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Site Response Analysis Using Forced-Vibration Tests on Hydraulic-Filled Soil Deposit

Dong-Soo Kim

Korea Advanced Institute of Science and Technology, Korea

Jin-Sun Lee

Korea Advanced Institute of Science and Technology, Korea

SITE RESPONSE ANALYSIS USING
FORCED – VIBRATION TESTS ON HYDRAULIC – FILLED SOIL DEPOSIT

Kim, Dong-Soo
Department of Civil Engineering
Korea Advanced Institute of Science and Technology
373–1, Kusong–dong, Yusong–gu, Taejon 305–701
Korea

Lee, Jin-Sun
Department of Civil Engineering
Korea Advanced Institute of Science and Technology
373–1, Kusong–dong, Yusong–gu, Taejon 305–701
Korea

ABSTRACT

Forced vibration tests and detailed site characterization were performed at a geotechnical experimental site in Yong–jong Island where Inchon International Airport being constructed. The vibratory motions generated by the combination of hydraulic hammer compaction and dynamic compaction were monitored by 3–component velocity transducers in the down hole array as well as on the ground surface. Spectral analyses of the recorded vibratory motions were performed to evaluate the resonant frequencies of the experimental site. The linear analysis and simplified method were also performed to evaluate the resonant frequencies of the experimental site based on the results of detailed site investigation. The resonant frequencies of the experimental site evaluated by spectral analyses based on forced vibration tests were in good agreement with those of the linear analysis and the simplified method.

KEYWORDS

Forced vibration tests, Hydraulic–filled deposit, Site characterization, Site response analysis, Spectral analysis.

INTRODUCTION

One of the most important and most commonly encountered problems in geotechnical earthquake engineering is the evaluation of ground response. The estimation of ground response during the earthquake have been performed by using analytic and numerical ground response analysis. The resonant period of the site is the most influential parameter in the amplification of soil strata during the earthquake(Kramer, 1996). The resonant period of the site is usually estimated by numerical analysis or simplified wave propagation equation. The engineering soil parameters used for ground response analysis can be acquired by field and laboratory tests, but the difficulty of using such parameters always exists. The uncertainties in determining of soil parameters can be classified as follows: 1. disturbance of soil specimen during the sampling procedure. 2. inherent experimental error. 3. obstacles in the in-situ tests. Because of the above mentioned problems, if the resonant period of the site can be estimated directly from the forced vibration test, it will be more reliable than the numerical analysis results. For those reasons, many researchers have tried to represent the earthquake motion artificially using such as rotating mass, explosion and etc. (Negmatullaev, 1999, Carrubba and Maugeri, 1999) The forced vibration tests were performed by using the combination of hydraulic hammer and dynamic compactions in the hydraulic–filled soil deposit at Yong–jong Island where Inchon international Airport being constructed. The detailed site characterization was performed using SPT, CPT and shear wave velocity measurements. The particle motions measured in the downhole array as well as on the ground surface during the forced vibration tests were used for estimating the natural period of the experimental site and the estimated periods of the site were compared with the results determined by the linear analysis and the simplified method.

TEST SITE

Geotechnical experiment site was constructed at Yong–jong Island where Inchon International Airport being constructed(Fig. 1). The site consisted of a reclaimed soil of about 6m, a weak alluvial clayey silt layer of about 20m, an alluvial stiff clay layer of about 15m, residual sandy soil, and bed rock (Fig. 2). The reclaimed layer, classified as SM, was required to be improved to resist a pavement structures for runway, taxiway and apron. The hydraulic hammer compaction with a tamper of 10ton and a drop height of 1.2m was employed to improve the reclaimed layer minimizing the size of the disturbed craters. The seismic test; SASW and down–hole tests were performed to acquire the shear wave velocity profile of the test site. The in–situ seismic tests results indicate increasing values of V_s with depth, from a minimum 100m/sec to a maximum 200m/sec in hydraulic fill and silty sand layer. In the clay layer, shear wave velocity is about 250~350m/sec. Standard penetration test with a energy ratio correction and cone penetration test were also performed to acquire the soil profile.

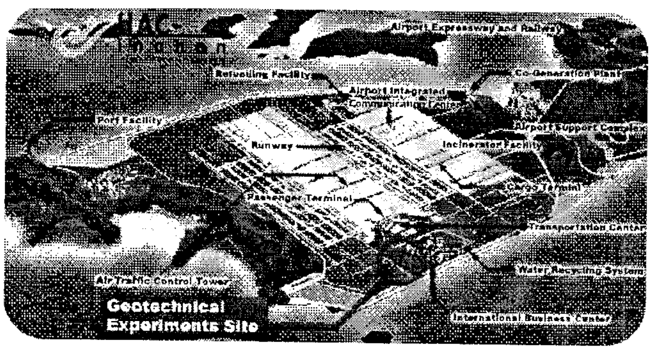


Fig. 1 Experimental site overview

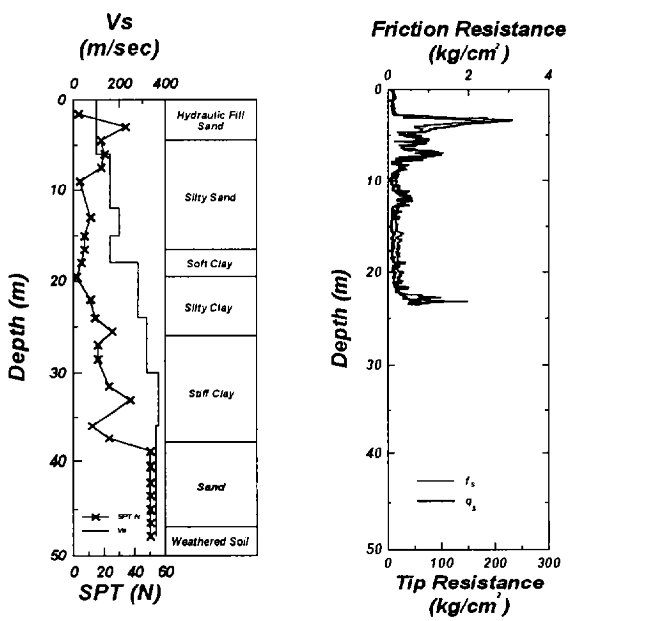


Fig. 2 Site profile determined by shear wave velocity, SPT N value and cone penetration resistance.

FORCED VIBRATION TEST

In order to represent the earthquake motion artificially, various forced vibration methods were employed(Carrubba and Maugeri, 1999, Negmatullaev et. al, 1999). The vibratory motion induced by rotating mass type footing usually has a narrow frequency bandwidth and the installation cost is very expensive. The explosion method generates the most similar vibratory motion compared with earthquake ground motion, but the explosion is somewhat dangerous and too much charge weight(about 10,000kg) is needed to simulate similar earthquake ground motion. Because of the above reasons, the dynamic compaction and hydraulic hammer compaction were selected for the vibratory source in this study. They are very simple and cost–effective vibratory sources to generate vibratory motion at the in–situ tests, but the generated ground motion was somewhat different from the earthquake ground motion. The stress waves generated by hydraulic hammer compaction and dynamic compaction mainly consist of body and surface waves and main energy is propagated by surface waves(Kim and Lee, 2000).

In order to generate the vibratory motion at the experimental site, the hydraulic hammer compaction and dynamic compaction were performed either simultaneously or separately at the ground surface of the site. The weight of the tamper and the drop height were presented in the Table 1.

Table 1. Specification of forced vibration sources

	Drop height (m)	Tamper weight (ton)
Hydraulic hammer	1.2m	10ton
Dynamic compaction	7 ~ 9m	7ton

The vibratory motions generated by hydraulic hammer and dynamic compactions were measured by 3–component velocity transducers in the down hole array as well as on the ground surface. The 3–component velocity transducer consists of three 4.5Hz velocity transducers installed in the three directions(vertical, longitudinal, transverse) to monitor 3–component vibratory motions. It is designed to be attached on the inner wall of the borehole by inflated rubber bag. The calibration curve for the 3–component geophone (with open shunt damping) which has a natural frequency of 4.5Hz is presented in Fig. 3.

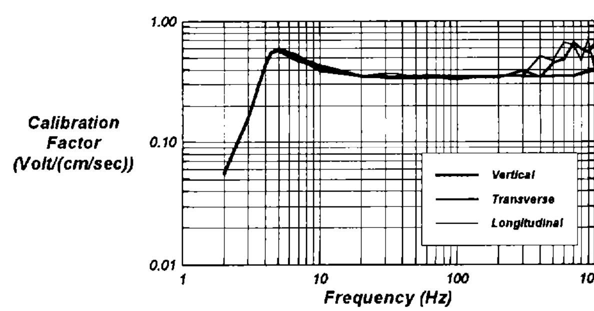


Fig. 3 Calibration curve for 3–component velocity transducer

Calibration factor is constant in frequencies approximately ranging from 10Hz to 500Hz, and the reliable range of vibration measurement using this transducer is about 4Hz and above. In order to measure the shearing strain of the soil layer as well as particle velocity, the three serial connected 3-component velocity transducers were used to construct down hole measurement array. The installation depth was selected to measure the shearing strain of silty sand layer in cooperated with excessive pore water pressure measurement. The forced vibration tests were performed using the following combinations in three ways: 1. Dynamic compaction only, 2. Hydraulic hammer compaction only, 3. Both dynamic hammer and hydraulic hammer compaction.

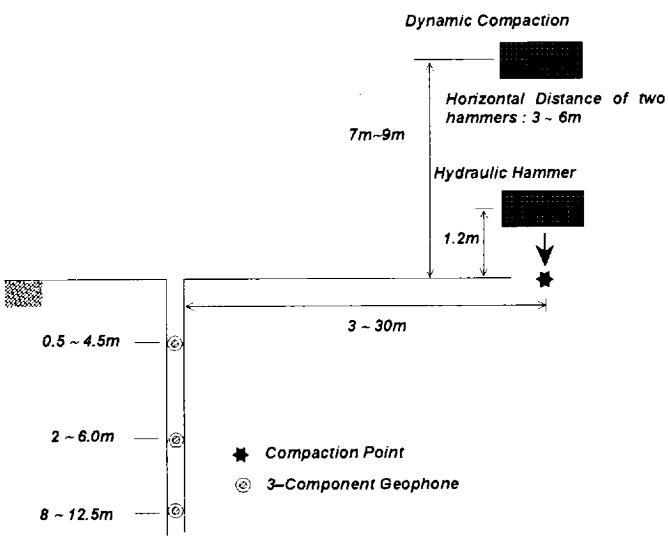
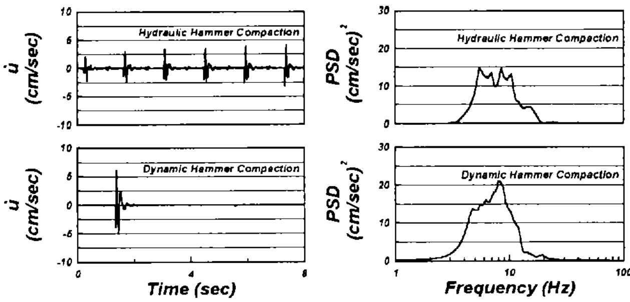
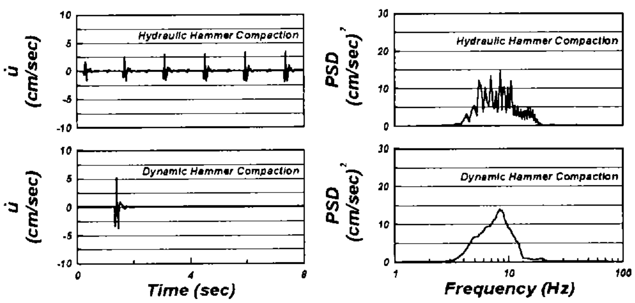


Fig. 4 Schematic diagram of experiments

Typical time records and Fourier spectra of the measured vibratory motion are presented in Fig. 5. From the measured signal at depth of 4m, the maximum particle velocities of longitudinal component vibratory motion at the 3m horizontal distance from source are about 3 ~ 6cm/sec and 6 ~ 7cm/sec for hydraulic hammer compaction and dynamic compaction, respectively. Most of the energy induced by hydraulic hammer compaction and dynamic compaction exists at frequencies between 4 ~ 20Hz. In case of the measured signal at depth 12m, the maximum particle velocities are about 2 ~ 4cm/sec and 6 ~ 7cm/sec for hydraulic hammer compaction and dynamic compaction, respectively. Most of the energy induced by both sources exists at the similar frequency ranges.



(a) horizontal distance from the source: 3m, depth : 4m



(b) horizontal distance from the source: 3m, depth : 12m

Fig. 5 Typical time records and power spectral density of the induced vibratory motion

SITE RESPONSE CHARACTERISTICS

Site resonant frequencies provide the tools for conducting simplified seismic response analyses(Kramer, 1996). The estimation of resonant period of soil deposit can be utilized to determine the design response spectrum. Spectral analyses based on the particle velocity records determined at the down hole array were performed to estimate the resonant period of the experimental site. The cross power spectrum $s_{v_iv_j}(f)$ of particle velocity $v_i(t)$ and $v_j(t)$ was estimated as follows:

$$s_{v_iv_j}(f) = V_i^*(f)\,V_j^*(f) \tag{1}$$

in which f = frequency; $V_i^*(f)$ = complex conjugate of the finite Fourier transform of $v_i(t)$; and $V_j^*(f)$ = the finite Fourier transform of $v_j(t)$.

In general, the peak amplitude of $s_{v_iv_j}(f)$ exists at either a resonant or an input frequency(an excitation spectrum peak). In this regard, the phase-angle spectrum provides a simple way to distinguish between these two types of peaks.
In order to become spectral peaks due to the resonance of soil

spectrum, the phase difference determined by two output measurements in the downhole array would be in phase or 180° out of phase with one another. This is illustrated for the cantilever–beam–type structure in Fig. 5.

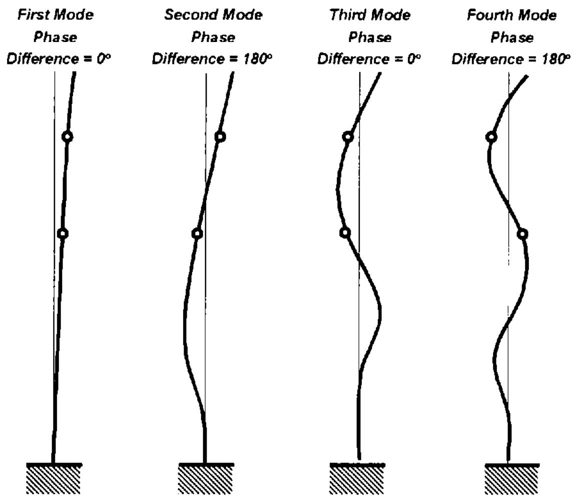


Fig. 5 First four normal modes of a cantilever structure (Bendat and Piersol, 1993)

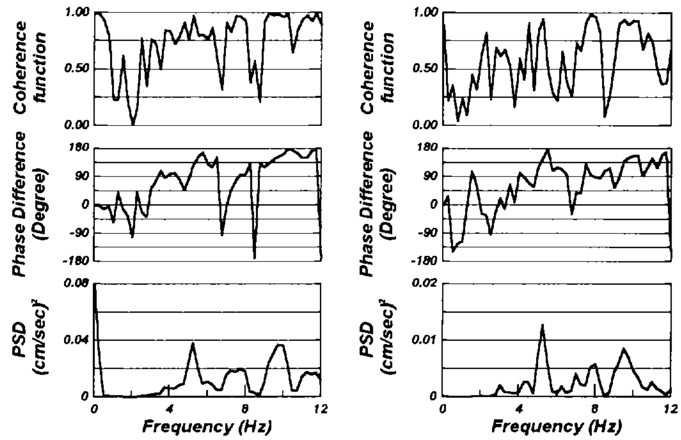
The phase difference can be calculated from the Eq. (2)

$$\theta_{ij}(f) = \tan^{-1}\left[\frac{\mathrm{Im}(S_{v_i v_j})}{\mathrm{Re}(S_{v_i v_j})}\right] \qquad (2)$$

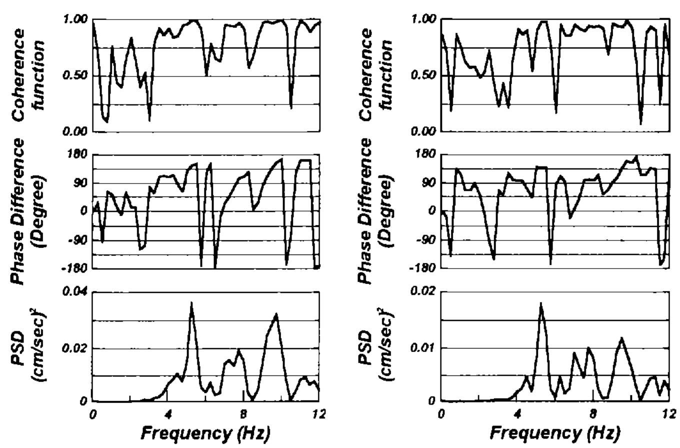
where, $\theta_{ij}(f)$ = phase difference between two measured signals of v_i and v_j.

The coherence function was utilized to ensure quality and accuracy of estimates, and to detect noise or non–linearities (Bendat and Piersol, 1993). Spectral analyses were performed for various experiment set-up changing the transducer installation depth, horizontal distance from the vibratory source, and vibratory source type(hydraulic hammer compaction and/or dynamic compaction).

Typical results are shown in Fig. 6, which displays cross power spectrum, phase angle difference, and coherence function monitored at downhole receiver array. Fig. 6(a) shows a spectral analysis result of receivers installed at depths of 4.5m and 12.5m and the horizontal distance from the vibratory source was 3m. Fig. 6(b) shows a spectral analysis result of receivers installed at depth between 0.5m(soil surface) and 8.0m and the horizontal distance from the vibratory source was 3m. In order to measure the natural frequency of the soil deposit induced by the shear wave propagation, the longitudinal motions were used for spectral analysis among the three component motions.



(a) Hydraulic hammer compaction
(Horizontal dist. : 3m, Depths of two receivers : 4.5 and 12.5m)



(b) Hydraulic hammer compaction and dynamic compactions
(Horizontal dist. : 3m, Depths of two receivers : 0.5 and 8.0m)

Fig. 6 Typical test results; Cross power spectrum, Phase angle difference and Coherence function

At the resonant frequencies, phase angle difference is generally in the vicinity of 0°(in phase) or ±180°(out of phase) and the coherence function which is used for checking the quality of measured signal(Elgamel et. al, 1995) approaches close to 1.0. From the results of spectral analyses, the frequencies of about 5, 7 and 9Hz show peaks in power spectral density, phase difference of close to 180°, and coherence function value of about 1. So, it can be estimated that the above frequencies are the natural frequencies of the soil deposit.

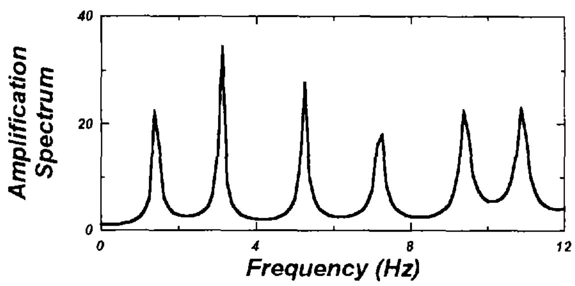


Fig. 7 Amplification spectrum between bedrock and soil free surface motion.

From the results of detailed site characterization, the one-dimensional linear ground response analysis was performed by the hypothesis that the soil deposit is layered and damped on elastic rock(Schnabel et. al, 1972). The shear wave velocity profile used in the analysis was based on the downhole test results at depth above 30m, while at depth below 30m shear wave velocity profile was estimated based on the spectral analysis of surface waves(SASW) test. For the response analysis, the soil deposit was divided in layers of 2m thickness based on the information determined by SPT samples. Fig 7. shows an amplification spectrum of the soil deposit. The peaks in the amplification spectrums represent the natural frequencies of the soil deposit. The peaks at 5, 7, 9Hz in the amplification spectrum matched well with the estimated natural frequencies from the spectral analyses based on forced vibration tests. However, the peaks at 1, 3Hz were not found in the spectral analyses. The reasons that the fundamental and second natural frequencies cannot be obtained from the spectral analysis, can be explained by the following two reasons. The one is that the energy generated by the vibratory source in this experiments did not exist sufficiently in frequency ranges below about 4Hz as shown in Fig. 4 and the reliable vibration measurement using the 3D-velocity transducer can be obtained at frequencies above 4Hz as shown in Fig. 2. Therefore, in order to determine the fundamental frequency and mode shape of the soil deposit, the source which enable to generate the lower frequency energy and the transducers which measure the reliable vibration at the corresponding lower frequencies are required, and the further studies are underway.

The equivalent shear wave velocity of the soil deposit can be calculated by the Eq. (3) and the natural frequencies of the soil deposit can be simply calculated by using the equivalent shear wave velocity and thickness of soil layer as Eq. (4):

$$\overline{V_s} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \tag{3}$$

in which, if the soil deposit is divided into n layers, $\overline{V_s}$ is the equivalent shear wave velocity, d_i is the thickness of each soil layer and v_{si} is the shear wave velocity of each soil layer.

$$\omega_n = \frac{\overline{V_s}}{H}\left(\frac{\pi}{2} + n\pi\right) \tag{4}$$

in which ω_n is the natural frequencies of soil deposit, $\overline{V_s}$ is the equivalent shear wave velocity of the soil deposit and H is the thickness of the soil deposit and n = 0, 1, 2, ·····, ∞.

From Eq. (3), the equivalent shear wave velocity of the experimental site is calculated 206.7m/sec and the thickness of the soil deposit is 50m. Therefore we can estimate the natural frequencies of the experimental site as 1.0, 3.1, 5.2, 7.2, 9.3, ···Hz. The natural frequencies 5.2, 7.2, 9.3Hz from the Eq. (4) matched well with the estimated natural frequencies from the spectral analyses, but the peaks at 1, 3.1Hz are still not found from the spectral analyses.

CONCLUSIONS

Forced vibration tests and detail site characterization were performed at the geotechnical experimental site in Yong-jong Island where Inchon International Airport being constructed. Spectral analyses of the recorded vibratory motions were performed to evaluate the resonant frequencies of the experimental site. The natural frequencies estimated from the spectral analysis were the 3rd natural frequency and the higher ones. Even though it was difficult to obtain the fundamental and 2nd natural frequencies because of the restrictions of transducer and vibratory source, the natural frequencies at higher modes determined by spectral analysis methods matched well with the natural frequencies of the soil deposit estimated from linear ground response analysis and simplified estimation method.

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