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Sliding Mechanism of Okimi Landslide and Analysis of Overall Blocks

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ABSTRACT:

The adverse effect of landslide to any nations, how advanced is she and her human society, is well understood from the past. However, the proper way for the countermeasure planning is still in the developing stage. There always appear many uncertainties, balanced by large value of factor of safety. In this regard, some geotechnical professionals derived various methods for the stability analysis, ranging from simple Swedish circle method to three dimensional method. These can be utilized to analyze the critical potential slope but can not directly be applied on the landslide area and back analysis method is continuously used since long time ago, in the past. Back analysis is simple method in the absence of soil test data on the sliding surface soil. Testing of the sliding surface soil is rare in these days, as very few drainage wells penetrate the sliding surface. Hence, back analysis method always assumes shear intercept (c), using thumb rule to derive the internal friction angle (ϕ). Until now, even the developed countries like Japan are applying the value of c with respect to depth of sliding surface and calculating the value of ϕ for whole landslide. The ground water level in the crosssection is also fixed from the monitoring data of very limited number of piezometers. In this condition, ground water position will also be uncertain parameter and should be assessed. In the developing countries, the budget is not sufficient for plenty of check borings and drainage wells are almost impossible. The balance in the stability of the landslide is always tried to maintain by retaining walls, due to the availability of cheap manpower and materials. Especially, in the case of repetitive rotational slides, residual shear strength plays an important role, rather than the peak one. Recent researches show that the residual shear strength is governed by the parallel most orientation of the clay minerals of the soil rather than the position, seepage condition, over consolidation pressure and so on of the soil. This made the analyzer easy to relate the shear strength of main scarp soil with the sliding surface soil, as both should have similar mineral composition. Present research is conducted to find the relationship between the shear strength of sliding surface and the main scarp soil.

Okimi landslide, situating in Maki village, Niigata prefecture of Japan is also suffering from the aforesaid problems. The landslide, extending over 70 ha area was first slided more than 100 years ago and is still moving inspite of the heavy countermeasures, applied after the extensive monitoring work since more than 30 years ago. The countermeasures in some blocks, were planned according to the calculations by back analysis method. Due to the lack of drainage wells constructed in recent years, the stability of individual blocks were checked, using tested c and ϕ of the main scarp soil. The result shows the lowermost block, bearing the mass of whole upper blocks is unstable. Inspite of heavy counter measures, the upper blocks are found to be moving with the speed of 0.2 to 0.7 m/year, according to the monitoring of the moving posts. This report deals with the analysis of whole landslide, using tested c and ϕ.

Keywords : landslide, investigation, prevention, residual shear strength, stability analysis, factor of safety.

1. Introduction

Inspite of the huge effort on investigation, expertise, research and countermeasure construction, Japan is not still getting rid of the landslides, that are occurring in various parts of the country. Lack of sufficient data for perfect planning always brings many uncertainties in landslide planning. Japan spent huge amount of budget for the investigation and countermeasure planning of the landslides. If we analyze the results obtained by them, only few landslides are fully countermeasured and many of them are still moving inspite of huge cost involvement for the countermeasures. Back analysis method is popularly used in these days for the analysis of landslide stability and to plan the appropriate countermeasures. This method roughly estimates the value of angle of internal friction (ϕ) with assigned shear intercept (c), using thumb rule. Although it was sustainable method in various conditions, it would obviously be better if both the values of c and ϕ could be assigned after the shear test. Recent developments on the concept of residual shear strength of the soil by many professionals, necessiated the utilization of residual shear strength in the analysis of long run repetitive rotational slides. Bromhead (1998) clearly mentioned the dependency of residual shear strength with the mineral composition, and not on the over consolidation stage of the soil mass. He also stated on possible unalteration of residual shear strength of soil either it is undisturbed or remoulded. This concept gave new idea to the landslide analyzer for measuring the residual shear strength of main scarp soil, in place of the undisturbed sliding surface soil. In fact, sliding surface soil is very difficult to obtain these days, as the drainage well hardly penetrates the sliding surface.

Okimi landslide is one of the chronic landslides in Japan and the investigation work is continuing there since about 30 years ago. Inspite of the application of various countermeasures, the landslide is still moving. In the past, countermeasures were planned according to the stability analysis made by the back analysis method, and were applied accordingly. However, the landslide is still moving inspite of extensive drainage network. Therefore, this study aims to analyze the shear strength characteristics of the soil from the main scarps of various landslide blocks of Okimi landslide and utilize these results for the stability analysis, using both c and ϕ from the tested data. Various researches were done to fix the testing criteria, first, before the finalization of shear test by ring shear test apparatus. The reason for the continuos sliding was found out and concept of possible counter measure planning was developed to apply in future as explained below.

2. Objective of present study

The main objective of the present study is to set the soil testing criteria for the main scarp soil in Okimi landslide and conduct necessary soil tests to utilize the tested data for stability analysis. After the soil test, the stability condition of various blocks of Okimi landslide will be analyzed. After reviewing the existing plan, recommendations will be made on the future activities to be conducted, in order to minimize the landslide movement.

3. Methods and procedure

To achieve the above mentioned objectives, following methods has been adopted in all steps i.e. during the data collection at field level, its preservation and during laboratory testing.

1. Data sampling

Data collection and soil sampling in the field were done carefully, according to the following steps.

1. Collection of the main report, displacement history, movement pattern, topographical map and crosssections of the landslide area from the concerned authorities: The main report of Okimi landslide was collected from the KOWA Consultant and other information were collected from Yasuzuka Civil Engineering Branch of Niigata Prefectural Office.
2. Field observation of the landslide blocks and sliding pattern. Consequently, the identification of the main scarp location from where, the surface soil will be collected.
3. Collection of sufficient soil from the main scarp of the landslide area: The organic materials are cleared off first from the scarp and fine particles are collected as far as possible, with the shovel and put in the sampling bag. The location and date were recorded well and preserved.
4. As no drainage wells are constructed these days, small quantity of boring core sample from the sliding surface soil was collected during vertical check boring.

2. Laboratory testing

After bringing the various samples in the laboratory, each samples have been treated separately as described below.

-The natural water content was measured immediately after reaching the laboratory.

-The sample was dried on natural temperature for several weeks until it reached the complete dry level.

-After complete drying, 1 Kg of the sample was sieved to get the grain size distribution. Finer particles were tested with the hydrometric analysis.

-Average specific gravity of the soil grains were measured.

-The remaining soils were sieved through a.) 2mm. and on the second step b) 425μ sized sieve. The finer particles were collected in separate tray.

-Atterberg's Limit Tests i.e. liquid limit and plastic limit tests were performed on the sample.

-While starting the ring shear test for the first time for few samples, first, dry soils were poured and compacted to specified level and then, water was filled up into the water jacket, which was sucked through the ring opening.

-In the further tests, the samples were mixed up with water slightly less than liquid limit and each readymade samples were tested by simple shearing apparatus to get the shear strength of the tested soil. A series of tests were performed for the normal stresses of 50, 100, 150, 200, 250, 300 kPa. Before each tests, the samples were overconsolidated with the normal pressure of 300 kPa. After each tests, the samples were dried and the dry densities after the tests, were measured.

-Similarly, the samples were prepared according to the aforesaid two methods, and were tested by ring shear apparatus, applying the normal stress of 50, 100, 150, 200 kPa, separately. In all the steps, the samples were overconsolidated with the normal stress of 200 kPa, the capacity of the machine.

-The samples were dried and weighed to get the dry density after the test.

-The data were plotted and compared with various parameters.

-Likewise, multistage ring shear tests were conducted for some samples, both, with increasing loading and decreasing loading trend. In increasing load test, the sample was, first, overconsolidated with the normal stress of 200 kPa and the normal stress was decreased to negligible value, i.e. 0.3 kPa, after steady state of consolidation was reached. Once the residual shear strength was reached, the normal load was increased and residual shear value was noted. This process was repeated gradually, up to the normal load of 200 kPa. In the decreasing load test, the sample was first consolidated, with the normal stress of 200 kPa and residual shear stress was noted for that normal stress. Then the normal stress was gradually decreased to negligible value (0.3 kPa) and residual shear strength was measured accordingly. The graph obtained from normal stress vs. shear stress gave the residual shear envelop.

- The sliding surface soil was remoulded, first and tested with the aforesaid methods.

4. Location and background of landslide

Okimi landslide is situated in Maki village of Niigata prefecture, 14 km west from Joetsu City (figure 1). Small portion of the landslide area was moved several hundred years ago. The major movement was observed on 18th March 1719, triggered by a great earthquake that destroyed about 20 houses. However, during the earthquake of 1876, the landslide did not show any movement. The landslide area then expanded in 1896, because of the heavy precipita-

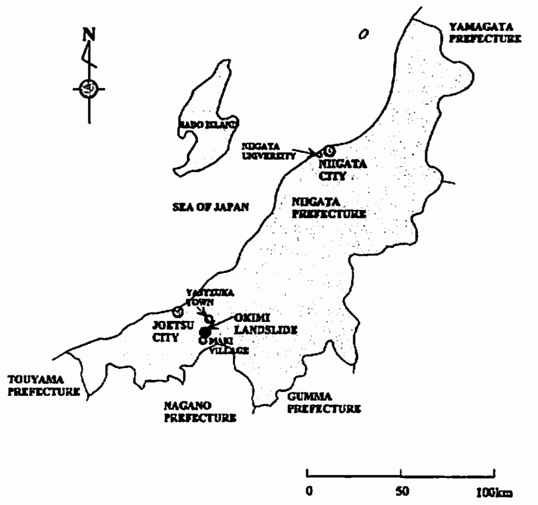


Figure 1 : Location map of study area

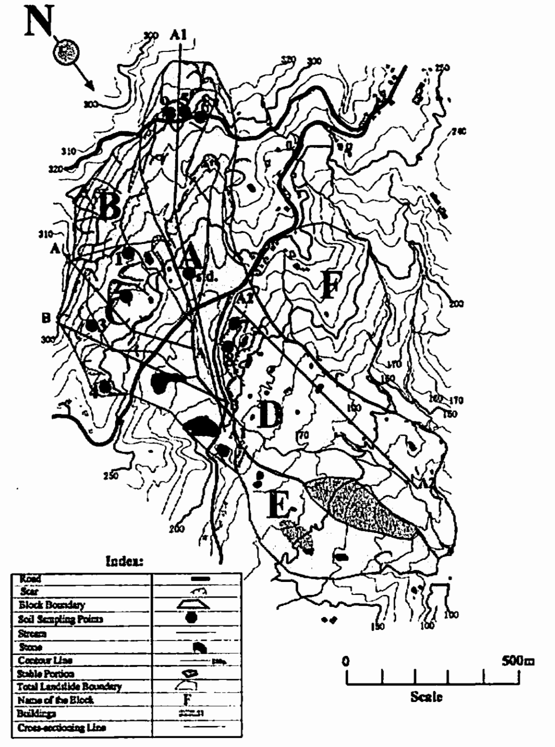


Figure 2 : Sampling points and cross-section on the topographical map

tion. After the earthquake of 23rd July 1905 , which was focused at Yasuzuka Town, several springs were observed on the landslide surface, and it seemed to be stable for about 4 years. The landslide was again moved during the earthquake of 21st November 1911. After the construction of sabo dam at the downstream river in 1932, the displacement area was minimized. After the continuous rainfall during 1944 and 1945, additional 10 ha area slided to make the area of 70 ha. Then, the relationship of the landslide movement to the under cutting of rivers was started to investigate since 1948 until 1952. However, much more detailed investigation began from 1970. Although several years have been passed since the start of investigation and periodic preventive measures have been applied, the landslide is still moving. According to the previous studies, topographical variation and movement data, total landslide area is divided into 6 major blocks, namely, A, B, C, D, E and F. Block A and C incorporates the total damaged road lengths whereas B is resting on A and C. Lower part below the road includes D, E and F block. Small portions between D and E block are found to be stable. The average width of the landslide area is about 500m whereas the total length is 1500m. It is extended to an altitude difference of 220m. Likewise, length and width of A, B, C, D and E blocks are about 900 x 200m, 300 x 200m, 400 x 300m, 600 x 250m, 500 x 300m and 310 x 200m respectively. F block is not moving now, but has a possibility to move in future.

Photo 1 : Overall View of landslide area

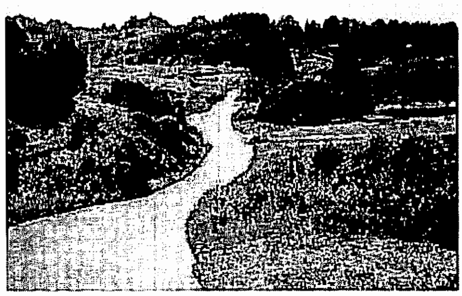


Photo 2 : Cracks on the road

5. Landslide Investigation System

The landslide area is under investigation since 1970 and counter measures after revised design are applied every year. The priority of the investigation was fixed to be A, D, C, B and E blocks respectively. Fifteen check borings were drilled in A block in 1970 and one drainage well was constructed for emergency countermeasure. Eight more check borings and 4 drainage wells were made in A block during 1971. The investigation was still concentrated on A block during 1972 and 8 additional check borings were done in addition to 4 drainage wells. 15 check borings were done at the central part of A-block and a drainage well was constructed in 1973 after analysis of the data from previous boring points. During 1974 and 1975, the research was concentrated on D block. 7 check borings were drilled whereas 2 more drainage wells were constructed at the lower part of A-block during these years. The investigation was further intensified from 1979 after a small break of some years. After knowing the movement of C-block, 7 check borings, throughout the block and a drainage well were constructed until 1980. 3 more check borings and an additional drainage well were constructed during 1981 and 1982. 4 more confirmative borings and 3 drainage wells were made in C-block during 1983 and 1984. 2 more confirmative borings were also made near the toe of A block at that time. During 1985 and 1986, 3 check borings were made to confirm the border of A and C block.

4 horizontal drainage borings and one drainage well were constructed in C-block at that time. 5 more confirmative borings at A block boundary and the toe as well as C-block boundary were made in 1987 and 1988. One drainage well at the head and horizontal drainage boring at the side were made at A-block for prevention work. The investigation at B-block was intensified in 1989. 4 check borings were done in addition to the horizontal drainage boring in one location. 2 more check borings were made on A-block in that year. 2 additional borings at A-block and one at C-block were done in 1990. 4 drainage borings were done at C-block and one drainage boring in addition to one drainage well were made in B-block at the that time. Two more check borings at A-block and one at C-block were done in 1991. 4 check borings with one horizontal drainage boring were made at B-block in 1992. Data from all the aforesaid borings were utilized to identify the landslide blocks and the sliding zones. With these data, surface drainage works were planned in some main streams and A-stream was covered up with gabion/PVC surface drainage works. Referring the topographical map and other investigation data, the landslide area has been equipped with numbers of moving posts and several subsurface observation equipment at suspicious locations. The area is equipped with 13 numbers of automatic and semiautomatic piezometers, 3 nos. of strain gauges, one inclinometer and 5 nos. of semiautomatic surface drainage discharge measurement unit. Monitoring has been done regularly. The rainfall data has been compared with the movement data. To measure the surface movement as well as to ease the block division, 7, 9 and 1 rows of moving peg network (total 170 nos.) were established on 1987, 1992 and afterwards respectively and have been surveyed regularly. The monitoring data has been compared with various parameters like precipitation, ground water table, displacement and so on. The rainfall data has been tapped from the near by station.

6. Analysis of the monitoring data

The landslide area is being monitored continuously. According to the monitoring data obtained from the moving posts, 6 clear blocks can be identified. Clear settlement on the road along the block boundary can be noticed. According to the data obtained from the monitoring of moving posts, the average displacement of A, B, C, D and E block in 1998 were less than 0.3m, 0.3m, 0.2m, 1.5m, 1.7m, respectively whereas average displacement per year in last 6 years were 0.7m, 0.5m, 0.7m, 1.7m and 1.5m respectively. If the movement pattern of the moving pegs will be carefully observed and tallied with the road deformation, the previously assumed A block and C block should be revised. C-block is residing on A1 (renamed) block and A1 is residing in A block. In 1998, ground water level was at minimum depth i.e. about 0.9m during early June at A-block. The ground water was nearly at the similar depth during first week of April in C-block. However, considerably low ground water level was observed even during rainy season. This shows high contribution of snow melts. The displacement is supposed to be prominent at about 10-11 m. depth in A block.

The bore logs from various check borings revealed the under ground condition of the landslide area. According to the assumed slip zone and available data on cross section and ground water condition, stability analysis was done to prepare countermeasure plan against the landslide. With the planned factor of safety of more than 1.15, the massive surface and subsurface drainage works as well as drainage wells were designed and implemented. Present study is targeted to check the stability of the whole landslide area with the laboratory tested data of c and ϕ. First, the stability analysis calculations, used in the past were checked with the laboratory test results. Stability analysis of C-block was done in 1979 and countermeasures were designed accordingly. It is said that the fact "stability of landslide is dependent on ground water condition" was first noticed in Japan from this landslide. The analysis was done with the arbitrarily assigned value of c i.e. 10 kPa, assuming the sliding surface at about 10m depth. Back analysis was done with safety factor of 1 to get the value of ϕ that was utilized for further countermeasure planning, afterwards. Those values were totally replaced by the laboratory test values to confirm the applicability of the procedure during our research work. Once it was confirmed, same process was applied for other untested blocks too. The soil test results are explained in the respective chapters below.

Soil samples were collected from various scarps of different blocks for the soil testing. 3, 4 and 2 sample points were selected at A, C and D block respectively. B-block and E block were not considered as these blocks were not analyzed in the past and are dependent fully on other blocks. Likewise, small quantity of the sample was also collected from the sliding surface of A block during check boring. It is really difficult to get sufficient undisturbed sliding surface soil from this landslide area as there are no plannings for drainage wells. Therefore, it was necessary to correlate the values of shear strength by ring shear and simple shear device. Besides, it was necessary to find the appropriate particle size of the sample for the test as well as, the consolidation ratio to get optimum efficiency. Setting of testing condition was supposed to be the most important for the future test programmes too. Hence, after getting the results by sieve and hydrometer analysis, consistency limit and water content, the soil was first sieved through 2 mm sieve and the finer ones were measured to make dry density of 1.7. Then it was poured into the apparatus and compacted to marked level (4.5 cm. thickness). The mass was made saturated by supplying the tap water through the water jacket and consolidated with the maximum normal stress of 200 kPa in all the cases. Then the samples were tested with the planned normal stresses, separately. Again, the sample, prepared through the similar method was tested by the simple shear apparatus, to verify the result. After that the sample passing through 425μ sieve was tested by ring shear apparatus, in similar condition, as done for 2mm down particles. However, the consolidation ratio of 4 was maintained in each test. After that, the soil sample passing through 425μ sieve was premixed with water weighing slightly less than the liquid limit, feed into the apparatus and consolidated. Then the multistage ring shear test was conducted, both for increasing load and decreasing load cases. At the same time, the x-ray diffractions of the soil samples were also done to identify the constituent minerals. Likewise, specific gravity test was also done.

7. Data Sampling and testing procedure

7.1 Data Sampling

As already mentioned above, utmost care was taken in data sampling procedure. As the main target of the study was to test the residual shear strength of the soil, collection of the undisturbed samples were not thought necessary, due to its unavailability. For the data sampling of main scarp soil of Okimi landslide, whole landslide area was reviewed first, with the displacement data and site survey. As C-block seemed to be less complicated and small, 4 scarp points (sample no. 1, 2, 3 and 4) were selected in C-blocks for the data sampling. Those points are almost equidistanced throughout the C-block. The sampling was done as explained in above articles. Almost 20 kg samples were collected from each point, after excluding the top organic layer carefully. Similarly, samples were collected from 3 points (sample no. 5. 6 and 9), at the topmost main scarp of A-block. About 2 kg soil sample was also collected from the sliding surface of the A-block during the check boring of bore hole 10-3. These samples were carefully transported to the laboratory after putting into the sampling bag. The fast moving, D-block, is also very important as it is the lowermost block of the whole landslide. It was necessary to check the relationship between A and D block. Hence, two samples i.e. sample no. 7 and 8 were collected from the main scarps of D-block too for the soil test.

7.2 Laboratory testing of the soil

After transporting the samples carefully to the laboratory, the water content was measured immediately and rest of the samples were dried naturally for 2-4 weeks. Then, sieve and hydrometer analysis, specific gravity test, consistency tests, x-ray diffraction test, simple shear test and ring shear tests were conducted on all the samples.

8. Test Findings

8.1 Water Content tests

Water contents were tested at various stages of the test. Immediately after the sample reached the laboratory, the natural water content (w_η) was measured. Then, water contents were measured during all, ring shear and simple shear test, stages as far as possible. For initial samples, water content of some tests were not measured. The water contents at various stages are described herewith separately.

Sample No. 1: For sample no. 1, the water content (w.c.) was measured during natural condition only. As it was the first test, w.c. was, unfortunately, not measured after the test. The natural w.c. was 34.2%. It is to be noted here that sample 1, 2, 3 and 4 were collected on September 1, 1998 and sample no. 1 and 3 were not collected and tested afterwards.

Sample No. 2: The natural w.c. of sample no. 2 on September 1, 1998 was 26.4% which was increased to 30.7% on October 6, 1998. During the individual ring shear test on 2mm down soil, with maximum overconsolidation pressure of 200 kPa, the w.c. after each tests for the normal stress of 50 kPa, 100 kPa, 150 kPa and 200 kPa were 43.5%, 41.2%, 36.8% and 40.5% respectively. During the ring shear test on 425μ down soil with over consolidation ratio of 4, the water contents after the test for the normal stress of 12.5, 25, 37.5 and 50 kPa were 41.4%, 42.2%, 45.4% and 47% respectively. During the multistage ring shear test, the water content during decreasing load, increasing load and one cycle loading were 40.3%, 34% and 27.8% respectively.

Sample No. 3: The natural w.c. of sample no. 3 on September 1, 1998 was 29.2%. During the individual ring shear test on 2mm down soil, with maximum overconsolidation pressure of 200 kPa, the w.c., after each individual tests for the normal stress of 50 kPa, 100 kPa, 150 kPa and 200 kPa were 45.2%, 39.4%, 35.9% and 35.1% respectively. This point was not resampled on October 6, 1998.

Sample No. 4: The natural w.c. of sample no. 4 on September 1, 1998 was 32.8%, which was decreased to 31.3% on October 6, 1998. During the individual ring shear test on 2mm down soil with maximum overconsolidation pressure of 200 kPa, the w.c., after each tests for the normal stress of 150 kPa and 200 kPa were 44.5% and 38.05% respectively. During the ring shear test on 425μ down soil with overconsolidation ratio of 4, the water content after the test with the normal stress of 12.5 kPa, 25 kPa and 50 kPa were 44.1%, 46.2% and 47.1% respectively. During the multistage ring shear tests, the water content during increasing load and one cycle loading were 40.1% and 28.3% respectively.

Sample No. 5: Sample No.5 was collected from landslide A block, on September 4, 1999. The natural water content was 32.2%. This sample was tested with multistage ring shear test only and the final water content after the test was 31.1%.

Sample No. 7: Sample No.7 was collected from D block, on September 4, 1999. The natural water content was 29.2%. This sample was not tested as sample 8 represented the strength.

Sample No. 8: Sample No.8 was also collected from D block, on September 4, 1999. The natural water content was 32.2%. This sample was tested with multistage ring shear test only and the final water content after the test was 29.2.

Sample No. 9: Sample No.9 was collected from the main scarp of A block, on September 4, 1999. The natural water content was 27.1%. This sample was not tested with ring shear test as sample no.5 could represent the strength of soil.

Sample from sliding surface of A-block: About 2 kg of the sliding surface soil was obtained after check boring at bore hole no. 10-3 of A-block. That sample was re-moulded and tested by the ring shear test machine. The water content after the multi-stage ring shear test was 46.2%.

8.2 Consistency tests

Consistency limit of the soil samples from C-block of Okimi landslide did not differ much. The liquid limit of sample no. 1, 2, 3 and 4 were measured to be 54%, 58%, 56% and 54% respectively (Table 1). Likewise, the plastic limit of those samples were 46%, 40%, 45% and 44% respectively. For the sample of A-block i.e. sample no. 5, the liquid limit and plastic limit were 45% and 34% respectively. For D-block, sample no. 8 was tested. The measured liquid and plastic limits were 48% and 32% respectively. The sliding surface soil, revealed by boring, was remoulded and consistency tests were performed. The liquid limit and plastic limit of that soil was measured to be 84% and 30% respectively.

Table 1 : Consistency Limit of the samples

Sample	LL (%)	PL(%)	PI (%)
Okimi 1	54	46	8
Okimi 2	58	40	18
Okimi 3	56	45	11
Okimi 4	54	44	10
Okimi 5	45	34	11
Okimi 8	48	32	16
Okimi, sliding surface	84	30	54

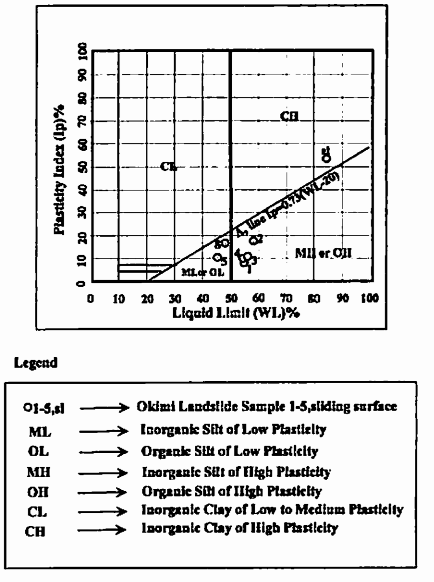


Figure 3 : Plasticity Chart of all samples

8.3 Grain Size Analysis

Grain size analysis was done for few representative soil samples after complete drying. First, 1 kg sample was sieved by consecutive set of sieves and hydrometer analysis was done for only small portion of the sieved soil samples. That might have measured the clay composition, in lower range than reality. Sieve analysis was done for the sample no. 2 and 4 only. According to the result, the percentage of clay, silt, sand and gravel at sample no. 2 of Okimi landslide were 4%, 7%, 69% and 20% respectively (Table 2). Likewise those for sample no. 4 were 8%, 9%, 50% and 33% respectively.

Table 2 : Soil Type by grain size

Sample	% of clay	% of silt	% of sand	% of gravel
Okimi 2	4	7	69	20
Okimi 4	8	9	50	33

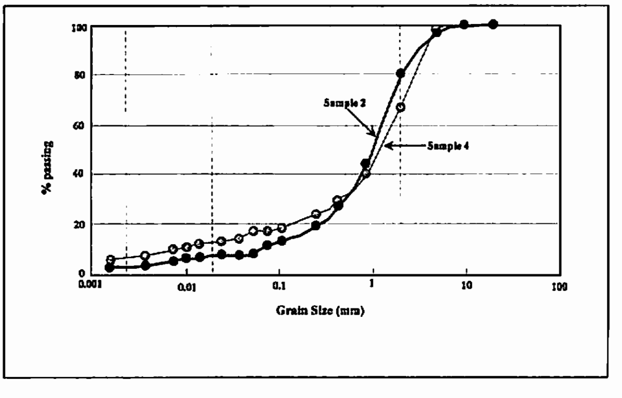


Figure 4 : Grain size Distribution Curve

8.4 X-ray Diffraction test

Representative soil samples were powdered well to pass through the x-ray. According to the diffracted result, all the soil samples at Okimi landslide, were mainly, composed of quartz, feldspar, montmorrilonite and kaolinite although their proportions were different. Soil sample no. 2 at Okimi landslide was composed of 60% quartz, 22% feldspar, 3 % kaolinite and 15% montmorrilonite (Table 3). Likewise, percentage of quartz, feldspar, montmorrilonite and kaolinite in sample no. 3 were 59%, 23%, 15% and 3% respectively. At sample no. 4, these proportions were 54%, 26%, 15% and 5% respectively. The sliding surface soil at Okimi landslide was composed of 65%, 18%, 13% and 4% of the aforesaid minerals respectively.

Table 3 : Mineral composition by x-ray diffraction

Sample	Quartz	Feldspar	kaolinite	Montmorrilonite
Okimi 2	60	22	3	15
Okimi 3	59	23	3	15
Okimi 4	54	26	5	15
Okimi, sliding surface	65	18	4	13

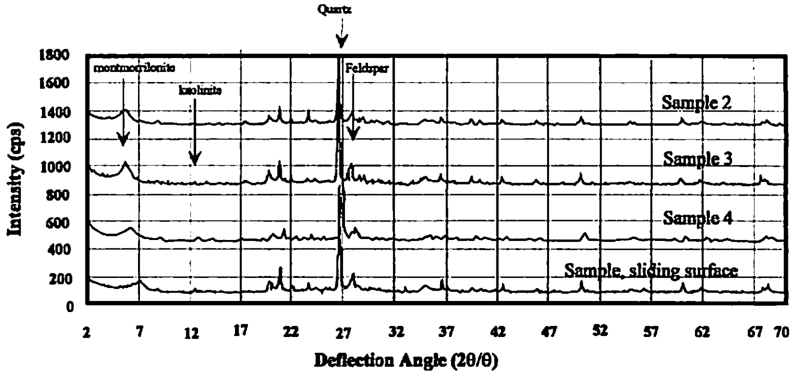


Figure 5 : x-ray diffraction result

8.5 Simple Shear Test

All samples were not tested by simple shear apparatus, as the main objective of shear testing, in this study, was to compare the value of residual ϕ after the test with that by the ring shear test. Therefore, the 2mm down samples, only from sample no. 1, 2 and 4, collected on September 1, 1998, were tested by simple shear apparatus. According to the test result (Table no. 4), peak and residual ϕ of sample no. 1 were 17° and 15° respectively. The value of peak and residual c were 29 kPa and 20

Table 4 : Internal Friction angle and Shear Intercept by Ring Shear and Simple Shear Apparatus

a. Residual Internal Friction Angle (ϕ)

Sample no.	2mm.dn with const. o.c.stress		475μdown with const o.c.ratio		Multi-stage	Simple Shear Test	
	Peak, deg.	Residual, deg.	Peak, deg.	Residual, deg.	Residual, deg.	Peak, deg.	Residual, deg.
Okimi 1	27	15				17	15
Okimi 2	29	16	38	22	16	26	15
Okimi 3	30	16					
Okimi 4	30	22	39	25	22	17	22
Okimi 5					12		
Okimi 8					19		
Okimi, sliding surface					10		

b. Shear Intercept(c)

Sample no.	2mm.dn with const. o.c.stress		475μdown with const o.c.ratio		Multi-stage	Simple Shear Test	
	Peak, kPa	Residual, kPa	Peak, kPa	Residual, kPa	Residual, kPa	Peak, kPa	Residual, kPa
Okimi 1	12	9				29	20
Okimi 2	12	5.6	11	5.6	6.6	5.1	12
Okimi 3	20	4					
Okimi 4	15	6	9	4	5.6	23	23
Okimi 5					5		
Okimi 8					7		
Okimi, sliding surface					6		

kPa respectively. Likewise the peak and residual ϕ of sample no. 2 were 26° and 15° respectively and value of c were 5.1 kPa and 12 kPa respectively. The peak and residual value of ϕ at sample no. 4 were 17° and 22° respectively. The value of peak and residual c there, were 23 and 23 kPa respectively.

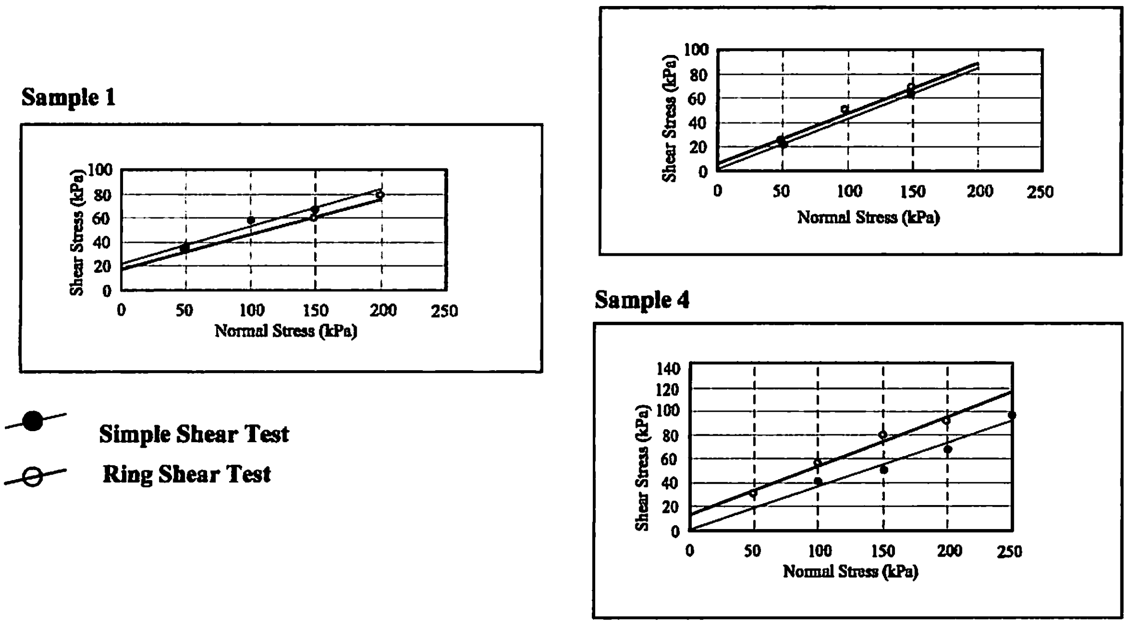


Figure 6 : Residual shear strength envelopes by both ring and simple shear test

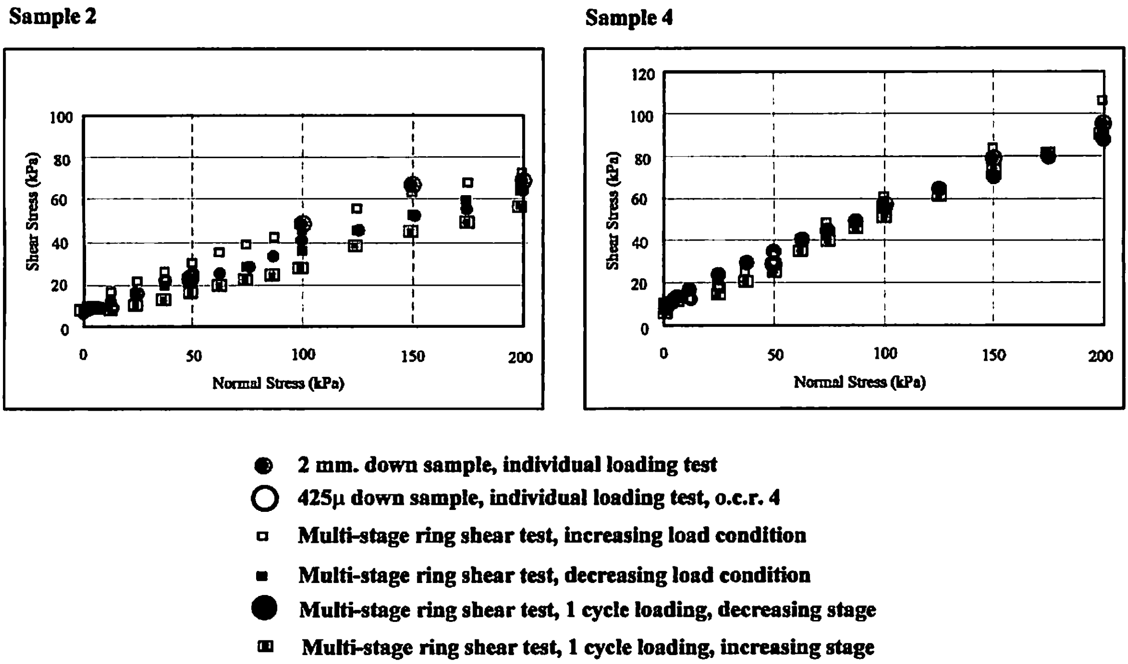


Figure 7 : Ring shear tests by all methods, Okimi 2 and 4

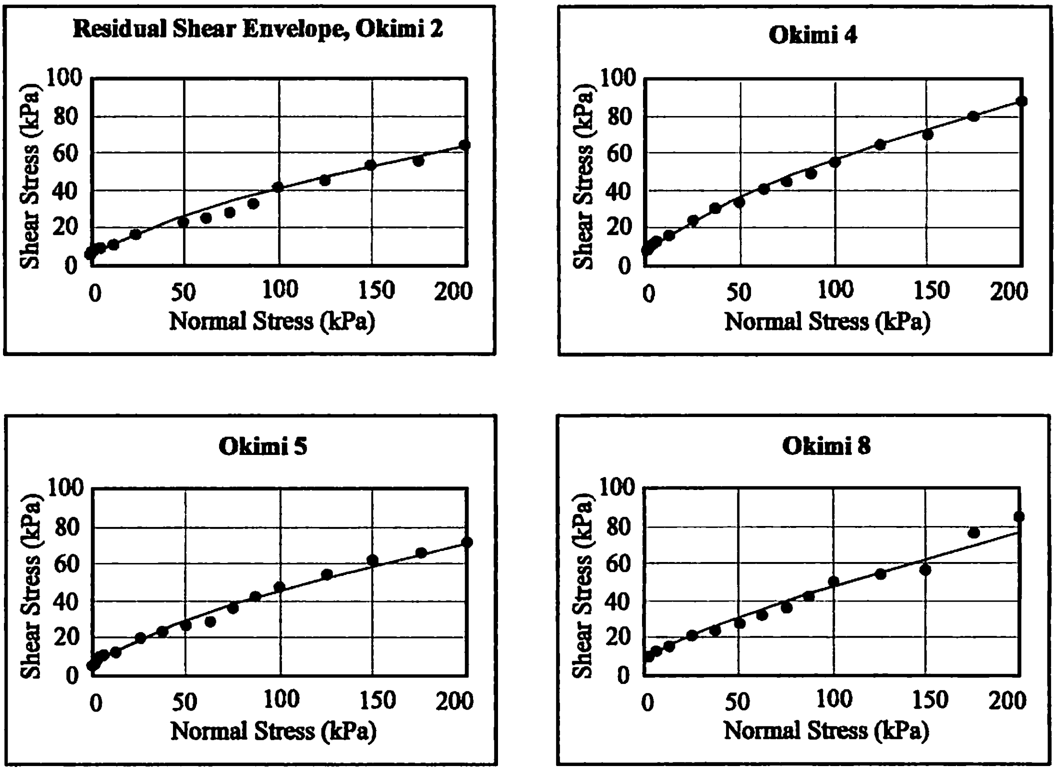


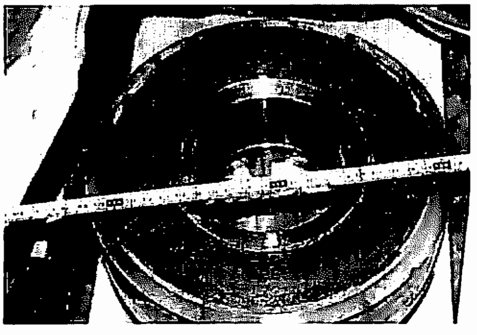
Figure 8 : Residual shear envelopes of sample no. 2, 4, 5 and 8 by multi-stage ring shear test

8.6 Ring Shear Test

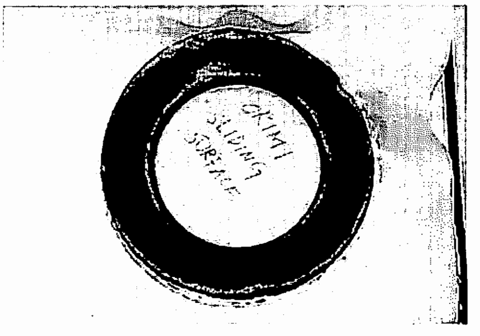
As the main target of this study was to check the residual shear strength of various soils, extensive ring shear tests were conducted, using various methods (Table no. 5). First, 2mm down soil particles of various samples were tested with individual ring shear test with varied normal stress, at the constant overconsolidation pressure of 200 kPa. The soil sample no. 1 exhibited the peak and residual ϕ of 27° and 15° respectively. The value of peak and residual c there, were 12 kPa and 9 kPa. respectively. Likewise, sample no. 2 gave peak and residual ϕ of 29° and 16°, respectively. Value of c in peak and residual stage for that sample were 12 kPa and 5.6 kPa. respectively. In sample no. 3, peak and residual ϕ were 30° and 16°, respectively, whereas c were 20 kPa and 4 kPa, respectively. While testing sample no. 4 with the similar method, it gave peak and residual ϕ to be 30° and 22° respectively and c to be 21 kPa and 6.5 kPa. respectively.

After the soil testing by the aforesaid technique, finer particles of the (425μ down) soils were tested with constant overconsolidation ratio of 4 to compare the result. For this purpose, the soil sample no. 2 and 4 were selected and were collected on October 6, 1998. After the soil test, the peak and residual ϕ at sample no. 2 were measured to be 38° and 22° respectively. The peak and residual c, there, were measured to be 11 kPa and 5.6 kPa. The peak and residual value of ϕ at sample no. 4 were measured to be 39° and 25°, respectively, whereas peak and residual c of that sample, were measured to be 9 kPa and 4 kPa. respectively.

After all the above mentioned tests, sample no. 2 and 4 were tested by multistage ring shear. This was done to overcome the possible errors due to dissimilarity of two different samples, while testing individual samples in former methods. "If the residual shear strength is mainly generated due to parallelmost orientation of the soil minerals, then this method would be the best one", was the concept to use this method. Therefore, 425μ down sized particles of sample no. 2 and 4 were tested again, by this method. Peak value can not be reliable in this method. The residual ϕ of sample no. 2 and 4 were found to be 16° and 22° respectively whereas the value of residual c in these two samples were measured to be 6.6 kPa and 5.6 kPa. respectively. Likewise, curved shear envelope, rather than the assumed straight lined one (assumed by Colomb's equation), was found by testing the soil up to the possible minimum value of normal stress. That selfexplained the reason for the increase of ϕ during the second test condition with constant O.C.R. of 4, which was done for low normal stress range only, due to the limitation of testing equipment. The value of residual shear strength was not so different, for different size of particles or different testing modes and different overconsolidation ratios. Hence, sample no. 5 for A block, 8 for D block and sliding surface soil from A block were tested by multistage ring shear test on 2mm down soil particles. According to the result, the residual ϕ at sample 5, 8 and sliding surface soil were measured to be 12°, 19° and 10° respectively whereas residual value of c were measured to be 5 kPa, 7 kPa and 6 kPa respectively.



Sample no. 2 after individual test



Sliding surface soil after multi-stage test

Photo 3 : Final shear plane formed after ring shear test (about 0.5mm. thickness)

9. Analysis of the test data

From the water content analysis of the soil samples from Okimi landslide, it is clear that (during constant consolidation loading with 200 kPa. on 2mm down soil), the water contents were decreased with the increase in normal load. This suggests that the increased normal stress might have decreased the void ratio and the water content have reached to the saturated water content. But while 425 μ down soils were tested with constant overconsolidation ratio, the water contents were increased for increased normal load. As the overconsolidation ratio was 4, the difference between the value of normal load and overconsolidation load for lower normal stress was lesser than that in higher ones. This might have given less swelling opportunity after the decrease in normal stress. In all the cases, the final water contents after multistage tests were quite less than the plastic limit and were almost similar in all cases except for sliding surface soil. Such possible differences in water contents, ascertained good drainage condition and less dilatency effect on residual shear strength.

There is good correlation between the liquid limit and residual ϕ. The tested data correspond well with the data given by Mesri and Diaz (1986). This indicated the possibility of rough estimation for residual ϕ by careful consistency limit test. Almost all of the samples were supposed to be inorganic silt with medium plasticity whereas sliding surface clay was supposed to be inorganic clay with high plasticity. This difference was occurred due to large amount of finer particles, while testing the consistency limit of the later. For consistency limit test, 2mm down particles were tested, except for sample nos. 2 and 4, which were tested with 425 μ down particles too.

The percentage of clays measured by grain size analysis were comparatively small. In fact, 2mm down soil particles were tested during constant consolidation stress ring shear test and 425 μ down soil particles were tested during constant overconsolidation ratio ring shear test. If the percentage of clay will be recalculated among the 2mm down particles only, then considerable clay percentage can be noticed. That might be responsible to provide about 0.5mm thick clear shearing zone after attaining the residual shear strength. The shape of grain size distribution curve for sample no. 2 and 4 were similar. According to the textural classification, both the soil samples were sand to sandy loam.

According to the mineralogical analysis, all the soils in Okimi landslide are rich in quartz mineral, if we analyze the whole soil. As finer particles are found to be responsible for the shear surface formation, the proportion of clay might play main role for residual shear strength. Due to this reason, clay fractions were separated in the laboratory using Stoke's method and diffracted through x-ray diffractometer. The result showed, the proportion of montmorrilonite was higher than others and were almost similar in all the samples, if total soil will be analyzed. Small proportion of kaolinite could also be detected in all the soil samples. The high proportion of montmorrilonite might be responsible for less residual shear strength.

Peak shear strengths were measured by both ring and simple shear apparatus. The value by ring shear apparatus was higher than the simple shear apparatus in many cases. That might be due to difference in testing conditions. In fact, there is no meaning to test the peak shear stress while analyzing the landslides, unless the undisturbed sample is to be tested.

The residual shear strength is found to be irrespective of testing conditions and mineral composition is found to be dominant for the residual shear strength. Sample no. 1 was tested on 2mm down particles only, with both simple and ring shear tests. Both tests gave the similar value of residual ϕ i.e. 15°. Sample no. 2 was tested with various methods i.e. on 2 mm down soil by both apparatus, ring shear test on 425μ down soil and multistage ring shear tests. All the tests gave the residual ϕ between 15° to 16°, except that by testing 425μ down soil with constant o.c.r. of 4 (22°). First, it was surprising result, but, after multistage test, the curved failure envelope was confirmed and the larger value of residual ϕ at lower normal stress range was proved obvious. Residual ϕ of 2 mm down soil from sample no. 3, by ring shear test, was 16°, which was similar to that of sample no.1 and 2. However, sample no. 4 always gave higher residual ϕ than others. Almost all of the test methods gave residual ϕ closer to 22°, except the test with o.c.r. of 4 (25°), which was understood obviously, as explained above for sample no. 2.

Due to the aforesaid reasons, the shear strength data provided by the multistage ring shear test of sample no.2 was used for the stability analysis of block C, Okimi landslide. The analyzed section is closer to this location. As it was understood that residual strength given by multistage ring shear test is most reliable one for landslide analysis of repetitive rotational slides, soil samples from the main scarps of other blocks were tested with multistage ring shear test after sieving them by 2 mm sieve. Sample no. 5, from the main scarp of block A, had the residual ϕ of 12° which was slightly higher than the remoulded sliding surface soil, which had residual ϕ of 10°. It was very difficult to test the sliding surface soil due to its stiffness and low permeability. The test was failed for many times. It took almost 10 times more test durations than the main scarp soils to yield the residual value. Although the residual value was almost equal with main scarp soil for lower normal stress range, it was slightly gentler for higher normal stress range. The main scarp soil of D-block yielded the residual ϕ of 19°. This value was utilized for stability analysis. It is to be noted here, that almost all of the samples yielded some small values of c, when the soil test was conducted for negligible normal stress. This was found to be real property of those soils rather than the effect of self weight.

10. Stability Analysis of Various Blocks

The tested data were utilized for the stability analysis of whole landslide area. The computer program, using c++ language was made to ease the calculations, while assigning the unknown water table during analysis. As the tested values of c and ϕ were most reliable and invariable, the only variable parameters might be water table. Therefore, the maximum position of water table was tried to found out during the first movement. Stability analysis of each blocks were performed as follows.

C-Block: Stability analysis of C Block along some surveyed crosssections was done, once, in 1979. During that time, stability analysis was done with c as 10 kPa (assuming slide surface depth of 10m) and ϕ was calculated by back analysis. Surveyed maximum water table was utilized during the analysis. With proposed factor of safety of 1.15, counter measures were planned and applied. Same section and the same data except c and ϕ were utilized in this study too. Here, c and ϕ obtained by the ring shear test on sample no. 2 was utilized. Stability analysis for both upper and lower sections of crosssection A-A were done. Slope 2, lying at steeper slope was found to be unstable during that time while slope no. 1 was in critical condition with the factor of safety of 1.03 (Table 7). But the water table, decreased after the construction of extensive drainage network, might have stabilized this block, although minor displacements of few cm per year were recorded. This small movement might be due to the movement of A block in oblique direction, which might have induced the loss in the toe support of C-block. The numbers of functioning piezometers in the landslide area, to confirm the ground water table after countermeasure application, are very less. Therefore, the data from the nearby piezometers were utilized for analysis.

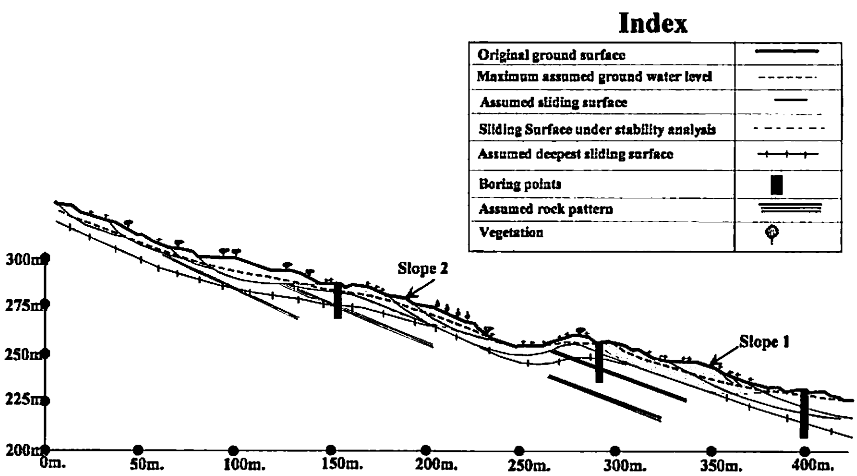


Figure 9 : Cross-section of Block C along A-A

Table 5 : Stability Analysis Chart for Slope 1 along Section A-A, Block C

s	h1,m	h2,m	d,m	A,m2	W,kN	θ,deg	sinθ	cosθ	wsinθ	wcosθ	h1',m	h2',m	u,kPa
1	0	4.5	6	13.5	224.9	34.4	0.565	0.825	127.1	185.6	0	0	0
2	4.5	7	9	48.875	814.3	37	0.602	0.799	490	650.3	0	0	0
3	7	7	3	22.4	373.2	28.5	0.477	0.879	178.1	328	0	0	0
4	7	9	4	29.6	493.1	25.5	0.431	0.903	212.3	445.1	0	1	18.13
5	9	8	7	57.8	962.9	20.3	0.347	0.938	334.1	903.1	1	3.2	139.94
6	8	9.3	10	83.04	1383	21.5	0.367	0.93	507	1287	3.2	6	432.77
7	9.3	13	14	156.1	2601	16	0.276	0.961	716.8	2500	6	5.5	788.9
8	13	11.5	10	122.5	2041	9.7	0.168	0.986	343.9	2012	5.5	4.8	504.7
9	11.5	9.2	4	43.47	724.2	5.9	0.103	0.995	74.44	720.4	4.8	4.2	185.22
10	9.2	6.2	9	69.3	1155	3	0.052	0.999	60.42	1153	4.2	3	317.52
11	6.2	2	14	57.4	956.3	0	0	1	0	956.3	3	0.6	246.96
12	2	0	8	7.5	125	−4.6	−0.08	0.997	−10	124.5	0.6	0	22.05
Σ				711.49					3034	11265			2656.2

Σl 102.1

c= 6.6 ϕ= 16 FS= 1.036 With tested c and ϕ

c= 10 ϕ= 13.15 FS= 0.999 back analysis method with c=10kPa

A-Block: A block was not analyzed, even in the past, although countermeasures were heavily applied. Section A-A was considered for the analysis. The sliding surface information and condition of water table were tapped from the recent check boring records. The tested soil strength parameters of the sliding surface soil were used for the analysis. With this, the factor of safety become 1.59, highly stable after the countermeasure implementation. But, A-block is moving with speed of more than 0.5 cm/ year. This made suspicion on the movement of D-block and stability analysis of D block was also done.

D-Block: No stability analysis was done for D-block too in the past. Hence, the section surveyed in the past with the ground water information at that time was used for stability analysis. Soil test data of sample 8, the main scarp of D block, were used. The block was found to be unstable, but, no countermeasures are applied in this block. The evidence of water accumulation in the form of pond was also found in this block. In fact, this block was recorded to be fastest in movement, with the rate of few meters per year. This movement might have caused the loss of the support for the toe of A-block, and A-block also moved inspite of heavy drainage works. No important land and infrastructures are located in D-block. That might be the main reason for not implementing any countermeasures in D-block.

Table 6 : Stability Result of All Blocks

Block No	Real test data			Back Analysis	
	c,kPa	ϕ, deg.	FS	c, kPa	ϕ, deg.
C, slope 1	6.6	16	1.03	10	13
C, slope 2	6.6	16	0.91	9	16
C, sec. B-B	6.6	16	1.6	11	10
A	6	10	1.37	26	3
D	4.9	19	0.99	13	12

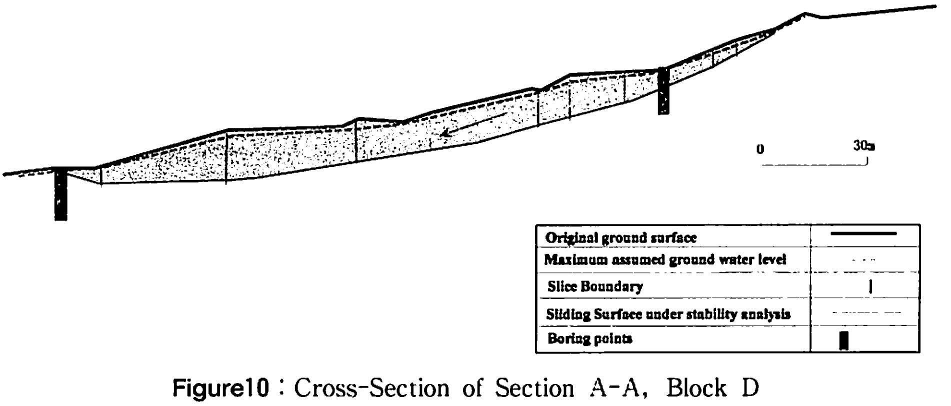


Figure10 : Cross-Section of Section A-A, Block D

11. Conclusion

As stated earlier, the countermeasures applied at A, B and C blocks of Okimi landslide are functioning well. The concept of the block division should slightly be revised as A and C block with the addition of a elongated small A1 block. The moderately moving but very important A block is stable in itself, but is moving due to the movement on D-block. The movement observed in C block might be due to the induction by the movement of A block. D block is the most unstable block and is constantly moving due to the toe erosion and ground water rise. This induced displacement in A-block, making the whole landslide unstable. If water table in D-block can be reduced, movement of D block can be controlled. This will, in fact, make the whole landslide stable. After that, E-block should be studied with similar analyzing method and countermeasures should be applied there too, if necessary. Several piezometers should be established in D-block to check the position of ground water before and after countermeasure application. It is supposed that the ground water reduction at D-block will significantly change the instability condition of Okimi landslide as a whole. Therefore, it is recommended to apply the surface and subsurface drainage network in D-block as early as possible and add some more monitoring devices to ensure the expected effect.

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