NONLINEAR BEHAVIOR OF SURFACE DEPOSIT
DURING THE 1995 HYOGOKEN-NAMBU EARTHQUAKE

IWAO SUE TOMI and NOZOMU YOSHIDA

ABSTRACT

Behavior of the ground during the 1995 Hyogoken-Nambu earthquake is investigated through various analyses on and comparisons with the earthquake records.

Behavior of the Holocene clay, a soft clay layer, as well as the fill, the liquefied layer, is shown to affect the response of the ground significantly through effective stress analysis. Earthquake response analysis carried out on the section passing the Sannomiya station indicates that the existence of the Holocene clay and its thickness controls the response of the ground surface to a great degree. It is also shown that there was no deamplification effect caused by the nonlinear behavior of the soil in the area called the damage belt, where damage to houses was especially severe.

Incident waves on the top of the upper Osaka Group formation are estimated for several sites where earthquake records were obtained by deconvolution analysis. Their waveforms on the base layer are very similar to each other in a narrow region a few kilometers wide, but are different from those at distant sites, although their PGV are nearly the same at about 80 cm/s.

It is shown that the ground shaking observed at the 1995 Hyogoken-Nambu earthquake was characterized mainly by the surface geology, and that it was are controlled by the nonlinear behavior of the soft clay layer and liquefaction of fill.

Key words: earthquake, liquefaction, nonlinear behavior, strong ground motion, transfer function (IGC: E8)

INTRODUCTION

Significant damage occurred over a widespread area in Hyogo prefecture and its vicinity during the 1995 Hyogoken-Nambu earthquake. Damage in Kobe city was especially severe; many houses completely or partially collapsed and port facilities were severely damaged due to liquefaction, for example. When looking at this damage in detail, one notices that the damage patterns are different depending on region. In the Rokko mountain region, damage itself was scarce. However, south of the mountain area, there is a region called the damage belt where more than 30% of the houses completely collapsed and the JMA seismic intensity scale was evaluated to be 7. In this region, damage to the houses seems to have been caused by the strong inertial force. The damage to houses becomes smaller in the region south of the damage belt zone. On the other hand, significant liquefaction of the fill was observed in areas along the shoreline, where damage to houses caused by the inertia force seems very rare, in contrast to damage caused by soil deformation.

These different damage patterns must be related to the ground shaking. In other words, ground shaking at sites which suffered different damage patterns must be different. In this paper, we intend to clarify why the difference of ground shaking occurs through numerical analysis, as well as by investigation of the observed earthquake records, especially focusing on the behavior of the surface deposit.

OVERVIEW OF NONLINEAR BEHAVIOR BASED ON OBSERVED RECORD

Two functions control ground shaking in soft ground. The first is amplification of ground shaking; amplification is larger in the soft ground than at a stiff site. The second is deamplification; transmission of the shear force upwards is prevented when the working stress reaches close to the shear strength. The effect of the first function has been confirmed in past earthquakes. For example, peak acceleration at the fill area in San Francisco was about 3 times as large as that in the adjacent rock site during the 1989 Loma Prieta earthquake, in which peak acceleration at the rock site was of the order of 0.1 g (Shakal et al., 1989). On the other hand, although the second function which occurs in large earthquakes has been
confirmed by one-dimensional dynamic response analysis (Idriss, 1990), it has not been strongly confirmed through observed earthquake records. Earthquake records, not only at the ground surface but also at the base or at an adjacent rock site are required for confirmation. This kind of record has, however, hardly been obtained at sites where significant nonlinear behavior has occurred, which is the reason why these have no confirmation.

Digital observation records have been published by many organizations, such as CEORKA (the Committee of Earthquake Observation and Research in the Kansai Area), JMA (the Japan Meteorological Agency), PWRI (the Public Works Research Institute, Ministry of Construction), PHRI (the Port and Harbour Research Institute, Ministry of Transport), RTRI (Railway Technical Research Institute), the Kansai Electric Power Co., Osaka Gas Co. and others. We calculated peak values of the records in the two-dimensional horizontal plane. Velocity is calculated by the integration in frequency domain, and frequency component higher than 10 Hz are neglected because high frequency components are not important in the engineering practice and are possibly affected by small disturbances of the soil condition beneath the ground surface. Figure 1 compares peak ground acceleration (PGA) and peak ground velocity (PGV) at the ground surface and at the base in stiff, medium and soft grounds. The ground classification was made by Sawada et al. (1996) based on the Specification of Highway Bridges (Japan Road Association, 1990) as shown in Table 1. If only the earthquake record at the ground surface is available, the base record at any location within 10 km from the site is used as the response at the base of the site. Observation sites used in Fig. 1 are shown in Fig. 2. Here, records at Port Island (GL-83.4 m), Kobe University, Technical Research Center of Kansai Electric Co. Ltd. (GL-97 m), Takasago Power Plant (GL-100 m), and Kainan Port (GL-100 m) are used as the base records.

The surface geology is shown to have more effect on amplification of the PGA than on that of the PGV. Liquefaction was supposed to have occurred at all soft deposit sites. This indicates that, although the term soft deposit includes both clay and sand layers, there is a surface sand layer at almost every soft deposit site. On the other hand, liquefaction was not observed in medium deposits. As seen in Fig. 1, a clear difference in acceleration amplifications is seen between the soft and medium deposits; those at medium deposits are larger than those at soft deposits. Ground types have frequently been classified into rock, stiff deposit and soft deposit in past research. Both soft and medium deposits in this study are classified to be soft deposit based on the basis of this classification. Therefore, both liquefied and nonliquefied sites were classified in the same category in the past classification, which makes it difficult to discuss the effect of

Table 1. Ground classification by Sawada et al. (1996)

<table>
<thead>
<tr>
<th>Classification in this study</th>
<th>Soil condition</th>
<th>Geological definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock</td>
<td>Group 1</td>
<td>Tertiary or older rock, or diluvium with $H&lt;10$ m</td>
</tr>
<tr>
<td>Stiff</td>
<td>Group 2</td>
<td>Diluvium with $H\geq 10$ m, or alluvium with $H&lt;10$ m</td>
</tr>
<tr>
<td>Medium</td>
<td>Group 3</td>
<td>Alluvium with $H&lt;25$ m including soft layer with thickness less than 5 m</td>
</tr>
<tr>
<td>Soft</td>
<td></td>
<td>Other than the above, usually soft alluvium or reclaimed land</td>
</tr>
</tbody>
</table>

$H$: thickness
liquefaction on the amplification. We can, however, distinguish the effect of liquefaction based on the classification in this study; sites in medium deposit did not liquefy whereas sites in soft deposit liquefied. As seen in past research, the amplification ratio of the PGA has a tendency to decrease as input motion at the base increases, which suggests the occurrence of non-linear behavior. This tendency is the clearest in sites where liquefaction occurred (soft deposit sites); the ratio reaches 1 and is sometimes less than 1. This is acceptable because the occurrence of liquefaction decreases shear strength; this does not occur in ground where liquefaction does not occur. It is noted, however, that a decrease in the amplification of the PGV hardly occurs, even if liquefaction occurs.

The amplifications are fairly large at three soft sites whose peak acceleration at the base are less than 200 cm/s². Occurrence of liquefaction was not confirmed at the points covered by earthquake observation, but it was observed close by, so liquefaction was supposed to have occurred in these sites as well. This fact may seem contrary to the common idea that acceleration is not amplified at sites where liquefaction occurs. This can be, however, explained as follows. One of the characteristics of the earthquake motion in this earthquake is that the large motion came first. Therefore, the peak response occurred prior to the occurrence of the liquefaction especially at the far site about 30 kilometers away from the fault. Since the amount of shaking is greater at far sites, liquefaction can occur even if the peak acceleration is small. On the other hand, at nearer sites, non-linear behavior as well as liquefaction occur even during the first large motion because of the large amplitude of shaking, resulting in a small amplification ratio.

NONLINEAR BEHAVIOR DUE TO LIQUEFACTION

The behavior of Port Island where severe liquefaction was observed and vertical array earthquake observation was conducted has been analyzed by the use of an effective stress dynamic response analysis code YUSAYUSA-2 (Yoshida and Towhata, 1991) which is an improved version of YUSAYUSA (Ishihara and Towhata, 1982). Equivalent linear analysis by SHAKE (Schnabel et al., 1972) was also carried out in order to compare the analyzed behaviors.

A hyperbolic model is used for shear stress-shear strain relationships in the effective stress analysis. A model proposed by Hardin and Drnevich (1972) is used in the equivalent linear analysis, and is expressed as

\[
\frac{G}{G_{\max}} = \frac{1}{1 + \frac{C_{\max}}{\tau_{f}}}
\]

\[
h = h_{\max} \left(1 - \frac{G}{G_{\max}}\right)
\]

where \(G\) denotes secant shear modulus, \(G_{\max}\) denotes shear modulus at strain levels of \(10^{-6}\), \(\tau_{f}\) denotes shear strength, \(h\) denotes damping ratio, and \(h_{\max}\) denotes maximum damping ratio. The backbone curves of the hyperbolic model and H-D model are the same, but damping characteristics are different.

PS-logging was made at the site when the earthquake observation system was installed. Shear wave velocity is reported to be constant within the same layer as shown in Fig. 6. The SPT \(N\)-value, however, increases with depth even in this layer, which is supposed to be a reflection of the confining pressure dependency of the SPT \(N\)-value. Namely, shear wave velocity is expected to change within the layer. This disagreement is not surprising because recent research has pointed out that elastic wave velocity obtained by the downhole PS logging, the logging done at this site, provides an average value (e.g., Kokusho, 1992). In addition, the importance for considering the effective confining pressure dependent characteristics in the nonlinear analysis has been also pointed out (e.g., Nakamura, 1994). Therefore we assume that the elastic modulus is proportional to the square root of the effective confining pressure. By assuming that the shear wave velocity by PS logging is the value at a half depth of each layer, we obtain

\[G_{\text{max}} = 8230\sigma_{0,5}^{\text{m}}\, (\text{kPa})\]  
\[G_{\text{max}} = 4430\sigma_{0,5}^{\text{m}}\, (\text{kPa})\]  
\[G_{\text{max}} = 9900\sigma_{0,5}^{\text{m}}\, (\text{kPa})\]

where \(\sigma_{0,5}\) denotes effective mean stress. Cyclic torsional shear tests were made on the fill material “masado”, a decomposed granite soil, which is deposited about 18 m from the ground surface, by using the undisturbed sample taken before the earthquake by thin wall sampling and disturbed samples. Test specimens for disturbed samples were made by air pulluviation. The former tests were
conducted under the undrained condition whereas the latter were under the drained condition. Both sets of results were similar to each other, as shown in Fig. 3. The internal friction angle \( \phi \) is chosen to be 25.5° and maximum damping ratio is set at 23%. It is noted that the value of \( \phi \) may not be the actual value of the internal friction angle but a mere parameter. As seen in Eq. (1), the \( G-\gamma \) relationship is uniquely determined by defining \( \tau_f = \sigma'_v \tan \phi \), where \( \sigma'_v \) denotes effective overburden stress. Therefore the value of \( \phi \) is determined so that a good agreement is obtained in \( G-\gamma \) relationships with test results at medium to large strains.

The parameters to define excess pore water pressure generation are determined by simulating the liquefaction strength test of the undisturbed samples conducted before the earthquake; the sample was taken at Port Island not far from the site of the earthquake observation. Figure 4 shows test results and simulated liquefaction strength, in which the coefficient of earth pressure at rest \( K_s \) is assumed to be 0.5. The simulation gives values \( B_\gamma = 2.5 \), \( B_\mu = 0.18 \) and \( \kappa = 0.06 \). Since the parameters used in YUSAYUSA are not common, see Ishihara and Towhata (1982), for example, on the meaning of the parameters.

Referring to past investigation (Kansai Branch of JSSMFE, 1995) of the relationship between shear modulus \( G_{\text{max}} \) and one-dimensional compression strength \( q_u \), we model the cohesion \( c \) of the Holocene clay (usually called Ma13) that exists between GL-18 and 28 m in the form

\[
c = \frac{q_u}{2} = 70.7 \log G_{\text{max}} - 30.9 \text{ (kN/m}^2\text{).} \tag{6}
\]

Maximum damping ratio is determined to give good agreement with test results, resulting in 21% for this layer. The comparison between test (Kansai Branch of JSSMFE, 1995) and simulated strain dependent characteristics is shown in Fig. 5.

Because no good data was obtained for the nonlinear characteristics of the Holocene gravel and Pleistocene gravel below the Holocene clay layer, and because we are especially interested in the behavior of surface deposits, we analyzed the subsoil above the location of the earthquake observation at a depth of 32.4 m, although earthquake data was also obtained at 83.4 m. Since the thickness of the Holocene gravel layer above GL-32.4 m is thin, the effect on the dynamic response is supposed to be less important because significant nonlinear behavior is not expected here. The internal friction angle is computed from the empirical formula (Architectural Institute of Japan, 1988). Under the average SPT \( N \)-value of 15, \( \phi \) is computed to be 32.3°. The maximum damping ratio is set at 22% based on the author’s experience.

Peak response values are shown in Fig. 6, and acceleration-time history at both the ground surface and GL-16.4 m where another seismometer is installed is shown in Fig. 7. Typical stress-strain relationships obtained by effective stress analysis are shown in Fig. 8; the result at the 18th layer (GL-17.2 to 18 m, fill) and the 23rd layer (GL-26 to 28 m, Holocene clay) are shown in the figure.

In the equivalent linear analysis, significant nonlinear behavior is observed in the clay layer; peak shear strains exceed 2%. Peak accelerations are nearly constant above this layer. This indicates that the peak value at the fill is predominantly controlled by the shear strength of the clay. Severe nonlinear behavior of the clay is also observed in the effective stress analysis, as shown in Fig. 8(b). It is clearly seen that shear stress reaches near shear strength at the first main motion.

The advantage of the use of effective stress analysis is clearly seen when comparing the acceleration time histo-
### Table: Soil Properties

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>SPT-N Value</th>
<th>Vs (km/s)</th>
<th>Mass Density (t/m³)</th>
<th>Acceleration (cm/s²)</th>
<th>Displacement (cm)</th>
<th>Shear Stress (kN/m²)</th>
<th>Shear Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>20</td>
<td>0.17</td>
<td>1.7</td>
<td>200 400 600</td>
<td>10 20 30</td>
<td>50 100 150</td>
<td>2 4 6</td>
</tr>
<tr>
<td>Clay (Ma13)</td>
<td>40</td>
<td>0.21</td>
<td>2.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel</td>
<td></td>
<td>0.18</td>
<td>1.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **Vs**: shear wave velocity at the center of each layer

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**Fig. 6.** Peak response at Port Island

**Fig. 7.** Comparison of time history at Port Island

- (a) Equivalent linear analysis
- (b) Effective stress analysis

**Fig. 8.** Stress strain relationship in effective stress analysis

(a) Fill (GL-17.2 to 18 m)

(b) Clay (GL-26 to 28 m)
ry shown in Fig. 7. In the ground surface response, the waveform by equivalent linear analysis is fairly smooth whereas that by effective stress analysis shows a sharp peak. This waveform has a characteristic that the slope of the acceleration time history increases although the absolute value of the acceleration increases, which is a quite different characteristic compared with the ordinary case in which the slope decreases with time when the absolute value of the acceleration increases. This waveform is made as the result of the cyclic mobility behavior of relatively dense sand. The same kind of peak is also seen in the acceleration time history observed at GL-16.4 m. Beside these details, equivalent linear analysis also grasps general features of the response at the ground surface.

There is disagreement between the two analyses and the observed record on peak acceleration at a depth of GL-16.4 m; those by the analyses are much smaller than that in the observed record. This peak occurs at about 7 seconds as a very sharp pulse in the observed record. The comparison between the effective stress analysis and observed record indicates that this peak may also be caused by cyclic mobility or local behavior, but it does not seem to affect the surface response, and is therefore not important in engineering practice.

Through this study, it can be concluded that the behavior of Port Island during the earthquake is evaluated better by effective stress analysis than by equivalent linear analysis, but general features are well grasped by both analyses. This is because the behavior of the soft clay layer lying beneath the fill strongly controls the behavior above it; only strongly filtered low-pass waves propagate above this layer.

**SIMULATION OF NONLINEAR RESPONSES FOR SANNOMIYA SECTION**

Severe nonlinear behavior is shown to have occurred on Port Island by the detailed analyses in the previous section. In this section, the behavior of the surface ground in the Kobe area is investigated. A section perpendicular to the shore line (roughly in the N-S direction) and passing the Sannomiya station is chosen for the analysis. The soil profile of this section is shown in Fig. 9 with borehole data (Yasuda et al., 1995). As described in the previous section, the material property of the Pleistocene soil is little known. In addition, the depth of the base is not in common agreement at present. On the other hand, the Pleistocene soil and the subsoil below it will not behave in a nonlinear manner. Considering these situations, we analyze only the surface deposit in which nonlinear behavior is expected.

One dimensional analyses are made of the locations where borehole data are shown in Fig. 9, for the following reasons. Although the soil layer seems to tilt in Fig. 9, this is a result of the difference of scale in the horizontal and vertical directions; the slope is about 1% on average. The damping ratio of the soft layer is greater than 1%, mainly several percent in the previous analysis;

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**Fig. 9. Analyzed section passing the Sannomiya station, and acceleration response spectra (h = 5%). The soil profile is shown with the borehole data (Yasuda et al., 1995).**
in this case, waves become smaller in several cycles, so wave propagation in the horizontal direction is small.

Equivalent linear analysis is employed because it has been frequently used practically, and, as shown in the previous sections, the general feature of the response can be evaluated by equivalent linear analysis. Shear wave velocity is evaluated based on the Specification of Highway Bridges (Japan Road Association, 1990). Empirical equations proposed by Yasuda and Yamaguchi (1985) are used for the strain dependent shear modulus and damping characteristics. The shear wave velocity in the base is set at 500 m/s so that the predominant period obtained by Uehan and Nakamura (1995) by means of microtremor coincides with the predominant period of the analyzed model. The incident wave at Port Island at a depth of GL-83.4 m (location of earthquake measurement) is computed by the equivalent linear method and used as the incident wave at the base.

Distributions of the calculated PGA, PGV and JMA intensity scale at the ground surface are shown in Fig. 10. PGVs do not change very much in all regions. On the other hand, the change in PGA is large; it keeps nearly constant on the Rokko mountain side, the left side of the figure, but decreases rapidly on the sea side. Referring to the soil profile shown in Fig. 9, it is clear that the existence of the Holocene clay layer contributed to the difference. Namely, as with as the result in the previous sections, the behavior of the Holocene clay layer affects the shaking at the ground surface. This phenomenon, i.e., PGA decreases by the nonlinear behavior but PGV does not, is inconsistent with the tendency in Fig. 1.

Distributions of the acceleration response spectra with 5% damping are also shown in Fig. 11. Distribution of the response in natural periods shorter than 0.8 second is similar to that of the PGA; it decreases rapidly on the sea side. On the other hand, distributions of 2 second natural periods appear similar to those of the PGV, while those of 1.2 second natural periods appear similar to the JMA intensity scale. JMA intensity scale is calculated with a filter which emphasizes the component of the periods at about 1.2 seconds, as shown in Fig. 12, and velocity has frequency characteristics of the reciprocal of circular frequency for acceleration. Because the soft deposits behave as a low-pass filter during the strong ground motion, as mentioned in the previous section, only short period components decrease on the sea side. The sea side boundary of the damage belt corresponds with the geological boundary where a soft clay layer (Ma13) either exists or doesn’t. Therefore, it is concluded that damage to structures whose natural periods were shorter than 1 second, such as wooden houses (Uehan and Nakamura, 1997), was not severe on the sea side.

The damage belt exists between 500 m and 1500 m in the soil profile in Fig. 9. Referring to Fig. 11, maximum accelerations in these regions are very large, about 3000 cm/s². In addition, the range of periods with acceleration higher than 2000 cm/s² is roughly between 0.5 and 1 second. This period range coincides with the predominant period of many low to medium rise structures and houses. This explains the appearance of the damage belt.

There is no soft soil such as the Holocene clay or fill in the damage belt region. This is important because earthquake disasters have usually been believed to occur in
soft ground, due to the large amplification of the earthquake motion. This conventional sense may be true when the input earthquake motion is not very large so that the amplification effect in soft soil plays an important role with structures too weak not to resist the amplified earthquake motion. On the other hand, when the input motion becomes very large, as in this earthquake, the deamplification effect of soft soil layer through the occurrence of the nonlinear behavior of soil becomes predominant. Therefore, if structures have sufficient strength under a deamplified ground motion, which is of course larger than an amplified ground motion under relatively weak earthquake input, the damage to the structures is expected to be small in a soft ground region, as seen in the damage in this earthquake.

The basin-edge effect of wave propagation is said to be the reason for the appearance of the damage belt (e.g., Motosata and Nagano, 1996; Kawase and Hayashi, 1996). Even if this is true, the ground shaking cannot be very large because of the existence of soft ground. Therefore, we should take into account the effects of nonlinear behavior of the ground in order to investigate the region of damage belt.

ESTIMATION OF INCIDENT WAVE ON THE TOP OF OSAKA GROUP FORMATION

In this section we will expand the investigated region from one section passing the Sannomiya station to the Kobe city area. It was shown in the previous sections, especially in Fig. 10, that nonlinear behavior is important in this region and its effect on the peak parameters is different depending on the soil type. Table 2 is a list of the peak ground motion parameters at 6 sites, i.e., Kobe University, Port Island, JMA-Kobe, Fukiai Gas Supply Depot., JR Takatori station and Shin-Kobe S.S., in the region. Peak values are calculated in the same way as Fig. 1. It is seen that the PGA is large in stiff and medium ground, the PGV and JMA intensity scale are large in medium ground, and the PGD is large in soft ground. Acceleration response spectra with 5% damping shown in Fig. 13 explain the reason; the long period component is

<table>
<thead>
<tr>
<th>Recording site</th>
<th>Soil condition</th>
<th>PGA (cm/s²)</th>
<th>PGV (cm/s)</th>
<th>PGD (cm)</th>
<th>JMA intensity scale</th>
</tr>
</thead>
<tbody>
<tr>
<td>JMA Kobe</td>
<td>stiff</td>
<td>848</td>
<td>105</td>
<td>26.8</td>
<td>6.41</td>
</tr>
<tr>
<td>Fukiai</td>
<td>medium</td>
<td>834</td>
<td>131</td>
<td>49.2</td>
<td>6.49</td>
</tr>
<tr>
<td>JR Takatori St.</td>
<td>medium</td>
<td>743</td>
<td>156</td>
<td>49.0</td>
<td>6.48</td>
</tr>
<tr>
<td>Shin-Kobe S.S.</td>
<td>stiff</td>
<td>669</td>
<td>94</td>
<td>29.5</td>
<td>6.18</td>
</tr>
<tr>
<td>Port Island</td>
<td>soft</td>
<td>426</td>
<td>103</td>
<td>49.3</td>
<td>5.93</td>
</tr>
<tr>
<td>Kobe Univ.</td>
<td>rock</td>
<td>322</td>
<td>56</td>
<td>15.1</td>
<td>5.66</td>
</tr>
</tbody>
</table>

Fig. 13. Acceleration response spectra (h = 5%) at objective sites. The damage belt of JMA intensity scale VII is also shown, in which more than 30% of houses were collapsed.
predominant on the sea side (Port Island), the short period component is predominant on the mountain side (JMA Kobe, Shin-Kobe S.S.), and the predominant period covers a wide range in the damage belt region (JR Takatori St., Fukiai). This observation is consistent with the finding in the previous section. If, however, the input earthquake motion is different in these regions, it will affect the ground shaking at the surface, therefore discussion in the previous section may have needed to consider another factor. In order to confirm this, we will evaluate the earthquake motion on the upper boundary of the Osaka Group Formation (Middle or Early Pleistocene series) or the layer whose shear wave velocity \((V_s)\) is similar to that at the site shown in Table 2. The component in the direction of N40W°–S40E°, that is in the direction perpendicular to the fault (Sawada et al., 1996), where ground shaking is expected to be predominant, is retrieved in the following discussion. The procedure to compute incident waves in these sites are as follows.

**JMA Kobe**

This site is located on a small hill top, so the topographic effect should be considered. Figure 14 shows a two-dimensional geological model, the upper part of the model shown by Kawase and Hayashi (1996). Shear wave velocity, density and damping coefficient are shown in Table 3. The amplification factor of the ground surface at the hill top to the base can be calculated under this condition by 2-D FEM linear analysis. The incident wave is, then, computed from the observed record by dividing the amplification factor.

**Port Island**

The incident wave is estimated from the record observed at GL-83 m by the equivalent linear method; this procedure is already described in the preceding.

**Shin-Kobe S.S.**

Matsumoto et al. (1996) conducted PS-logging and dynamic deformation tests to obtain the modulus and damping after the earthquake, and calculated the incident wave at GL-65 m by FDEL (Sugito, 1995). Here, we retrieve the incident wave at GL-33 m, which is the upper boundary of the weathered granite whose shear wave velocity is 450 m/s.

**Fukiai**

Both borehole data near the site (Kobe Municipal Office, 1980) and a few aftershock records are available. The same aftershocks are also recorded at the Port Island vertical array that is located about 2 km from the site. The S-wave velocity structure at the site is determined by fitting the transfer function calculated by the one-dimensional multiple reflection theory to the ratio of the vector spectrum (Nakamura, 1995) of the aftershocks at Fukiai to that at Port Island (GL-83 m) shown in Fig. 15, all of which is shown in Table 4. Strain dependent characteristics of the shear modulus and damping coefficient obtained at Port Island (Kobe City, 1995) and Kobe Univ. of Mercantile Marine (Iwata et al., 1996) are used in the analysis.

Figure 16 compares the ratios of the observed records during the main shock and the transfer functions calculated by the equivalent linear method. Here, the improved equivalent linear method, FDEL (Sugito, 1995), and SHAKE are used. Amplification factors are much smaller than the observed values in the frequency range higher than 3 Hz in the analysis by SHAKE, whereas

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**Table 3. Soil profile model at JMA Kobe**

<table>
<thead>
<tr>
<th>No.</th>
<th>Soil type</th>
<th>Density ((\text{t/m}^3))</th>
<th>(V_s) ((\text{m/s}))</th>
<th>(V_p) ((\text{m/s}))</th>
<th>Damping coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>weathered diluvial sand</td>
<td>1.80</td>
<td>165</td>
<td>1800</td>
<td>0.02</td>
</tr>
<tr>
<td>2</td>
<td>upper diluvial clay</td>
<td>1.50</td>
<td>250</td>
<td>1800</td>
<td>0.02</td>
</tr>
<tr>
<td>3</td>
<td>upper diluvial sand</td>
<td>1.85</td>
<td>320</td>
<td>1800</td>
<td>0.02</td>
</tr>
<tr>
<td>4</td>
<td>upper diluvial gravel</td>
<td>1.90</td>
<td>375</td>
<td>1900</td>
<td>0.02</td>
</tr>
<tr>
<td>5</td>
<td>upper Osaka group</td>
<td>1.95</td>
<td>450</td>
<td>1900</td>
<td>0.01</td>
</tr>
</tbody>
</table>

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**Fig. 15. Spectral ratios of records at Fukiai to those at Port Island (GL-83m) during the main shock and two aftershocks. Transfer function is estimated by fitting to the spectral ratios during the aftershocks.**
hoose by FDEL show better agreement. Therefore, we used FDEL in evaluating the incident wave at Fukiai.

\textit{JR Takatori St.}

Borehole data and predominant frequency estimated by microtremor observation are reported by Nakamura et al. (1996). The S-wave velocity structure is determined as shown in Table 5. Strain dependent characteristics are determined in the same way as Fukiai and FDEL is used for evaluating the incident wave at the base layer.

Figure 17 shows all the transfer functions used for calculating the incident wave on the upper Osaka Group formation. The predominant periods are quite different from each other. This result shows that the effects of the characteristics of shallow soil deposits are very important in evaluating ground shaking at the surface.

The PGA, PGV, PGD and JMA intensity scale of the incident waves (2E) at 5 sites are shown in Table 6. Peak values except PGA are the largest at JR Takatori St. This may be caused by the source process, but the information on the soil profile deeper than 20 meters is limited for JR Takatori St., so careful consideration is necessary. The PGVs at other sites are about 80 cm/s. Considering it, the ground motions at the top of the upper Osaka Group formation are well deconvoluted.

Figure 18 shows acceleration response spectra of incident waves (2E) with 5% damping. The acceleration responses are identical at about 500 cm/s² in the period from 0.1 to 1.0 second. This indicates that if there exist predominant periods shorter than 1 second at the surface, they are generated by shallow site amplification. Predominant periods longer than about 1 second (0.8
The behavior of the surface ground during the 1995 Hyogoken-Nambu earthquake was investigated through various ways, which are summarized as follows:

1) The source effect and basin-edge effect are seen in the incident waves on the top of the base, but, when focused on the region whose size is several kilometers large, the incident waves are almost the same. Even in this narrow region, however, the earthquake motion at the ground surface and damage pattern are sometimes quite different. This indicates that the behavior of the ground strongly affects the earthquake motion on the ground surface, hence the damage pattern.

2) Strong nonlinear behavior occurred in both Holo-

![Graph showing acceleration response spectra](image)

**Fig. 18.** Comparison of acceleration response spectra \((h=5\%)\) of estimated incident waves

![Graph showing ground surface waveforms](image)

**Fig. 19.** Acceleration time histories of calculated incident waves \((2E)\) on the top of Osaka group formation band-pass filtered 0.5-5 sec. (n.b. times are not accurate.)
cane clay (Ma13) and fill, which distinguish the damage pattern. The ground in the damage belt zone is classified as medium ground and there is no soft material. The ground shaking in this region was severe because of the large input motion and amplification from the base, resulting in significant damage to structures. The sea side boundary of the damage belt zone coincides with the appearance of the soft clay layer, and because of the deamplification effect due to nonlinear behavior in this layer, damage to houses is smaller to the south of the damage belt zone than in the zone itself. In the regions along the shore line, in addition, there is fill that liquefied significantly, contributing more deamplification of the motion. As a result, houses that collapsed because of the inertia force were few in this regions although damage caused by the large ground deformation was frequently seen.

3) The PGA and JMA intensity scale change depending on the subsoil condition, but the effect of the surface geology on the PGV seems much smaller than it does on the PGA and JMA intensity scale. This is confirmed not only in the analysis in the Kobe region but also at fairly far sites. Since the JMA intensity scale was developed based mainly on damage to houses, it is natural that distribution of the JMA intensity scale agrees with the damage pattern, and this agreement confirms the accuracy of the analysis, too. It represents the mid-period component from 0.5 to 3 seconds in the earthquake motion. On the other hand, the PGA represents period components shorter than the JMA intensity scale, i.e., 0.8 second or shorter, and the PGV represents period components longer than the JMA intensity scale of about 2 seconds or longer, neither of these agrees with the predominant period of ordinary houses. Therefore, the above mentioned statement is consistent with the observed damage pattern.

ACNOWLEDGMENTS

The authors would like to express their deep appreciation to the Japan Meteorological Agency, the Committee for Earthquake Observation and Research in Kansai Area, Kobe City, Kansai Electric Power Company, Osaka Gas Co., and Railway Technical Research Institute for providing the earthquake records.

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