Transactions of the Korean Nuclear Society Autumn Meeting
Gyeongju, Korea, October 28-30, 2015

Site Response Analysis Using DeepSoil: Case Study of Bangka Site, Indonesia

Eko Rudi Iswanto [a*], Eric Yee [a]
[a] *KEPCO International Nuclear Graduate School (KINGS)*
1456-1 Shinam-ri, Seosaeng-myeon Ulju-gun, Ulsan, South Korea
[*] *Corresponding author: e_rudii@yahoo.co.id*

1. Introduction

Based on the long-term energy planning of Indonesia, it is concluded that the energy demand in year 2025 will become twofold compared to the energy demand in year 2000. Many studies reported that in order to cope with the fast-growing demand, the application of energy mix needs to be introduced. Therefore, Indonesia is considering building Nuclear Power Plant (NPP) by 2025. Indonesia government declared through Act No. 17 year 2007 on the National Long-Term Development Plant Year 2005-2025 and Presidential Decree No. 5 year 2006 on the National Energy Policy (Indonesia 2007; Indonesia 2006), that nuclear energy is stated as a part of the national energy system. In order to undertake the above national policy, National Nuclear Energy Agency of Indonesia, as the promotor for the utilization of nuclear energy will conduct site study, which is a part of infrastructure preparation for NPP construction.

Thorough preparation and steps are needed to operate an NPP and it takes between 10 to 15 years from the preliminary study (site selection, financial study, etc.) up to project implementation (manufacturing, construction, commissioning). During project implementation, it is necessary to prepare various documents relevant for permit application such as Safety Evaluation Report for site permit, Preliminary Safety Analysis Report and Environment Impact Assessment Report for construction permit.

Considering the continuously increasing electricity energy demand, it is necessary to prepare for alternative NPP sites. Within the framework of identifying NPP sites, site surveys are performed in West Bangka Island of Bangka-Belitung province. The safety requirements of NPP's are stringent; amongst the various requirements is the ability to safely shut down in the wake of a possible earthquake. Ground response analysis of a potential site therefore needs to be carried out, parameter that affect the resistance of an NPP to earthquakes such as peak strain profiles is analysed. The objective of this paper is to analyse the ground response of the selected site for a NPP, using The M_w 7.9 in Sikuai Island, West Sumatra on September 12, 2007 as present input motion. This analysis will be carried out using a ground response analysis program, DeepSoil. In addition to this, an attempt was made to define the site specific input motion characteristics of the selected site for use in DeepSoil (DeepSoil 5.0).

2. Site Description

West Bangka (WB) site is selected as the preferred area for the first NPP sites for some reasons in terms of their acceptability such as safety, suitability, and construction cost, and other considerations. It is situated in Bangka Island of Bangka-Belitung Island Province which is located at 104° 50' - 109° 30' E and 0° 50' - 4° 10' S and can be seen in Figure 1.

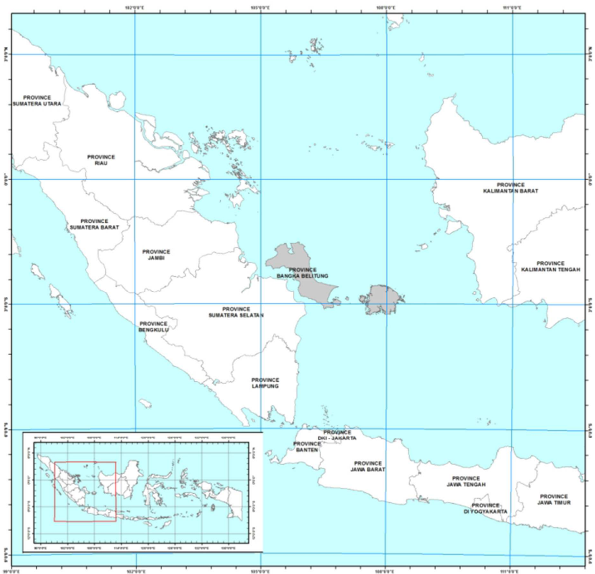


Figure 1. Map of Bangka-Belitung Island Province, Indonesia

According to literature study and field confirmation, WB site is free from exclusions factors and therefore they can be considered as area which is potential to be developed further. It has been considered as a stable area among other islands in Indonesia with relatively low seismicity and there are no significant earthquakes in this area. The value of peak ground acceleration on the base rock for the past 500 years period is based on Manual for Seismic Resistance Designing for Building Construction (SNI, 2002) No. 1726 is very small, about 0.03 g.

2.1 Results of Geotechnical Exploration

A site exploration program was performed at WB site that included the drilling of boreholes, in-situ testing such as standard penetration testing (SPT) with energy measurements, and suspension logging of seismic velocities, and laboratory testing. In this project, geotechnical site investigation data are collected from the borehole, namely BBH 06 where the reactor will be placed. The location of BBH 06 at the WB Site is shown in the Figure 2. The borehole was drilled using rotary wash procedures with a diameter of 116 mm for the upper 20 m and a diameter of 86 mm thereafter.

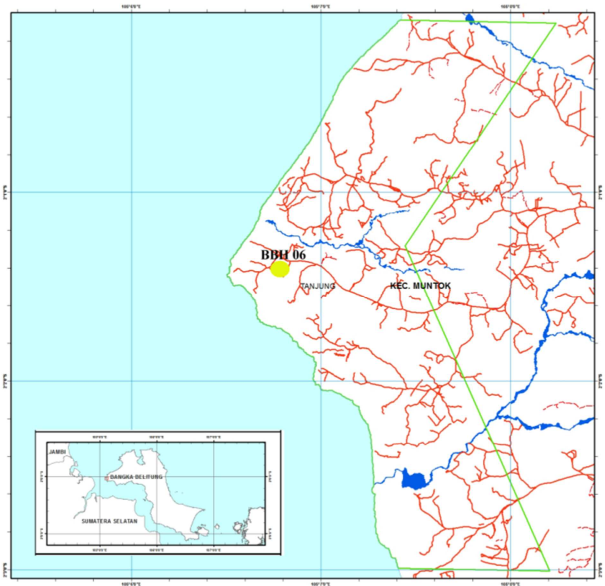


Figure 2. The location of BBH 06 at the WB Site

The SPT is performed during a soil boring to obtain an approximate measure of the dynamic soil resistance. The procedures for the SPT are detailed in ASTM D 1586 (ASTM 2000). The SPT involves the driving of a hollow thick-walled tube into the ground and measuring the number of blows to advance the split-barrel sampler a vertical distance of 300 mm. A drop weight system is used for the pounding where a 63.5-kg hammer repeatedly falls from 0.76 m to achieve three successive increments of 150-mm each. The first increment is recorded as a “seating”, while the number of blows to advance the second and third increments are summed to give the N-value ("blow count") or SPT-resistance (reported in blows/0.3 m). Table 1 shows the result of SPTs at BBH 06.

Geophysical methods are used in geotechnical investigations to evaluate a site's behaviour in a seismic event. The dynamic response of that soil can be estimated by measuring a soil's shear wave velocity. There are a number of methods used to determine a site's shear wave velocity such as PS suspension logging, surface wave reflection, surface wave refraction, and multichannel analysis of surface waves. For this project BBH 06 is prepared using SPT, then PS suspension logging was used to measure shear wave (S wave) and compression wave (P wave). The geologic log and the PS suspension logging of BBH 06 can be seen at Figure 3.

Table 1. Results of index tests

Depth (m)	SPT blow counts, N
3	13
6	9
9	12
12	23
15	52
17.55	54
20.55	49
23.55	56
26.55	58
30	>50

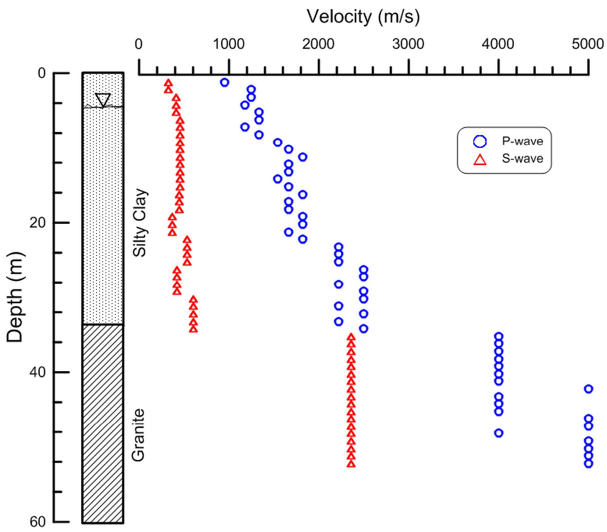


Figure 3. Geologic log at BBH 06 and results of penetration and suspension logging geophysical testing.

2.2 Laboratory Testing

Laboratory testing of soils is a fundamental element of geotechnical engineering. Soil samples that are collected during the drilling of borehole were tested using sieve analysis, plastic limit, specific gravity and direct shear test for physical, chemical and engineering characteristics. The results of index tests including grain size distribution, atterberg limits, and maximum and minimum unit weight and the results of direct shear tests are shown in the Table 2 and Table 3 respectively.

Table 2. Results of index tests

Depth (m)	Unit weight (kN/m^3)	Specific gravity	Water content (%)	Grain size distribution	Atterberg limit
0 - 34	17.96	2.63 – 2.68	27.33 – 29.10	Sand: 39.42 – 43.12%	WL: 59 – 71%
				Silt: 35.6 – 44.88%	Ip: 12 – 40%
				Clay: 12 – 24%	
34 - 52	18.22	2.63 – 2.64	0.45		

Table 3. Results of shear tests for BBH 06

Test	Angle of internal friction - ϕ (°)	Apparent cohesion- c (kN/m^2)
UU Triaxial	7–29	24.52–48.05
Direct Shear	7–29	20.59–41.19

3. Input Motion

For present analysis, The Sumatra Earthquake, so called Sikuai2 with M_w 7.9 in Sikuai Island, West Sumatra on September 12, 2007 is used as input motions. This earthquake occurred off the southern coast of Sumatra about 167.7 km at 2.5060ºS, 100.9060 ºE with a depth of 30.0 km. It has PGA value 0.13g. Record of accelerograph of horizontal component of input motion at PSKI station is shown at Figure 4.

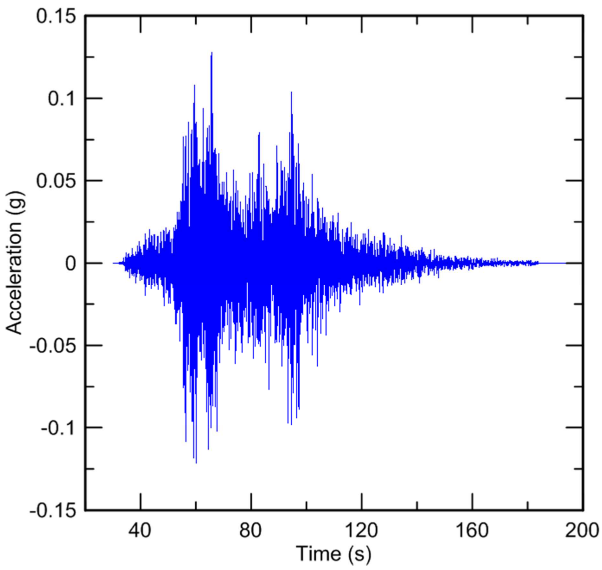


Figure 4. Acceleration histogram recorded at PSKI station during Sumatra Earthquake, NS component

4. Ground Response Analysis

Ground response analysis is the process of calculating the shear wave propagation due to seismic loading through borehole BBH 06 at the WB site. The wave propagation problem is solved using one-dimensional analysis which is simpler and significantly reduce the required computational effort. One dimensional ground response analysis is mainly performed using an equivalent linear (EQL) method in which the wave equation is solved in the frequency domain and a nonlinear (NL) method in which employing non-linear hysteretic soil models is solved in the time domain using numerical integration (Kramer 1996). These methods have been carried out using DeepSoil, a one-dimensional site response analysis program that can perform EQL method and NL method. It can feature a spontaneous graphical user interface and has capability of deriving a number of strong motion parameters often required for engineer.

4.1. One-dimensional analysis

One-dimensional analysis is based on the assumption that all boundaries are horizontal and that the response of a soil deposit is generally caused by SH-waves propagating in the vertical direction from the underlying bedrock. The soil and bedrock surface are considered to extend infinitely in the horizontal direction.

After a fault ruptures below the earth's surface, body waves travel away from the source in all direction. They are reflected and refracted as they reach boundaries between different geologic materials. By the time the rays reach the ground surface, multiple refractions have often bent them to a nearly vertical direction.

4.1.1. Equivalent Linear (EQL) method

Iddris and Seed (1967) first proposed the EQL method for ground response analysis that calculates an approximate nonlinear response through a linear analysis with soil layer properties adjusted to account for the softening during seismic loading. The layer properties are adjusted through an iterative process involving a series of linear analyses that can be performed either in the frequency or the time domain. The value of shear modulus and damping ratio is used to calculate the linear soil behaviour then the peak strains in the soil layers are computed. An effective shear strain is then calculated for each layer by multiplying the peak shear strain by an effective shear strain ratio. This strain value is used to determine modulus reduction and damping ratio of each layer.

4.1.2. Nonlinear (NL) method

The NL method simulates the hysteretic stress-strain response of the soil. It is capable of representing the actual behaviour of soils much more accurately and more realistic than the EQL method. The soil profile can be modelled using either lumped masses or finite element. In the lumped-mass approach, the soil layers are lumped into adjacent nodal masses, which are connected by springs that model the soil stress-strain behaviour in shear. The input ground motion is applied at the base of the borehole, and the dynamic equations of motion are integrated using the Newmark-β method in order to calculate the response of the soil layers. Similar models can be built using finite element program using the available element types and material models. The hysteretic material models are characterized by (1) the backbone curve, and (2) a set of hysteresis rules.

The base of the soil profile can be modelled either as a transmitting or reflecting boundary. It depends on the type of ground motion input and the impedance ratio at the base of the soil profile. The transmitting boundary condition is applied by modelling the bedrock using viscous damper that absorb the radiating energy, and the input ground motion is applied as a history of shear force (Lysmer, 1978). The reflecting boundary condition is modelled by directly applying the input acceleration time series at the base.

4.2. Dynamic soil properties and material modeling for analysis

The borehole BBH 06 at WB site comprises a 34 m layer of silty clay (V_s = 328 – 601 m/s), and granite (V_s > 2000 m/s) thereafter. A unit weight of 17.96 KN/m^3 is assigned to a depth of 8 m and a saturated unit weight of 18.22 KN/m^3 was used from 12 to 34 m.

Site response analysis is performed using a maximum frequency. The maximum frequency is the highest frequency that the layer can propagate and is calculated by the equation:

$$f_{max}=\frac{V_s}{4H} \qquad (4.1)$$

where V_s is the shear wave velocity of the layer, and H is the thickness of the layer. To increase the maximum frequency, the thickness of the layer should be decrease. For all layers, the maximum frequency should fall between a range of a minimum of 25 Hz and a maximum of 50 Hz.

Soil exhibit nonlinear behaviour, a hyperbolic relationship can be used to relate the shear stress and shearing strain in modelling dynamic soil behaviour (Hardin and Drnevich, 1972). In DeepSoil the hyperbolic model is used to define the backbone curve, which is given by the equation,

$$\frac{G}{G_{max}}=\frac{1}{1+\beta\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}} \qquad (4.2)$$

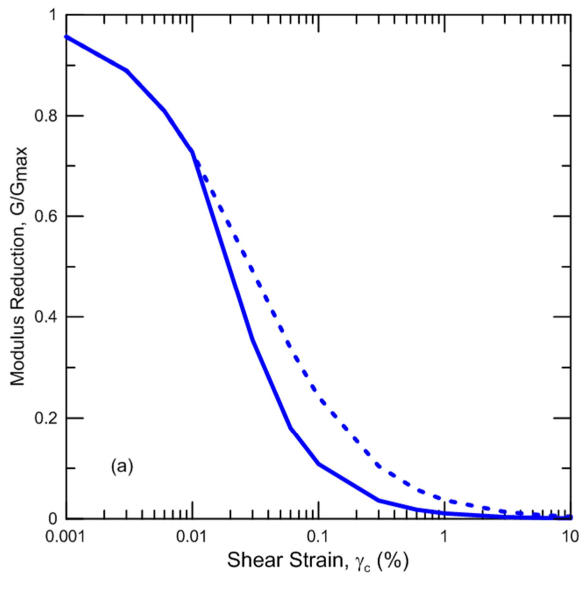
where $\frac{G}{G_{max}}$ = modulus reduction; γ = shear strain; γ_r = pseudo reference strain; and α = 0.92 and β = 1 as fitting coefficient (Darendeli 2001). The pseudo reference strain describes the backbone curve at small strains (γ < ~0.3-0.5%). However, the hyperbolic model breaks down at large strains, where it tends to produce biased shear strength estimation.

4.3. Modification of Backbone Curve

An adjustment procedure of soil backbone curve is utilized to capture a specific shear strength at large strain while conserving the small strain behaviour (Yee et al. 2013). For $\gamma < \gamma_l$, a first hyperbola is constructed, where γ_l is a user-specified transitional shear strain, while $\gamma > \gamma_l$, a second hyperbola is used having an initial modulus that is the tangent modulus of the first hyperbola at γ_l. The second hyperbola moves toward the shear strength (τ_{ff}) at large strain. The shear strength is taken as $c+\sigma' \tan\phi$, where c and ϕ are taken from laboratory testing as 0.49 and 29° respectively. The modulus reduction is calculated as

$$\frac{G}{G_{max}}=\frac{\frac{\gamma_l}{1+(\gamma_l/\gamma_r)^{\alpha}}+\frac{(G_{\gamma l}/G_{max})\gamma'}{1+(\gamma'/\gamma'_{ref})}}{\gamma} \qquad (4.3)$$

In this paper, new values of shear strain and damping are used to generate modulus reduction and damping curve as a target curve for each layer. Then, the fitting procedure in DeepSoil is performed for each sub layer of the soil model. This procedure is utilized a target curve as given previously. The new backbone curves are input into DeepSoil as shown in Figure 5.



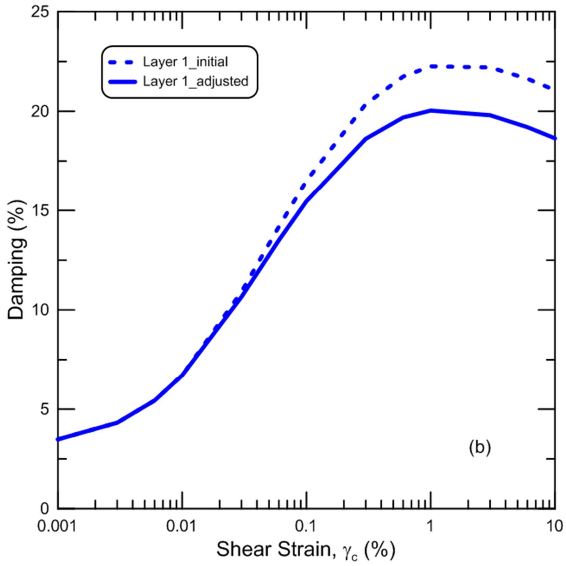


Figure 5. Comparison of the initial and strength-adjusted of (a) modulus reduction and (b) damping model

4.4. Ground response analysis (result and observation)

Ground response analyses are performed for BBH 06 subjected to the corresponding Sikuai2 input motions, strength-adjusted backbone and damping curves using DeepSoil. Responses are presented in term of surface acceleration response spectra and the peak strain profiles as can be seen in Figure 6 and 7 respectively.

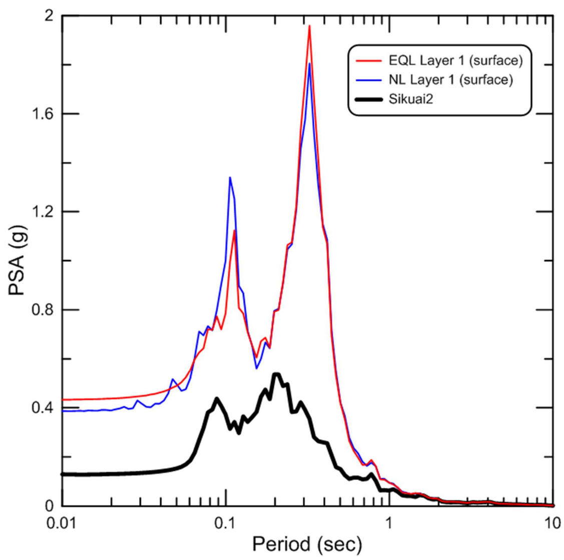


Figure 6. Surface acceleration response spectra for EQL and NL method subjected to Sikuai2

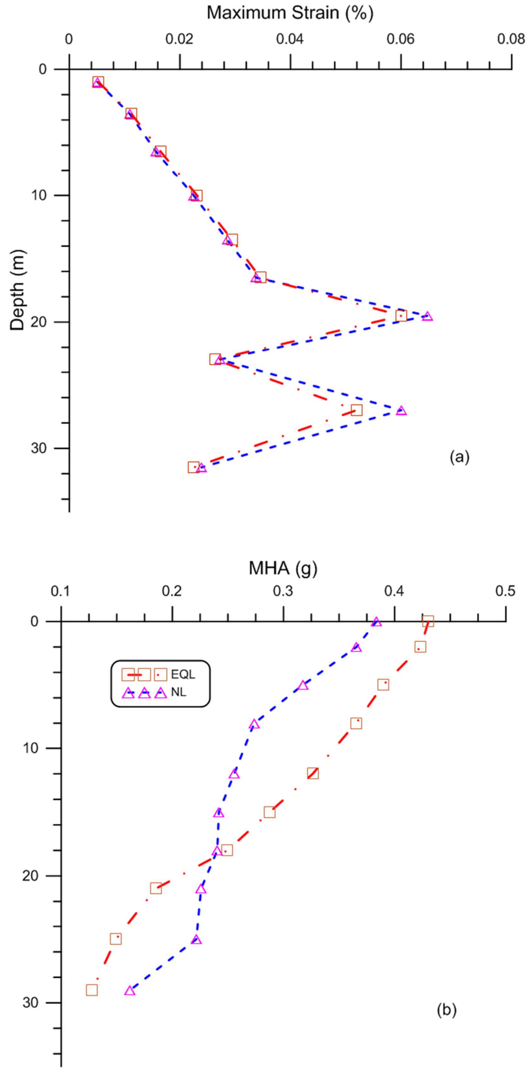


Figure 7. Results of NL and EQL method of ground response analyses (a) maximum strain and (b) maximum horizontal acceleration

The difference between EQL and NL responses varies with soil profile, boundary type and the ground input motion. The following observation can be made by comparing EQL and NL responses.

1. In general, results of the analyses with EQL method show similar trends to those with NL method.
2. The peak strain profiles calculated using EQL method are close to those calculated by NL method on a rigid boundary. The peak strain profiles show many spikes through the depth of soil profile. One of the highest spikes is formed by strain in the soil layer at depth 18-20m which has smaller shear wave velocity compare to other layers.
3. At the surface layer, the acceleration values for EQL and NL methods are resulted as 0.425g and 0.375g respectively, when Sikuai2 is used as input motion.

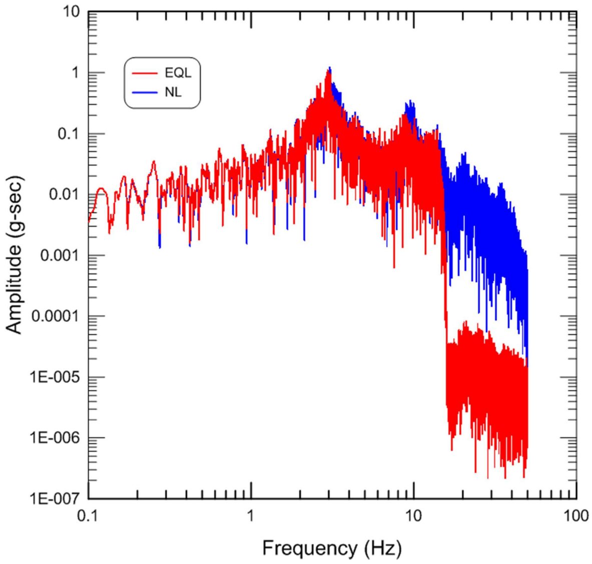


Figure 8. Fourier transforms of the surface spectral accelerations subjected to Sikuai2 input motions in EQL and NL method

Figure 8 illustrates Fourier transforms of the surface spectral accelerations subjected to Sikuai2 input motion. The spectral accelerations calculated using EQL method is increasingly different at shorter period. It indicates EQL method is unable to reproduce the higher frequency response. The EQL method involves a linear analysis with a constant shear modulus for each layer. On the other hand, NL method analyses the shear modulus of each layer and may range from the low-strain to the high-strain shear modulus. Spectral acceleration in the higher frequencies is amplified and the continuous changing of soil properties (shear modulus) also excites the higher vibration modes. These modes are not captured in EQL method. Either rigid boundary or elastic boundary shows the same trend.

5. Summary and Conclusion

A site investigation at the WB site was performed primarily on the PS Logging Test (Downhole Seismic Method) and Standard Penetration Test results. The soil profiles consist of a 34 m layer of silty clay, and granite thereafter. The shear wave velocity varies each layer. The groundwater is known at depth of about 4 m. The EQL and NL ground response method was modelled with DeepSoil using dynamic soil properties and Sukai2 input motion was subjected to WB site.

When the ground response analyses show large-strain response, backbone curves should be adjusted to predict the shear strength at large strains. This correction will be needed for both method, either EQL method or NL method. For research purposes in the feature, we suggest to use local input motion so the ground response from analysis reflects the real condition of the site.

6. Acknowledgements

This research was supported by the 2105 Research Fund of the KEPCO International Nuclear Graduate School (KINGS), Republic of Korea.

REFERENCES

ASTM. (2000). "Standard test method for standard penetration test and split-barrel sampling of soils," *D1586*, West Conshohocken, PA.

Darendeli, M. (2001). “Development of a new family of normalized modulus reduction and material damping curves.” Ph.D. dissertation, Dept. of Civil Engineering, Univ. of Texas, Austin, TX.

DeepSoil 5.0 [Computer software]. Urbana, IL, Univ. of Illionis.

Hardin, B. O., and Drnevich, V. P. (1972). “Shear modulus and damping in soil.” J. Soil Mech. and Found. Div., 98 (7), 667-692.

Iddriss, I.M., Seed, HB., (1967). Response of Horizontal Soil Layers During Earthquakes. Soil Mechanics and Bituminous Material Research Laboraatory, University of California, Berkeley, Berkeley, CA.

Indonesia, (2006). The Presidential Decree No. 5 Year 2006 on the National Energy Policy. Jakarta

Indonesia, (2007). The Act No. 17 yaer 2007 on the National Long-Term Development Plant Year 2005-2025. Jakarta

Kramer, S.L., (1996). Geotechnical Earthquake Engineering. Prentice Hall, Upper Saddle River, New Jersey.

Lysmer, J., (1978). Analytical Procedures in Soil Dynamics. Earthquake Engineering Research Center, University of California, Berkeley, Berkeley, California.

Manual for Seismic Resistance Designing for Building Construction (SNI). (2002). "Standar Perencanaan Ketahanan Gempa Untuk Struktur Bangunan Gedung, " No. 1726. Bandung: DPU

Yee, E., Stewart, J.P., Tokimatsu, K., (2013). Elastic and large-strain nonlinear seismicsite response from analysis of vertical array recordings. J. Geotech. Geoenviron. Eng. 139, 1789–1801