Chapter 5. Ground and Building Motion Simulation in Golcuk

5.1 Introduction

The Kocaeli earthquake of August 17, 1999 destroyed over 60,000 masonry houses, and residential and office buildings in the northwest area of Turkey. In Golcuk, Kocaeli Province, in particular, a large number of medium-rise buildings sustained either partial or complete collapse typically of a soft first story (for example, Photo 5.1). Figure 5.1 shows spatial distribution of collapsed building ratios in Golcuk, which is based on the results of the reconnaissance survey performed by Architectural Institute of Japan (AIJ) team (AIJ Reconnaissance Team, 1999). The damage to buildings was concentrated at several areas in the north of Ataturk street, which is the main street running in the east-west direction. The concentration of the building damage could be due to the effects of surface geology on ground motions, i.e., so-called "site effects." In fact, most of the northern area of the main street is located on a plain while the south is on a hill, where the building damage was slight.

In order to evaluate the site effects quantitatively, two- or three-dimensional shear wave velocity (V_S) profiles down to the bedrock should be properly determined. It is, however, difficult to estimate multi-dimensional deep V_S profiles using conventional geophysical methods with boreholes. For this purpose, the microtremor horizontal-to-vertical (H/V) spectrum method originally proposed by Nakamura (1989) has been developing for two- or three-dimensional soil profiling. Recent studies, for example, have indicated that the H/V spectrum of microtremor at a site reveals that of Rayleigh or surface waves (e.g., Tokimatsu, 1995; Arai and Tokimatsu, 2000) and that V_S profile at a site can be estimated by inverse analysis of microtremor H/V data (Arai and Tokimatsu, 1998). Based on the improved H/V method, microtremor measurement was performed in Golcuk and the two-dimensional V_S structure across the damaged area was estimated. With this V_S structure, in this report, ground motions during the main shock were simulated by two-dimensional dynamic response analysis, and then strength demands of the ground motions were computed for a simplified building system.

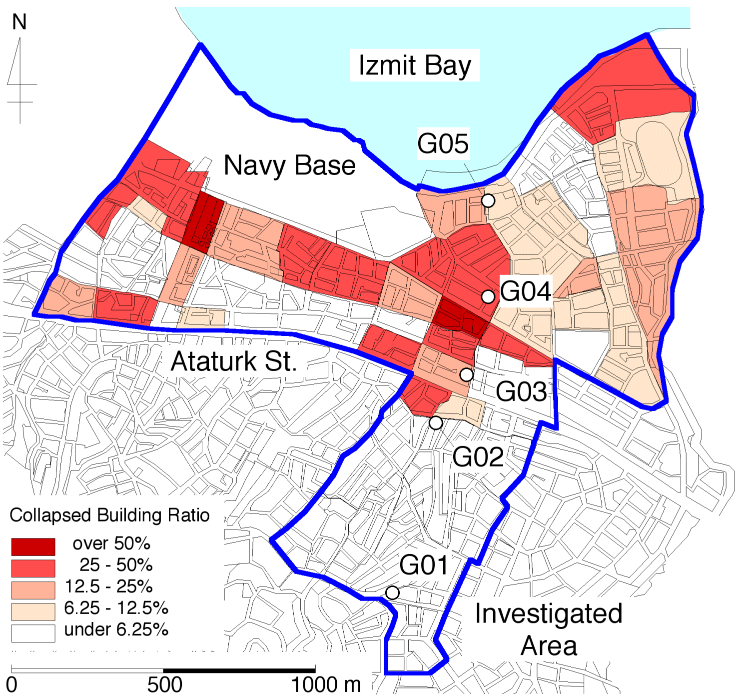


Figure 5.1. Map showing microtremor observation sites and distribution of damage to buildings in Golcuk (AIJ Reconnaissance Team, 1999; Refer to color Figure 11).

Photo 5.1. Damage to buildings in the northern area of Ataturk street.

The objectives of this chapter are to introduce the use of microtremor H/V inversion method for two-dimensional V_S profiling and to examine the effects of the V_S structure on the ground motion characteristics and the building damages during the 1999 Kocaeli earthquake.

5.2 Theoretical Background of Microtremor H/V Method

After the pioneer work by Kanai and Tanaka (1961), microtremor measurement using one station have been used for estimating dynamic characteristics of sites as well as natural site periods. They assumed that the microtremor horizontal motions at periods less than 1s consist mainly of shear waves, and that the spectra of the horizontal motions reflect the transfer function of the ground at a site. However, many researchers, e.g., Udwadia and Trifunac (1978), indicated that microtremor spectra often pointed the exciting function rather than the transfer function of the site.

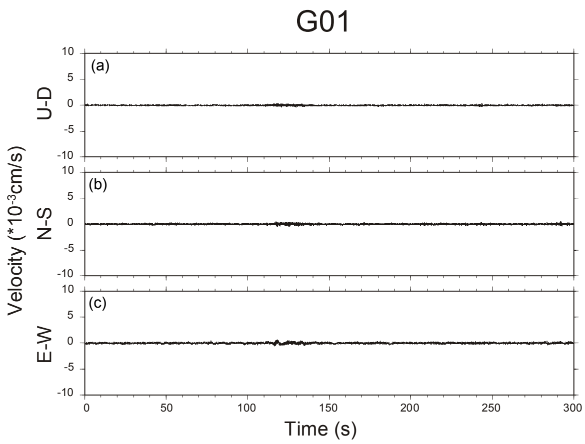
Nakamura and Ueno (1986) proposed a revised method in which the effect of source function might be minimized by normalizing the horizontal spectral amplitude in terms of the vertical one. Assuming that the S-waves dominate in microtremors, they indicated that the horizontal-to-vertical (H/V) spectral ratio of microtremors at a site roughly equals the S-wave transfer function between ground surface and bedrock at the site. This mean that the H/V peak period and the peak value itself respectively correspond to the natural site period and the amplification factor. This method has the potential to make site periods more reliable, however, it rests on tenuous assumptions (e.g., Finn, 1991; Horike, 1993). In fact, recent studies have shown that Frequency-wave number (F-k) spectral analysis (Capon, 1969) and Spatial Auto-Correlation (SAC) analysis (Aki, 1957; Matsushima and Okada, 1990) of microtremor records measured with arrays at a site can yield dispersion characteristics of Rayleigh waves (and Love waves under a certain particular condition), and the inverse analysis of the dispersion data successfully results in V_S profile at a site (e.g., Horike, 1985; Okada and Matsushima, 1986; Tokimatsu *et al*., 1992). These indicate that microtremors consist mainly of surface (Rayleigh and Love) waves.

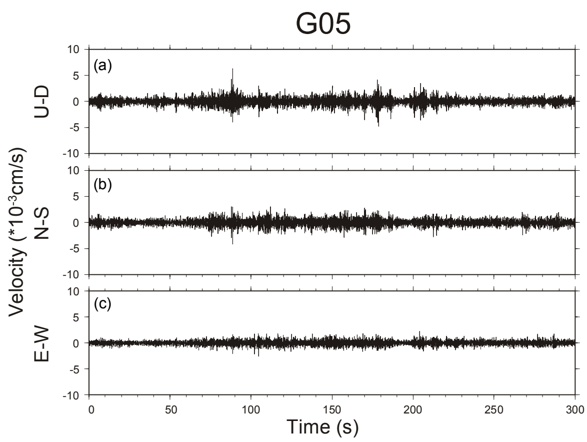
Based on the above findings, for example, Tokimatsu and Miyadera (1992) pointed out that the variation with frequency of the H/V ratio of microtremors corresponds to that of the fundamental Rayleigh mode at a site. However, the H/V values of microtremors are not comparable with those of fundamental Rayleigh wave. It is considered that this disagreement may be caused by the influence of other surface waves in microtremors, i.e., higher Rayleigh modes and Love waves (e.g., Tokimatsu and Tamura, 1994; Lachet and Bard, 1994). In order to solve this problem, Tokimatsu and Arai (1998) presented theoretical formulas for simulating microtremor H/V spectra using surface (both Rayleigh and Love) waves propagating on a layered half-space, considering the effects of fundamental and higher modes. Using these formulas, Arai and Tokimatsu (1998) also presented an inverse analysis of microtremor H/V spectrum for estimating layer thickness of subsurface soils, in case that a prior information of V_S values at a site is given. In this report, these microtremor H/V methods revised by Tokimatsu and Arai (1998) and Arai and Tokimatsu (1998) are used for determining V_S profiles.

5.3 Microtremor Measurement

Microtremor measurement with a three-component sensor was conducted at five sites in Golcuk, hereinafter called Sites G01-G05. These sites are fallen on the line crossing the damaged area and extend from the southern hill northward to the shore of Izmit bay as shown in Figure 5.1. The distances between two adjacent observation stations were about 240-550m. Sites G03-G05 are located in the heavily damaged area while Sites G01 and G02 are on the southern hill, where the building damage was slight. There was little traffic density at each observation site, except Site G03 where was close to the main street.

The measurement system used consists of amplifiers, 16-bit A/D converters and a note-type computer, which are built in a portable case, with a three-component velocity sensor unit whose natural period is 1s (Photos 5.2 and 5.3). At each site, microtremor motions were measured for 5 minutes, and digitized with equi-interval of 0.01s. Figures 5.2 and 5.3 show the velocity time series of the vertical and two orthogonal horizontal (NS and EW components) microtremor motions recorded at Sites G01 and G05. Ten sets of data with 2048 points each were selected from the digitized motions excluding traffic-induced vibrations, and used for the following analyses.





Figures 5.2 and 5.3. Velocity time series of microtremor ground motions at Sites G01 and G05.

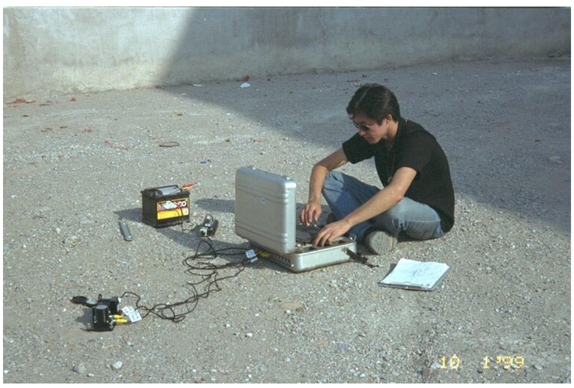


Photo 5.2. Test equipment used in microtremor measurement.

Photo 5.3. Microtremor measurement at Site G01 (Refer to color Photo 17).

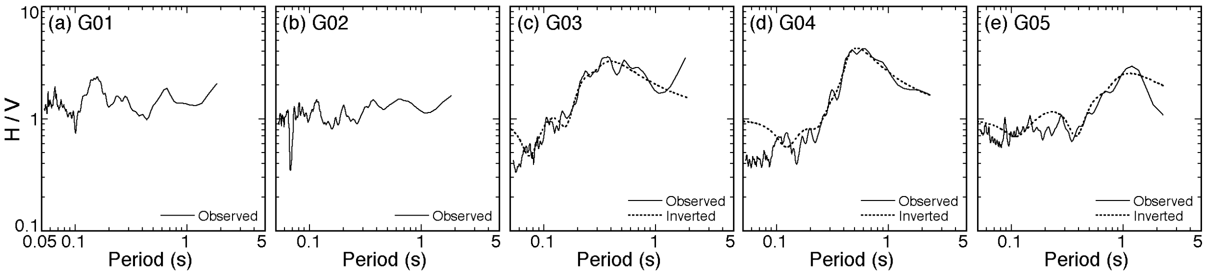


Figure 5.4. H/V spectra of microtremors compared with those of Rayleigh waves theoretically computed for the inverted soil layer models at Sites G01-G05.

5.4 H/V Spectra of Microtremor

The microtremor H/V spectral ratio, $(H/V)_m$, used in this report is defined as:

$$(H/V)_m = (S_{NS}S_{EW})^{1/2} / S_{UD} \tag{5.1}$$

where S_{UD} is Fourier amplitude of vertical motion, and S_{NS} and S_{EW} are those of two orthogonal horizontal motions. In this definition, $(H/V)_m$ could correspond to the H/V spectral ratio of Rayleigh waves at a site (Tokimatsu, 1995; Arai and Tokimatsu, 2000).

The microtremor H/V spectra at Sites G01-G05 are shown in Figures 5.4(a)-(e) by solid lines. At Sites G01 and G02 located on the hill, the observed H/V spectra have no significant peaks (Figures 5.4(a) and (b)). This suggests that the bedrock could outcrop at these sites. At Sites G03-G05 located on the plain, which are in the heavily damaged area, on the other hand, the observed H/V spectra have significant peaks (Figures 5.4(c)-(e)). The H/V peak period increases northward, i.e., from 0.4s at Site G03 to 1.2s at Site G05. The variation of H/V peak period suggests that the V_S profile varies drastically along the line passing through Sites G01-G05.

5.5 V_S Profiling Using Microtremor H/V Spectra

5.5.1 Inversion Procedure

The soil model is assumed to be a semi-infinite elastic medium made up of *N* parallel, solid, homogeneous, isotropic layers (Figure 5.5). Each layer is characterized by thickness, H, mass density, ρ, P-wave velocity, V_P, and S-wave velocity, V_S. The misfit function *F* to be minimized in the inversion is defined as following:

$$F = \sum_j \{(H/V)_m(f_j) - (H/V)_R(f_j)\}^2 W_j \tag{5.2}$$

where $(H/V)_R$ is the H/V ratio of theoretical Rayleigh waves computed from an inferred soil layer model considering the effects of fundamental and higher modes (Tokimatsu and Arai, 1998), and W_j is a weighting factor defined by adaptive biweight method (Tukey, 1974) at a given frequency of f_j. Adaptive biweight method is a kind of maximum likelihood estimation, and is also of robust estimation.

In this inversion, both genetic algorithm (GA) and non-linear least-squares method are used. First, the soil profile is searched by GA method using elite selection and dynamic mutation techniques (Yamanaka and Ishida, 1995). Since the GA method is a kind of probabilistic approach, the GA solution is defined by averaging the models from 10 GA inversions, where each inversion uses different initial model. The details of the GA

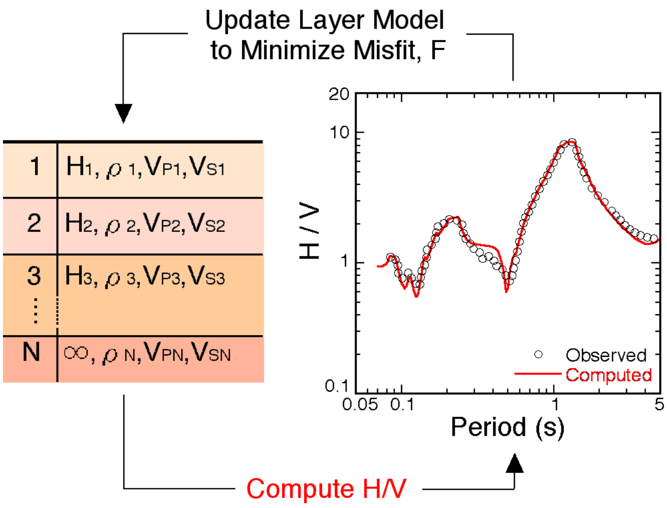


Figure 5.5. Inversion procedure of H/V spectrum.

inversion can be found elsewhere (e.g., Yamanaka and Ishida, 1995). With the GA solution, non-linear least-squares inversion is performed, in which modified Marquardt technique (Marquardt, 1963) and singular value decomposition method (e.g., Matsu'ura and Hirata, 1982; Horike, 1985) are used. The iteration procedure is terminated when the norm of the modification vector is converged into a sufficiently small value, and then the final solution is determined (see Figure 5.5). The details of the non-linear least-squares inversion can be found elsewhere (e.g., Wiggins, 1972; Matsu'ura and Hirata, 1982; Horike, 1985; Okada and Matsushima, 1986; Tokimatsu *et al.*, 1992; Yuan and Nazarian, 1993; Arai and Tokimatsu, 1998).

5.5.2 Estimated V_S Structure

According to the inverse analysis introduced above, the V_S profiles at Sites G01-G05 are estimated using the observed H/V data shown in Figure 5.4. In the inverse analyses, the following assumptions are made: (1) soil model at Sites G01 and G02 is represented by a half-space while that at the other sites a four-layered half-space; (2) soil models at these sites have a common bedrock; and (3) V_S values of the soil layers are those from the results of microtremor array measurement near Navy Base (Kudo *et al.*, AIJ Reconnaissance Team, 1999). The above assumption leaves only the thickness of the top three layers at Sites G03-G05 unknown, thus, which were sought so that the misfits between the H/V ratios of observed microtremor and theoretical Rayleigh waves be minimized.

Figures 5.6(a)-(c) show the inverted V_S profiles with standard errors (e.g., Matu'ura and Hirata, 1982) of layer thickness at Sites G03-G05. The dashed-lines in Figures 5.4(c)-(e) indicate the theoretical H/V spectra for the inverted profiles at the sites. The theoretical H/V spectra are in good agreement with the observed ones. Besides, the evaluated errors of layer thickness at the sites are sufficiently insignificant (Figures 5.6(a)-(c)). Figure 5.6(d) shows the V_S profile near Navy Base estimated by the AIJ Team (Kudo *et al.*, AIJ Reconnaissance Team, 1999). The estimated depths to the bedrock at Sites G04 and G05, which are close to Navy Base, are consistent with that at Navy Base by the AIJ Team. These results indicate that the microtremor H/V method used in this report is effective for estimating the S-wave velocity structure and the estimated profiles could be reliably reasonable.

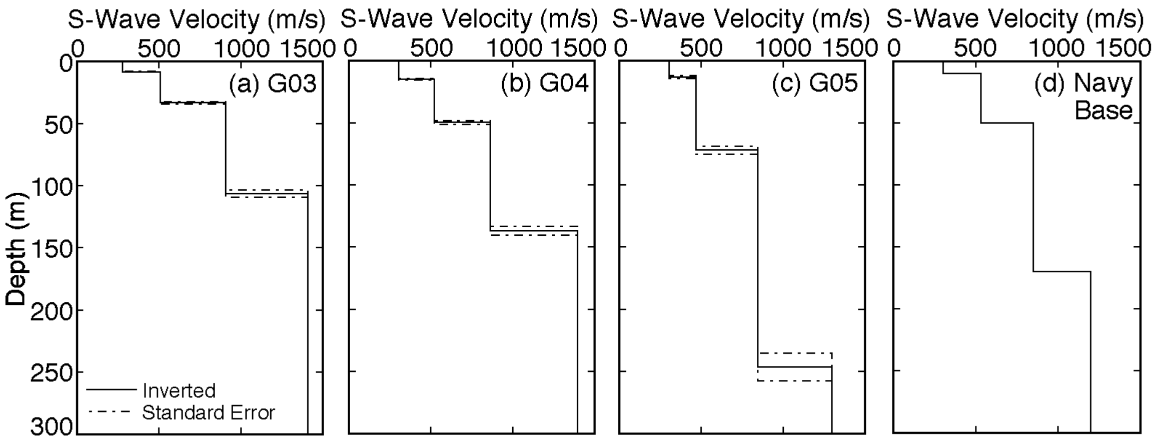


Figure 5.6. (a)-(c) V_S profiles estimated from microtremor H/V method at Sites G03-G05 compared with (d) that from array method near Navy Base (Kudo *et al*., AIJ Reconnaissance Team, 1999).

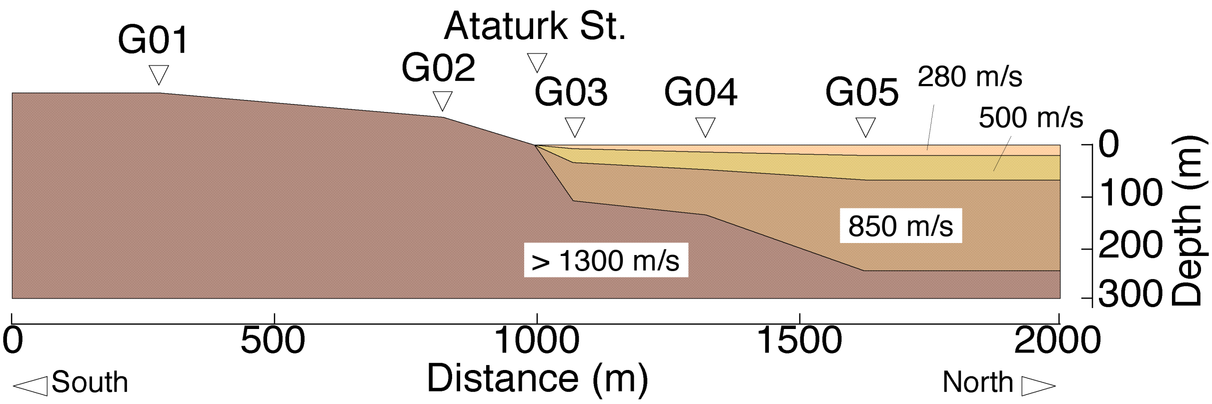


Figure 5.7. Estimated two-dimensional V_S profile along the line passing through Sites G01-G05 (Refer to color Figure 12).

The determination of the V_S profiles can result in a geophysical cross section along the line passing through Sites G01-G05 as shown in Figure 5.7. On the south of the main street, the bedrock with V_S over 1300m/s outcrops. The bedrock, however, dips on the immediate north of the main street creating a vertical gap of about 100m and then slopes gently northward. The estimated depths to the bedrock in the heavily damaged area ranges about 100-200m.

5.6 Ground Motion Simulation in Golcuk

5.6.1 Two-Dimensional Dynamic Response Analysis

With the inverted V_S structure shown in Figure 5.7, two-dimensional equivalent-linear dynamic response analysis was performed using the finite element method in frequency domain (Lysmer *et al.*, 1975). Maximum element size of each layer was determined so as to keep the effective frequency range up to 10Hz for shear waves, and transmitting and viscous boundaries were used on both sides and at the bottom of the soil model, respectively. In order to take the nonlinear behavior of subsurface soils into account, Hardin-Drnevich (H-D) model was

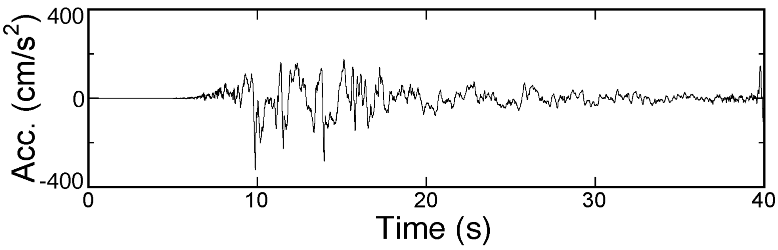


Figure 5.8. Input motion acceleration used in the dynamic response analysis (NS horizontal motions at Yarimca station, Kandilli Observatory, Bogazici University, 1999).

Table 5.1. Soil parameters used in the dynamic response analysis.

$\rho(t/m^3)$	$V_P(m/s)$	$V_S(m/s)$	γ_r	h_{max}
1.7	1500	280	5×10^{-3}	0.25
1.8	2000	500	1×10^{-2}	0.25
1.9	2500	850	2×10^{-2}	0.25
2.0	3000	1300	5×10^{-2}	0.25

inferred for strain-stiffness and strain-damping relationships of soils. Soil parameters of each layer used in the analysis are shown in Table 5.1, in which γ_r is reference strain and h_{max} is maximum damping factor.

The stronger NS horizontal motions recorded at Yarimca (YPT) station during the main shock, which is located about 10km northwest of Golcuk, was used as the input motion; thus, which is the vertically incident S-waves with in-plain particle motions. The input motion acceleration used is shown in Figure 5.8. Peak acceleration and velocity of the input motion are 322cm/s^2 and 73cm/s, respectively.

5.6.2 Ground Motion Characteristics during the 1999 Kocaeli Earthquake

Figure 5.9 shows the variation of peak ground accelerations (PGA) and velocities (PGV) of the simulated horizontal motions along the observation line. The simulated PGV are almost constant with locations and their values are 90-100cm/s. On the other hand, the PGA varies considerably along the line and their values lie in the range of 250-600cm/s^2. Figure 5.10 shows the variation of the collapse ratios of medium-rise reinforced concrete (R/C) buildings along the line (AIJ Reconnaissance Team, 1999). Comparing Figure 5.9 with Figure 5.10, the PGA values in the heavily damaged zone near Sites G03 and G04 exceed 500cm/s^2, and are larger than those in the other zone. This suggests that the peak ground accelerations could have a significant effect on the damage to buildings in the area.

To investigate the possible reason why the PGA values in the heavily damaged zone near Sites G03 and G04 are largely amplified, amplitude ratios between Fourier spectra of the simulated ground motions and those of the input motions, i.e., amplification factors were computed at Sites G01-G05 and are shown in Figure 5.11. The amplification factors at Sites G01 and G02 are almost unity for all periods, while those at Sites G03-G05 are larger than unity, and their peak periods are 0.2, 0.5, and 0.8s, respectively. These values are equal to or slightly less than the natural site periods of the vertically incident S-waves computed for the inferred soil profiles shown in Figures 5.6(a)-(c). This indicates that the large amplification of the PGA in the heavily damaged zone near Sites G03 and G04 could be due mainly to those for vertically propagating S-waves with periods less than 1s, i.e., so-called “site effects.”

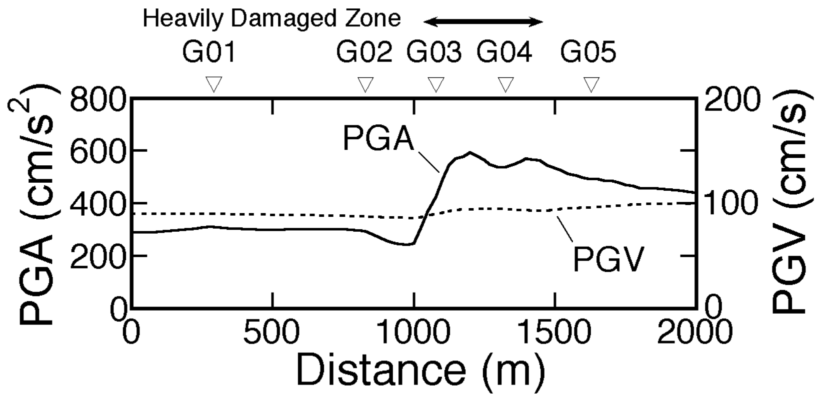


Figure 5.9. Variation of PGA and PGV of the simulated ground motions along the observation line (Refer to color Figure 13).

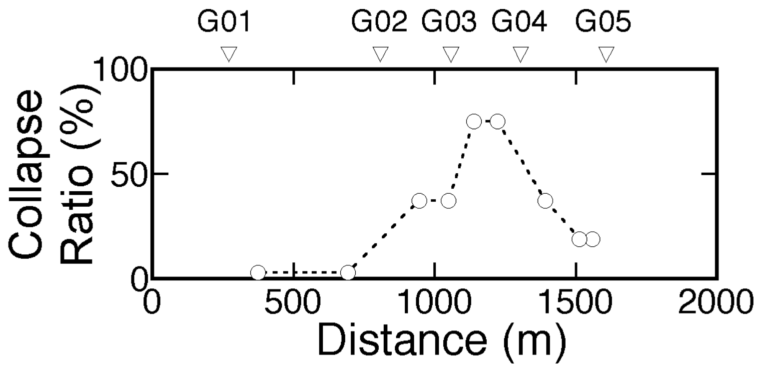


Figure 5.10. Variation of the collapse ratios of medium-rise R/C buildings along the observation line (AIJ Reconnaissance Team, 1999; Refer to color Figure 14).

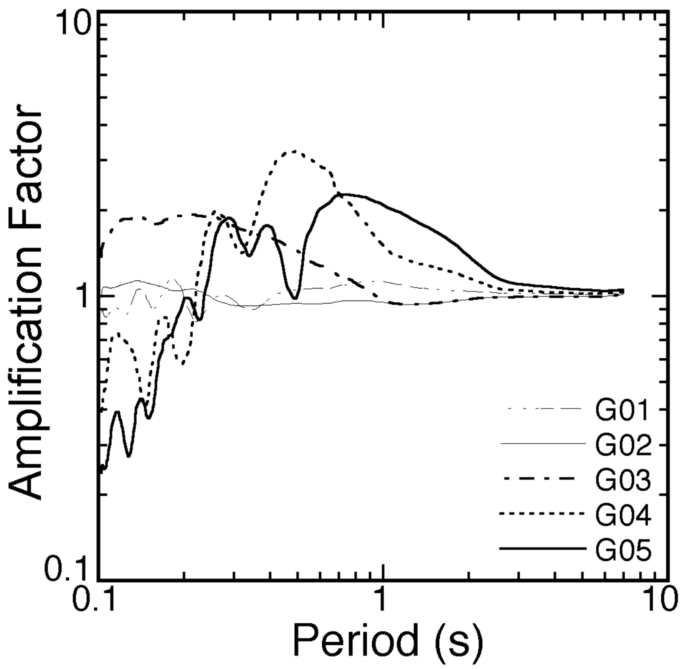


Figure 5.11. Amplification factors between Sites G01-G05 and the bedrock for vertically incident S-waves.

5.7 Building Damage Evaluation in Golcuk

5.7.1 Building System Characterization

In order to examine the effects of the PGA on the building damage, strength demands of the simulated ground motions are computed for simplified single-degree-of-freedom (SDOF) building systems with elasto-plastic force-displacement relations. To determine displacement ductility demands and damping factors of the systems required in the analyses, microtremor measurement was performed at two R/C buildings for residential use; one is located near Site G01 and the other about 1km east of Adapazari downtown. These buildings are both five-storied, almost squared, and have a similar structural system, but Golcuk one is no damaged and the other heavily damaged (Photos 5.4-5.6). The damaged one in Adapazari subsided by about 0.5–1m, and this subsidence might be due to soil liquefaction. At the top and ground floors of the both buildings, the two orthogonal horizontal (longitudinal and transverse) microtremor motions were observed. The test equipments, sampling conditions, and data processing used in the measurement were all the same as before.

Photo 5.4. Non-damaged building that microtremor measurement was performed.

Photo 5.5. Heavily damaged building at which microtremor measurement was performed.

Photo 5.6. Subsidence and damaged infill wall of the building shown in Photo 5.5.

Amplification Factor

Figure 5.12 shows the amplification factors between the top and ground floors of these buildings estimated from spectral analyses of the microtremor data, which are in the most damaged direction of the observed two horizontal motions. Fundamental period of the non-damaged building could be 0.3s, while that of the damaged one be almost 0.7s. Assuming that the effects of soil-structure interaction on the observed amplification factors are insignificant and that the dynamic response characteristics of the both buildings before the earthquake are almost same, it is suggested that fundamental period of the medium-rise R/C building could vary two times larger due to structural damage. Thus, it is indicated that the maximum response ductility of medium-rise R/C buildings might be about/over 4 during the 1999 Kocaeli earthquake. Damping factor of the non-damaged building was also estimated from the observed amplification factors, and the estimated value is 0.06.

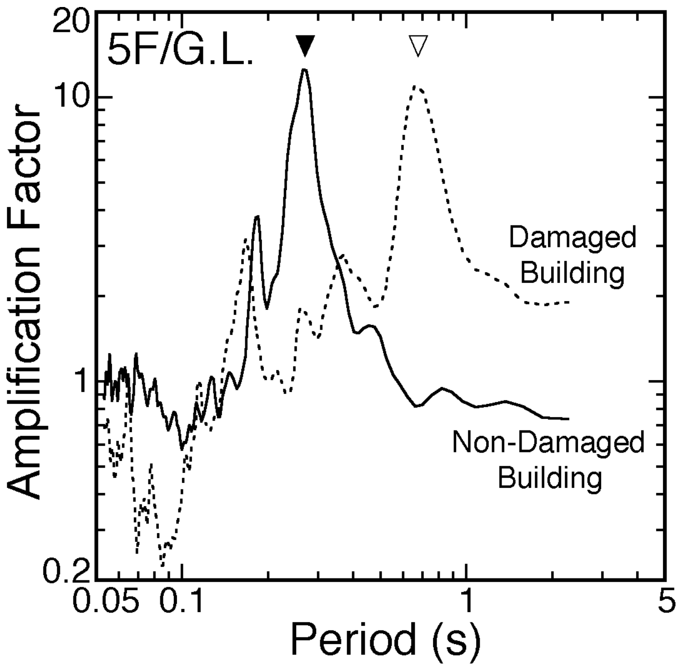


Figure 5.12. Amplification factors between the top and ground floors of the damaged and non-damaged buildings, which are in the most damaged direction of the observed two horizontal motions.

Fundamental Period

In Turkish building code established in 1975 (Ministry of Public Works and Settlement, Turkey, 1975), approximate fundamental period of building, T (s) is presented as:

$$T = min\ (0.09H / \sqrt{D}, \lambda N) \quad (5.3)$$

where H is height of building, D is width of building in motion direction, N is number of stories, and λ is constant (0.1 or 0.07 for lower or higher stiffness). As for the non-damaged building, H, D, and N are 15, 8, and 5, respectively, and λ could be 0.1 considering the lower stiffness due to the infill wall. Using Eq. (5.3), the fundamental period T for the non-damaged building is then approximated, and the resulted value is 0.48s. This approximated value is larger than the fundamental period from microtremor measurement as shown in Figure 5.12. One of the causes of this disagreement might be due to the stiffness of infill wall, which might have an effect on micro-vibration characteristics of the building.

5.7.2 Effects of Ground Motion Characteristics on Building Damage

Based on the above results, displacement ductility demands μ used in the strength demand analyses are assumed to be equal to 2, 4, and 6, and damping factor of the SDOF system is taken equal to 0.06. In the analysis, hysteresis rule of the force-displacement relations of the system was defined by degrading bi-linear model. The base shear coefficient, i.e., strength demand, of the SDOF system was determined using an iteration procedure so that the misfit between the computed and given displacement ductility be sufficiently insignificant.

Figure 5.13(a) shows spatial variations of the computed strength demand spectra for the simulated ground motions along the line using the gradations shown in the legend, in which μ is taken equal to 4. Strength demand spectra in the heavily damaged zone have a significant peak at a period of 0.3s. This peak period is consistent with the observed natural period of the medium-rise R/C building shown in Figure 5.12. Then, spatial variations of the strength demands computed for the SDOF system with a natural period of 0.3s are shown in Figure 5.13(b), in which μ are taken equal to 2, 4, and 6. In each ductility demand, the spatial variation of the computed strength demands is in fairly good agreement with both that of the collapse ratios of medium-rise R/C buildings shown in Figure 5.10 and that of the PGA shown in Figure 5.9. This reveals the fact that the estimated V_S profiles and ground motions are reliably reasonable and that the damage distributions of the medium-rise R/C building in Golcuk could be mainly controlled by the PGA during the earthquake, in other words, by the site effects as discussed previously.

Most of the damaged buildings in this area could be designed by the Turkish building code established in 1975. According to the design code, base shear coefficient of medium-rise R/C building could be 0.1 in this area. In Figure 5.13, however, the values of the computed strength demands for the buildings on the southern hill, where building damage was slight, are 0.3-0.4, which are larger than those of the design code. This indicates that the medium-rise R/C buildings in the area might have horizontal strength more than 0.3-0.4. The same suggestion could be found in the report on the damage investigations of the 1992 Erzincan, Turkey earthquake (Architectural Institute of Japan, 1993).

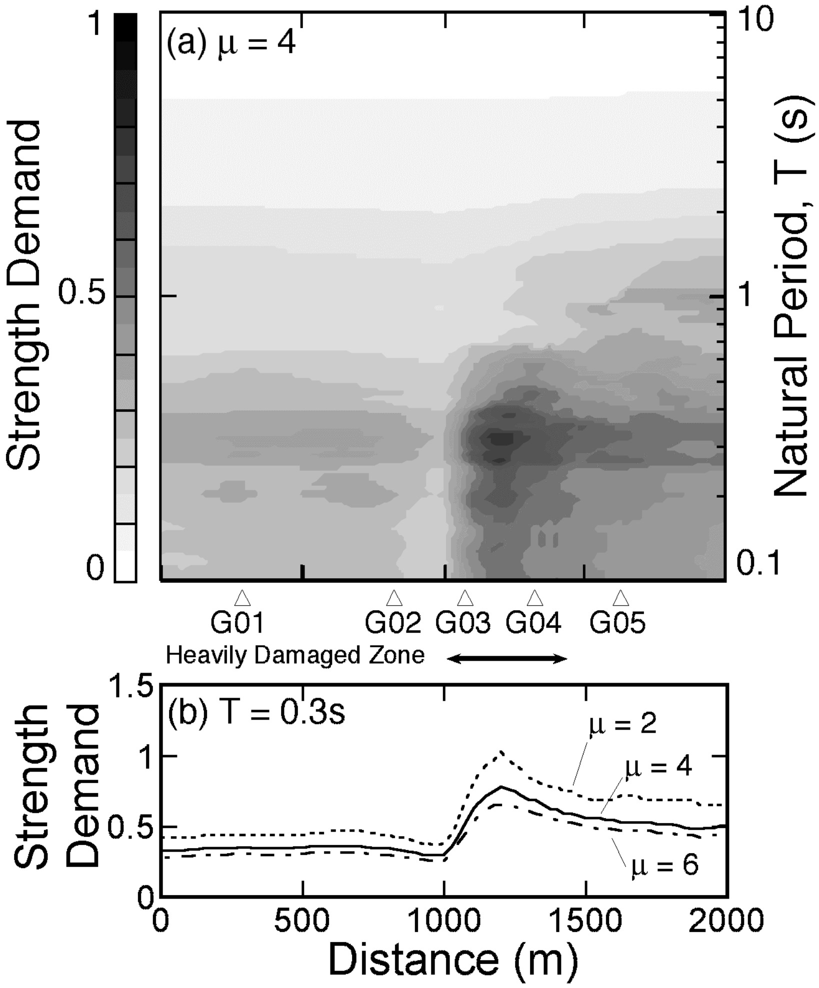


Figure 5.13. Variation of strength demand spectra computed for the simplified SDOF building systems along the observation line (Refer to color Figure 15).

5.8 Summary

The two-dimensional V_S profile in Golcuk across the damaged area due to the 1999 Kocaeli earthquake was evaluated using the microtremor H/V inversion method. With this profile, ground motions during the earthquake were simulated by a two-dimensional response analysis, and then strength demands of the ground motions were computed for simplified SDOF building systems. The evaluated ground and building responses are consistent with the observed damage distribution in the area. This result indicates that the estimated V_S profile and ground motions are reasonable and reliable, and that the microtremor H/V method employed in this study could be reliable means for estimating the S-wave velocity profiles and the effects of surface geology on ground motion characteristics and building damages during earthquakes.

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