GEOTECHNICAL SIMULATION TEST FOR THE NIKAWA LANDSLIDE INDUCED BY JANUARY 17, 1995 HYOGOKEN-NAMBU EARTHQUAKE

FA-WU WANG\(^0\), KYOJI SASSA\(^0\) and HIROSHI FUKUOKA\(^0\)

ABSTRACT

By employing an undrained cyclic loading ring-shear apparatus, a series of tests to reproduce the dynamic behavior of the Nikawa landslide induced by the January 17, 1995 Hyogoken-Nambu earthquake, is conducted. The test sample is Osaka-group coarse sandy soil taken from the landslide. The initial stress condition acting on a soil element in the sliding surface is applied to the sample. Based on the seismic records monitored at the JR Takarazuka Station, the input seismic wave is synthesized to reproduce the seismic stress acting on the sliding surface. The test results show that the soil failed due to the dynamic loading of the earthquake. The most important results are the excess pore water pressure generation and the acceleration of shear displacement continuing after the main shock. Combined with the grain crushing at the shear zone and the volume reduction in the drained constant-speed ring-shear test, the mechanism of this landslide is interpreted as, shear displacement causing grain crushing in the shear zone and volume reduction, and then resulting in a localized liquefaction phenomenon, "sliding-surface liquefaction". This geotechnical simulation test provides a reasonable interpretation of this highly mobile landslide.

Key words: excess pore water pressure, grain crushing, landslide, (localized) liquefaction, (ring-shear test), (seismic record), (simulation test) (IGC: E6/E8/D6)

INTRODUCTION

The January 17, 1995 Hyogoken-Nambu earthquake in Japan killed more than 6,430 persons, destroyed about 200,000 houses, and induced many landslides. The Nikawa landslide was one of the largest geo-disasters of the earthquake. It destroyed 11 houses and killed 34 persons. According to Sassa et al. (1995), the landslide volume was 110,000 - 120,000 m\(^3\), and the moving distance was 175 m. No observation data of the sliding speed is available. However, it is believed that it was a high-speed landslide, because no one was evacuated from the destroyed houses and all 34 residents were killed. Concerning the time of the landslide, Mr. Tsuchito Tanaka, a witness who lived near the Nikawa landslide said: "the announced occurrence time of the Hyogoken-Nambu earthquake is from 5:46:51.6 on the morning of January 17, 1995. The occurrence time of the landslide was one or two minutes later. I can also remember there were two strong shocks of the earthquake. It is difficult to tell the exact time of the landslide, I can just say that the landslide occurred as soon as the second shock of the earthquake came."

As an example of rapid landslide induced by earthquake, the Nikawa landslide was studied by Sassa et al. (1996), with an undrained ring-shear apparatus, DPR1-3. In that study, normal stress was kept as constant, and shear stress was applied as a sine wave at 0.1 Hz with the amplitude increasing cycle by cycle. In the undrained condition, the sample was sheared and the effective stress path was obtained. Based on the study, Sassa et al. (1996) proposed a concept termed "sliding-surface liquefaction." Compared with normal (mass) liquefaction, sliding-surface liquefaction can take place even in medium and dense soil layer; it emphasizes the build-up of excess pore water pressure due to grain crushing in the shear zone and the resulting volume reduction.

On the subject of landslides during earthquake, Seed (1968) gave a comprehensive report at the fourth Terzaghi lecture. The mechanism of landslides induced by earthquake was concluded to be due to soil liquefaction. Thereafter, using triaxial compression apparatus, a great deal of research on soil liquefaction was carried out. On the mechanism of liquefaction and flow failure during earthquakes, great developments were achieved (Ishihara, 1993; Verdugo and Ishihara, 1996). However, because of the limit of shear strain in triaxial compression tests, the post-failure behavior of sandy soils has been poorly understood until now. On the other hand, besides the sliding mechanism of the landslide triggered by earthquake, shear displacement is another important parameter. In this respect, the "sliding-blocking theory" proposed by Newmark (1965) is commonly used. However, the reduction of the shear strength caused by

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the built-up of excess pore water pressure was not generally considered. The dynamic elasto-plastic finite element method (FEM) to simulate the dynamic behavior of a slope under seismic loading has the same disadvantage. When analyzing sliding displacement of slopes during earthquake by FEM code, Ugai et al. (1995) concluded that although FEM can effectively predict the limited shear displacement of a dry slope caused by earthquake, it is difficult to simulate a long runout landslide. Against this background, an investigation that can not only measure the stress and pore water pressure variation before failure, but also monitor the post-failure behavior becomes necessary. Concerning the effect of seismic frequency on the dynamic behavior of slope, it is believed that frequency of cyclic loading has no obvious effect on the excess pore water pressure generation in the undrained condition, and a loading frequency between 0.05 Hz and 1 Hz is generally used as the frequency of cyclic loading in undrained triaxial tests (Tatsuoka et al., 1986), simple shear tests (Peacock and Seed, 1968) and ring-shear tests (Yoshimi and Oh-oka, 1975). However, according to Newmark’s “sliding-block theory” (Newmark, 1965) and Arias’ intensity measure (also termed acceleration energy, is the sum of the energy absorbed by a population of simple oscillators evenly spaced in frequency (Arias, 1970; Kayen and Mitchell, 1997)), a cycle of cyclic-loading with low frequency means high energy. According to these theories, loads with lower frequencies correspond to greater energy to the soil than those given by an actual earthquake. Therefore, a controversy has arisen. To clarify the question and investigate the dynamic behavior of landslide during earthquakes, it is necessary to perform a test using a real earthquake record. To meet these requirements, an improved undrained ring-shear apparatus, DPRI-5, which makes it possible to load a real seismic wave and which can conduct long distance undrained shear tests was developed in 1995 by Sassa (1997).

The grain crushing property of sandy soils has a great effect on pore water pressure generation in undrained shearing. Other than Sassa’s proposed concept of “sliding-surface liquefaction,” there have been some papers presented concerning the effect of grain crushing on the built-up of excess pore pressure. The concept of “steady state of sand” also emphasized the completion of all of the grain crushing at the corresponding effective stress level. Fragaszy and Voss (1986) conducted compressive loading tests on Monterey No. 0 and Eniwetok sands and found that significant grain crushing and cracking occurred in the Eniwetok sand and to a lesser extent in the Monterey sand. Based on undrained compression tests on both sands, it was concluded that the plastic volume changes causing residual excess pore water pressure under compression loading appear to come from the crushing of the individual particles.

Based on the background described above, the main purpose of this research is to study the mechanism of landslides triggered by earthquakes, observe the generation of shear displacement, analyze the post-failure behavior of an actual slope failure triggered by an earthquake, and investigate the relationship between grain crushing and the residual excess pore water pressure generation of used sandy soils. To search the critical condition for slope stability is beyond the scope of this study. The stress state of a representative point in the sliding zone was selected and reproduced. As a simulation test in a ring-shear box, this study aims to supply a representative view of the whole landslide from one point. Two tests were performed in this investigation. One is the simulation test loaded with the seismic record monitored in the January 17, 1995 Hyogoken-Nambu earthquake, to examine the mechanism of the Nikawa landslide induced by the earthquake, and the other is a drained ring-shear test to investigate the grain crushing susceptibility of the tested sample.

**SETUP OF THE UNDRAINED RING-SHEAR APPARATUS**

Initially, the modern ring-shear apparatus was developed by Bishop et al. (1971) to study the residual strength of soil. It is a drained speed-control ring-shear piece of apparatus. Because there is no limit in shear displacement, it is widely used to study the mechanical behavior of landslide motion (Tika et al., 1996). A series of stress-control ring-shear pieces of apparatus was developed and improved in Disaster Prevention Research Institute (abbreviated as DPRI), Kyoto University by Sassa et al. (Sassa, 1995; 1996 and 1998); four of these (DPRI-3 to 6) can conduct undrained test. The basic structure of the four is the same, but the size of shear box, stress level, shear speed and loading frequency are all different.

In this test, the improved smaller size apparatus (DPRI-5) was used. The most important features of this apparatus are listed in Table 1. The essential technology for the undrained ring-shear test is in the design of the shear box, which is illustrated in Fig. 1. The lower half is rotated while the upper half is retained by means of a pair of load cells for shear resistance measurement. Normal stress is provided through the loading plate rolled by “O”-rings. The rubber edge in the lower half, coated by Teflon and vacuum silicon grease, is always pressed onto the upper edge at a certain pressure greater than the expected maximum pore-water pressure to keep the shear box in undrained state. The contact pressure of the rubber edge is automatically controlled by maintaining a gap.

| **Table 1. The most important features of the DPRI-5 ring-shear apparatus** |
|---|---|
| Shear box | 120 |
| —inner diameter (mm) | 180 |
| —outer diameter (mm) | 2.0 |
| Maximal normal stress (MPa) | 10 |
| Resolution of gap-control system (mm) | 0.001 |
| Maximum data acquisition rate (readings/sec) | 200 |
| Maximum frequency of cyclic loading (Hz) | 5 |
| Capability of undrained test | Yes |
| Capability of loading with monitored seismic wave | Yes |
| Automatic safeguard and alarm system for mishandling | Yes |
between the two halves of the shear boxes using a servo-control system with a precision of 1/1000 mm. The friction of the rubber edge is measured and subtracted from

the monitored shear resistance. Pore water pressure is measured by a pore-pressure transducer, which is connected through water to the gutter (4 × 4 mm²) cut along the entire circumference of the outer-upper ring. The gutter is covered with two metal filters with a felt cloth between them and located at 2 mm above the shear surface. Theoretically, the pore-pressure transducer measures the pore-water pressure generated at the shear surface. During undrained tests, normal stress, shear resistance, pore water pressure, shear displacement and vertical displacement are measured and stored automatically by the data acquisition system. Further details of the equipment and procedures are given by Sassa (1998).

THE PROPERTIES OF THE NIKAWA LANDSLIDE

The details of the Nikawa landslide were introduced by Sassa et al. (1996). Figure 2 is a photograph showing the whole area of the Nikawa landslide. It was triggered by the Hyogoken-Nambu earthquake and moved for a long distance to the nearby residential area. Figure 3 shows a plan of the landslide area before the landslide occurred. The sliding direction of this landslide is roughly N60°E. Location of borings and excavation pits P1 and P2 for observation and sampling point S1 are plotted. Standing ground water was observed at secondarily moved debris. Although January in this area was very dry, the Osaka-group layer composing the slope retained ground water. The table of ground water was confirmed later by ground water level monitored in bore-holes, because the ground water takes a very important role in landslide motion.

Figure 4 is the A-A’ section shown in Fig. 3. It is

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Fig. 1. The undrained shear box of the ring-shear apparatus (DPRI-S)

Fig. 2. Photograph of the Nikawa landslide (Taken by K. Sassa, January 21, 1995)
drawn based on the ground surface survey and boring investigation. The average angle of the sliding surface was about 20 degrees. The base rock of the slope was granite. Above the base rock was an Osaka-group layer, whose deposit is a limnetic and marine deposit of the Pliocene to the Mid-Pleistocene distributed around the Osaka
area, Japan (Itihara, 1996). Overlaid on it are terrace deposits distributed on the slope. In this section, the Osaka-group layer is shown as a single layer. In fact, three sub-layers were observed in the field: a rocky sub-layer at the base (O₁ sub-layer), a sandy soil sub-layer (O₁s, O₂s and O₃s), and a clayey sub-layer (O₂c). In the upper part and lower part of the slope, the Osaka-group layer was overlaid by the terrace deposit. On the old slope surface, landfill consisted of the distributed Osaka-group layer. The ground water was observed to flow out from the headscarp of the landslide. To investigate the depth of the ground water table, some boreholes were drilled immediately after the landslide. The ground water table existed 6–7 m below the ground surface near and outside of the landslide area in three boreholes from February to March, 1995 (Kawasaki Geology Corporation, 1995; Sassa et al., 1996). The landslide occurred in the landfill. According to the standard penetration test, the N value at the landfill was smaller than 10, which showed a relatively small resistance. No other evidence suggests the soil layer was overconsolidated. Thus, it is assumed that the soil layer was normally consolidated in a dry condition, and the lower part was saturated by ground water. A test sample was taken from the landslide mass just above the sliding surface by excavating the landslide debris, namely the same materials (Osaka-group coarse sandy soil) in which the sliding surface was formed.

**INPUT OF SEISMIC LOADING**

There are many earthquake records of the Hyogoken-Nambu earthquake from observations at different locations. Unfortunately, none of these were located at the Nikawa landslide area. Theoretically, it would be best to use a seismic record monitored at the landslide site, but it does not exist. Among the seismic records that do exist, the seismometer at the JR Takarazuka Station is the nearest one to the Nikawa landslide. Therefore, the real seismic wave monitored at the JR Takarazuka Station was used in this study (courtesy of the Japan Railway Technical Research Institute).

There are two important elements in the input seismic wave. One is the peak ground acceleration, and the other is wave shape. Concerning the peak ground acceleration, an attenuation equation (Eq. (1)) proposed by Fukushima and Tanaka (1992) was employed.

\[
\log A = 0.42 M_w - \log (R + 0.025 \times 10^{0.42 M_w}) - 0.0033 R + 1.22
\]

where \(A\) is the average of two peak horizontal accelerations in cm/s², \(M_w\) is the moment magnitude and \(R\) is the distance from an observatory station to the fault rupture in km. Although this equation is for horizontal acceleration, because vertical acceleration also attenuates following the same law, the correction of the vertical acceleration is also processed with this equation.

According to the “Active Fault Map in Urban Area”, published by the Geographical Survey Institute of Japan (1996) and Irikura and Fukushima (1995), the distance from the active fault, the Koyo active fault, to the JR Takarazuka Station and the Nikawa landslide are about 7 km and 0.5 km, respectively. The peak acceleration for the moment magnitude (\(M_w\)) value for the Hyogoken-Nambu earthquake was 7.0. By calculation, the peak ground acceleration at the Nikawa landslide was about 1.4 times that at the JR Takarazuka Station.

Fukushima and Tanaka (1990) also found that, in general, the mean curve of the peak ground acceleration for loose soil was 140% times the average curve. The seismic measurement system was set in a stiff layer at the JR Takarazuka Station, while, as mentioned above, the Nikawa landslide mass consisted of landfill of Osaka-group coarse sandy soil that had a low standard penetration number in the standard penetration test, i.e., a low N value. So the peak acceleration in the landslide consisting of landfill was assumed to be 1.4 times that measured by seismometers at Takarazuka Station. Considering the two factors, \(2.0 \times 1.4 \times 140\% \approx 2.0\) was selected as the amplifying coefficient of the peak ground acceleration. This means that the applied peak ground acceleration is 2.0 times that monitored at the JR Takarazuka Station.

Concerning the wave shape and loading magnitude, through the following procedure (based on Fukuoka et al., 1998), the seismic wave acting on the soil element in the sliding surface was obtained.

1. Determine the horizontal seismic acceleration \(a_{HR}(t)\) in the sliding direction of the landslide by means of NS components \(a_{NS}(t)\) and EW components \(a_{EW}(t)\) of the earthquake (Fig. 5(a)).

\[
a_{HR}(t) = (a_{NS}(t) + a_{EW}(t))^2 / 2 \cos \delta.
\]

2. Determine the two components in parallel direction and normal direction to the sliding surface, by summing the horizontal component \(a_{NR}(t)\) and the vertical component \(a_{SV}(t)\) (Fig. 5(b)). The component in the normal direction to the sliding surface, \(a_{SN}(t)\), and the component in the shear direction of the sliding surface, \(a_{SH}(t)\), are obtained as:

\[
a_{SN}(t) = a_{SV}(t) \cos \theta - a_{HR}(t) \sin \theta
\]

\[
a_{SH}(t) = a_{SV}(t) \sin \theta + a_{HR}(t) \cos \theta.
\]

3. The three components of the earthquake, i.e., UD, EW and SN monitored at the JR Takarazuka Station were used to calculate \(a_{SR}(t)\) and \(a_{SR}(t)\), the acceleration parallel and normal to the sliding surface of the Nikawa landslide. Then, considering the distance effect and the difference caused by bedrock and landfill, these two components were multiplied by 2.0, and then two acceleration components, i.e., the component in the normal direction to the sliding surface, \(A_{SN}(t)\), and the component in the shear direction of the sliding surface, \(A_{SH}(t)\), are obtained as:

\[
A_{SN}(t) = 2.0 \times a_{SR}(t), \quad A_{SH}(t) = 2.0 \times a_{SR}(t).
\]

4. Calculate the initial stress and the seismic stress on
the sliding surface. A soil column with a unit width along the slope was considered in Fig. 5(c). So, the self-weight of a unit column is:

$$W = \gamma H \cos \theta, \quad m = W/g.$$  \hspace{1cm} (6)

The initial stress condition can be determined by:

$$\sigma_0 = W \cos \theta, \quad \tau_0 = W \sin \theta.$$  \hspace{1cm} (7)

Moreover, the seismic stresses can be determined by:

$$\Delta \sigma(t) = m A_{\text{SR}}(t), \quad \Delta \tau(t) = m A_{\text{SH}}(t).$$  \hspace{1cm} (8)

(5) Calculate the test stress parameters. Here, we use the following values based on the field investigation: $H=14$ m, $\theta=20^\circ$, $\gamma_1=17.6$ kN/m$^3$, $\gamma_2=20.6$ kN/m$^3$ (Kawasaki Geology Corporation, 1995). The ground water table above the sliding surface was assumed to be about 7 m (at least), because the ground water table outside the landslide area was 6 - 7 m below the ground surface. Accordingly, the initial normal stress $\sigma_0$ was 236.1 kPa, the initial shear stress $\tau_0$ was 85.9 kPa, and the initial pore water pressure $u_0$ was 60.6 kPa.

Eventually, the dynamic-loading input was obtained as shown in Fig. 6. The seismic wave lasts for 40 seconds. The main shock is distributed between 3 seconds and 7 seconds. After the seismic wave is over, the stress state of the soil element returns to the initial total stress state.
Figure 7 is the total stress path obtained from applied normal stress and shear stress. The failure line drawn in this figure is the residual failure line obtained from the effective stress path in the following simulation test. Some points are located above the failure line, meaning that the earthquake loading gives greater stress over the residual failure line.

SAMPLE PREPARATION AND TEST PROCEDURE

The sample of Osaka-group layer taken from the Nikawa landslide site is an angular sandy soil consisting of weathered granite and is composed of 77 percent quartz and 23 percent feldspar and mica. Grains greater than 4.75 mm make up about 7% of the sample. Considering the size of the shear box, they were eliminated after being dried in an oven. The physical properties of the sample are shown in Table 2. It should be mentioned that the measurements of these properties followed the Japanese Geotechnical Society Standard (Japanese Geotechnical Society, 1990). Because the sample is not suitable for the test method for maximum and minimum densities of sands, which is restricted by the standard No. JSF-T-161-1990, these values for the sample are just for reference.

It was assumed that the soil mass at the sliding zone before the occurrence of the landslide was normally consolidated. So, in the sample preparation process, the sample was normally consolidated at first, and then an initial pore water pressure was applied considering the existence of the groundwater. The seismic loading simulation test on the Nikawa landslide was performed with the following procedures:

1. Weigh and set the oven-dry sample in the shear box using the free-fall deposition method, and then saturate it. Consolidate the sample under 50 kPa normal stress at first, and confirm the degree of saturation by applying a normal stress increment, \( \Delta \sigma = 50 \) kPa, in the undrained condition, and measuring the increment of the excess pore pressure, \( \Delta u \). The pore pressure parameter \( B_0 (\Delta u / \Delta \sigma) \) is then calculated (Sassa, 1985). A high \( B_0 \) value indicates a high degree of saturation. The \( B_0 \) value for the sample in this test was 0.99, which means a high degree of saturation was achieved.

2. Consolidate the sample at the initial normal stress, \( \sigma_0 \) and then apply the initial shear stress, \( \tau_0 \), at drained condition slowly (0.1 kPa/sec) to avoid the built up of excess pore water pressure in the shear zone.

3. Apply the initial pore water pressure \( u_0 \) from the upper drain line to simulate the ground water condition. The relative density of the sample is then 121.2 percent. Following this procedure, the actual initial stress state is reproduced.

4. Change the shear box to the undrained condition, because the soil element at the sliding surface should be under the undrained condition during earthquake loading. Load the dynamic waves of normal stress and shear stress simultaneously, as shown in Fig. 6. During the test, the data acquisition rate is set as 200 readings/sec, the maximal data acquisition rate.

![Diagram of simulation test](image)

**Table 2. Physical properties of the test sample of the Osaka-group coarse sandy soil**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity, ( G_s )</td>
<td>2.60</td>
</tr>
<tr>
<td>Maximum void ratio, ( e_{max} )</td>
<td>1.107</td>
</tr>
<tr>
<td>Minimum void ratio, ( e_{min} )</td>
<td>0.589</td>
</tr>
<tr>
<td>Uniformity coefficient, ( U_r )</td>
<td>75</td>
</tr>
<tr>
<td>Average grain size, ( D_{av} ) (mm)</td>
<td>0.50</td>
</tr>
</tbody>
</table>

**Fig. 8. Time-series data of the simulation test on the Nikawa landslide.** \( B_0=0.99, \sigma_0=121.2 \) percent: (a) Normal stress, (b) Pore water pressure, (c) Shear resistance and shear displacement.
mic wave. This difference suggests that the excess pore water pressure could not accumulate when no plastic shear deformation is generated. Figure 8(c) shows the variation of shear resistance and shear displacement. During the main shock, the sample failed because the loaded shear stress exceeded the shear strength of the soil. After the soil failed, the shear resistance became much smaller than the applied shear stress (Fig. 6). When soil failed and the slope began to move, the modified shear stress (the value of 2.0 was adopted as the amplifying coefficient) could not act on the soil mass completely. It can be concluded that the amplifying coefficient mainly affects the dynamic behavior before the sample fails and has no obvious effect on the post-failure behavior. Shear displacement was small during the main shock, while it increased rapidly after the main shock. This time lag that shows the large shear displacement occurred after the main shock corresponds to the testimony of the eyewitness of Mr. Tsunehito Tanaka. The mobilized shear resistance changed rapidly, but the maximum value did not decrease so much during the main shock. It is quite remarkable that the shear displacement accelerated. This apparently resulted from the decrease of shear resistance to a certain low value, reasonably corresponding to the tendency of excess pore water pressure built-up after the main shock.

To check the monitored data more precisely, the time series data from the start to 10 seconds are presented in Fig. 9. Pore water pressure varied with the same phase with the normal stress with an increasing tendency. From 3.5 second, the shear displacement was generated, meaning that the sample failed. From about 4 second, the shear displacement increased rapidly. However, the shear displacement was only 5 mm when the main shock was over (at about 7 second). At the end of the seismic loading at 40 second, the shear displacement increased to about 350 mm. A very important phenomenon is that, the shear displacement continued to increase after the main shock, and the excess pore water pressure continued to be built-up, as clearly found in Fig. 9(b). This naturally resulted in the reduction of shear resistance, and caused the rapid high-mobility landslide. This failure is different from (mass) liquefaction however. The reduction of the shear resistance is gradual and there is no sudden drop-down of the shear resistance in the time series curve. In other words, this failure is not caused by structure collapse, but by another mechanism. Considering the susceptibility of the Osaka-group (weathered granitic coarse sands) to grain crushing during shearing under a certain normal stress, it is believed that the grain crushing plays an important role in this process. For dense sandy soils difficult to collapse, grain crushing always causes the volume reduction during shearing (or moving); under the undrained condition, it in turn causes the excess pore water pressure generation.

Figure 10 shows the stress path obtained in the simulation test. ESP means the effective stress path, while TSP means the total stress path. Because of possible delay of pore water pressure measurement during the period of high frequency, some stress points are distributed above the failure line. Although it is difficult to follow the process of each stress path with this figure, referring to the time series data, it is reasonable to describe the stress path as follows. At first, the stress path reached the peak strength failure line ($\phi_p = 39.6^\circ$) and the soil failed. The state of shear zone then became residual. During the progress of shearing after failure, the effective stress path turned to the residual failure line ($\phi_r = 35.5^\circ$). Shearing under a high effective stress should cause the grain crushing and then result in the generation of excess pore water pressure. Thereafter, with the generation of great excess pore water pressure, the effective stress path descended along the residual failure line to a very low effective stress level. The grain crushing process goes to close, until the effective stress became small enough that grain crushing cannot take place any more. This is somewhat different from the usual liquefaction, in which the stress path instantaneously reduces to a very low stress level without reaching the peak strength failure line. It is consistent, however, with the phenomenon of "sliding-surface liquefaction" where the excess pore water pressure caused by grain crushing in the shear zone during shearing is more important. The grain crushing leads to a volume reduction, and results in the built-up of excess pore water pressure. To examine the concept, the built-up of residual excess pore water pressure with shear displacement is presented here, and the grain crushing property of the sample will be investigated later.

According to our recent findings (Wang and Sassa,
1998; Sassa and Sekiguchi, 1999), in the undrained condition, the excess pore water pressure in the shearing process can be separated into two components: cyclic component and residual component. In a fully saturated sample, the cyclic component of excess pore water pressure is equal to the cyclic component of normal stress (= $\Delta \sigma(t) B_0$), and is affected only by the saturation degree. The residual component is decided by the properties of the soil material, for example, the initial density and the grain crushing susceptibility. It has no obvious relation with the applied dynamic stress pattern, and this makes it possible to compare the residual component of excess pore water pressure in the undrained condition to the grain crushing susceptibility.

Figure 11 shows the build-up process of the residual excess pore water pressure. The residual excess pore water pressure ratio ($r_{\omega}(t)$) is obtained by Eq. (9).

$$r_{\omega}(t) = \frac{(u(t) - u_0) - \Delta \sigma(t) B_0}{\sigma_0}$$  \hspace{1cm} (9)

where, $r_{\omega}(t)$ is the residual excess pore water pressure ratio; this has the maximal value of unity. The residual excess pore water pressure ratio is independent of the initial pore water pressure and the loaded normal stress, and can be used as a parameter to evaluate the liquefaction degree. $u(t)$ is the monitored excess pore water pressure, $u_0$ is the initial pore water pressure, $\Delta \sigma(t)$ is the applied normal stress increment, and $\sigma_0$ is the initial effective stress. $B_0$ is the pore-pressure coefficient in the direct shear state. It is shown that, although there is a possible measurement lag of pore water pressure between 0.2 mm and 3 mm, the increase trend of $r_{\omega}(t)$ with shear displacement is clearly observed. Especially, after 10 mm, it is clear that excess pore water pressure is generated with increasing shear displacement. When the shear displacement exceeds 1000 mm, the excess pore water pressure ratio becomes greater than 0.8. This means that the liquefaction phenomenon occurred with the progress of shear displacement. The relationship between shear displacement and liquefaction in the shear zone is further confirmed.

**GRAIN CRUSHING PROPERTY OF THE TESTED SAMPLE**

After the simulation test, the drained constant-speed ring-shear test was carried out on the same sample to investigate the grain crushing property of the tested sample.

Grain crushing is cited as one of the subjects of major importance in the soil mechanics field and is being investigated by many researchers, in connection with practical problems such as the possibility of erosion (Marsal, 1967), the bearing capability of piles and the stability of earth structures (Bishop, 1966; Lee and Farhoomand, 1967; Leung et al., 1996). It has mainly been studied with triaxial compression tests, one-dimensional compression tests, Gyratory tests and compaction tests (Fukumoto, 1992). Fukuoka (1991) used ring-shear apparatus to study the
grain-crushing property of crystallized schist in the sliding zone of the Zentoku landslide. According to a previous study (Lade et al., 1996), grain crushing is mainly affected by 1) the stress level (proximity to failure), the stress magnitude and the stress path, 2) grain size, 3) particle angularity, 4) uniform coefficient, 5) mineral hardness, and 6) introduction of water.

Concerning the relation of the volume change in the drained state and the excess pore water pressure in the undrained state, it has been found that, at the same initial density, samples generating larger contraction in drained shearing will always generate larger excess pore water pressure in the undrained shearing (Ishihara, 1993). The dilatancy property of samples matches well with the excess pore water pressure generation (Wang, 1998). Based on this background, the drained ring-shear test was performed to investigate the volume change and the grain crushing property of the tested sample. Under a normal stress of 196 kPa and shear speed of 3 mm/sec, the sample was sheared for 42 m in drained condition. Figure 12 is the sample height change during the shearing. During the shear displacement from the start to 10 mm, the change of the sample height is very small. From 10 mm, the sample height reduced with the shear displacement at a direct logarithm relation. Although volume reduction may occur without grain crushing, it will soon terminate within a small shear displacement, as proved by triaxial tests and shear box tests. The volume change presented in Fig. 12 seems to be greatly different from that; such a large volume reduction is unlikely to have been caused without grain crushing. Wang and Sassa (1998) conducted a series of tests to compare the grain crushing susceptibility of Toyoura standard sand, silica sand and Osaka-group coarse sandy soil taken from the Takarazuka Golf Club landslide, which was also induced by the January 17, 1995 Hyogoken-Nambu earthquake. It was found that the Osaka-group coarse sandy soil taken from the Takarazuka Golf Club landslide was easily crushable. Comparing the result in this study with those results, it can be seen that the Osaka-group coarse sandy soil taken from the Nikawa landslide is also highly susceptible to grain crushing.

In order to investigate the state of grain crushing in the shear zone, the sheared sample after the drained test was excavated and a cross section was exposed. Figure 13 is a photograph showing the cross section of the sample in the shear box. It is clear that a grain crushing zone was formed at the shear zone. The pins show the upper and lower boundary of the grain crushing zone. Then, the samples in the shear zone, and those at the upper part and lower part of the shear box were taken out, and grain size distribution analyses were conducted on them. Figure 14 is the result, compared with the original sample. The samples at the upper part and lower part have the same grain size distribution as the original one, while that in the shear zone is much finer. It is clear that grain crushing only took place in the shear zone, and the sample height change primarily resulted from this grain crushing. Based on this result, it is reasonable to conclude that

**Fig. 12.** Sample height change with the shear displacement of Osaka-group coarse sandy soil under the drained ring-shear test. Normal stress = 196 kPa, shear speed = 3 mm/sec. Initial relative density = 108.3 percent

**Fig. 13.** Photograph of the sample in test. A trench cut in the shear box. Pins show the upper and lower boundary of the grain crushing zone

**Fig. 14.** Grain size distribution analysis results for the original sample, sample at the shear zone, samples at the upper part and the lower part of the shear box after sheared for 42 m under 196 kPa normal stress, shear speed = 3 mm/sec
the built-up of excess pore water pressure resulted from the grain crushing in the shear zone, and the “sliding-surface liquefaction” phenomenon can be distinguished from the usual liquefaction.

CONCLUSION

The mechanism of the Nikawa landslide triggered by Hyogoken-Nambu earthquake was investigated using the undrained ring-shear apparatus. Earthquake loading modified from the real seismic record was loaded and the post-failure behavior of the tested sample was observed. Test results show that shear displacement started during the main shock, but it was very small. The major phenomenon representing the rapid motion of the long run-out landslide occurred after the main shock. Shear displacement increased rapidly, excess pore water pressure was built-up continually and shear resistance reduced to a very low value after the main shock. The grain crushing process was estimated from the sample height change during shearing and confirmed carefully by grain size distribution analysis of the soils taken from the shear zone. The variation of the residual excess pore water pressure ratio in the undrained dynamic loading shows a good correspondence with the sample height change in the drained shearing. The mechanism of this long run-out landslide can be deduced as: after the slope failed due to the peak stress in the main shock, shear displacement was generated along the shear zone. Because the Osaka-group coarse sandy soil is easily crushed, under the undrained condition, the grain crushing resulted in the built-up of excess pore water pressure. With the high excess pore pressure and the resulting low shear resistance, the long run-out landslide occurred.

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