1099

DYNAMIC VARIATION OF SOIL RIGIDITY BASED ON DOWN-HOLE ARRAY OBSERVATION

Makoto KAMIYAMA[1], Tadashi MATSUKAWA[2] And Masaru YOSHIDA[3]

SUMMARY

This paper provides a method for estimating dynamic variations of rigidity and damping of soils related intimately to the non-linear earthquake responses of ground. The down-hole array records obtained at Port Island during the 1995 Kobe Earthquake, Japan are used to investigate the capability of the method. First the records obtained both during the main-shock and after-shocks of the Kobe event are comparatively analyzed based on the spectral ratio method, cross correlation method and so on. These comparative analyses showed that non-linear responses with an extreme reduction of soil rigidity and increase of soil damping occurred during the main-shock while linear responses prevailed during the after-shocks. Such a reduction of soil rigidity was found to have time-varying characteristics, so the subsequent topic of this paper focussed on a method for analyzing non-stationary variation of soil rigidity and soil damping. This paper uses the concept of the "Complex Envelope" for the purpose. The time histories of stresses and strains in the soils were numerically obtained from the observed array records, and then they were dealt with by the Complex Envelope method so that the time variations of soil rigidity and damping are estimated. In contrast with the conventional method in which soil rigidity and damping are estimated graphically based on the locus between stress and strain, this method provides more precise estimates of soil rigidity and damping and can follow complicated stress-strain relations. The resultant variations of soil rigidity and damping at Port Island showed, for the surface layer of 0-16 m depth during the main-shock, an extreme reduction of rigidity with a factor of about 50 in a short duration of about 5 sec starting at S wave motions. As opposed to such an extreme variation for the main-shock, the after-shocks motions showed rather stationary variations of soil properties with less non-linearity. This paper finally concludes that the Complex Envelope method is quite effective to estimate non-stationary variation of soil rigidity and damping and can be applied extensively to various phenomena associated with non-linear behaviors of earthquake motions.

INTRODUCTION

Local soil conditions exert a great influence on strong ground motions. Among various local soil effects, the non-linear effect of soils has unknown factors compared with other effects. The purpose of this paper is to provide a method for exactly estimating dynamic variations of rigidity and damping related intimately to the non-linearity of soils. This is a kind of inverse analysis to obtain material's properties from observed data. In order to estimate inversely the rigidity and damping of soils from observed strong-motion records, these records are required to be resulting from an array observation system. An ideal set of records suitable to such an inverse analysis was obtained at Port Island during the 1995 Kobe Earthquake, Japan. The Port Island system, which is featured by its down-hole array observation, obtained strong-motion records both during the main-shock and some after-shocks of the Kobe event. Since the motion levels of these main-shock and after-shocks are quite different from each other, they provide us with an important information about the degree of non-linear responses.

[1] *Department of Civil Engineering, Tohoku Institute of Technology, Sendai, Japan Email: mkamiyam@titan.tohtech.ac.jp*
[2] *Department of Civil Engineering, Tohoku Institute of Technology, Sendai, Japan*
[3] *Department of Civil Engineering, Tohoku Institute of Technology, Sendai, Japan*

Using the Port Island records, several researchers studied on the non-linear behaviors of soils. For example, Elgamal et al. and Kazama et al. made separately an inverse analysis of soil moduli from the array records, showing some remarkable decrease of rigidity and increase of damping during the main-shock relative to the after-shocks. Both of them estimated graphically the soil moduli in terms of the locus of stress and strain that were numerically obtained from the records. However, these graphical methods not only lack quantitative estimates of soil moduli but also fail to follow complicated variations of the locus. In view of the importance of material's properties for earthquake responses, it is necessary to develop more sophisticated method for exactly estimating the dynamic variations of soil rigidity and damping not based on graphical methods. The fact that strong ground motions generally have non-stationary variations in time enhances such a necessity, so this paper places emphasis on a method of quantitatively and exactly estimating the non-stationary variations of soil rigidity and damping.

NON-LINEAR RESPONSE OF THE DOWN-HOLE RECORD

Figure 1 shows the array profile of the observation system at Port Island. As shown in Fig.1, the observation system consists of tri-axial accelerometers (the NS, EW and UD components) located at 0m, 16m, 32m and 83m depths. It recorded strong motions during the main-shock of the Kobe Earthquake. In addition, it also obtained motion records during some after-shocks of the event. It is expected to detect a difference in the degree of non-linear response between the main-shock and after-shocks because they differ remarkably from each other in their motion levels. Figures 2 and 3 show the strong-motion records obtained during the main-shock and a representative aftershock, respectively. In the main-shock records, the horizontal ones at 0m depth differ considerably in the wave forms from those at 16m, 32m, and 83m depths whereas the after-shock records demonstrate less difference in their wave forms. This indicates that the horizontal motions near the surface were due highly to the non-linearity of soils, especially during the main-shock.
Figure 4 shows the spectral ratios of records at 0m depth to those at 83m depth, separately for the NS, EW and UD components. In Figure 4, the spectral ratios for the main-shock are plotted along with the ones for representative aftershocks, to compare their differences depending on the motion directions. These spectral ratios also make it clear that the main-shock provided non-linear responses in the horizontal motions, being quite different from the after-shocks that showed rather uniform features regardless of the events.

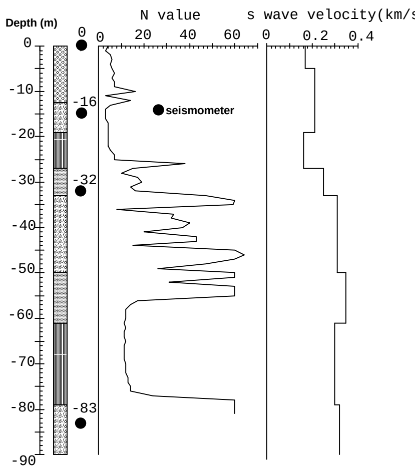

Figure 1: Array profile of the observation system at Port Island

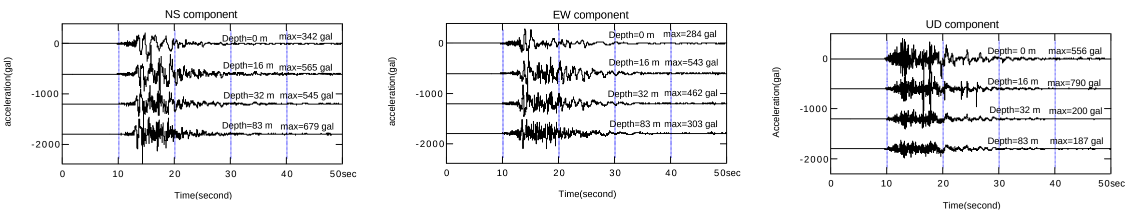

Figure 2: Strong-motion records during the main-shock(NS, EW and UD components)

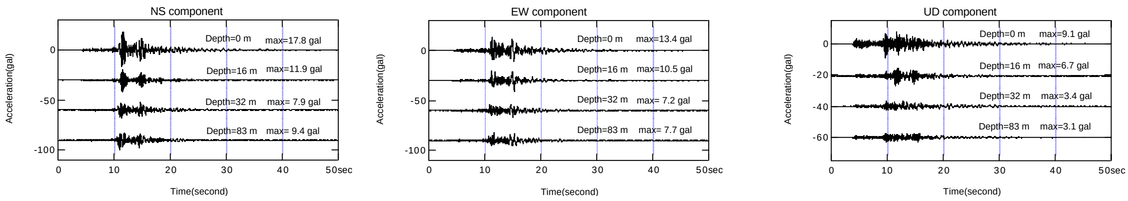

Figure 3: Strong-motion records during an after-shock(NS, EW and UD components)

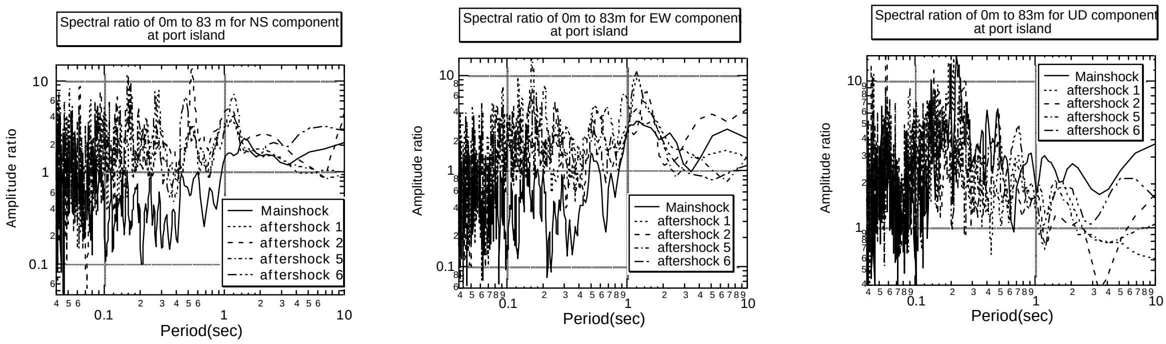

Figure 4: Spectral ratios of the 0 m records to 83 m records (NS, EW and UD components)

NON-LINEAR RESPONSE OF THE DOWN- HOLE RECORD

A cross correlation technique was used to estimate the average velocities of S and P waves between the down-hole stations at Port Island. This technique may give an insight into a possible reduction of rigidity of the soils that might have culminated in the characteristic amplification spectra for the main-shock. At the same time, this technique's results provide a calibration basis for the non-stationary variation estimates of soil rigidity and damping that will be described later, because they are obtained as some stationary values of rigidity and damping averaged over the total duration of motions. The detailed process of the cross correlation estimates can be referred to Kamiyama et al., 1998. In this paper, only the results are shown to compare them with the ones due to the non-stationary method.

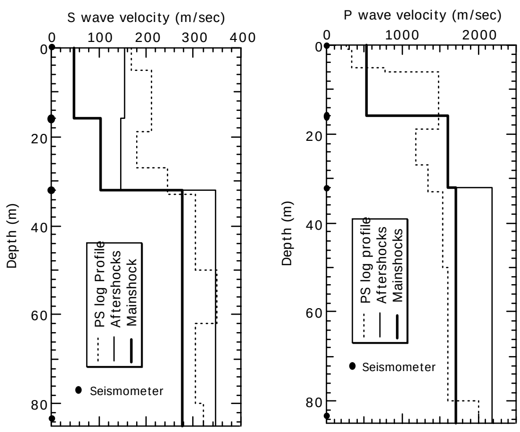

Figure 5: S and P waves' velocities estimated from the cross correlation analyses

Figure 5 shows the S and P waves' velocities estimated by the cross correlation analysis. Figure 5 includes the S wave and P wave velocities estimated independently of the main-shock and after-shocks together with the velocities of S wand P waves resulting from the PS logging method that represents propagation velocities of waves with extremely small strain of soils. In Figure 5, the S wave velocities for the main-shock is averaged between the NS and EW components and the S wave velocities for the aftershocks are similar average over all the after-shocks. It is found in Figure 5 that the S wave velocity in the 0m-16m layer for the main-shock reduces to about 30 % against the one from the after-shocks whereas the P wave velocity shows less change between the main-shock and after-shocks. This result also concludes that the rigidity of soils at Port Island was degraded only in the horizontal direction, compatible to the results of spectral ratio. On the other hand, Figure 6 shows the damping ratios estimated using the S wave's velocities in Fig.5 as well as the spectral fitness method [Kamiyama et al., 1998]. These damping ratios also indicate stationary values averaged over the total duration of the main-shock and after-shocks' motions. We can see from Fig.6 that the damping ratio of the 0-16 m layer increased by a factor of 5 for the main-shock in accordance with the reduction of S wave velocity that had a factor of 3.

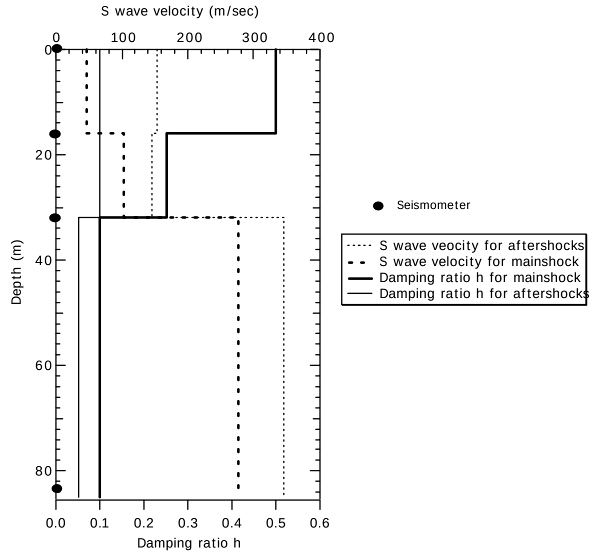

Figure 6: Damping ratios estimated from the spectral fitness method with the reproduced velocities

NON-STATINARY METHOD ESTIMATING DYNAMIC SOPIL PROPERTIES

The results in the preceding sections made it clear that the soils at Port Island behaved non-linearly especially in the horizontal direction during the main-shock. However, they provide no information of the time-varying characteristics of non-linearity. Non-linear behaviors of soils generally result from the reduction of rigidity and increase of damping of materials in response to incident earthquake motions that vary in a non-stationary manner, this thus suggests that the dynamic variations of rigidity and damping occur in a time-dependent manner. Based on this suggestion, we made an attempt to detect time-varying variations or non-stationary variations of soil rigidity and damping from the down-hole array records at Port Island. Until now, there have been several studies in which the dynamic variations of soil rigidity and damping were estimated from observed down-hole records as described in the introduction section. Most of them attributed their estimates to a graphical method with aid of the locus between the stress and strain induced in soils. However such a graphical method does not necessarily give exact estimates enough to follow complicated variations existing actual ground motions. As opposed to the graphical method, this paper deals with another technique, which has a numerical basis apart from the graphical basis, possible to estimate more precisely the dynamic variations of rigidity and damping.

Assuming that the ground motions are due mainly to the vertical propagation of S wave and the soils have the non-viscous and hysteric damping [Ishihara, 1996], the motion equation is given as a simultaneous form:

$$\rho(z)\frac{\partial u^2}{\partial t^2}=\frac{\partial \tau(t,z)}{\partial z} \tag{1}$$

$$\tau(t,z)=G(t,z)\{1+i2h(t,z)\}\gamma(t,z) \tag{2}$$

in which ρ(z) is the density, u(t,z) is the displacement, τ(t,z) is the shear stress, γ(t,z) is the shear strain, G(t,z) is the rigidity modulus, h(t,z) is the damping ratio, t is the time and z is the depth.

In principle, equations (1) and (2) enable us to inversely estimate the rigidity modulus and damping ratio after obtaining numerically the stress and strain. The most difficult issue in such inverse estimates of G(t,z) and h(t,z) is that they are expressed by the imaginary number system as shown in equation (2). Therefore it is necessary to find some skills overcoming this difficulty. The "Complex Envelope"[Fanbach, 1975] is used here as the most appropriate skill because it transfers a time history into its corresponding imaginary number system with a reasonable manner. The Complex Envelope is defined as an imaginary number system in which the real part consists of a time history while the imaginary part stems from the histories' Hibert transform. For example, the complex envelope for the stress τ(t,z) and strain γ(t,z) are, respectively, given as follows:

$$T(t,z)=\tau(t,z)+iH[\tau(t,z)] \tag{3}$$

$$\Gamma(t,z) = \gamma(t,z) + iH[\gamma(t,z)] \tag{4}$$

in which H[ ] is the Hilbert transform.

By substituting equations (3) and (4) instead of $\tau(t,z)$ and $\gamma(t,z)$ into equation (2), we can obtain

$$h(t,z) = \frac{1}{2}\tan\{\varphi(t,z) - \psi(t,z)\} \tag{5}$$

$$G(t,z) = \frac{1}{\sqrt{1+(2h(t,z))^2}}\frac{|T(t,z)|}{|\Gamma(t,z)|} \tag{6}$$

in which

$$T(t,z) = |T(t,z)|\exp\{i\varphi(t,z)\} \tag{7}$$

$$\Gamma(t,z) = |\Gamma(t,z)|\exp\{i\psi(t,z)\} \tag{8}$$

The theoretical verification for equations (3) and (4) can be easily given by use of a harmonic motion for $\tau(t,z)$ and $\gamma(t,z)$[Kamiyama et al., 1999].

NON-STATIONARY VARIATIONS OF SOIL PROPERTIES AT PORT ISLAND

The above method for estimating the dynamic variations of soil properties was applied to the down-hole records at Port Island. The shear stress $\tau(t,z)$ and shear strain $\gamma(t,z)$ were first numerically obtained at the mid-depth between each accelerometer in Fig.1, assuming that the acceleration and displacement vary both in a linear manner between the accelerometers' points. When obtaining the stress and strain, the observed acceleration records were treat through a band pass filter to avoid numerical errors that result from the down-hole configuration as the "spatial ailiasing". In this paper, the filter's band was set to be 0.1 to 1.5 Hz for the main-shock's records and 0.5 to 3.0 Hz for the after-shocks' records. Following the estimates of stress and strain, the non-stationary variations of h(t,z) and G(t,z) were next obtained according to equations (5) and (6). Equations (5) and (6) show that a phase relation estimates these variations, it is therefore possible that they fluctuate too rapidly in time. To avoid spurious fluctuations of h(t,z) and G(t,z), the resultant variations of them were smoothed using the method of moving average with a time window of 1.0 sec.

Figures 7 to 9 show the variations of rigidity modulus and damping ratio estimated at the mid-depth between each accelerometer from the NS component's records. Similar variations were obtained from the EW component's records, but they are omitted here because of the space consideration. The variations in Figs.7 to 9 are plotted for a time interval ranging from 13.1 sec to 43.0 sec, regarded to be principally the S-wave portion. In Figures 7 to 9, the average values of rigidity modulus and damping ratio deduced from the cross correlation analysis are also plotted as stationary variations to gauge the capability of the present non-stationary method. Figures 7 to 9 indicate that the present method's results are relatively well compatible with the ones by the cross correlation analysis, regarding the latter values represent an average variation in time. Conversely, the comparisons in Figs. 7 to 9 claim that the present method analyzes well the non-stationary variations of soil properties peculiar to actual strong motions. On the other hand, the variations of rigidity modulus and damping ratio at each depth are reproduced together in Figure 10 to compare their differences. It is clear in Fig. 10 that the time-varying characteristics of G(t,z) and h(t,z) are remarkable at the depth of 8m as opposed to the less variations of them at the other depths. Especially Figure 10 shows that the reduction of soil rigidity at 8m-depth occurred with a factor of 50 in a short duration of about 5 sec starting at the S-wave motions.

To correlate these non-stationary variations of rigidity and damping with the relation between stress and strain, the stress-strain orbits are shown in Figure 11. It is seen in Fig.11 that the orbit in the surface layer of 0 m to16 m depth is much more complicated than the ones in deeper layers. A complicated orbit such as in Fig.11 is clearly beyond the scope of graphical method, but in contrast the present method enables to detect exact variations of rigidity and damping to some extent, manifesting its effectiveness.

As shown above, the main-shock of the Kobe event caused conspicuously non-linear response to the soils at Port Island. On the other hand, the soils experienced smaller motions during some after-shocks. Figure 12 shows an example of array-motion records during a representative after-shock. The non-stationary variations of rigidity modulus and damping ratio of the soils were similarly estimated as shown in Figs. 13 and 15. Figures 13 and 15 show that the after-shock caused little variations of rigidity and damping in all the layers, demonstrating rather linear responses in comparison with the main-shock. This is consistent with the stress-strain orbits that were obtained from the observed records.

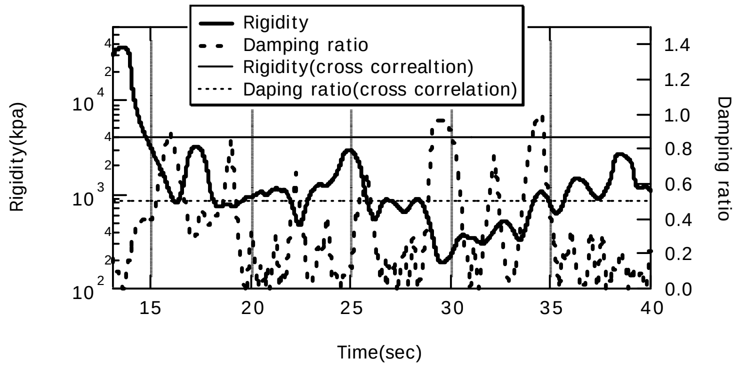

Figure 7: Non-stationary variation of rigidity modulus and damping ratio estimated at 8 m depth

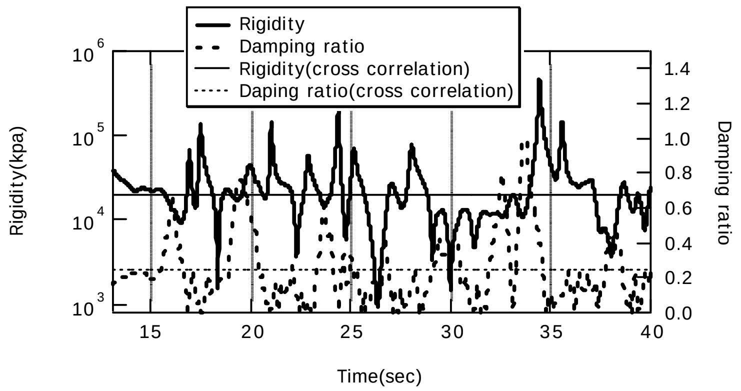

Figure 8: Non-stationary variation of rigidity modulus and damping ratio estimated at 24 m depth

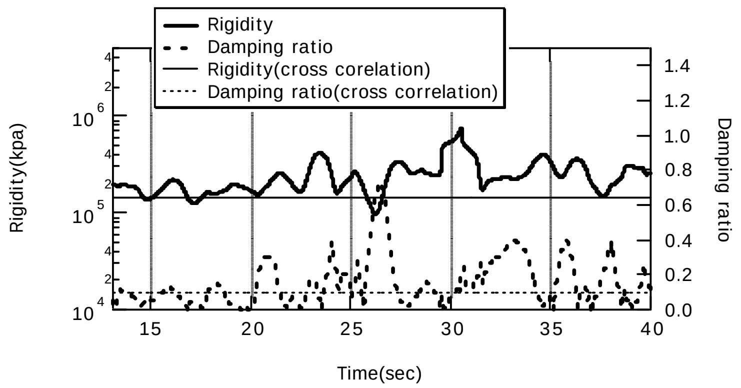

Figure 9: Non-stationary variation of rigidity modulus and damping ratio estimated at 57.5 m depth

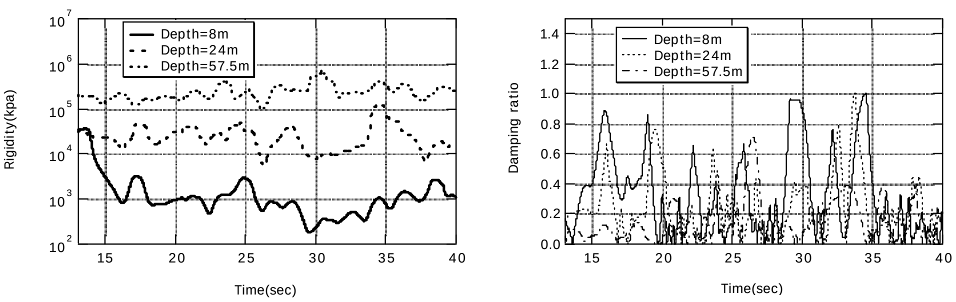

Figure 10: Comparisons of rigidity modulus and damping ratio at each depth

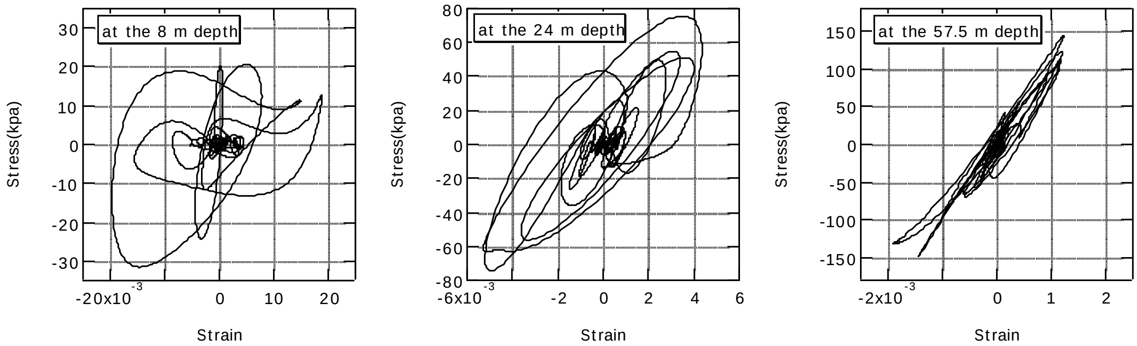

Figure 11: Stress and strain orbits at each depth

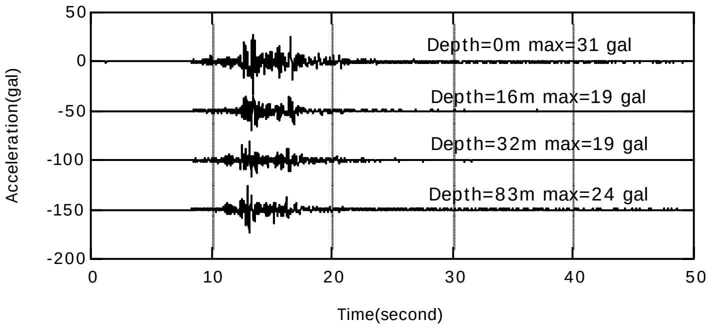

Figure 12: Strong-motion records during an after-shock (EW component)

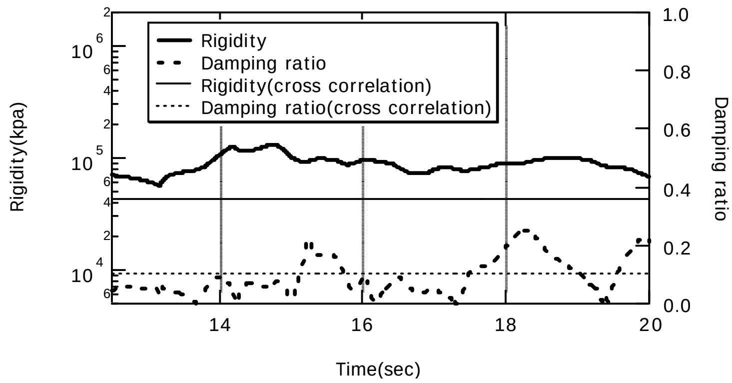

Figure 13: Rigidity modulus and damping ratio estimated at 8 m depth for an after-shock

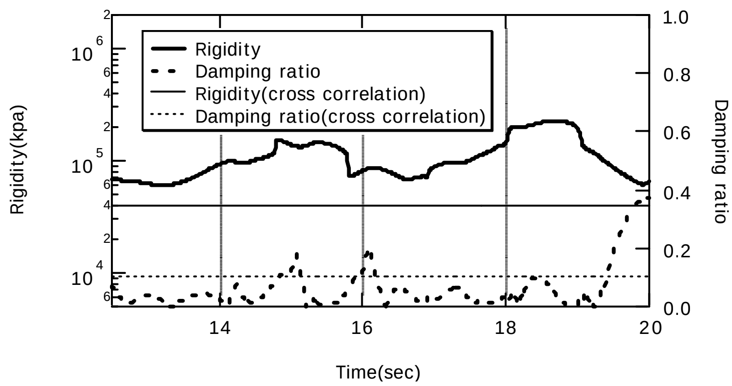

Figure 14: Rigidity modulus and damping ratio estimated at 24 m depth for an after-shock

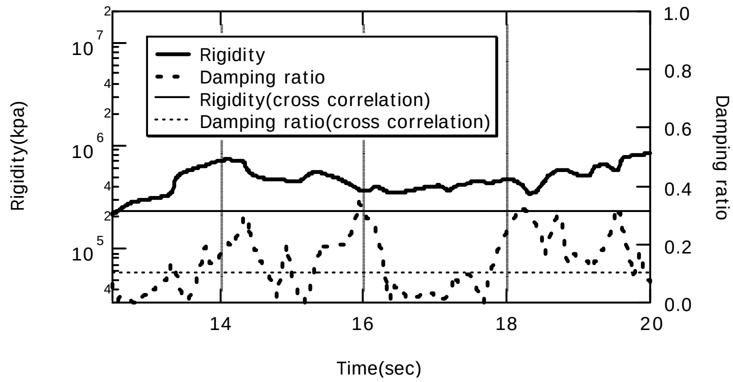

Figure 15: Rigidity modulus and damping ratio estimated at 57.5 m depth for an after-shock

CONCLUSIONS

1. The Complex method presented here is quite effective to exactly estimate the non-stationary variations of rigidity and damping associated with non-linear responses of earthquake motions. The method, not based on the graphical technique conventionally used, enables to follow complicated motions common in actual strong-motion records. It is easily applied to any experimental data having the stress-strain relation obtained in laboratory as well as down-hole arrays of ground motions obtained in situ as exemplified in this paper.
2. The main-shock motions at Port Island showed remarkably non-linear responses in the horizontal directions with less non-linearity in the vertical direction whereas the after-shocks' motions behaved with a linear response in both directions. The non-linearity is highly related to the reduction of rigidity and increase of damping in the surface layer from 0 m to 16 m depths.
3. The reduction of rigidity and increase of damping occurred in a non-stationary manner in time. Especially the rigidity of the surface layer reduced by a factor of 50 in a short duration of about 5 sec after the S wave's arrival at the site. In contrast with such a non-stationary variation of soil properties during the main-shock, the after-shocks showed rather stationary variations of rigidity and damping, compatible with their linear responses.

REFERENCES

1. Elgamal, A. W., Zeghal, M. and Parra, E. (1995), Identification and modeling of earthquake ground response, *In K. Ishihara(ed), Earthquake Geotechnical Engineering*, Vol.3, pp1368-1406.
2. Fanbach, J. S. (1975), The complex envelope in seismic signal analysis, *Bull. Seism. Soc. Am.*, Vol.65, No.4, pp95-962.
3. Ishihara, K. (1996), *Soil behavior in earthquake geotechnics*, Oxford University Press, pp16-39.
4. Kamiyama, M. and Matsukawa, T. (1998), Non-linear response in the downhole strong-motion records at Port Island during the 1995 Kobe Earthquake, Japan, *Proc. of the 11th European Conference on Earthquake Engineering*, CD-ROM.
5. Kamiyama, M. and Yoshida, M.(1999), Non-statinary analysis of soil rigidity and damping by use of array strong-motion records, submitted to *Structural Eng. / Earthquake Eng.* (in Japanese).
6. Kazama, M., Yanagisawa, E., Inatomi, T., Sugano, T. and Inagaki, H., (1996), Stress strain relationship in the ground at Kobe Port Island during 1995 Hyogo-Ken Nanbu Earthquake inferred from strong motion array records, Journal of Geotechnical Engineering No.547, JSCE, pp171-182 (in Japanese).