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Horizontal and vertical components of earthquake ground motions at liquefiable sites

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Abstract

Field observations on ground motions from recent earthquakes imply that current knowledge is limited with regard to relating vertical and horizontal motions at liquefiable sites. This paper describes a study with the purpose of clarifying this emerging issue to some extent. A series of numerical analyses is carried out on a liquefiable soil deposit with a verified, fully coupled, nonlinear procedure. It is shown that the transformation of vertical motions in the deposit differs considerably from the transformation of horizontal motions. Both the amplitude and frequency content of the horizontal motions are strongly dependent on the shaking level or the associated nonlinear soil behavior. The transfer function for vertical motions is however likely to be independent of the intensity of input motions; no reduction in the amplitude occurs even in the case of strong shaking. The results are shown to be in consistence with the laboratory observations on shaking table tests and recent field observations that less nonlinearity exists for vertical motions. It is also shown that the possibility exists for using information on spectral ratios between the horizontal and vertical surface motions to quickly identify in situ soil behavior and liquefaction that are not readily covered by conventional field or laboratory experimentation procedures. © 2002 Elsevier Science Ltd. All rights reserved.

Keywords: Site response; Ground motion; Vertical motion; Nonlinear analysis; Liquefaction

1. Introduction

One of the most important problems in earthquake geotechnical engineering is the evaluation of site response to earthquakes. Through site response analyses one may predict ground surface motions for developing design response spectra, determine the earthquake-induced forces that can cause instability of earth structures, and evaluate dynamic stresses and strains for assessment of liquefaction potentials. So far a number of procedures for site response analysis have been developed and extensive studies have been conducted, e.g. see Refs. [1–6]. These studies have produced useful understanding that advanced the practice of earthquake geotechnical engineering. However, it is to be realized that most of the studies have been concerned with horizontal ground motion. In this case, site response is usually regarded as the consequence of vertical propagation of shear waves in a horizontally layered system. Although it has long been recognized that in reality the ground is simultaneously subjected to shaking in both horizontal and vertical directions, attention to the vertical ground motion is quite limited and hence available understanding is much less than that for horizontal ground motion.

During the 1994 Northridge and the 1995 Kobe earthquakes, field observations [7,8] on vertical ground motions showed quite different features from those of the horizontal motions. For example, the three-dimensional downhole array records at the reclaimed Port Island in Kobe (see Fig. 1) showed that while the horizontal peak accelerations were reduced as seismic waves traveled from bottom to surface, the vertical motion was significantly amplified at the surface, resulting in the ratio of peak vertical to horizontal acceleration at the surface as large as 1.5 to 2. This value substantially exceeded 2/3, a value commonly used in engineering practice. In a recent North America–Japan workshop [9], it was recognized that current knowledge is limited with regard to relating horizontal and vertical motions in soft soil sites, especially in liquefiable sites.

Vertical ground motion, as a first approximation, may presumably be considered to be mainly related to P waves although in a realistic seismic environment there exist other contributions (e.g. surface waves and converted SV waves).

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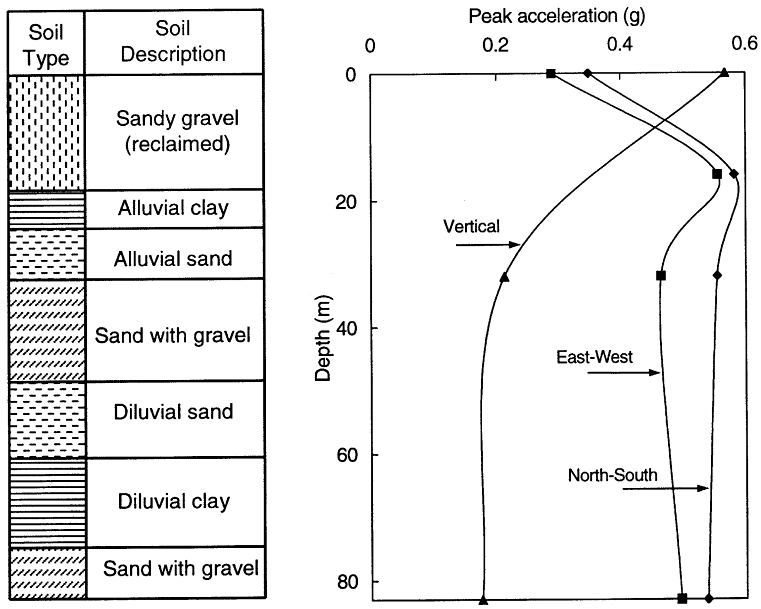

Fig. 1. Distribution of peak acceleration with depth: Port Island records.

Since the velocity of P waves in soil is strongly affected by pore water saturation, it is more reasonable to treat the soil media as a multiphase system in considering the vertical ground motion. Yang and Sato [10,11] and Yang et al. [12] have conducted analyses in detail drawing attention to the typical case from the Kobe array site mentioned earlier. The results convincingly revealed that the characteristics of ground motions in both horizontal and vertical components were closely associated with the surficial liquefied soils. Stimulated by these analyses, a research need appears in clarifying some issues that are fundamentally interesting in relation to vertical and horizontal ground motions, such as (1) what is the performance of seismic amplification in both horizontal and vertical components at soft liquefiable sites for a broad range of shaking levels, (2) whether or not the characteristics of vertical ground motion are affected by the liquefaction/nonlinearity of soils, and (3) how the horizontal ground motion is influenced by the inclusion of vertical excitation. These points would also be of interest for a technique for site evaluation generally known as H/V, which is based on the interpretation of field observations on both horizontal and vertical components of microtremors/ground motions [13,14] and has increasingly drawn attention in engineering practice.

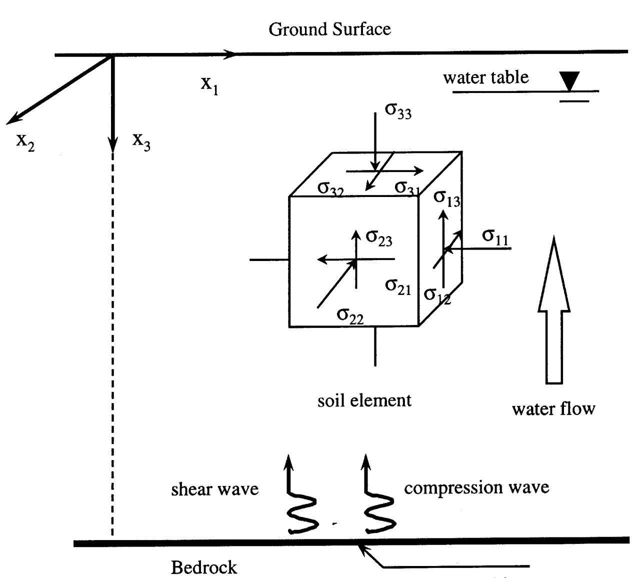

Fig. 2. Physical configuration for site response analysis.

In this paper an analytical study is described with the purpose of clarifying the aforementioned issues to some degree. By using a verified, fully coupled, inelastic finite element procedure, a series of numerical analyses is carried out on a hypothetical sand profile subjected to a variety of levels of horizontal and vertical motions scaled from actual near-field recordings. The results are discussed in detail and compared with experimental observations.

2. Numerical procedure and constitutive model

The numerical procedure for earthquake ground response analysis was formulated on the basis of vectored motion, effective stress, transient pore fluid movement and generalized material stiffness [15,16]. As illustrated in Fig. 2, in this procedure ground response is regarded as the consequence of vertical propagation of shear and compression waves induced by multidirectional earthquake motions originating primarily from the underlying rock formation [17]. A bounding surface hypoplasticity model [18] was incorporated into the procedure. The model was formulated in a three-dimensional space within the framework of bounding

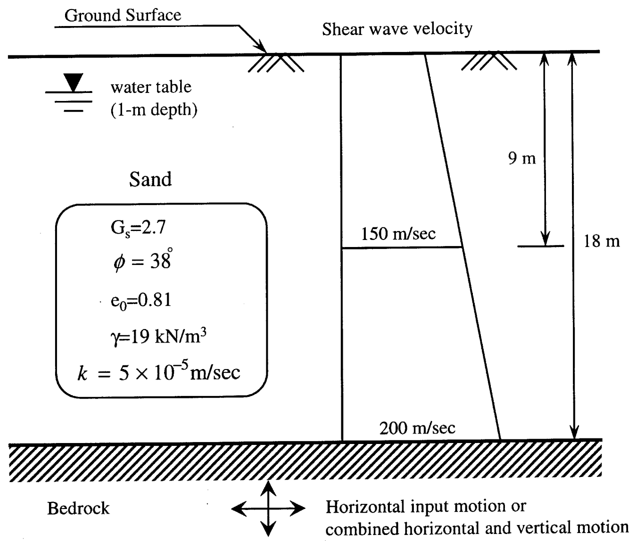

Fig. 3. A hypothetical sand deposit used in numerical analyses.

surface hypoplasticity [19], which characterizes incremental nonlinearity of the stress–strain rate relations based on the postulation that the stress–strain rate relationship depends not only on the current stress state but also on the stress rate itself. This model is capable of realistically simulating the soil behavior under a wide range of loading conditions such as the compression- and dilation-induced effective stress change; particularly, it may successfully simulate the soil response to a class of complicated loading conditions known as rotational shear that are the cases induced by multidirectional earthquake loading. Specified by the loading conditions related to the free field response, the model may take a reduced-order form and has nine model parameters to be determined for a particular soil as described in Appendix A. Predictive capability of this procedure has been verified using the three-dimensional field observations, such as the records of moderate level of shaking due to the 1986 Lotung earthquake [20] and, particularly, the records at the liquefied reclaimed site subjected to the strong ground motions due to the Kobe earthquake [12]. The predictive capability of this procedure has also been verified using laboratory centrifuge tests through the VELACS project [21]. A complete description of the procedure and the constitutive model is beyond the scope of this paper. Details can be found in the references given earlier.

Table 1
Model parameters

Parameter	Value
ϕ	38°
G_0	190.4
λ	0.0085
κ	0.0017
R_p/R_f	0.75
h_r	0.119
d	1.0
k_r	0.15
b	2
h_p	35

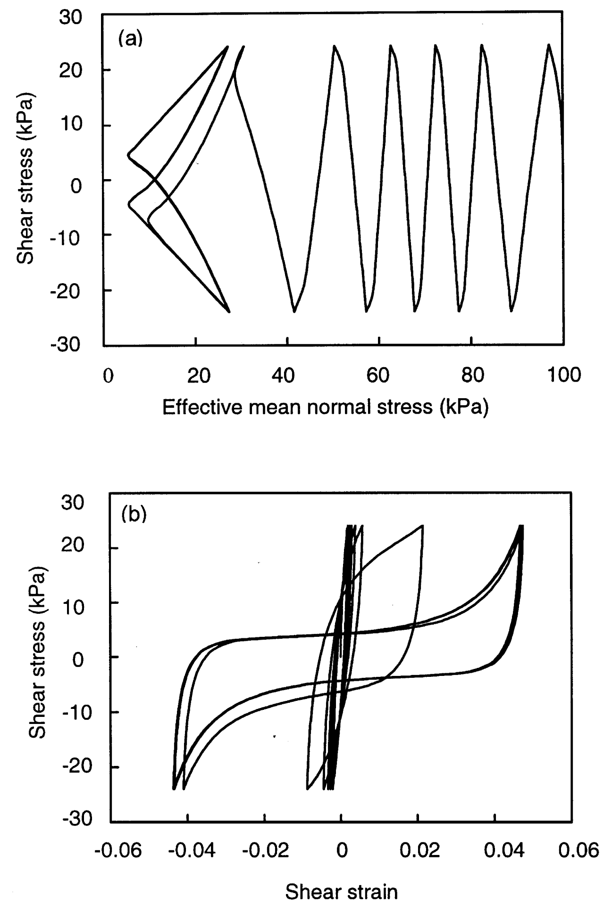

Fig. 4. Undrained cyclic behavior of sand simulated by constitutive model.

3. Sand deposit and input motions

A hypothetical sand deposit of thickness 18 m, as illustrated in Fig. 3, is used in the numerical tests. The deposit is assumed to rest on rigid bedrock that is impermeable. The water table is specified at 1 m depth below the ground surface. At that depth the pore water is free draining and below that depth the sand is fully saturated. The shear wave velocity of the deposit is assumed to vary linearly with depth. Fig. 4 shows the undrained behavior of the sand under cyclic simple shear, which is simulated by the hypoplasticity bounding surface model described previously. The basic properties of the sand are given in Fig. 3 and the model parameters are listed in Table 1. It is clear that the cyclic mobility effect and the significant reduction of stiffness associated with large shear strain upon liquefaction are well reproduced.

The original motions in horizontal and vertical components employed in the analyses are shown in Fig. 5,

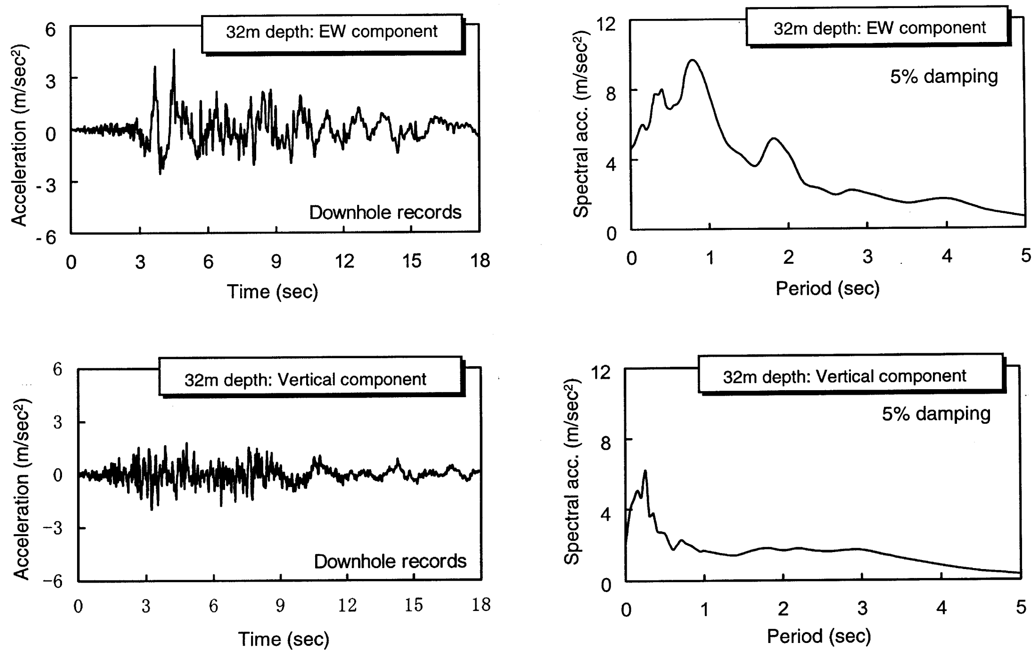

Fig. 5. Horizontal and vertical acceleration records employed.

together with their response spectra at a damping of 5%. These records are taken from the actual acceleration histories in the east–west and vertical directions at a depth of 32 m at the reclaimed array site during the Kobe earthquake. The peak accelerations in the east–west and vertical directions were about 0.471 and 0.204 g, respectively. The actual recordings are considered to suitably represent some characteristics (e.g. amplitude, frequency content and wave forms, etc.) of near field motions. In the numerical analyses a variety of levels of input motions in horizontal and vertical components are produced by scaling.

4. Results of analyses

4.1. Amplification of horizontal and vertical ground motions

To investigate the seismic amplification in both the horizontal and vertical directions and the influence of vertical base excitation, a series of case studies is carried out for the deposit subjected to, respectively, the combination of horizontal and vertical input motions and the horizontal input motion only. In Fig. 6(a), the peak horizontal acceleration at the ground surface is presented as a function of peak horizontal acceleration at the base. To illustrate the influence of the vertical excitation, both the case of horizontal input motion only and the case of simultaneous horizontal and vertical input motions are included in this plot. In Fig. 6(b) the peak vertical acceleration at the surface is plotted against the peak vertical acceleration at the base.

The results indicate that the influence of vertical motion seems to be small on the amplification in horizontal component, especially when the peak horizontal acceleration at the base is below about $0.2g$. In both the case of horizontal input motion only and the case of simultaneous horizontal and vertical input motions, almost the same amplification is observed in the horizontal component: the amplitude of horizontal acceleration is amplified at the surface when the peak horizontal acceleration input is below about $0.2g$, whereas the amplitude is reduced when the peak horizontal acceleration at the base is beyond $0.2g$. It is interesting to note that, in the range of strong excitation, a difference exists between the case of horizontal input motion only and the case of combined horizontal and vertical input motions, implying a possible coupling effect might exist between these two components in the range of strong motion. As far as the vertical motion amplification is concerned, the results shown in Fig. 6(b) clearly indicate that vertical motion is significantly amplified at the surface in all the cases of input motion. No reduction in the amplitude takes place when the seismic waves travel from the base to the surface.

The variation of the peak horizontal acceleration at the surface with that at the base appears to be related to the nonlinear soil behavior, being consistent with the field observations obtained in past earthquakes [22,23] as shown in Fig. 7. For liquefiable sites, particularly, the nonlinear performance is directly associated with the build-ups of pore pressure in subsoil generated by a strong shaking; this has been well demonstrated by the analysis [12] for the Port Island array site which liquefied during the Kobe earthquake. The results on the transformation of horizontal motions are also found to be in good agreements with the observations on the shaking table tests performed by Mori et

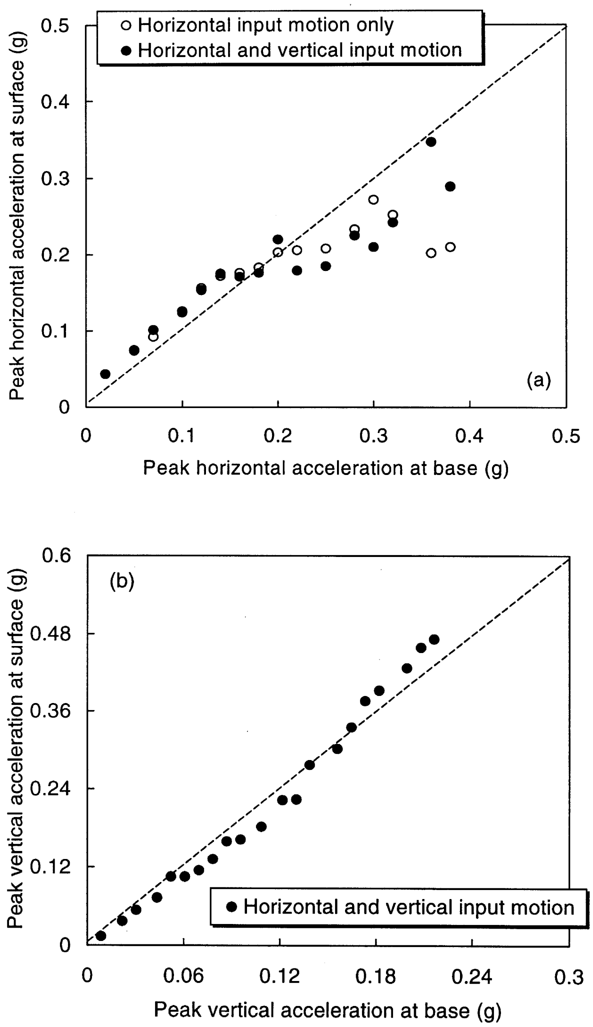

Fig. 6. Peak acceleration at surface versus peak acceleration at base: (a) horizontal component, and (b) vertical component.

al. [24]. Fig. 8 shows the instrumented model ground in the shaking table. The model ground is composed of a saturated granular soil whose properties are given in Table 2. According to the similitude law this model ground represents a saturated deposit of a thickness of 9 m at prototype scale. For the sake of comparison, Fig. 9(a) shows the test data for the peak horizontal acceleration recorded at the surface versus the peak acceleration input at the bottom in the case of horizontal shaking only and in the case of both horizontal and vertical shaking.

On the other hand, the findings on the transformation of

Table 2
Physical properties of the granular soil used in shaking table tests [24]

Density of grains ρ_s (kg/m^3)	2646
Maximum void ratio e_{max}	1.059
Minimum void ratio e_{min}	0.608
Maximum size of particles D_{max} (mm)	4.75
Mean size of particles D_{50} (mm)	0.48
Uniform coefficient U_c	5.9

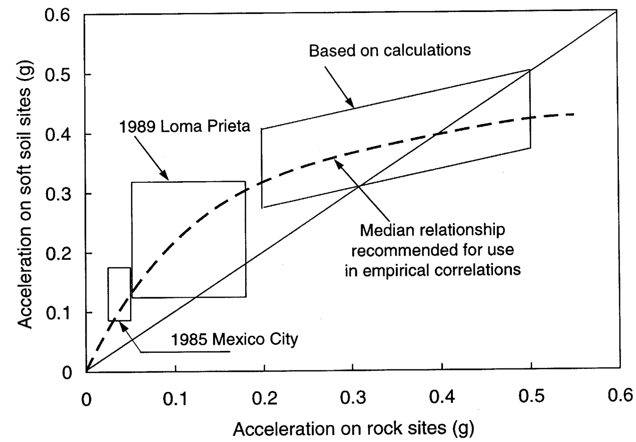

Fig. 7. Relationship between maximum acceleration on rock and soft soil sites [23].

vertical motions apparently are consistent with those obtained from the shaking table tests shown in Fig. 9(b) The less nonlinearity for vertical motions has also been displayed by field observations in past earthquakes such as the Northridge and Kobe earthquakes [7,8], although in the meantime significant nonlinearity was noticed for horizontal motions during these events. Moreover, it is to be noted that, compared to the test results, a larger amplification in vertical motions is observed in the numerical analyses. This difference is considered to be caused by the fact that the model ground in the numerical analyses consists of a 1 m top layer of dry soil whereas in the shaking table tests the model ground is totally saturated.

The distributions of peak acceleration with depth for four levels of input motion (with input peak horizontal accelerations of 0.05, 0.1, 0.2, and 0.3g denoted as Levels 0, 1, 2, and 3, respectively) are shown in Fig. 10. Fig. 10(a) shows the distribution of peak horizontal acceleration for the case of horizontal input motion only, Fig. 10(b) depicts the distribution of peak horizontal acceleration for the case of simultaneous action of horizontal and vertical loading, and

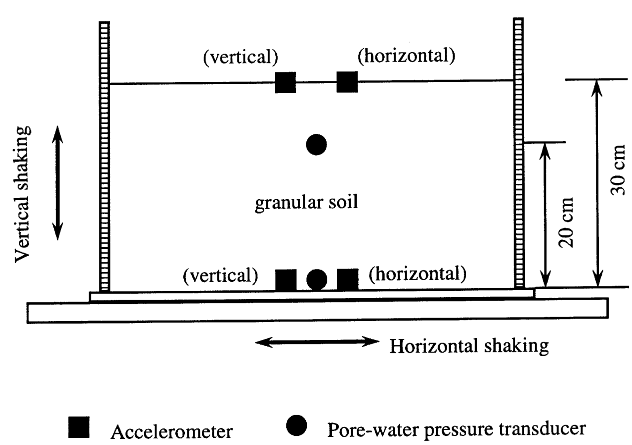

Fig. 8. Instrumented model ground in shaking table tests [24].

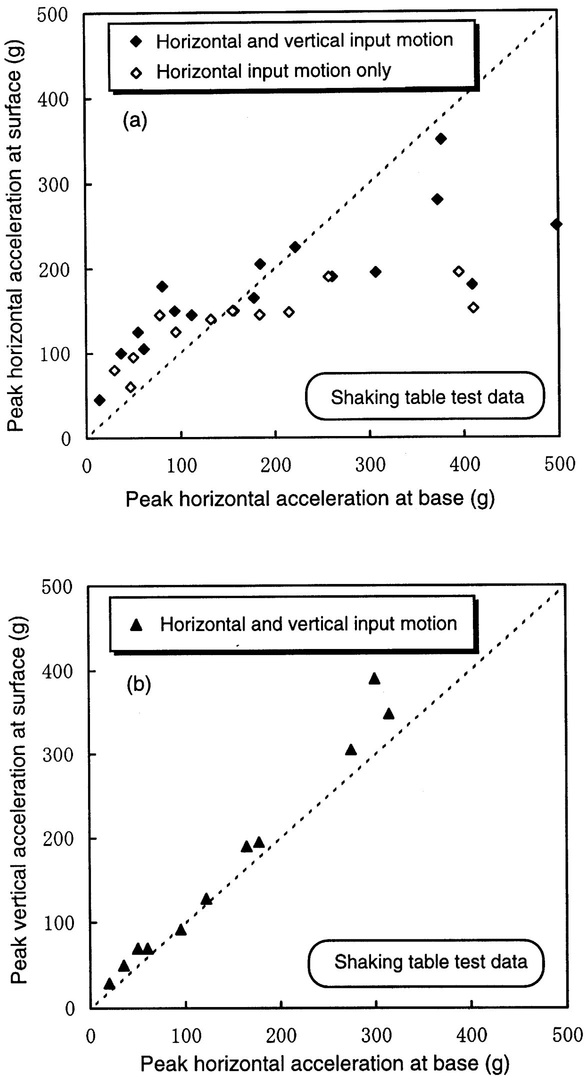

Fig. 9. Peak acceleration recorded at surface versus peak acceleration input at base: (a) horizontal component, and (b) vertical component.

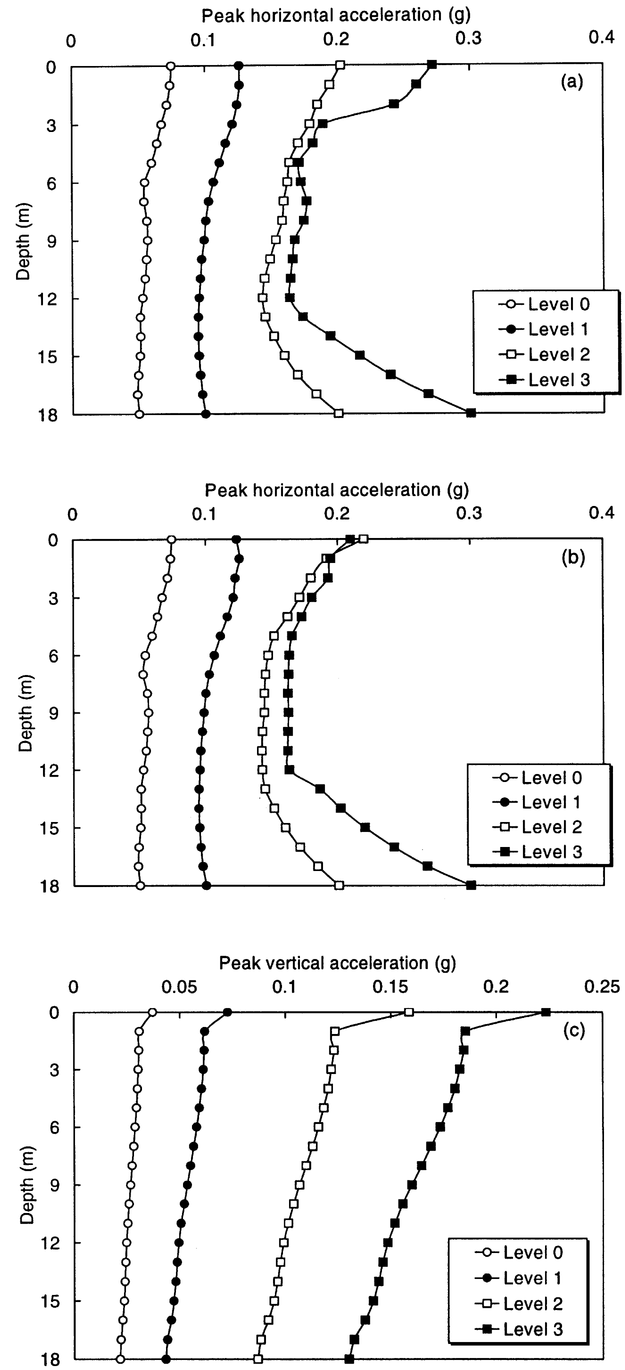

Fig. 10. Distribution of peak acceleration with depth at four levels of shaking.

Fig. 10(c) shows the distribution of peak vertical acceleration. A comparison of Fig. 10(a) and (b) suggests that the trend of the distribution in the case of horizontal input motion only is similar to that in the case of combined horizontal and vertical input motions. The distribution of peak horizontal acceleration for the strongest shaking exhibits a different behavior from that for lower levels of shaking. This difference is attributed to the liquefaction-induced nonlinearity as will be shown later. Whereas for the vertical motion, for all the four levels of input motion, the distribution generally exhibits an identical manner, namely, with decreasing depth the peak vertical acceleration increases.

A further investigation of the characteristics of the Fourier spectral ratios between the motions at the surface and at the base (i.e. transfer functions) for both the horizontal and vertical components is useful in understanding the performance of ground motion amplification described earlier. In Fig. 11(a) the spectral ratios for horizontal motion in the case of horizontal input motion only are shown. Two typical levels of shaking (i.e. Level 0 and Level 3), which are considered to represent low and strong intensities, respectively, are presented. The corresponding results for the case of simultaneous action of horizontal and vertical motions are illustrated in Fig. 11(b). Fig. 11(c) shows the spectral ratios for vertical component of motion.

It is observed from Fig. 11(a) that when the level of input motion is relatively low, the predominant frequency is

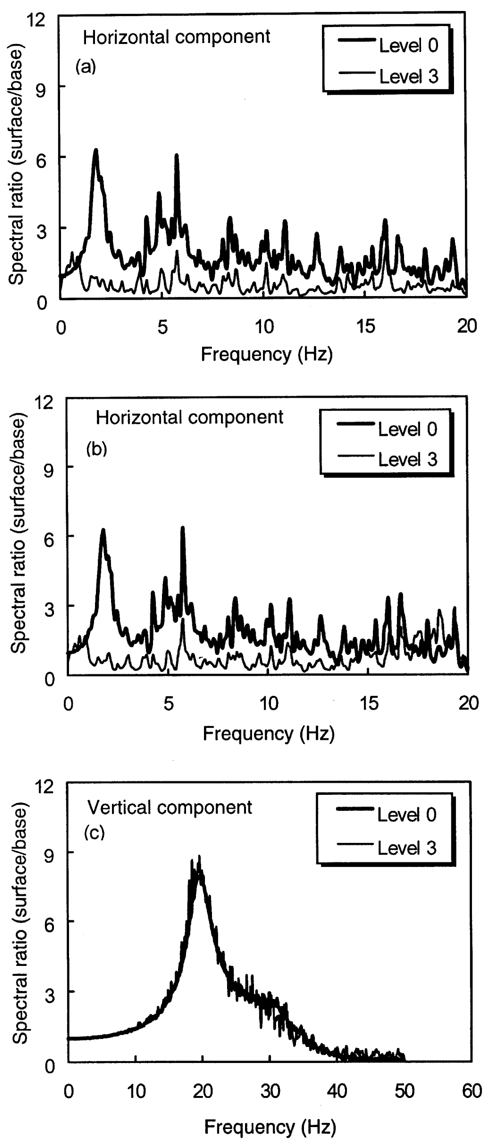

Fig. 11. Spectral ratios (surface/base) in horizontal and vertical components.

located at around 2 Hz. The average shear wave velocity for the deposit corresponding to this frequency can be identified as about 144 m/s, a value that is almost identical with the average initial shear wave velocity as shown in Fig. 3. This implies that for this level of shaking the soil responds generally in a slight nonlinear manner. When the level of shaking is stronger (for Level 3), the spectral ratios exhibit a significantly different behavior: high-frequency components are now substantially reduced and the peak frequency is shifted to the low frequency end, about 1 Hz. The evaluated shear wave velocity is 72 m/s, showing a substantial reduction associated with the nonlinear behavior of the sand. In addition, by comparing Fig. 11(a) with (b), it is found that they are generally very similar, suggesting a minor influence of vertical excitation on the amplification behavior in horizontal component.

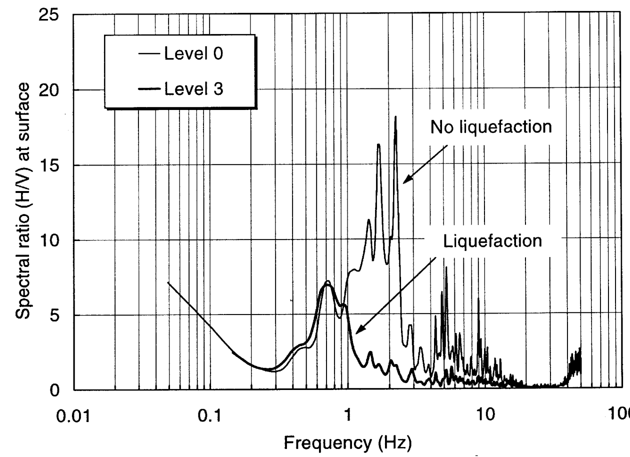

Fig. 12. Spectral ratios (H/V) at ground surface at low and high levels of shaking.

As far as the vertical motion is concerned, a quite different behavior from that of the horizontal component is noticed in Fig. 11(c). The spectral ratios in both the cases of shaking levels are almost identical, with the peak frequency located at around 20 Hz. This frequency corresponds to the average velocity of P waves in the deposit with the value 1440 m/s, typical for the water-saturated sand considered. This finding implies that the amplification in vertical motion may not be affected by the nonlinear soil behavior associated with strong shaking. It also implies that it would be appropriate to simulate the vertical motion based on the assumption of linear soil behavior in compression even though the site is subjected to strong level of shaking. The less nonlinearity in vertical motions was indeed displayed by field recordings at the liquefied, downhole array site in Ref. [8].

The performance of the spectral ratios between the horizontal and vertical components (referred to as H/V) at the ground surface is of considerable interest, because it may provide some useful information on soil behavior with only one observation station such that one can make use of it to quickly identify in situ soil behavior. In Fig. 12 the spectral ratios H/V at the surface for the two typical cases of shaking levels are shown. It is clear that, when the input motion is weak (Level 0), the peak value of the ratio is very large accompanied by a higher peak frequency (about 2 Hz); whereas for the strong input motion that may cause nonlinear response of soils, the peak value is substantially reduced with a significantly reduced peak frequency 0.7 Hz. As will be shown later, the sand indeed exhibits quite different stress–strain behavior and pore pressure buildup for these two shaking levels; in the case of Level 0 the deposit does not liquefy, but in the Level 3 case it completely liquefies. The finding suggests that it would be possible to detect in situ nonlinear soil behavior or site liquefaction through the analysis of spectral ratios H/V at the ground surface.

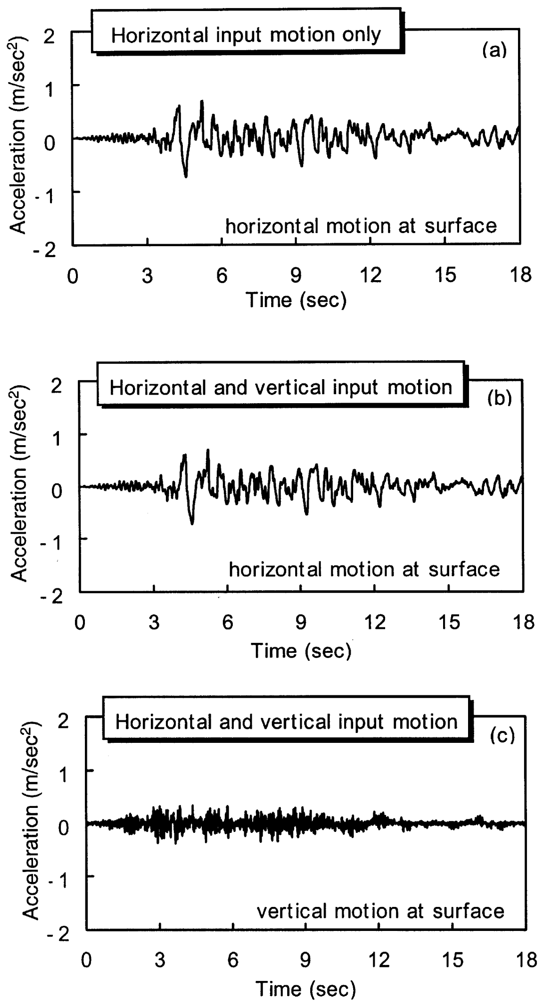

Fig. 13. Horizontal and vertical acceleration time histories at ground surface (case of Level 0).

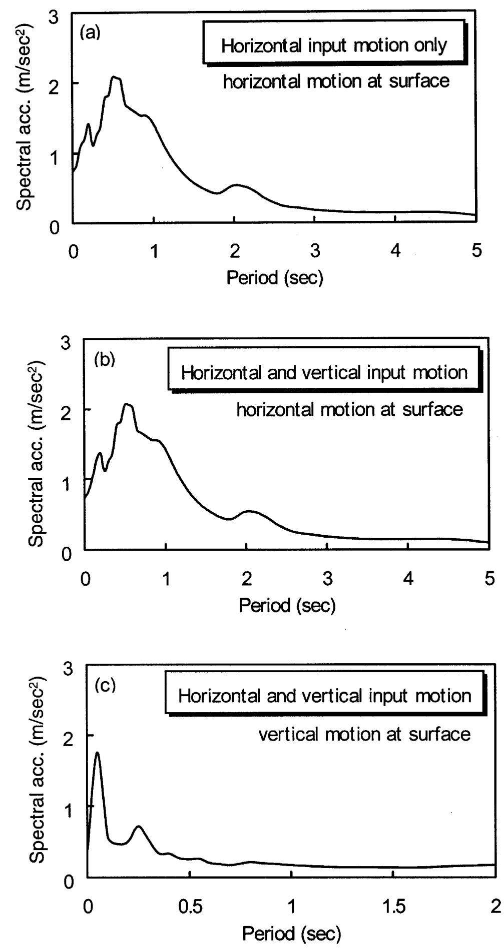

Fig. 14. Response spectra at a damping of 5% (case of Level 0).

4.2. Correlation of ground motion with soil behavior

The correlation of the characteristics of ground motions with the soil behavior such as the stress–strain history and the pore pressure buildup is investigated in this section. The influence of vertical motion on soil response will also be studied. Fig. 13 shows the acceleration time histories in horizontal and vertical components at the surface of the deposit for the case that the peak horizontal acceleration at the base is $0.05g$. Fig. 13(a) depicts the horizontal acceleration in the case of horizontal input motion only, Fig. 13(b) shows the results in the case of simultaneous action of horizontal and vertical motions, and Fig. 13(c) presents the acceleration time histories in the vertical component. The corresponding response spectra are shown in Fig. 14(a)–(c), respectively. The excess pore pressures at a depth of 4.5 m for the two cases of excitation are shown in Fig. 15, and the associated stress–strain histories at this depth are illustrated in Fig. 16.

In general, these results suggest that the influence of vertical base excitation is slight on the soil behavior. In either the case of horizontal input motion only or the case of horizontal and vertical input motion, the generated residual pore pressure ratio at the depth of 4.5 m is about 0.4. The inclusion of vertical excitation only causes some high-frequency oscillations in the response, as confirmed by the shaking table observations [24,25]. The lack of influence of vertical motion on the development of residual pore pressure is understandable. As described previously, in the level ground condition the soil element is not allowed to deform in horizontal direction. As a result, the propagation of vertical motion (corresponding to P waves) through saturated soils induces almost only compression stress while the induced deviatoric stress is nearly equal to zero. Since the compression stress is mainly transmitted through pore water, vertical motion can only produce changes in total stress which in turn cause instantaneous changes in pore pressure. This consideration is commonly accepted in geotechnical earthquake engineering [26,27].

The stress–strain histories exhibit a very small nonlinear manner, with a low level of shear strain (below 0.1%) and a slight reduction of shear modulus. This observation agrees well with the observation directly achieved on the basis of

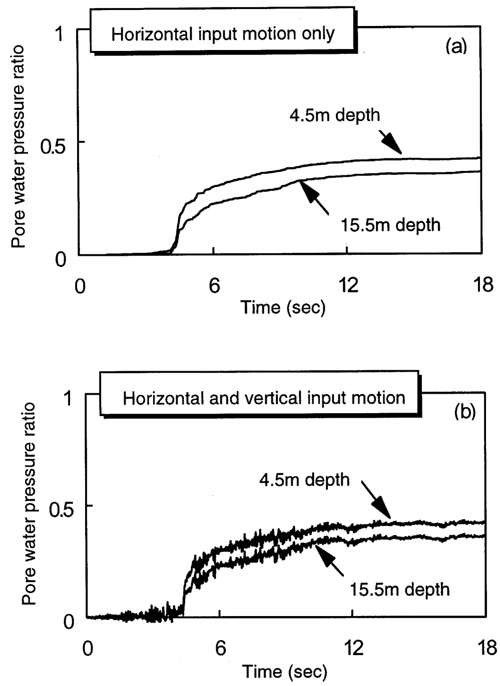

Fig. 15. Pore pressure response at a depth of 4.5 m (case of Level 0).

spectral ratio analysis as described previously. Corresponding to this shaking intensity, the response spectra for the horizontal surface motion take a peak value at a relatively low period (about 0.5 s), as shown in Fig. 14.

The results for acceleration time histories, response spectra, pore pressures, and stress–strain histories for the case that the peak horizontal acceleration at the base is 0.3g are shown in Figs. 17–20, respectively. Even for this strong level of shaking, the influence of vertical motion on the soil behavior is still slight, except causing some high-frequency oscillations in the later part of the surface horizontal motion. Because the shaking is strong, the generated pore pressure at the depth of 4.5 m now approaches the level of overburden pressure and meanwhile the level of shear strain is as large as 2.5%. An abrupt loss of soil stiffness upon liquefaction at 7 s can be observed. This significant nonlinear behavior is also manifested by the back-analysis based on the spectral ratios as shown before. Different from the case of weak shaking, long-period waves now become dominant in the horizontal motion in the case of strong shaking. The peak value appears at a period over 1 s in the response spectra. The correlation of soil behavior with the characteristics of ground motions described earlier has been displayed at several well instrumented realistic sites, especially at the liquefied downhole array site in Ref. [12]. As far as the vertical motions are concerned, the results imply that they are almost not affected by the liquefaction-induced nonlinearity, with very similar response spectra in both cases of shaking intensity.

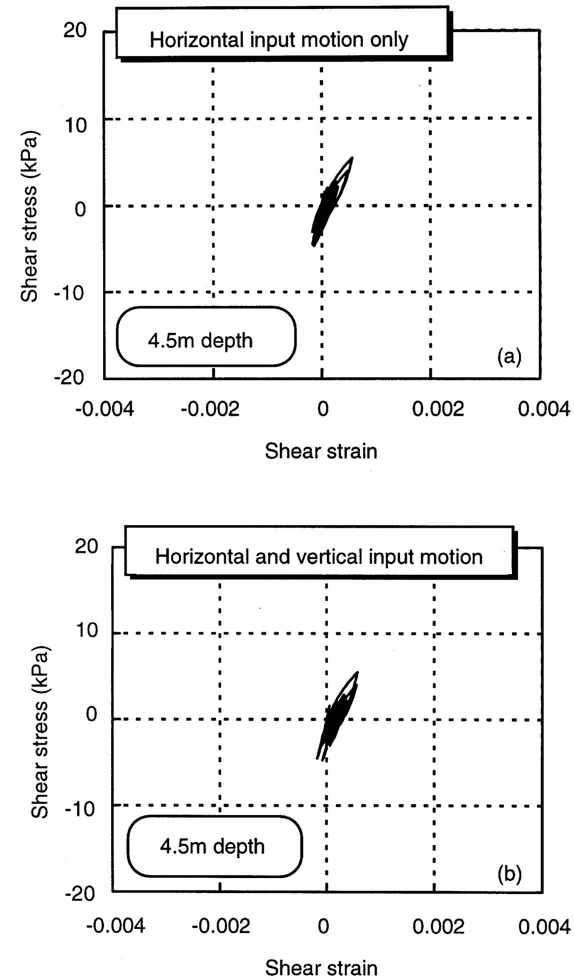

Fig. 16. Stress–strain histories at a depth of 4.5 m (case of Level 0).

5. Conclusions

An analytical study has been presented to identify the characteristics of earthquake motions in both the horizontal and vertical components at a liquefiable deposit. The results of analyses are summarized as follows:

1. The influence of vertical motions is generally small on the transformation of horizontal motions. In either the case of horizontal input motion only or the case of combined horizontal and vertical input motions, the horizontal motion is amplified at the ground surface for low levels of shaking, whereas for strong levels of input motion, the liquefaction-induced nonlinearity often prevents the horizontal acceleration from developing peak values as large as those input at the base.
2. The behavior of seismic amplification in the vertical component differs considerably from that in the horizontal component. In all the cases of shaking levels considered, the vertical motion is found to be amplified significantly at the surface. No reduction in amplitudes occurs when seismic waves traveling

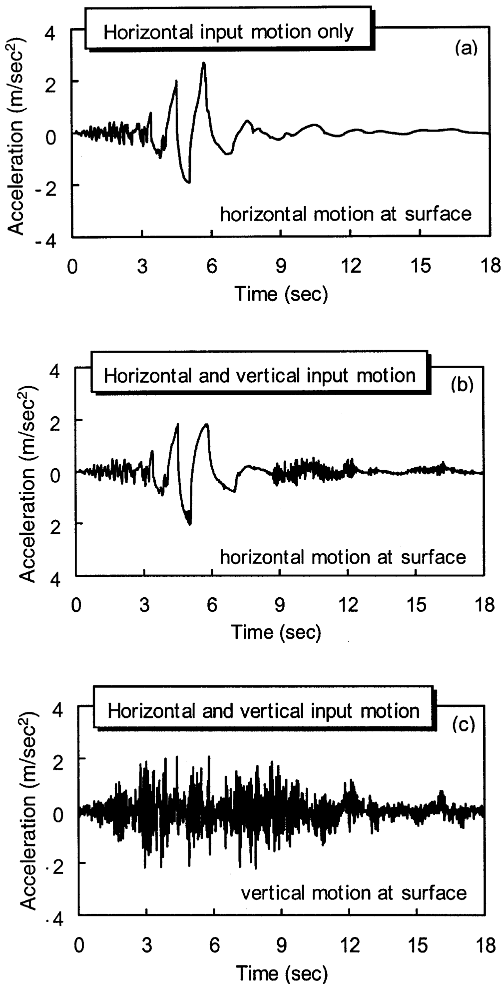

Fig. 17. Horizontal and vertical acceleration time histories at ground surface (case of Level 3).

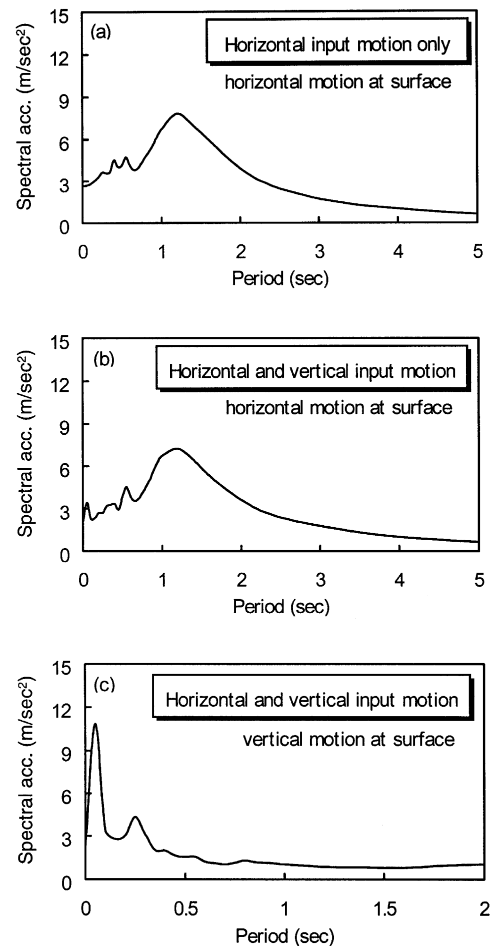

Fig. 18. Response spectra at a damping of 5% (case of Level 3).

from the base to the surface, even though soil liquefaction takes place as the result of strong shaking. This finding agrees well with the observations on the shaking table tests.

3. The transfer function for vertical motions is likely to be independent of the intensity of shaking or the associated nonlinear soil behavior, implying that the approach to simulating vertical motions based on the assumption of linear behavior in compression would be appropriate even in the case of strong shaking.
4. The inclusion of vertical motions produces a minor influence on stress–strain histories as well as pore pressure buildups, aside from causing some high-frequency oscillations in the response. This observation is consistent with laboratory findings that the influence of vertical motion on soil liquefaction is slight.
5. The spectral ratios of horizontal to vertical motions (H/V) at the ground surface show a substantial difference in both amplitude and frequency content between the case of low level of shaking and the case of strong shaking. The difference could be used to identify instrumented sites that may have liquefied but have not shown the usual surface manifestations.

Vertical ground motion and its relation to horizontal ground motion are beginning to appear to be important. The study reported here represents a preliminary contribution to this topic. There remain a number of issues to be studied continuously by means of analytical and experimental methods, for example, the effects of partial saturation of soil [10,28], the contribution of surface waves and the diffraction effects due to surface topography [29]. It is hoped that the present study would draw more attention and effort to this topic so as to improve current understandings and engineering practice.

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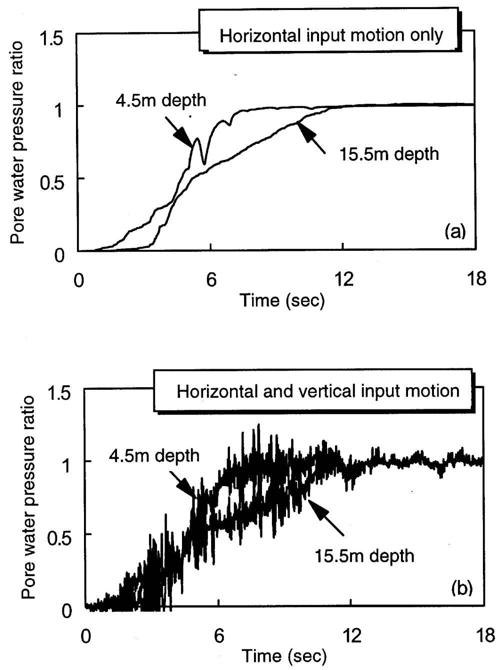

Fig. 19. Pore pressure response at a depth of 4.5 m (case of Level 3).

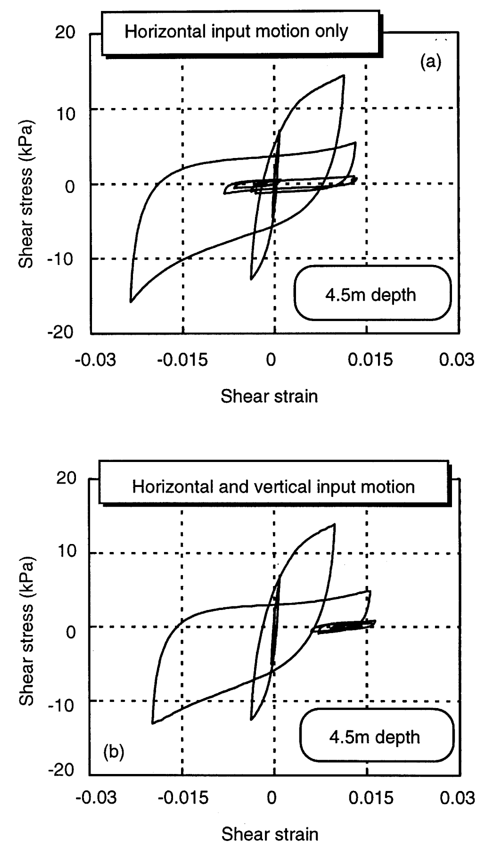

Fig. 20. Stress–strain histories at a depth of 4.5 m (case of Level 3).

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Appendix A. Parameters for constitutive model

The model parameters are described as follows: (1) a coefficient G_0 defining elastic shear modulus using the following equation $G_{max} = G_0\sqrt{pp_{atm}}(2.973 - e_0)^2/(1 + e_0)$, where e_0 is the initial void ratio, p is the effective mean normal stress and p_{atm} is atmospheric pressure; (2) the slope λ of the virgin compression line; (3) the slope κ of the rebounded line; (4) the ratio R_p/R_f between the slope R_p of the phase transformation line and the slope R_f of the failure line; (5) the term h_r characterizing the relationship between shear modulus and shear strain magnitude; (6) the term d, which characterizes the rate of the effective mean normal stress change caused by shear unloading; (7) the term k_r, which characterizes the amount of the effective mean normal stress change caused by shear loading; (8) b, a parameter affecting the shape of the stress paths of the virgin shear loading; and (9) h_p, a parameter controlling the amount of the shear strain increment due to the change of the maximum effective mean normal stress. Among these parameters, h_p is active only when the mean normal stress exceeds its maximum value in the loading history. During earthquakes the mean normal stress in soil is almost always less than its initial value, thus h_p is inactive for ground response analyses, with a typical value of 35. The parameter b is also inactive in the computation, with a typical value of 2. The remaining parameters should be calibrated specifically for a given soil either by laboratory tests or from field data.

References

[1] Schnabel PB, Lysmer J, Seed HB. SHAKE: a computer program for earthquake response analysis of horizontally layered sites, Report UBC/EERC 72-12. Berkeley: Earthquake Engineering Research Center, University of California, Berkeley, 1972.

[2] Lee MKW, Finn WDL. DESRA-2: dynamic effective stress response analysis of soil deposits with energy transmitting boundary including assessment of liquefaction potential, Soil mechanics series No. 38. Vancouver: University of British Columbia, 1978.

[3] Prevost JH. Effective stress analysis of seismic site response. Int J Numer Anal Meth Geomech 1986;10(6):653–65.

[4] Seed RB, Dickinson SE, Mok CK. Recent lessons regarding seismic response analyses of soft and clay sites. Proceedings of Seminar on Seismic Design and Retrofit of Bridges. California Department of Transportation, 1992.

[5] Finn WDL, Ventura CE, Wu G. Analysis of ground motion at Treasure Island Site during the 1989 Loma Prieta earthquake. Soil Dynam Earthquake Engng 1993;12(7):383–90.

[6] Oka F, Sugito M, Yashima A, Furumoto Y, Yamada K. Time dependent ground motion amplification at reclaimed island after the 1995 Hyogo-Nanbu earthquake. Proceedings of the 12th World Conference on Earthquake Engineering. Auckland, paper no. 2046, 2000.

[7] Trifunac MD, Todorovska MI. Nonlinear soil response—1994 Northridge, California, earthquake. J Geotech Engng, ASCE 1996; 122:725–35.

[8] Iwasaki Y, Tai M. Soils Foundations 1996:29–40 Special Issue on Geotechnical Aspects the January 17 1995 Hyogoken-Nanbu Earthquake.

[9] Bardet JP, Adachi N, Idriss IM, Hamada M, O'Rourke TD, Ishihara K. North America–Japan workshop on the Kobe, Loma Prieta, and Northridge earthquakes. Report to National Science Foundation and Japanese Geotechnical Society, 1997.

[10] Yang J, Sato T. Interpretation of seismic vertical amplification observed at an array site. Bull Seismol Soc Am 2000;90(2):275–85.

[11] Yang J, Sato T. Analytical study of saturation effects on seismic vertical amplification of a soil layer. Geotechnique 2001;51(2): 161–5.

[12] Yang J, Sato T, Li XS. Nonlinear site effects on strong ground motion at a reclaimed island. Can Geotech J 2000;37(1):26–39.

[13] Nakamura Y, Ueno M. A simple estimation method of dynamic characteristics of subsoil. Proceedings of the Seventh Japan Earthquake Engineering Symposium, 1986 p. 265–70.

[14] Theodulidis NP, Bard PY. Horizontal to vertical spectral ratio and geological conditions: an analysis of strong motion data from Greece and Taiwan (SMART-1). Soil Dynam Earthquake Engng 1995;14: 177–97.

[15] Biot MA. Theory of propagation of elastic waves in a fluid saturated porous solid. J Acoust Soc Am 1956;28:168–91.

[16] Zienkiewicz OC, Shiomi T. Dynamic behaviour of saturated porous media: the generalized Biot formulation and its numerical solution. Int J Numer Anal Meth Geomech 1984;8:71–96.

[17] Li XS, Wang ZL, Shen CK. SUMDES, a nonlinear procedure for response analysis of horizontally-layered sites subjected to multi-directional earthquake shaking. Davis: Department of Civil Engineering, University of California, 1992.

[18] Wang ZL, Dafalias YF, Shen CK. Bounding surface hypo-plasticity model for sand. J Engng Mech, ASCE 1990;116:983–1001.

[19] Dafalias YF. Bounding surface plasticity, I. Mathematical foundation and the concept of hypoplasticity. J Engng Mech, ASCE 1986;112: 966–87.

[20] Li XS, Shen CK, Wang ZL. Fully coupled inelastic site response analysis for 1986 Lotung earthquake. J Geotech Geoenviron Engng, ASCE 1998;124:560–73.

[21] Arulanandan K, Scott RF. VELACS: verification of numerical procedures for the analysis of soil liquefaction problems. Amsterdam: Balkema, 1993.

[22] Seed HB, Muraka R, Lysmer J, Idriss IM. Relationships between maximum acceleration, maximum velocity, distance from source and local site conditions for moderately strong earthquakes. Bull Seismol Soc Am 1976;66:1323–42.

[23] Idriss IM. Response of soft soil sites during earthquakes. Proceedings of the Symposium to Honor Professor H.B. Seed, Berkeley. 1990, p. 273–89.

[24] Mori S, Miwa S, Numata A. Shaking table tests to elucidate liquefaction behavior of reclaimed lands of weathered granite during the 1995 Hyogo-ken Nanbu earthquake. J Struct Mech Earthquake Engng, JSCE 1996;549/I-37:231–48 in Japanese.

[25] Yoshikawa M, Arano M, Yamada Y. Liquefaction characteristic of a model pile foundation during vertical and horizontal ground motion. Soils Foundations 1987;27:189–99.

[26] Ghaboussi J, Dikmen S. Liquefaction analysis for multidirectional shaking. J Geotech Engng Div, ASCE 1981;107:605–27.

[27] Ishihara K. Soil behaviour in earthquake geotechnics. Oxford: Clarendon Press, 1996.

[28] Yang J. Saturation effects on horizontal and vertical motions in a layered soil–bedrock system due to inclined SV waves. Soil Dynam Earthquake Engng 2001;21:527–36.

[29] Bard PY. Strong motion and site effects: general report. Proceedings of the Second International Conference on Earthquake Geotechnical Engineering, Lisbon, 1999.