SLOPE FAILURES AT YOKOWATASHI AND NAGAOKA COLLEGE OF TECHNOLOGY DUE TO THE 2004 NIIGATA-KEN CHUETSU EARTHQUAKE AND THEIR ANALYTICAL CONSIDERATIONS

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\textbf{ABSTRACT}

Numerous natural or artificially embanked slope failures were caused by the 2004 Niigata-ken Chuetsu Earthquake. Characteristics of large scale natural slope failures that occurred at folding hills, river terraces, etc., were classified from a geometric structural view point. It was noted that surface failures occurred at steep cliffs and landslides occurred at gently slanting dip slopes. The cause of a landslide in a dip slope at Yokowatashi, Ojija city was analytically investigated based on cyclic shear test results of the laminar sand at the bedding plane. This study revealed that the safety factor of the slope stability became momentarily smaller than 1.0 several times during the earthquake due to the small undrained shear strength of the tuff sand seam and the landslide behavior was simulated by an elastoplastic dynamic finite element analysis where the strength decrease of the sand seam with increasing number of cycles was taken into consideration. The cause of destructive damage of a school building at Nagaoka National College of Technology (NNT) was next examined by another elastoplastic dynamic FEM. From the analysis results, it was confirmed that the building was pulled and twisted by the landslide of the bank shotcrete accompanied with pile fracture.

\textbf{Key words:} bedding plane, cutting and filling ground, dip surface, earthquake, effective stress, finite element analysis, organic soil, shear strength, slope failure (IGC: D6/E6/E8)

\textbf{INTRODUCTION}

Landslides in natural slopes and slumping of embankments during earthquakes have become of relatively major consequence after the Chilean earthquake of 1960 and the Alaskan earthquake of 1964. Since then remarkable studies intensively started in such field, such as Seed (1966), Seed and Wilson (1967), etc. Newmark (1965) outlined methods for predicting maximum downhill movement during typical earthquake ground motion, and a lot of research has been conducted by others on earthquake-induced displacements of slopes as well as considerations to be taken into for deciding upon the safety of an earth dam during an earthquake. Ugaï (1987) reviewed briefly these studies and summarized equations to calculate displacement for various shapes of sliding surface. Japanese Geotechnical Society organized a research committee on the unstabilization mechanism and design method of a slope during an earthquake and published the committee report (1999) and the proceedings of symposium on the same topics (1999). Although various slope failure mechanisms were studied and design considerations were proposed mainly for artificially filled lands and soil structures such as earth dams and embankments, case studies of natural or artificial slope failure based on the intact soil properties are still not sufficient.

There was serious damage to many important secondary roads as well as destruction of residential areas and blockage of rivers due to landslides during the 2004 Niigata-ken Chuetsu Earthquake. In addition, several major transportation routes, such as the Joetsu line of East Japan Rail, Routes 17 and 117, were severely damaged due to the large scale collapse of river terrace along the Shinano River beside which most of these transportation lines run. Similar but more severe damage has continuously happened during earthquakes at inland mountainous areas in Japan as well, such as the 1848 Zenkoji earthquake (M7.4) (Akabane and Kitahara, 1989).
by the Niigata-ken Chuetsu Earthquake was the tremendous number of landslides which occurred at filling soil grounds for residential lands or transportation routes constructed by filling old valleys. As an example of this sliding type, the cause of severe damage to Building No. 3 at Nagaoka National College of Technology (NNCT) was studied here by an elasto-plastic effective stress dynamic FEM.

**GEOLOGICAL CONSIDERATION OF YOKOWATAKI SLOPE**

As seen in the geological map shown in Fig. 1, the Shinano River joins the Uono River at Kawaguchi town near the epicenter of the earthquake and flows northward from Shiroiwa, Nagaoka city after scraping the right coast at Yokowataki, Ojiya city. The river encroaches on the attacking rock slopes ranging from Shiroiwa to Myoken, Nagaoka city along the right coast. Neogene deposits, such as Shiroiwa layer, and Uonuma layer sedimented by early Diluvium distribute as hills and terraces from Kawaguchi town to Nagaoka city along the Shinano in this area. These layers fold several times forming the Higashiyama hill areas and the axis directions of these anticlines and synclines are almost South-North in direction. The Shinano locates to the west end of the anticline axis. The slopes inclining west and facing the river are dip slopes from a geological structure view point. Geological characteristics of typical landslides which occurred during the earthquake in the vicinity of Ojiya city were tabulated as shown in Table 1. According

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Table 1. Slope failure and geological structure along the Shinano River

<table>
<thead>
<tr>
<th>Failure type</th>
<th>Feature of slope</th>
<th>Geological structure</th>
<th>Location</th>
<th>Direction</th>
<th>No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Landslide</td>
<td>Slope of gentle slant</td>
<td>Dip slope</td>
<td>Shiroiwa*</td>
<td>West</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Yokowataki**</td>
<td>West</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Unoki**</td>
<td>West</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Shiodono**</td>
<td>West</td>
<td>4</td>
</tr>
<tr>
<td>Surface failure</td>
<td>Steep cliff of about 50° slant</td>
<td>Slope Perpendicular to layer strike</td>
<td>Nishikura</td>
<td>East</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Shiodono**</td>
<td>East</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>JH Echigo Kawaguchi SA</td>
<td>North</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Ushigashima***</td>
<td>South</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>West to Echigo Kawaguchi RS</td>
<td>South</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Nishikura***</td>
<td>South</td>
<td>10</td>
</tr>
</tbody>
</table>

*Nagaoka city, **Ojiya city, ***Kawaguchi town.

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2003) and the 1891 Nobi earthquake (M8.0) (Muramatsu et al., 2002; Sorimachi, 1978; Takahashi and Oyagi, 1986).

In this paper, the types of many natural slope failures which occurred along the Shinano River during the 2004 Niigata-ken Chuetsu Earthquake were examined first in order to study the cause of each failure. The cause of a dip slope failure was next investigated as an example of natural slope failure by means of a stability analysis based on the limit equilibrium method. An elasto-plastic dynamic finite element analysis was also applied to the slope failure for reproducing the long distance slide taking strain softening and decrease in strength due to cyclic shear loading into consideration.

The other remarkable feature of slope failure induced
STABILITY OF YOKOWATASHI SLOPE DURING EARTHQUAKE

Seismic Stability of Infinite Slopes

Figure 3 shows a schematic cross section of an infinite slope with water flow in the soil. Now let us imagine the sliced element of width $B$ and depth $Z$ on the imaginary sliding plane having an inclination of $\beta$ and parallel to the soil surface. The forces $E_1$, $E_2$ acting on the vertical sides of the element are assumed to exactly balance each other. When the depth of the water table is $z$ and the horizontal and vertical seismic intensity act to the mass of the soil and water are, $K_s$ and $K_v$ (downward; positive), respectively, the horizontal inertias are $H_i = K_s W_i$ and $H = K_v W$, and the vertical ones are $V_i = K_s W_i$ and $V = K_v W$ due to seismic motions, where $W_i = \gamma B Z$ and $W = \gamma B (Z - z)$. The resultant acceleration, $\alpha$, parallel to the equipotential line, pb, of seepage water flow is; $\alpha = (1 + K_v) \cos \beta - K_v \sin \beta / \cos \beta$. The water pressure, $u$, on the sliding plane is; $u = \gamma_z (1 + K_v) \cos \beta - K_v \sin \beta / \cos \beta$. The total water force, $U$, perpendicular to the sliding plane between b and c is therefore;

$$U = \gamma_z (Z - z) B ((1 + K_v) \cos \beta - K_v \sin \beta)$$

(1)

The effective normal force $N'$ and shear force $T$ on the sliding plane are indicated by Eqs. (2) and (3), respectively.

$$T = [(1 + K_v) \sin \beta + K_v \cos \beta] (W_i + W) B$$

(2)

$$N' = [(1 + K_v) \cos \beta - K_v \sin \beta] (\gamma Z + \gamma' (Z - z)) B$$

(3)

The effective normal stress, the shear stress and the factor of safety, $F$, are,

$$\sigma' = \frac{N'}{B} \cos \beta, \quad \tau = \frac{T}{B} \cos \beta$$

(4)
Features of Yokowatashi Landslide

Figure 2(a) is a photograph taken from a northwest direction. As seen at far end of this picture, a part of the upper Shiroiwa layer and the surface earth with high trees remain as they were on the bedding plane. The remaining upper Shiroiwa layer of soft silt rock exposes its side face. The other part of the upper Shiroiwa layer which made up the opposite side of the slid area is visible on site. The portion of the upper Shiroiwa layer between them had covered the planer tectonic dip surface which is clearly seen in the picture, and it has slid more than 72 m to the west toward the Shinano River. The inclination, $\beta$, of the bedding plane facing to almost west is slightly large, with a high of 22.4° at the relatively southern portion of the slid area and averaging approximately 22° overall. The thickness of the slid Shiroiwa block at the south end is about 4 m and those of earth on the block ranges from 20 cm to 1 m. The height of upper Shiroiwa layer remaining at the north side is about 2.5 m with earth cover of 60 cm thick near the ridge of the slope. Figure 4 is a close picture showing the border between the bedding plane and the exposed face of the remaining Shiroiwa layer at the south end of the slide area. Both upper and lower Shiroiwa layers were excavated and scraped slightly for intact soil sampling just after the slide. A thin seam layer of 5–10 mm thick was sandwiched between the upper and lower Shiroiwa layers. The material of the sand seam is tuff sand. Both Shiroiwa layers were gray, weathered, and changed their color to brown up to about 8 cm inside from the boundary of the sand seam. As is evident from the discoloration belts with a thin crevice in the center of each belt, there were a lot of joints in the upper Shiroiwa layer. Plant roots expanded in the joints and the bedding plane to get water as seen in this figure.

Shear Strength of the Sand Seam at the Bedding Plane

Figure 5 shows an intact sample consisting of the upper and lower Shiroiwa soft rocks and the tuff sand seam in between. The sample was subjected to a cyclic direct shear test under the constant volume condition. The sand portion of the sample was unsaturated with water. Although suction was not measured there, the normal stress observed in the test was considered to be equivalent to the effective one under the undrained shear condition, provided the capillary force can be negligible. Figure 6 shows the shear stress vs. displacement of block samples A and B during cyclic loading. As seen in the figure, the shear stress decreases with an increasing number of loading-unloading cycles for the same shear displacement. The strength parameters were determined from the peak strengths of virgin loading process for different consolidation pressures, as seen in Fig. 7. The average strength parameters of the sand concerning effective stress are that $c' = 35.3$ kN/m² and $\phi' = 17.2^\circ$. Although there is a large discrepancy in the peak values of $\tau$ for $\sigma = 120$ kN/m², those concerning total stress are $c = 23.8$ kN/m² and $\phi = 30.9^\circ$. The unconfined compression strength, $q_u$, of both Shiroiwa soft rock is 5.3 MN/m² and the unit weight is $y = 18.0$ kN/m³.

Safety Factor of the Slope during the Earthquake

The average thicknesses of the slid Shiroiwa layer and the earth cover were assumed to be equal to 3.5 m and 50 cm in conversion to the unit weight of Shiroiwa rock, respectively. As aforementioned, the thin tuff sand seam is considered to be filled with water because of the existence of plant's roots. Moreover, this district was suffered by heavy rain due to Typhoon No. 23 three days before the earthquake. The sand was thus saturated and
behaved in an undrained condition during the earthquake.

Figure 8 shows the time histories of acceleration in EW and NS directions observed by the Japan Meteorological Agency at Takezawa in Old Yamakoshi village. East and North directions are denoted by positive values. Figure 9 shows the time history of $F$ calculated by Eq. (5) using $K_s$ and $K_v$ values converted from Fig. 8. Here, $Z = 4$ m, $z = 3.99$ m and $\gamma_l = \gamma_{sm} = 18.0$ kN/m$^3$. According to Fig. 9, the value of $F$ before the event was considered to be 2.16. It fell below 1.0 first at 2.28 sec after the beginning of the event and several times by 10 sec. Although this time history of stability factor shows clear possibility of sliding, it is necessary for pursuing sliding amount to examine numerically by an elasto-plastic analysis method taking strain softening of the laminar sand into consideration.

**EVALUATION OF THE SLIDE BY AN ELASTO-PLASTIC DYNAMIC FEM**

**Analysis Model**

Since Yokowatashi landslide was a typical two dimensional failure and the cause of the long distance sliding was conjectured to be undrained strength reduction of the laminar sand due to seismic motion, it seems to be suitable to apply an undrained strength model with strain dependency in the two dimensional finite element analysis. The finite element mesh consisting of eight nodes per
each element were shown in Fig. 10. The upper and lower soft rock layers were assumed to be elastic material and the sandwiched tuff sand layer of 10 mm thick was assumed to be elasto-plastic material taking strain softening into consideration. The surface soil at the foot of the slope was assumed to be sand and gravel spreading down to the Shinano.

The basic concept of the elasto-plastic model used here is the same as the cyclic loading model originally proposed by Wakai and Ugai (2004). The undrained strength parameters, \(c\) and \(\phi\), which specify the upper asymptotic line of the hyperbolic skeleton curve of their model was modified as the decreasing function, Eq. (6), of accumulated maximum plastic strain, \(\gamma^p\), to incorporate the strain softening characteristics (Wakai et al., 2005) and used;

\[
\tau_i = \tau_{i0} + \frac{\tau_{i0} - \tau_{i0}}{A + \gamma^p} \gamma^p
\]  

Here, \((\tau_{i0}, \tau_{i})=\) (initial, residual) undrained shear strength and \(\tau_i/\tau_{i0}\) is called the residual strength ratio.

The shear modulus, \(G_o\), is also assumed to decrease in proportion to the decrease of shear strength. The cyclic loading model disregarding strain softening was used for the sand and gravel layer. The constants of Rayleigh damping were assumed to be basically \(\alpha=0.171\) and \(\beta=0.00174\) which are equivalent to a damping ratio of about 3% for a vibration period of 0.2 through 2.0 s. The material properties used in the analysis are summarized in Table 2.

![Diagram](image1)

Fig. 11. Examples of comparison between tested (left column) and simulated (right column) hysteresis loops

<table>
<thead>
<tr>
<th>Layer</th>
<th>Shiroiwa</th>
<th>Sand seam</th>
<th>Sand and gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus (E) (kN/m²)</td>
<td>100000</td>
<td>30000</td>
<td>30000</td>
</tr>
<tr>
<td>Poisson's ratio, (v)</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Cohesion (c) (kN/m²)</td>
<td>—</td>
<td>24</td>
<td>0</td>
</tr>
<tr>
<td>Internal friction angle, (\phi) (deg)</td>
<td>—</td>
<td>30.9</td>
<td>35</td>
</tr>
<tr>
<td>Dilatancy angle (\psi) (deg)</td>
<td>—</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>(b) (\gamma_{co})</td>
<td>—</td>
<td>5.0</td>
<td>0.85</td>
</tr>
<tr>
<td>(n)</td>
<td>—</td>
<td>1.5</td>
<td>5.0</td>
</tr>
<tr>
<td>Unit weight (\gamma) (kN/m³)</td>
<td>18</td>
<td>18</td>
<td>18</td>
</tr>
</tbody>
</table>

| Strain softening parameters | Residual strength ratio | — | 0 | — |
|—— | A | 1.8 | — | — |

Table 2. Input parameters

Figure 11 compares the simulated hysteresis loop and the tested loop of each specimen as mentioned before. The axis of abscissas is written in strain or displacement and 0.1 strains correspond 1.0 mm since the thickness of the sandwiched layer is 10 mm. Although they do not perfectly coincide, they are roughly similar to each other for the respective consolidation pressure.

**Analyzed Results and Discussions**

The acceleration record in EW direction observed at Takezawa was used in the analyses. Two cases of analysis were conducted to examine the influence of seismic intensity on the inducement of sliding. One was the analysis for which the observed acceleration record was input as it was at the base of the analysis area, and the other was the one for which the input acceleration amplitude was compressed to one half of the actual wave record. Figure 12 shows the time histories of horizontal displacement at the foot of the slope, namely Point A, in Fig. 10. As seen in this figure, the slope does not fail in the case of half acceleration amplitude. Contrarily the large-scale slope failure occurs in the case of actual acceleration amplitude. The sliding amount in horizontal direction is 1.4 m at \(t=30\) s and almost 28 m at \(t=50\) s. Since the shear strength became smaller than the shear stress induced only by the self weight of upper Shiroiwa layer, continu-
DESTRUCTIVE DAMAGE OF SCAOO BUILDING IN NNCT

Nagaoka National College of Technology has suffered from serious damage by the 2004 Niigata-ken Chuetsu Earthquake. Many damaged buildings and facilities located at the peripheral zone of the campus, were constructed by filling of soil. Since the school building No. 3 has been built on the site that consists of cutting and filling ground at the exit of old valley topography, the vibration of the ground was very complicated.

In order to examine such complicated vibration of the ground, a simulation model of a nonlinear effective stress dynamic finite element analysis was used. The present study aims to clarify the main cause of the split destruction of Building No. 3 through comparing the observation results and the analyzed ones.

SEISMIC DISASTER IN NNCT BY EARTHQUAKE

Outline of the Site

The campus of NNCT at Nishikatagai, Nagaoka-city is located at a corner of Mt. Yukyu near the west edge of the Higashiyama hills. The campus was originally a small hill with the summit; 75 m in elevation. The axis of the ridge locates almost Northwest-Southeast in compass as shown in Fig. 15. Several buildings, located along the southeast edge of the campus, are superimposed on the contour lines of original geography in this figure. The original stratigraphy was as follows; the volcanic ash cover, Oyama layer of Pleistocene deposit and Uonuma layer sedimented by early Diluvium, from the top to down.

In 1960, the northwestern area of the hill was improved and terraced: At the elevation (hereafter: EL) of 62.8 m for the athletic field, at EL = 65 m for the main campus, and at EL = 68.2 m for the level ground in the southeastern area. In Fig. 15, the gray and white color areas show the cutting ground and the filling ground respectively. In the first cutting and filling ground construction, the west slopes and valleys were filled with cut soil for the enlargement of each terrace in both directions; west and southwest.

The construction of the present campus shown in Fig. 16 had been completed in 1968 after the second filling construction for further enlargement of the site at each elevation. The figures shown in ellipse in Fig. 16 indicate the elevation at respective areas. The slopes of northeast and northwest have partially natural shapes of the original hill.

From the comparison between Figs. 15 and 16, we can see two deep and long hidden valleys, which must have been already filled many years ago. One is buried under the athletic field and the other is under the middle of the campus. Both of the old valleys open into the southwest edge of the present campus.

The filling ground spreads to the area where the original elevation is lower than 65 m. The boundary between the filling ground and the cutting ground lies in

Fig. 12. Time history of horizontal displacement of upper Shiroiwa layer

Fig. 13. Relationship between shear stress and shear strain of the sand layer during earthquake

Fig. 14. Slide at 50 sec after the beginning of the event

ous sliding on the bedding plane started at an elapsed time of about 11.5 s. Figure 13 shows the relationship between the mean shear stress and the mean accumulated shear strain of the sand seam, both of which were averaged in all over the sandwiched layer. The accumulated plastic strain converged at $\gamma = 2.5$ and $\tau = 38 \text{ kN/m}^2$ in the case of half acceleration amplitude. On the contrary, the shear stress decreases continuously with increasing $\gamma$ in the actual wave case. The accumulated displacement by 50 s after the beginning of the seismic motion was shown in Fig. 14. The long distance sliding of the upper Shiroiwa layer along the bedding plane can be seen discontinuously at the sand seam in this cross section. The large-scale slide occurred on site was thus reproduced quantitatively through the present analysis.
the following line; the southwest corner of Building No. 6—almost all of Computer center—the southeast edge of Library—the north end of Connection passage between Buildings No. 2 and No. 3—the northwest portion of Building No. 3—the northwest edge of Athletic club house—the northeast part of Second gymnasium—the southwest edge of Snow and Ice Laboratory Center. The connecting passage and the southwest half of Building No. 3 is located at the exit of the old valley which sweeps deeply off to the east direction. All buildings that are located on the filling ground are supported by bearing piles except for connecting passage and machinery factory. All buildings and facilities locating on the filling ground except Building No. 6 and Computer center displaced and/or inclined due to the earthquake, and were demolished afterward.

**Feature of Building No. 3 and the Ground**

Figure 17(a) shows the picture, taken from southwest direction, of Building No. 3 behind the parking lot for
bicycles. Gymnasium No. 2 is seen on the right. It should be noted that there is a difference of 5.2 m in ground elevation between the front and the back, both of which are filling ground. Figure 17(b) is a view from south direction, where the left hand side of Building No. 3 is of three-stories and the right hand side is two-stories. The two-story portion of Building No. 3 and athletic clubhouse, shown at the front, are located at the cutting ground, although the three-story portion of Building No. 3 is on the filling one.

**Damage of Building No. 3 due to the Earthquake**

Figure 18 is the plan of the damaged Building No. 3 and the displacement due to the earthquake. The three-story portion; 40 m long at northwest side and the two-story portion; 24 m long at the middle portion of the building located on the ground of EL = 65 m. The single story portion; 16 m long at southeast side is located on EL = 68.2 m for the convenience to connect to the second floor of the middle portion.

During the earthquake, the three-story portion turned anticlockwise, and the displacement of the northwest end
of the building reached 77 cm to southwest direction. The displacement of the ground reached 95 cm to southwest direction because of the crack, 18 cm in width, beside the great beam. At the same time, the subsidence of the ground reached 47 cm and 59 cm at the points b and c, respectively.

Figure 19 shows the footing, and two of the three-combined piles at the corner. They are reinforced concrete piles with a diameter of 30 cm. As shown clearly in the pictures, the movement of the piles is larger than that of the footing. The displacement of the piles toward southeast is under 20 cm, and that toward northwest is over 35 cm, which is larger than those of the footing. The pile heads were destroyed as long as 60 cm. From the result shown in Fig. 19, we can see that the ground has moved together with the piles.

Figure 20 is the picture of the column and the second floor at Point A, taken from the direction of the arrow shown in Fig. 18. The column was displaced 26 cm, resulting in an opening in the floor. Through the opening of the second floor, we can see a part of the column of the first floor. The floor is made up of double reinforced concrete with the thickness of 13 cm, and the rebar, 9 mm in diameter, with 17 cm in spacing.

SEISMIC RESPONSE ANALYSIS OF THE GROUND

Analysis Model

The analysis model shown in Fig. 21(a) was applied to the ground in the vicinity of Building No. 3. Figure 21(b) shows the FEM mesh configuration, where the viewing directions are different between (a) and (b). The gradient of the slope in the third quadrant is in the range from 25° to 28°, and the high step (between Building No. 3 and the bicycle parking lot) is retained by the stone block retaining wall.

The boring log at the point indicated in Fig. 21(a) is shown in Table 3, together with the input soil properties for FE analysis. There is the extremely weak organic soil layer whose N value is 0 just under the filling loam layer of about 8 m thick, having a N value of less than 6. This organic soil had covered the original hill of Oyama layer before the filling construction for campus extension, and it spreads to the border between the cutting and filling ground. The depth of water table is 3.5 m.

The analysis model shown in Fig. 21(a) is assumed to be a part of the infinite model, and all boundaries are surrounded by the special elements deformable only for
shear. In other words, the displacements in three directions of the all peripheral nodes were assumed to be equal to those of the corresponding nodes at one line inside by applying the Multi-Point Constraint function.

**Method of Analysis**

The configuration of soil and the shape of ground is three dimensional, and the behavior of the saturated organic soil play a key role. It is, therefore, reasonable for this particular case to apply a 3-D effective stress analysis. The dynamic analysis conducted here is the one that satisfies this condition, "HIPER" (Fukutake, 1997). The modified Ramberg-Osgood model extended to three dimensions was used for the stress-strain relationship, and the bowl model was used for the strain-dilatancy relationship (Fukutake and Matsuoka, 1993). The values of R-O model's parameters in Table 3 were assumed based on the existing $G/\gamma - \gamma$ and $h - \gamma$ relationships (Imazu and Fukutake, 1986). In the bowl model, the monotonously compressive component of dilatancy was expressed by the following cumulative shear strain, $G^*$, and the reversibly dilative component, namely the vibration component, was expressed by the following resultant shear strain, $\Gamma$:

$$G^* = \sum \Delta G^* = \sum \Delta \gamma_{xx} + \Delta \gamma_{yy} + \Delta \gamma_{zz} + \Delta \gamma_{xy} + \Delta (\varepsilon_x - \varepsilon_y)^2 + \Delta (\varepsilon_y - \varepsilon_z)^2 + \Delta (\varepsilon_z - \varepsilon_x)^2$$

$$\Gamma = \sqrt{\Delta \gamma_{xx}^2 + \Delta \gamma_{yy}^2 + \Delta \gamma_{zz}^2 + \Delta \gamma_{xy}^2 + \Delta (\varepsilon_x - \varepsilon_y)^2 + \Delta (\varepsilon_y - \varepsilon_z)^2 + \Delta (\varepsilon_z - \varepsilon_x)^2}$$

The dilatancy, $\varepsilon_i$, is accordingly expressed as Eq. (9).

$$\varepsilon_i = A \cdot i^n + \frac{G^*}{C + D \cdot G^*}$$

Applicability of this constitutive model for clay materials has already confirmed by Tamura et al. (2006). The values of $A$, $B$, $C$ and $D$ used in the present analysis are $-0.2, 1.4, 2.5$ and $40.0$, respectively, and the value of $C_i/(1 + e_0)$ is assumed to be 0.05 for all layers, where $C_i$ is the swelling index. These values were determined based on the study done by Fukutake (1997) and Tamura et al. (2006). The minimum value of the liquefaction resistance, $R$, namely $X_0$, is 0.05 for the organic soil and 0.1 for all other layers. The shear modulus for each layer is calculated from the $V_s$ value.

The seismic waves observed at the base rock of 100 m deep in the ground at Nagaoka KiK-net station (2004) in Nagaoka Institute of Snow and Ice Studies, which is just about 700 m far from NNCT, were used with the slight correction of the coordinate direction from compass, shown in Fig. 21(a). Figure 22 shows the horizontal accelerations being input as $E + F$ waves at the base of the analysis model. We can see the long period components due to surface waves at the latter half in this figure. The vertical acceleration was disregarded because of its negligible influence on the calculation results shown in the previous study (Fukutake et al., 2005). The major axis of the input acceleration almost coincides with $y$-direction, and the maximum acceleration values in $x$- and $y$-components are 313 Gal and 477 Gal, respectively. The predominant frequencies of $x$-direction are 0.5 Hz and 2.0 Hz, and those of $y$-direction are 0.6 Hz and 2.0 Hz.

**Analysis Results and Discussions**

Figure 23 shows the results of the modal analysis. The displacements both in the $x$-direction and the $y$-direction, at the slope in the third quadrant are remarkable in the first and the second inherent modes respectively. Figure 24 shows the distribution of the maximum accelerations in the whole duration. The $y$-components almost coincide with the major axis, and they are larger than the $x$-components. The largest acceleration was 720 Gal, which is observed along the top of the high step of Oyama layer between Building No. 3 and the tennis court in the fourth quadrant. We can see that the maximum accelerations of the filling ground (on the old surface layer of weak organic soil) are relatively small in comparison.
with those of the cutting ground. This means that the old surface layer played the role of an isolation layer. Figure 25 is the panel diagram, which shows the distribution of the maximum pore water pressure ratio, $r_s$. The largest $r_s$ value exceeds 70% in the old surface layer. The $r_s$ is larger than 50% at the bottom of the filling loam and at the top of Oyama layer between which the old surface layer is sandwiched. Figure 26 shows the distribution of the maximum resultant shear strain, $\gamma$. The largest value of $\gamma$ reached as large as 15% in the old surface layer, which indicates that the landslide of this layer has occurred.

Figure 27 shows the deformation of the ground surface at the end of vibration, $t = 15$ s. The rectangle in this figure shows the three- and two-story portions of Building No. 3. The left two third of it locates on the filling ground, whereas the right one third stands on the cutting one. As shown in this figure, although the deformation of the cutting ground is small, the filling ground moved remarkably to both left and lower directions. This indicates that the filling ground slid together with the piles, and twisted the three-story portion of the building anticlockwise. During this movement, the pile heads were broken as shown in Figs. 19(b) and (c). The maximum analyzed displacement of the ground at the origin of the coordinates is 11 cm, whereas the actual movement was more than 95 cm. This discrepancy will be attributed to the following imperfection in the analysis; 1) The present analysis can not evaluate the magnitude of the cracks in the ground. 2) All the layers on the water table are united as a continuum. 3) In the FEM analysis, discontinuous displacement like a sliding failure is difficult to treat.

Figures 28 and 29 show the time histories of ground surface displacement and the orbit of horizontal displacement at the origin of the coordinates, respectively. As is clear from these figures, the ground at the northwest corner of Building No. 3 moved toward the $y$-direction mainly, namely downstream of the buried valley, with subsidence.

Figure 30 shows the stress-strain relationships in the old surface layer just below the origin of the coordinates.
Both shear strains, especially $\gamma_{yz}$, increased in the same direction as the initial shear having acted before the earthquake.

Figure 31 shows the time history of the excess pore water pressure ratio in the old surface layer just below the origin of the coordinates. It rapidly reached 60% by 4.5 s, namely soon after the beginning of the seismic motion and prior to the following several large shocks.

CONCLUSIONS

The types of many slope failures occurred along the Shinano River during the 2004 Niigata-ken Chuetsu Earthquake were classified from a geometric structure view point. The cause of the slope failure at Yokowatachi, Ojiya city was examined based on the cyclic shear properties of the sandwiched material on the bedding plane and its long distance sliding was reproduced analytically. Destructive damage of Building No. 3 at Nagaoaka National College of Technology was next investigated by an elastoplastic effective stress dynamic FEM. The following conclusions were obtained from the present work;

1) Surface failures occurred at steep cliffs and landslides occurred mainly at dip slopes of gentle slant.
2) Regarding Yokowatachi slope failure;
   i) There exists a tuff sand seam at the bedding plane of Yokowatachi dip slope structure and the safety factor of the slope stability was confirmed to become smaller than 1.0 at the seam several times during the earthquake.
   ii) The shear strength and stiffness of the sand decrease markedly with increasing number of cycles and the sand seam finally lose its shear strength enough to support just the self weight of soils above it.
   iii) The long sliding distance of the slope failure was reproduced through the elastoplastic dynamic finite element analysis taking the cyclic shear properties of the sand into consideration.
3) Regarding causes of destruction of Building No. 3 at NNCT;
A landslide occurred at the filling ground where one half of the building was located. No sliding occurred at the neighboring cutting ground where the other half of the building was located. The sliding occurred in the old surface layer of weak organic soil, which had covered the original hill before filling construction on it. The filling soil slid with piles, and the piles pulled and rotated the building. As a result, the disconnection of the building occurred at the border between the cutting and filling ground.

According to the analysis, the sliding occurred approximately 4 s after the beginning of the seismic wave recording. The horizontal accelerations at the ground surface are the double of those at the base rock.

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