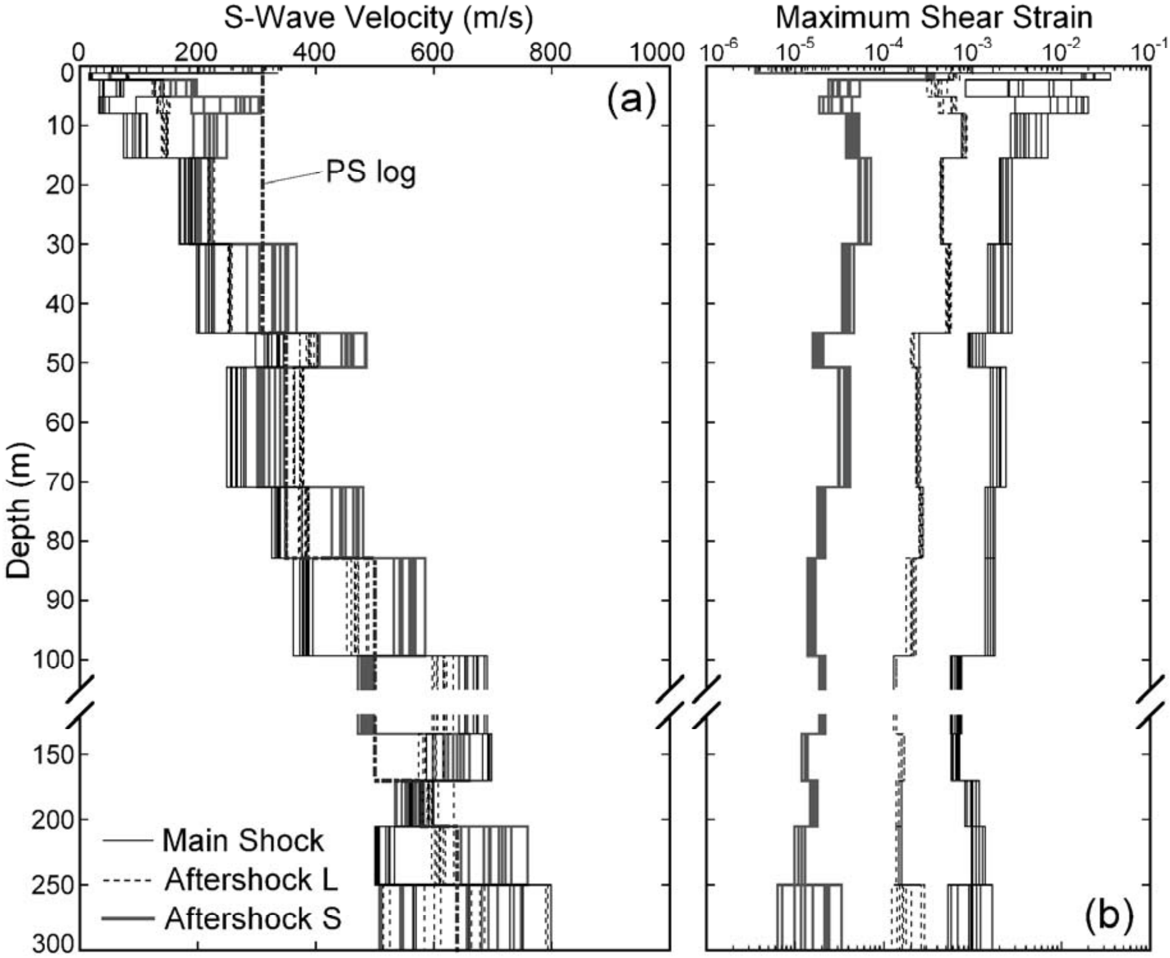

Figure 9. Back-calculated Vs profile and shear strain

Figure 9 shows the distribution of back-calculated equivalent shear wave velocity and maximum shear strain with depth for the three events compared with an available V_s profile determined by PS logging. The maximum shear strains during the main shock are either one or two orders of magnitude greater than those of the aftershock L or S. The estimated shear wave velocities at depths smaller than about 70 m are therefore significantly smaller in the main shock than in either of the two aftershocks. In contrast, those at deeper depths for the three events are almost identical. The back-calculated shear wave velocities at deeper depths are consistent with the available shear wave velocity profile, whereas those at the shallow depths even for the aftershock S are significantly smaller than the available V_s values. This poses a question about the accuracy of the available V_s profile at the shallow depths.

Figure 10 shows the back-calculated strain-dependent shear modulus and damping ratios at three depths for the three earthquakes. The shear modulus has been normalized with respect to the elastic shear modulus estimated using $G_o = \rho V_s^2$ in which V_s is the average of the back-calculated values for the aftershock S. The shear modulus ratios in the sand dune deposits at depths smaller than about 70 m decrease to 0.01-0.8 and their damping ratios increase to about 20-35% with increasing shear strain up to $2\text{x}10^{-3}$ - $3\text{x}10^{-2}$ or with increasing ground shaking. Also shown in the figures are the laboratory test data for sand tested under confining pressures (Kokusho, 1980) similar to those of the dune sands. The back-calculated strain-dependent shear modulus ratios are consistent with those of the previous study but the back-calculated damping ratios are slightly higher than those of the previous study. The shear modulus and damping ratios at depths greater than 70 m are about 1.0 and less than 5% and do not show any significant change irrespective of shear strain amplitude that varies depending on ground shaking from $1\text{x}10^{-5}$ to $2\text{x}10^{-3}$.

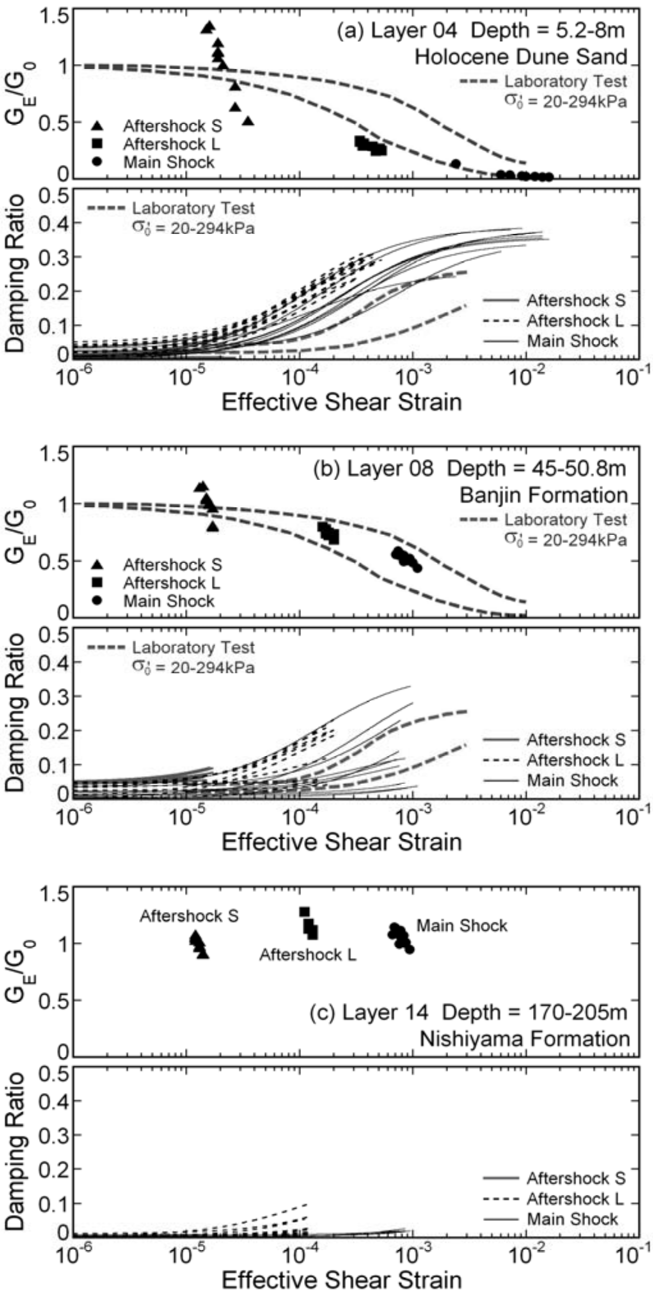

Figure 10. Back-calculated strain-dependent shear moduli and damping ratios

5. CONCLUSIONS

The strain-dependent shear modulus and damping ratios have been estimated based on the inverse analysis of the strong motion downhole array records at the Service Hall of the Kashiwazaki-Kariwa nuclear power plant during the 2007 Niigata-ken Chuetsu-oki earthquakes. The following conclusions may tentatively be made:

1) The computed spectral ratios and acceleration time histories show a good agreement with the observed ones, indicating that the back-calculated nonlinear soil properties and rock outcrop motion are reasonably reliable.

2) The Holocene and Pleistocene dune sands with V_s = 200-350 m/s at depths smaller than about 70 m showed strong nonlinear behavior with shear modulus ratio of about 0.01-0.8 and damping ratios of about 20-35 % at

maximum shear stains of about $2x10^{-3}$ - $3x10^{-2}$, during the main shock.

3) The back-calculated shear modulus ratios at depths smaller than 70 m during the main and aftershocks are consistent with previous laboratory test results for sand consolidated under similar confining pressures but the back-calculated damping ratios seem to be slightly higher than that of the previous studies.

4) The mudstone with V_s =500 m/s located at depths below 70 m did not show any strong nonlinearity during the main shock and aftershocks, with a shear modulus ratio of about 1.0 and a damping ratio of up to 5 % at a maximum shear strain up to about $5x10^{-4}$ - $2x10^{-3}$.

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