RESPONSE OF A DENSE SAND DEPOSIT DURING 1993 KUSHIRO-OKI EARTHQUAKE

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ABSTRACT

In the 1993 Kushiro-oki Earthquake of Richter magnitude 7.8, simultaneous recording of earthquake motions was successfully made at the ground surface and at a depth of 77 meters in a dense saturated sand deposit. The peak horizontal acceleration was 0.47 g on the ground surface and 0.21 g at a depth of 77 meters. The acceleration record at the ground surface showed a distinctive ground response, which consisted of a cyclic motion having a period of about 1.5 seconds overlain by a spike at each peak of the motion.

In order to study the mechanism of this peculiar ground response, effective stress analysis was conducted on the dense saturated sand deposit. The model used for this study was a strain space multiple mechanism model, which takes into account the effect of principal stress axis rotation. The recorded earthquake motion at a depth of 77 meters was used as the input earthquake motion for the analysis. Sampling after in-situ freezing was done in order to evaluate the properties of the sand. The results of the analysis indicated that the observed ground response was due to the effect of dilatancy of sand, which plays a significant role in the response of the dense saturated sand deposits during strong earthquake motions.

Key words: constitutive equation of soil, dilatancy, dynamic, earthquake, liquefaction, sand (IGC: E8/D7/E13)

INTRODUCTION

It has been shown in laboratory tests that dense saturated sand deposits often exhibit spiky acceleration in response to strong cyclic motions (e.g. Lee and Schofield, 1988). This is attributed to the cyclic mobility of the sand, in which contraction and dilation of sand alternately make the stiffness of sand soften and stiffen during cyclic loading. While laboratory test data are abundant to quantify the cyclic mobility of sand, available in-situ data during strong earthquake shaking have been limited to date. An exception may be the one reported by Holzer et al. (1989) of a loose silty sand deposit during the 1987 Superstition Hills Earthquake in California.

The authors and their colleagues have operated a strong-motion observation network in Japanese port areas since 1962 (e.g. Kurata and Iai, 1992). As of December, 1993, the network consists of 87 recording stations at 56 ports throughout Japan. During the 1993 Kushiro-oki Earthquake of magnitude 7.8, a set of strong motion records was obtained from a dense saturated sand deposit. The peak horizontal acceleration was 0.47 g at the ground surface and 0.21 g at a depth of 77 meters. The record at the ground surface showed a distinctive ground response, which consisted of a cyclic motion having a period of about 1.5 seconds overlain by a spike at each peak of the motion.

In the following pages, the results of the recording will be presented in detail, followed by an effective stress analysis to examine the notion that the observed ground response is due to the effect of cyclic mobility of the dense saturated sand under strong earthquake shaking. The model used for this study is a strain space multiple mechanism model (Iai et al., 1992).

RECORDING STATION AND INSTRUMENTATION

The recording station is located at the central part of Kushiro Port as shown in Fig. 1. As shown by the broken line in the figure, the station is on the former beach line at the estuary of the Kushiro River. The history of strong motion recording at Kushiro Port dates back to July 1964 when the recording station was located about 2 km southeast of the present location. In November 1983, the recording station was relocated to the present site and, in October 1992, it was upgraded to the current instrumentation.

The current instrumentation at this site consists of two

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units of 3-component servo-type seismographs; one installed at the ground surface (to be called the ground surface seismograph), the other embedded at a depth of 77 meters below the ground surface (to be called the downhole seismograph). Plan and cross section of the recording station are shown in Fig. 2.

As shown in this figure, the downhole seismograph is installed through a bore hole, which is lined with a casing of 112 mm in diameter down to a depth of 75.5 meters, below which coarse sand backfill is installed to fix the seismograph in place. The downhole seismograph is encased in a cylindrical aluminum capsule 90 mm in diameter, 550 mm in length and weighing 250 N. The direction of the downhole seismograph was adjusted to a precise azimuth during its installment at the bottom of a bore hole with an aluminum rod temporarily attached to the seismograph. The electrical cables from the seismograph capsule were extended to a digital recorder in a recording house as shown in Fig. 2.

The ground surface seismograph is installed together with an AD converter and a digital recorder in an aluminum container, the dimensions of which are 540 mm long by 540 mm wide by 380 mm high. The container is installed on a concrete base 560 mm by 560 mm as shown in Fig. 3. The overall unit weight of the concrete base was

Fig. 1. Locations of epicenter of the earthquake, Kushiro City, recording station and boring sites

Fig. 2. Plan and cross section of recording station; (a) plan and (b) cross section

Fig. 3. Concrete base for a unit containing ground surface seismograph and digital recorder
adjusted to be 20 kN/m$^3$ by allowing hollow spaces in the concrete base as shown in the figure in order to minimize any effect of the base on recording. The ground surface seismograph is installed in a recording house as shown in Fig. 4; the floor of the recording house being separated from the base of the ground surface seismograph as shown in the same figure.

The ground surface seismograph is a force balance servo type and the downhole seismograph a velocity feedback servo type. Both of the seismographs are similar to each other in frequency response, covering a frequency range from 0.01 to 35 Hz as shown in Fig. 5. Signals from these seismographs are transmitted to a digital IC (i.e. integrated circuit) cartridge type memory. The memory is 16 bits/data with a maximum acceleration capacity of 2.0 g. A total of six component records are stored simultaneously in the recorder. Electrical power supply for the digital recorder is switched on continuously and, whenever a tremor greater than 0.001 g occurs at this site, time histories of accelerations will be stored in the digital IC memory for a duration of 180 seconds at a time step of 0.01 seconds. The digital recorder is equipped with a pre-event memory to record earthquake motions commencing ten seconds before triggering.

**SUBSURFACE SOIL CONDITIONS**

- Soil investigation at the recording station was conducted about three months before the earthquake when the downhole seismograph was installed at the site. The boring log is shown in Fig. 6. In this investigation, the standard penetration test (SPT) was conducted with a free fall method, which is commonly called “tombi” in Japanese. Unit weights of soils were measured by radio
In order to supplement the data on the subsurface soil conditions, additional soil investigation was conducted about nine months after the earthquake. Sites chosen for the investigation, designated as Site Nos. 2 and 3 in Fig. 1(c), are located about 50 and 100 meters west and east of the recording station, respectively. In this investigation, velocity profiles were obtained in addition to SPT N-values and unit weights of the soils. The velocity logging was obtained by a downhole method with suspension technique. The results are shown in Figs. 7 and 8.

The shear wave velocities in this investigation correlate with the SPT N-values as shown in Fig. 9, which may be approximated by the field correlation proposed by Imai et al. (1975).

Comparing the three boring logs shown in Figs. 6 through 8 with each other, a distinctive difference in the soil types depicted in the boring logs shown in Figs. 7 and 8 from those in Fig. 6 is noted. In Figs. 7 and 8, fine sand and gravelly sand are seen whereas those in Fig. 6 consist

![Fig. 6. Boring log at the recording station](image)

![Fig. 7. SPT N-values and velocity logging at Site No. 2](image)
of coarse sand to a depth of about 20 meters from the ground surface. The actual difference between these soils, however, is marginal in terms of gradation as explained later in the next section of this paper.

**SAMPLING AND LABORATORY TESTING**

Sampling and laboratory testing of soils in the vicinity of the recording station were conducted in 1982 (Zen et al., 1988). The locations of the sampling are designated as Site Nos. 4 and 5 in Fig. 1(c). Boring logs obtained at these sites are shown in Fig. 10 with the depths of sampling. The range of gradation of sands at these sites is shown in Fig. 11. Undisturbed samples were obtained with an Osterberg piston tube sampler. The sampler consists of an inner piston and a thin wall sampling tube 75 mm in diameter, the latter penetrating the ground statically for sampling while the inner piston is kept fixed in the same position as before the penetration (e.g. Ishihara et

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**Fig. 8.** SPT $N$-values and velocity logging at Site No. 3

**Fig. 9.** Correlation between SPT $N$-values and S-wave velocities at Site Nos. 2 and 3

**Fig. 10.** Boring logs and depths of sampling at Site Nos. 4 and 5

**Fig. 11.** Grain size distribution of sands at Site Nos. 4 and 5
al., 1979). The samples obtained with the Osterberg piston tube sampler will hereafter be designated "tube samples."

To supplement these investigations, in-situ freezing sampling was conducted at Site No. 2 about nine months after the earthquake. Gradation of the soils sampled at this site are shown in Fig. 12. The samples were obtained from a depth ranging from 4.0 to 9.0 meters as indicated in Fig. 7. The sands and gravelly sands at this depth contained less than 10% fines as shown in Fig. 12. The in-situ freezing sampling technique is known to be ideal for sampling clean sands (Yoshimi et al., 1989). Using this technique, the ground was initially frozen with liquid nitrogen to form a frozen mass about 0.8 meters in diameter from about 3 to 10 meters in depth. Samples of the frozen sands were then obtained with a double tube core sampler 150 mm in diameter. The samples obtained using the in-situ freezing technique will be hereafter designated "frozen samples."

The gradation curves shown in Figs. 11 and 12 indicate that the ranges of fine, coarse and gravelly sands are partly overlapping, posing a difficulty in classifying these soils into clearly defined soil types. For example, the soils shown in Fig. 7 are classified as fine, gravelly and coarse sands. Comparison of the gradation curves in Fig. 12 with each other, however, indicates only marginal difference among some of the soils classified into different types. To be more specific, the soils at depths of 6, 7, 12, 14 through 16 meters in Fig. 7, those at depths of 10 and 11 meters, and those at 18 and 19 meters are classified as fine, gravelly and coarse sands, respectively. The gradation curves shown in Fig. 12, however, indicate that only marginal difference exists among these soils. Thus, as mentioned earlier, despite the apparent difference in the soil types depicted in the boring logs in Figs. 6 through 8, actual difference in the soils is very slight in terms of gradation.

In order to evaluate the properties of the soils under cyclic loading, the soil samples mentioned earlier were taken to the laboratory, carved into specimens 50 mm in diameter and 125 mm high for cyclic triaxial tests. Before cyclic loading, the specimens were consolidated under isotropic confining pressures equal to the in-situ vertical effective stresses. An example of the cyclic triaxial test results for the tube samples is presented in Fig. 13 in terms of the cyclic stress ratio versus the number of load cycles required to cause 5% axial strain in double amplitude and those to cause initial liquefaction (i.e. a state at which the excess pore water pressure equals the initial confining pressure). The cyclic stress ratio is defined as the ratio of cyclic deviator stress $\tau_d = (\sigma_2 - \sigma_1)/2$ to initial confining pressure $\sigma'$. A summary of the test results for all the samples is shown in Fig. 14 referring to the state of 5% double amplitude in axial strain. Similarly, the cyclic triaxial test results for the frozen samples are presented in Fig. 15. To summarize these test results, the cyclic stress ratios causing 5% strain in double amplitude at 15 load cycles were obtained from Figs. 14 and 15 except for Fig. 15(b). For the results in Fig. 15(b), the cyclic stress ratio
causing 2% strain in double amplitude was obtained instead of that causing 5% strain, which causes some difficulty in accurate estimation.

These stress ratios were plotted in Fig. 16 related to the SPT N-values. Following Seed et al. (1985), the SPT N-values were normalized with respect to a confining pressure of 98 kPa (1.0 kgf/cm²). Energy corrections were also made with respect to the rod energy ratio of 78 percent, which represents the average rod energy ratio of the free fall method used in Japanese SPT practice. Parallel coordinates specified by the equivalent N-values and the equivalent accelerations are those widely used in ports and coastal areas in Japan (Iai et al., 1989).

Shown also in the same figure are the field correlations between SPT N-values and cyclic shear stress ratios obtained by Seed et al. (1985), the first author and his colleagues (Iai et al., 1989) and Yoshimi (1993). The γ shown in the figure for the correlation by Seed et al. (1985) indicates a single amplitude of shear strain at 15 load cycles. The Fγ shown in the figure for the correlation by Iai et al. (1989) indicates a factor of safety sufficient to attain the state of liquefaction. The 2ε shown for the correlation by Yoshimi (1993) indicates a double amplitude of shear strain at 15 load cycles.

Figure 16 indicates that these field correlations are basically consistent with two of the data points for the frozen samples obtained at depths from 3.95 to 5.00 meters and from 5.00 to 6.00 meters. One data point for the frozen sample obtained at a depth from 6.10 to 7.10 meters is not consistent with these field correlations. This sample slightly differs in its gradation from the other two samples as shown in Fig. 12(b). The specific reason why this data point is not consistent with those field correlations, however, is not clearly known to the present authors.

Additional data obtained using the in-situ freezing sampling technique support those field correlations rather than the results obtained using tube sampling (e.g. Yoshimi et al., 1989). In the present study, two data points for the frozen samples and the data points for the tube samples support this assumption. Consequently, it
was decided in the present study to evaluate the liquefaction resistance of the soils at the recording station from the SPT N-values by referring to the level of the stress ratio at 15 load cycles obtained using the field correlation proposed by Yoshihi (1993).

EARTHQUAKE AND RECORDED MOTIONS

An earthquake of a Richter-Japan Meteorological Agency (JMA) magnitude of 7.8 named the 1993 Kushiro-oki Earthquake took place on January 15, 1993 with a focal depth of about 107 km and an epicenter located about 15 km south of Kushiro City as shown in Fig. 1(c). The intensity of shaking in the area surrounding Kushiro City registered 6.0 in terms of the Japanese Meteorological Agency scale. Among many seismographs that have registered the motions of this earthquake, the one installed at Kushiro Port was of particular interest because it gave simultaneous recording of motions on the ground surface and at a depth of 77 meters. There was no evidence of ground deformation and failure at the recording station such as cracking, water spouting or sand boiling.

In Kushiro City, the average temperature falls below freezing during winter. When the earthquake occurred, the ground surface was said to be frozen with a thickness of about 0.5 to 1.0 meters in Kushiro City. At present, the authors have not been able to obtain definite evidence to determine whether the ground surface at the recording station was frozen at the time of the earthquake. When

Fig. 17. Recorded accelerations of the mainshock on January 15, 1993
the earthquake occurred, the ground at Kushiro City was not covered with snow. It began to snow the next day and the ground was covered with snow with a thickness of about 0.2 meters.

The recorded time histories of acceleration are shown in Fig. 17. In this figure, the major portion of the earthquake record is plotted from the original record obtained for a duration of 180 seconds; the origin of time is ten seconds before the triggering. The peak accelerations on the ground surface were 4.68, 3.44 and 3.42 m/s² in North-South (N-S), East-West (E-W) and Up-Down (U-D) directions whereas those at a depth of 77 meters were 2.04, 2.65 and 1.10 m/s², respectively. Loci of these motions in a horizontal plane, shown in Fig. 18, indicate that the velocity and displacement of the earthquake motion are predominant in the N-S direction, which is consistent with the fault movement of the earthquake reported by Kasahara et al. (1993).

By comparing the ground surface motion in the N-S direction shown in the first panel in Fig. 17 with downhole motion at a depth of 77 meters in the same direction shown in the fourth panel, a distinctive change in the response of ground after about 30 seconds can be observed; most of the high frequency motions are filtered out and instead cyclic motion becomes predominant with a period of about 1.5 seconds overlain by a spike at each peak of the motion. Less remarkable is the change in the E-W direction probably due to smaller shaking in both velocity and displacement.

In contrast to the distinctive response of ground during the mainshock of January 15, 1994, no peculiarity was observed in the ground response at a smaller level of excitation at the same site. As an example, a record of an aftershock with a magnitude 4.9 on February 4, 1993 is shown in Fig. 19. Frequency transfer functions were obtained for the mainshock and the aftershock as Fourier spectral ratios of the ground surface motions over the downhole motions as shown in Fig. 20. The figure indicates that the natural frequency of the fundamental dynamic response mode for sand deposit is about 1.0 Hz during the weak motion at the aftershock of February 4, 1993 whereas the natural frequency becomes 20 or 30 percent lower during the strong motion at the mainshock of January 15, 1993. This indicates non-linear response of the subsurface ground.

**CONSTITUTIVE MODEL OF SOILS**

To study the mechanism of the observed distinctive response of ground during the mainshock, effective stress analysis was conducted on the dense saturated sand deposit. Since the motion was predominant in the N-S direction in velocity which represents the predominant direction of kinetic energy of ground response, the analysis was made on a one dimensional soil column in this direction. The effective stress model for soils used in this
study was a multiple shear mechanism model (Iai et al., 1992a). With the effective stress and strain vectors in plane strain condition expressed as

\[ \{\sigma'\}^T = \{\sigma_x', \sigma_y', \tau_{xy}\} \]  
\[ \{\varepsilon\}^T = \{\varepsilon_x, \varepsilon_y, \gamma_{xy}\} \]  

then the basic form of the constitutive relation is given by

\[ \{d\sigma'\} = [D] \{[ds] - \{de_p\}\} \]  
\[
[D] = K \{n^{(1)}\} \{n^{(1)}\}^T + \sum_{i=1}^{I} R_{ij}^{(i)} \{n^{(i)}\} \{n^{(i)}\}^T.
\]

In this relation, the term \{de_p\} in Eq. (3) represents the volumetric strain increment due to dilatancy \(de_p\) and is given by

\[ \{de_p\}^T = \{de_x/2, de_y/2, 0\} \]  

and the first term in Eq. (4) represents the volumetric mechanism with rebound modulus \(K\) and the direction vector given by

\[ \{n^{(1)}\}^T = \{1, 1, 0\} \].

The second term in Eq. (4) represents the multiple shear mechanism. Each mechanism \(i=1, \cdots, I\) represents a virtual simple shear mechanism, with each simple shear plane oriented at an angle \(\theta/2 + \pi/4\) relative to the \(x\) axis. The tangential shear modulus \(R_{ij}^{(i)}\) represents the hyperbolic stress strain relationship with hysteresis char-
characteristics. The direction vectors for the multiple shear mechanism in Eq. (4) are given by the expression
\[ n^{(0)} = \begin{bmatrix} \cos \theta_i & -\cos \theta_i & \sin \theta_i \end{bmatrix} \]
(for \( i = 1, \cdots, I \)) \tag{7}

in which
\[ \theta_i = (i-1)\Delta \theta \quad \text{(for} \quad i = 1, \cdots, I) \quad \tag{8} \]
\[ \Delta \theta = \pi / I. \quad \tag{9} \]

The loading and unloading for shear mechanism are separately defined for each mechanism by the sign of \((n^{(0)})^T(d \varepsilon)\).

The multiple shear mechanism model realistically represents the effect of rotation of principal stress axis directions (Iai et al., 1994), the effect of which is known to play an important role in the cyclic behavior of anisotropically consolidated sand (Iai et al., 1992b).

**DETERMINATION OF MODEL PARAMETERS**

The present model has ten parameters; two of which specify the elastic properties of soil, another two specify plastic shear behavior, and the rest determine dilatancy. These parameters were determined from the in-situ velocity logging, the SPT N-values and the results of the undrained cyclic loading tests described earlier. The details in determining the soil parameters were as follows.

In the analysis, the subsurface soil was idealized into eight soil layers as indicated in Fig. 21. In the finite element computation, these eight soil layers were further divided into a stack of 77 element layers, each with a thickness of one meter. The elastic shear modulus \(G_{0m}\) at a reference confining pressure of \((-\sigma_{cm}')\) was determined by referring to the correlation between SPT N-values and S-wave velocities shown in Fig. 9 based on the in-situ velocity logging in the vicinity of the recording station. The reference effective confining pressure of \((-\sigma_{cm}')\), which links elastic shear modulus to in-situ confining pressure for each soil layer, was determined by referring to the \(K_s = 0.5\) condition. The use of the reference effective confining pressure permits a gradual increase in the elastic shear modulus in accordance with a gradual increase in confining pressure within each soil layer. Shear resistance angle \(\phi_f\) was estimated by referring to a correlation with SPT N-values (Dunham, 1954). The hysteretic damping factor for an infinitely large shear strain \(H_m\) was determined to be 30% by referring to the typical laboratory results summarized by Ishihara (1982).

The rebound modulus of soil skeleton \(K_s\) was determined by assuming that Poisson's ratio is 0.33. The phase transformation angle \(\phi_p\), which separates dilative and contractive zones in stress space, was assumed to be 28 degrees by referring to the typical value adopted for effective stress analyses of saturated sand (Ishihara et al., 1989). The remaining five parameters \(p_1, p_2, w_1, S_1\) and \(c_1\), which specify dilatancy of sand, were determined by referring to the cyclic loading test results, levels of which were determined by referring to the SPT N-values as discussed earlier. An average of the normalized SPT-N values of Layer 2 was about 26, corresponding to a cyclic stress ratio of 0.41 at 15 load cycles; similarly the cyclic stress ratio of Layer 3 was estimated as 0.95 at 15 load cycles. In order to simplify the procedure for parameter determination, soil layers below Layer 3 were assumed to generate no pore water pressures. The basis for this simplification is (1) liquefaction resistance of Layers 5 and 8.
with SPT N-values about 50 or greater should be too high to generate pore water pressures as indicated by the field correlations in Fig. 16, and (2) smaller SPT N-values in Layers 4, 6 and 7 may be due to a mixture of silty materials, which generally exhibit higher cyclic resistance than clean sands with similar SPT N-values. More details in the procedure for parameter determination as well as the definition of each parameter can be found in a paper by Iai et al. (1990).

The soil parameters determined in this manner are shown in Table 1. Figure 22 for example shows a computed liquefaction resistance curve and cyclic behavior of sand in Layer 2.

### Table 1. Model parameters for response analysis

<table>
<thead>
<tr>
<th>Layer no.</th>
<th>$H$ (m)</th>
<th>$\rho$ ($\text{t/m}^3$)</th>
<th>$V_r$ (m/s)</th>
<th>$G_m$ (kPa)</th>
<th>$-\sigma'_{ref}$ (kPa)</th>
<th>$\phi_r$ (deg)</th>
<th>$\phi_p$ (deg)</th>
<th>Parameters for dilatancy</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>2.0</td>
<td>1.54</td>
<td>249</td>
<td>106600</td>
<td>37</td>
<td>40</td>
<td>-</td>
<td>$-\sigma'_{ref}$ = 7.0, $\rho_i = 0.5$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\rho_2 = 0.65$, $c_i = 3.97$</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.0</td>
<td>1.72</td>
<td>249</td>
<td>106600</td>
<td>37</td>
<td>40</td>
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<td></td>
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<td></td>
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<td></td>
<td></td>
<td>$\rho_i = 3.5$, $c_i = 0.5$</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\rho_2 = 0.4$, $c_i = 3.68$</td>
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<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>$S_i = 0.01$</td>
</tr>
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<td>1.73</td>
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<td>121500</td>
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<td>1.76</td>
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<td>204700</td>
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<td>44</td>
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<td>-</td>
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<td>1.70</td>
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<td>44</td>
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<td>-</td>
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<td>201200</td>
<td>354</td>
<td>44</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

$H$: layer thickness; $\rho$: density; $V_r$: shear wave velocity; $G_m$: elastic shear modulus at a confining pressure of $(-\sigma'_{ref})$; $-\sigma'_{ref}$: reference confining pressure; $\phi_r$: shear resistance angle; and $\phi_p$: phase transformation angle

![Computed liquefaction resistance curve and computed cyclic behavior of sand for Layer 2](image)

(a) Liquefaction resistance curve, (b) Stress paths under cyclic loading, and (c) Stress-strain relation under cyclic loading.
EARTHQUAKE RESPONSE ANALYSIS

Before the earthquake response analysis, a static analysis was conducted by gravity in order to simulate the initial stresses acting in-situ before the earthquake. The same constitutive model was used as in the earthquake response analysis but under drained condition. The analysis resulted in an initial earth pressure coefficient at rest \( K_0 \) being about 0.5. During the earthquake response analysis, an internal constraint on the lateral normal strain was imposed to simulate the behavior of horizontally layered soils.

With these initial conditions and internal constraint, an earthquake response analysis was conducted on the dense saturated sand deposit. The N-S component of the recorded motion at a depth of 77 meters was used as the input motion. The analysis was conducted with the undrained conditions (Zienkiewicz et al., 1982) in order to simplify the analysis. The time integration was numerically done using the Wilson-\(\theta\) method (\(\theta=1.4\)) using a time step of 0.01 seconds. Rayleigh damping (\(\alpha=0\) and \(\beta=0.0005\)), which was proportionally decreasing with the degree of cyclic mobility, was used to ensure stability of the numerical solution process.

RESULTS OF THE ANALYSIS

The effective stress analysis results in accelerations in the dense saturated sand deposit as shown in Fig. 23. In order to facilitate comparison between the computed and the recorded accelerations on the ground surface, these time histories were obtained from Figs. 17 and 23 and reproduced in Fig. 24. The computed acceleration at the ground surface captures essential features of the recorded acceleration. The computed and recorded displacements at the ground surface also show consistency as shown in Fig. 25, in which displacements are defined as a relative value compared to those at a depth of 77 meters.

A closer look may reveal a tendency in the computed results to become softer at a later stage, say after 45 seconds, than the recorded results as seen in the relative displacements in Fig. 25. This may be improved either by including the effect of seepage flow between the soil layers or by finely tuning the model parameters such as \( P_2 \) which control the dilatancy in the vicinity of the zero effective stress.

By looking back at the computed response of soil layers in Fig. 23, a significant change in the response is noted between the motions at depths of two meters and nine meters. Between these two depths, Layer 2 exists.
Fig. 25. Recorded and computed displacements at the ground surface relative to the base at a depth of 77 meters.

Fig. 26. Computed stress path and stress-strain relation at a depth of 3.5 meters in Layer 2; (a) stress paths and (b) stress-strain relation for which the stress path and stress-strain relation at a depth of 3.5 meters are shown in Fig. 26. The stress strain relation, being initially close to a linear relation as shown in the central part of Fig. 26(b), gradually changes into a relation of a hysteretic hardening spring type due to the effect of dilatancy. This clearly indicates that Layer 2 is undergoing cyclic mobility. As shown in Fig. 27, the maximum shear strain in Layer 2 reaches a level of about 1 to 2% whereas those in other layers are all less than 0.3%, which is much less than those strain levels to exhibit cyclic mobility as shown in Fig. 26. Consequently, the recorded spiky acceleration at the ground surface is considered to be due to the cyclic mobility of the dense sand deposit at a depth ranging from two to nine meters.

Fig. 27. Maximum shear strains in the response analysis.
PARAMETRIC STUDY

Despite the reasonable agreement between the recorded and computed motions, the agreement might have been obtained by sheer coincidence. To resolve this point to some degree, a more general picture of the analysis will be presented by varying some of the conditions assumed in the analysis.

The first of the parametric studies was to examine the sensitivity of the liquefaction resistance of Layer 2 to the results of the analysis. In this study, the liquefaction resistance curve of Layer 2 was varied based on Cases 2 through 4 in Fig. 28 from the original curve designated Case 1, which was used for the original analysis described earlier. Varied model parameters to achieve the variation in the liquefaction resistance curves were only two or three of those used for controlling the dilatancy of sand as shown in Table 2; all the other parameters remained the same as those shown in Table 1. The results of the analyses, shown in Fig. 29, indicate that spiky accelerations at the ground surface can still be seen in Cases 2 and 3 but the distinctive change in the acceleration wave form begins about five seconds earlier than in Case 1. When the liquefaction resistance becomes as low as that adopted for Case 4, the analysis resulted in a much smaller peak acceleration than that of the recorded acceleration.

Another series of parametric studies was conducted to examine the possibility of liquefaction in Layers 4, 6 and 7. In the present study, these layers were assumed to generate no pore water pressures based on their silty nature. If these layers were clean sands, then the average normalized SPT N-values for these layers ranging from 16 to 17 would indicate lower liquefaction resistance than assumed in the present study. As a parametric study, liquefaction resistance of these layers was assigned by referring to the field correlation for clean sands by Yoshimi (1993) shown in Fig. 16. The assigned cyclic stress ratios ranged from 0.22 to 0.23 at 15 load cycles. In this parametric study, Layer 4 at a depth ranging from 19 to 28 meters liquefied, resulting in inconsistency in the acceleration at the ground surface as shown in Fig. 30. Although it was not possible to evaluate the liquefaction resistance of Layers 4, 6 and 7 with undisturbed sampling, the result of the parametric study implies that these silty sand layers have much higher liquefaction resistance than assumed in the parametric study. The

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Table 2. Varied model parameters for dilatancy for Layer 2 in the parametric study

<table>
<thead>
<tr>
<th>Case</th>
<th>$w_1$</th>
<th>$p_1$</th>
<th>$p_2$</th>
<th>$c_i$</th>
<th>$S_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original Study</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Case 1*</td>
<td>7.0</td>
<td>0.5</td>
<td>0.65</td>
<td>3.97</td>
<td>0.01</td>
</tr>
<tr>
<td>Parametric study</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Case 2</td>
<td>3.5</td>
<td>0.5</td>
<td>0.65</td>
<td>1.0</td>
<td>0.01</td>
</tr>
<tr>
<td>Case 3</td>
<td>1.2</td>
<td>0.5</td>
<td>0.65</td>
<td>1.0</td>
<td>0.01</td>
</tr>
<tr>
<td>Case 4</td>
<td>0.35</td>
<td>0.5</td>
<td>0.87</td>
<td>1.0</td>
<td>0.01</td>
</tr>
</tbody>
</table>

*The same as those shown in Table 1 for Layer 2
result of the parametric study supports the assumption made in the present study of either no pore water pressures, or at least not a high enough pore water pressure to cause liquefaction, were generated in these silty sand layers during the earthquake.

The last series of parametric studies was conducted by varying the level of input motion at a depth of 77 meters with peak accelerations ranging from 0.50 to 2.50 m/s² including the peak acceleration of 2.04 m/s² recorded during the earthquake. The results are shown in terms of time histories at the ground surface in Fig. 31 and Fourier spectral ratios of the ground surface motions over the input motion at a depth of 77 meters in Fig. 32. These results indicate that the present model has the potential ability to simulate the change in the response characteristics of dense sand deposits in accordance with a change in intensity level of shaking.

CONCLUSIONS

The present paper reported in detail the simultaneous recording of a dense saturated sand deposit at the ground surface and at a depth of 77 meters. This recording was of particular interest because of the intensity and the distinctive wave form of the motions; the motion on the ground surface was a spiky acceleration with a peak value of 0.47 g while at a depth of 77 meters peak acceleration was 0.21 g. Effective stress analysis of the dense saturated sand deposit leads to the following conclusions.

1. The recorded spiky acceleration at the ground surface is due to the cyclic mobilities of the dense sand deposit, in which contraction and dilation of sand alternately make the stiffness of the sand soften and stiffen during cyclic loading. The recorded acceleration is one of few in-situ records to clearly indicate that the dilatancy of sand plays an important role in the response of a dense saturated sand deposit during strong earthquake motions.

2. A strain space multiple mechanism model used in the present study has the potential ability to simulate the change in the response characteristics of a dense sand deposit in accordance with a change in intensity level of shaking. In particular, the model has an ability to capture the essential features of the response of ground including the spiky response motions.

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REFERENCES


