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Damage analysis of a compound retaining wall collapsed during the 2014 earthquake in Northern Nagano

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Abstract

A lot of structures were damaged at Horinouchi area of Hakuba village, Nagano prefecture due to the 2014 earthquake in Northern Nagano (Mw=6.3), occurred on November 22th, 2014. In this area, we found that a compound retaining wall, which has a vertical wall on a block stacking wall, has collapsed. The total height of the wall is about 5m, which has a block stacking wall about 3.5m high and a vertical wall about 1.5m high.

In this research, we focused on the compound wall, analyzed the cause of damage based on field investigations. First we conducted field investigations on the compound wall. Three investigations were conducted. The first one was just after the earthquake, on November 28th, 2014. We observed the damage situation around the wall. The second one was after a month, on December 21st, 2014. We performed microtremor measurements, and sampled soils at 5 points with thin walled tube to perform static compression tests. The third one was after five months, on April 21st, 2015. We performed some simple measuring to confirm a shape and a size of the wall, and two in-situ investigations, portable dynamic cone penetration (PDCP) tests and surface wave survey. We also sampled soils at more 5 points with thin walled tube to perform cyclic triaxial tests, and soils at 36 points using core cutter to examine water content and density of the soil.

In the field investigations and laboratory tests, we found out two typical features of the site. First, the back ground of the wall has alternation strata that consist of cohesive soil, whose fine contents are about 80%, and sandy soil, whose fine contents are about 30%. We can also find some traces of piping at the back ground of the wall. Second, the back ground and the foundation ground have a poor subsurface layer and steep dip in a basement layer. The poor layer whose shear velocity is 100m/s or less, from the surface to about 2m in depth. Focusing on the layer whose shear velocity is over 200m/s, in the back ground it is found from the surface to about 9m in depth, while in the foundation ground it is found from the surface to more about 12m in depth. These results reveal that the dip of the basement layer around the wall is steep, and thick poor soil deposit on the basement layer. From the result of microtremor measurements, we can find that the ratio of the horizontal component at the top of the wall and the one at the bottom of the wall is about six times in a frequency band of around 1Hz. From the above results, it is supposed that the ground motion greatly amplified at the back ground of the wall, and it seems the main factor of the damage.

Keywords: retaining wall, in-situ test, laboratory test

1. Introduction

1.1 Area of the research

Figure 1 shows the area of our research. We surveyed the Horinouchi district in the village of Hakuba, Nagano Prefecture. What is notable about the disaster is that most of the vibration-related damage, such as the collapse of houses and the surfacing of buried pipes caused by soil liquefaction, was reported there, although the district is slightly south of the epicenter [1].

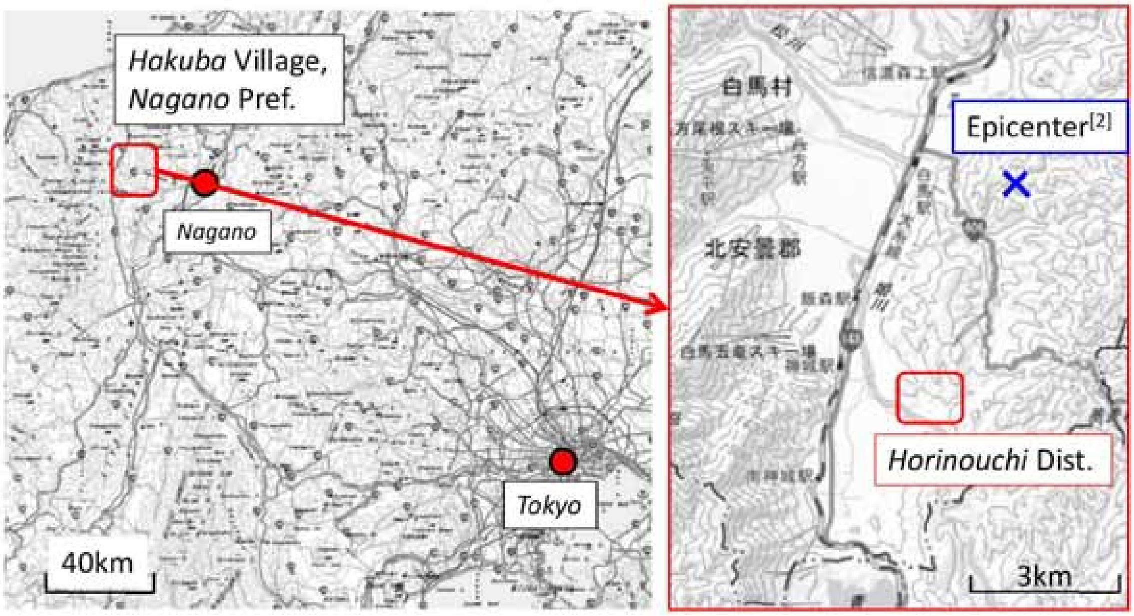


Fig. 1 – Area of the research (from GSI Digital Map [3])

1.2 Damage situation

Figure 2 and 3 show the retaining wall before and after the earthquake. The retaining wall, as described above, is a compound retaining wall consisting of a 4-meter-high block wall at the base and a 2-m gravity retaining wall on it. The upper gravity retaining wall collapsed after the earthquake, causing the backfill soil behind it to partially collapse.

A simple measurement conducted on the site showed that the collapsed part of the retaining wall totaled about 42 m in length. While there is a vinyl greenhouse above the retaining wall, which measures 37 m in length and runs parallel to the retaining wall, no problems were found with the greenhouse. After the earthquake, the distance between the greenhouse and the top of the backfill soil behind the retaining wall measured about 50 centimeters. Meanwhile, before the retaining wall collapsed, the greenhouse is estimated to have been situated about 2 m from the top part of the wall based on the location of the remaining part of the retaining wall. It is therefore estimated that a 1.5 m-long area on the backfill soil slid down from the top part of the wall.

We also found some piping traces at the background of the wall. Figure 4 shows the piping traces. The trace size was about from 1m to 3m in width.

Fig. 2 – Compound retaining wall before earthquake (from Google Map [4])

Fig. 3 – Compound retaining wall after earthquake

Fig. 4 – Piping traces of background

2. Details of Investigation

2.1 In-situ test

The (1) to (4) in-situ tests below were conducted to study the ground condition around the retaining wall. The locations of the in-situ tests are shown in Figure 5.

(1) Portable dynamic cone penetration (PDCP) test

The penetration test was conducted at a total of three points - the top, middle and bottom parts of the retaining wall. A rod was inserted at a right angle to the retaining wall to find the border between the backfill soil and the foundation ground based on changes in penetration resistance for the grounds around the wall. The PDCP test was conducted according to the Japanese Geotechnical Society Standards (JGS 1433), and the number of blows (N_d) was converted into the N-value using the method of Okada et al.[5].

(2) Soil strength test rod [6]

The soil strength test was conducted to measure the shear strength (cohesion and internal friction angle) of the backfill soil. The testing was carried out at a total of four locations very close to where the undisturbed samples were collected (around the top of the retaining wall) so that its results can be compared with those of a separate survey conducted to measure the shear strength of the backfill soil.

(3) Surface wave exploration

The surface wave method was adopted for two measuring lines - one in the upper part of the retaining wall and the other in the lower part - to estimate the shear wave velocity (V_s) structure below the ground. The survey was conducted to estimate the velocity structure of a zone between the backfill soil behind the retaining wall and the foundation ground.

(4) Microtremor measurements

Microtremor measurements were conducted at two spots - the upper and lower parts of the retaining wall - to estimate the predominant period and identify the amplification characteristics for the backfill soil behind the retaining wall and the foundation ground.

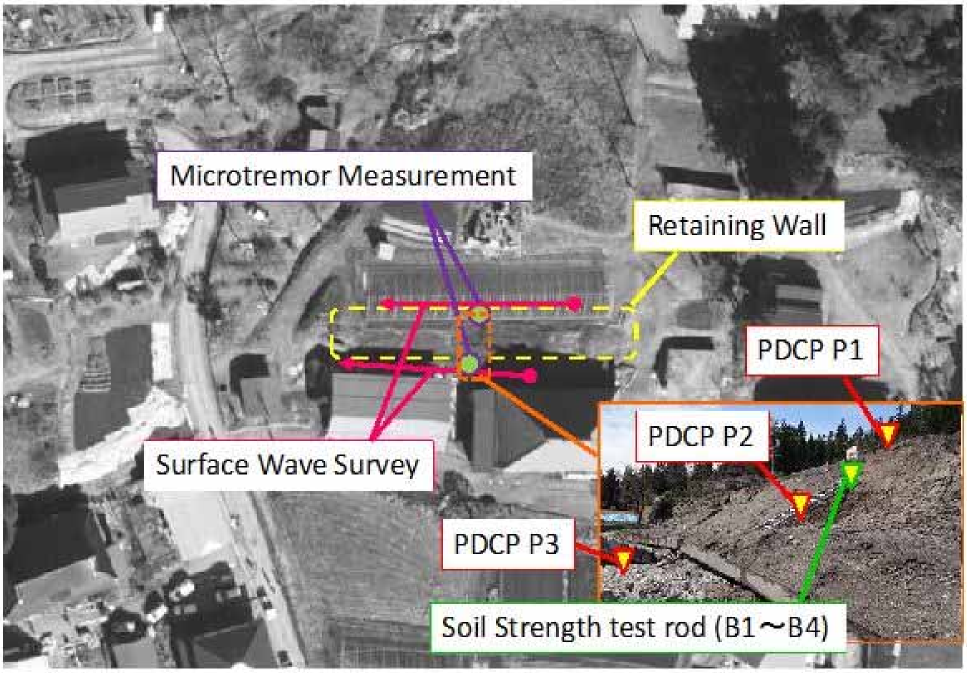


Fig. 5 – In-situ test locations

2.2 Laboratory test

In addition to in-situ tests, we collected undisturbed samples with thin-wall sampler and disturbed samples with core cutter from the backfill soil behind the retaining wall surveyed in the earlier study. We performed physical laboratory test, unconfined compression test and triaxial compression test conducted by using the samples. Figure 6 show the sampling area. We collected 34 distributed samples with core cutter and 10 undistributed samples with thin-wall sampling. We collected distributed samples at 50 cm spaces on a 3 survey lines. And we collected 5 undistributed samples at each 2 points, one is a bed of silt, the other is sand.

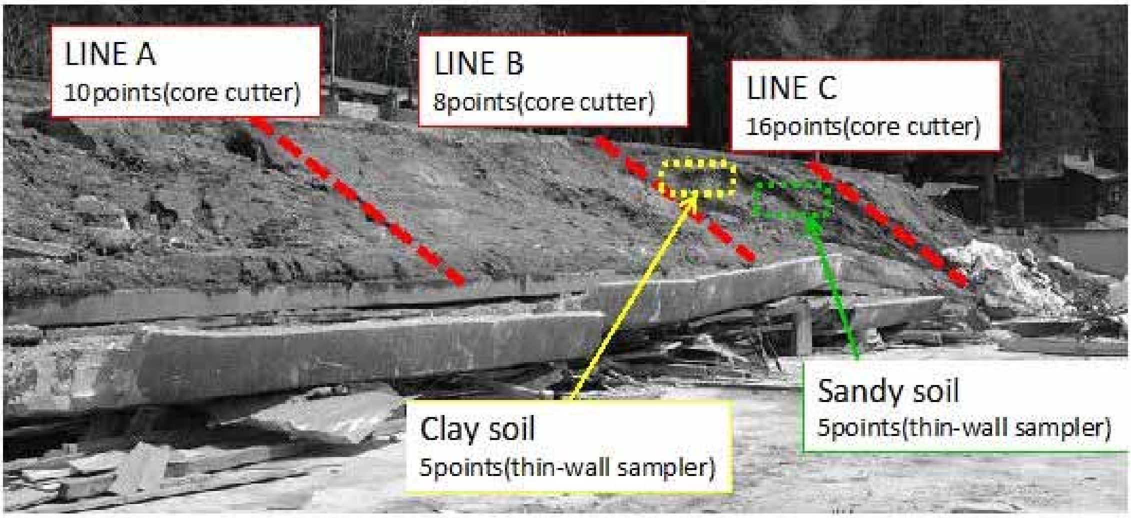


Fig. 5 – Sampling area

3. Results and findings

3.1 Subsurface structure

Figure 6 show the results of the PDCP tests, and Figure 7 show the results of the surface wave tests.

According to the results of the PDCP test for P1 and the surface wave test for the A-A' measuring line, the area between the top of the retaining wall and the base of the gravity retaining wall is made up of soft soil (*N*-value of less than 10, V_s = 80 to 150 m/s), while the ground deeper than the base of the gravity retaining wall is made up of harder soil (*N*-value of 40 or higher, V_s = 150 to 200 m/s).

And according to the results of the PDCP test for P3, the *N*-value for the foundation ground at the base of the retaining wall is around 10 at a depth of up to 1.5 m or so below the ground surface, while hard soil with an *N*-value of about 30 appears at a depth of more than 2.0 m. The surface wave test for the B-B' measuring line, which is slightly distant from the base of the retaining wall, showed the underground soil is soft at a depth of up to 5.0 m or so with V_s being 80 to 150 m/s.

Figure 8 show the results of microtremer measurements. When the H/V spectrum (H represents the geometric mean of orthogonal horizontal components and V represents the vertical component) was calculated based on the results of microtremor measurements, low-frequency waves were predominant for the grounds around the retaining wall. Significant amplification was also reported between the upper and lower parts at around 1 Hz - a level at which the retaining wall structure becomes vulnerable to vibrations. For example, the H/V spectrum was nearly 1.0 for the ground near the K-NET Hakuba (NGN005) measurement point.

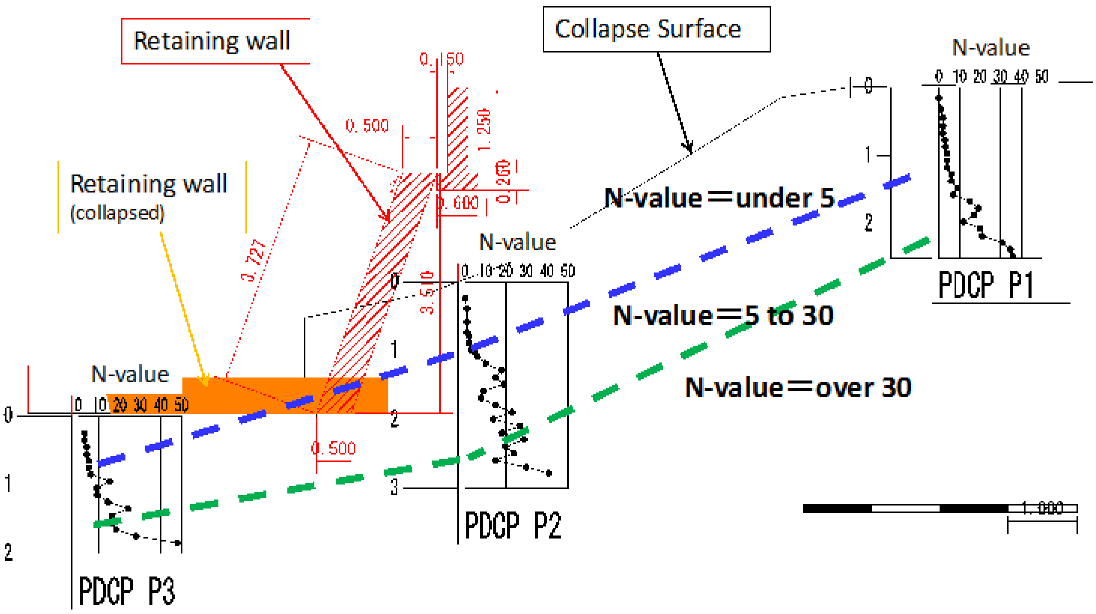


Fig. 6 – Results of PDCP tests (cross section of the wall and background)

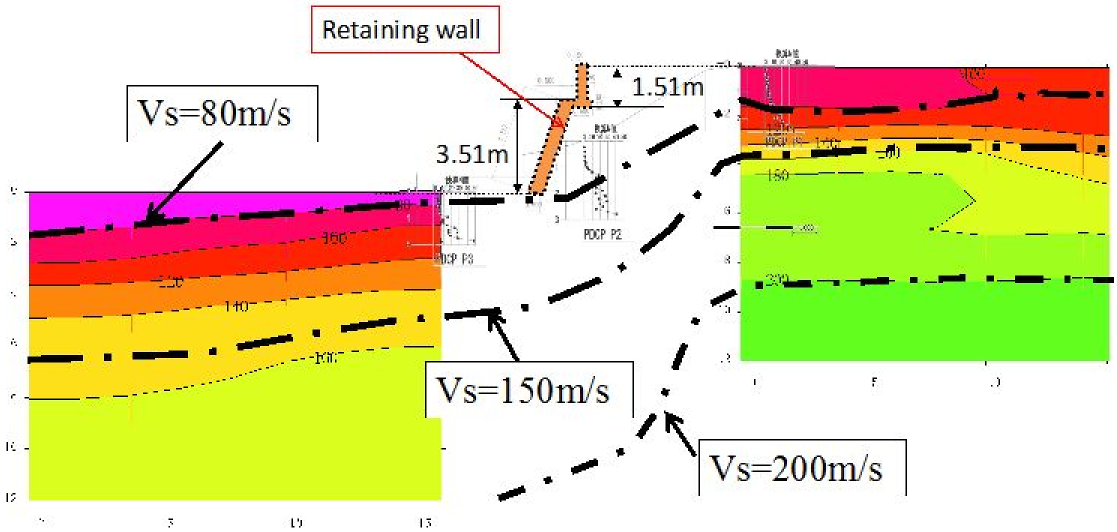


Fig. 7– Results of surface wave tests (cross section of the wall and background)

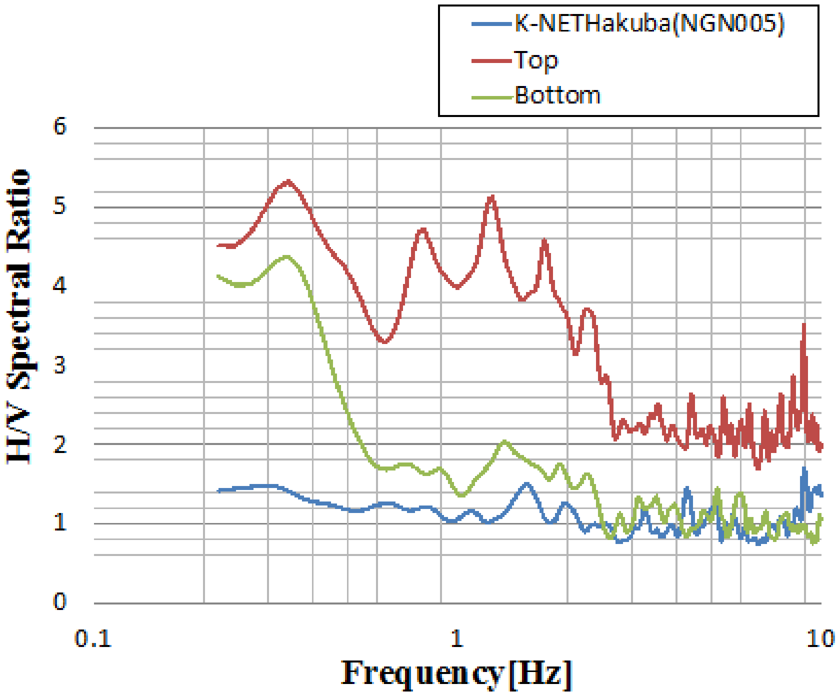


Fig. 8– Results of microtremer measurements (H/V spectral ratio and H/H spectral ratio)

3.2 Soil property of back ground

Figure 9 shows stratum structure of background, and Figure10 shows the particle size distribution curve for the studied specimens which sampled with core cutter.According to the results, we found out that the bakground soil has a wide variety of soils, such as sandy soil, silty soil, and clay. Furthermore, these soils has formed alternating layer, as shown in Figure 9. The thickness is about 1m if divided with a point where fine fraction contents(Fc) equal 35%.

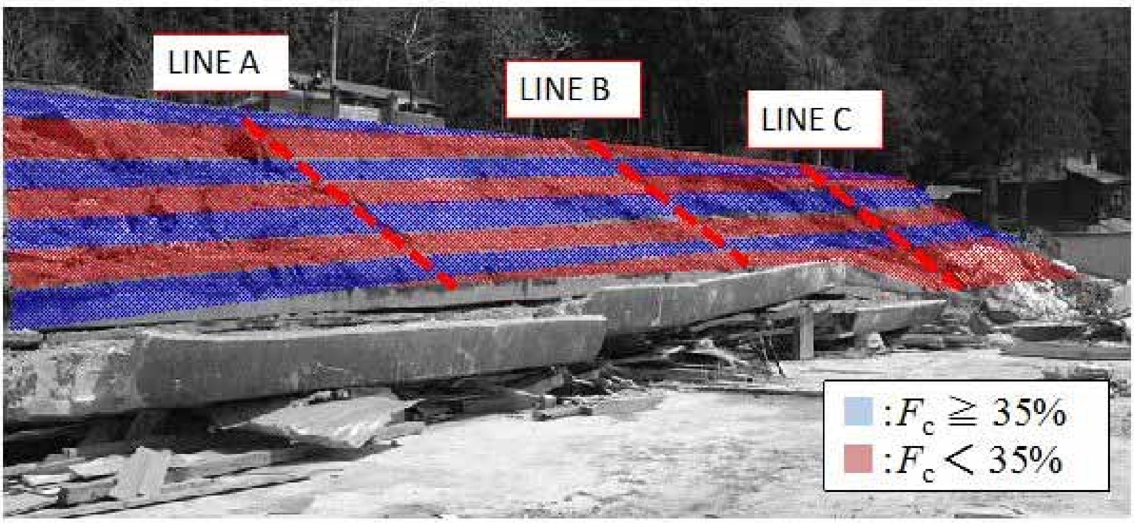
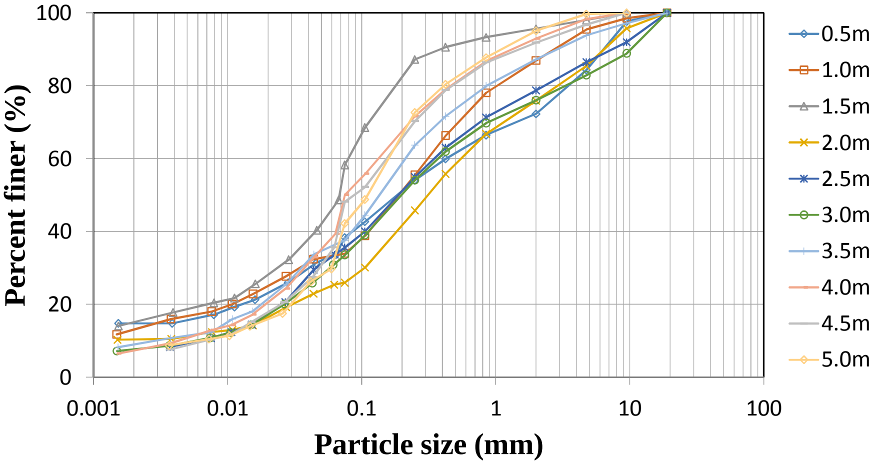
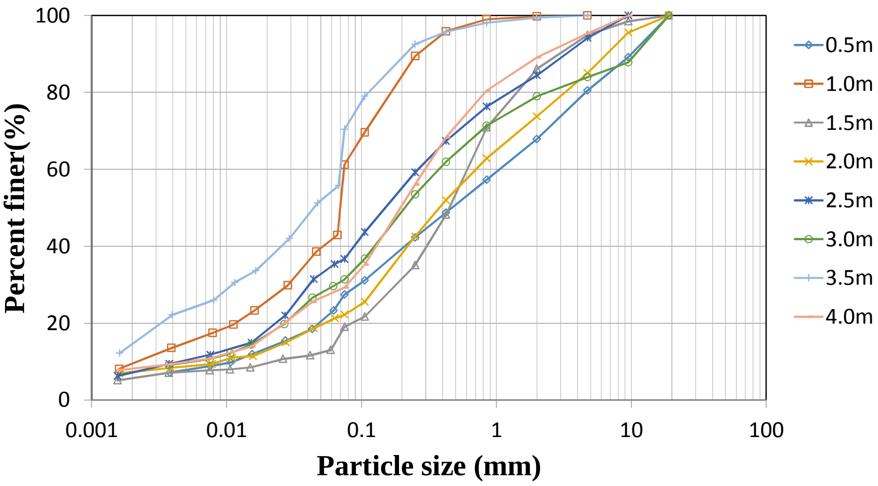


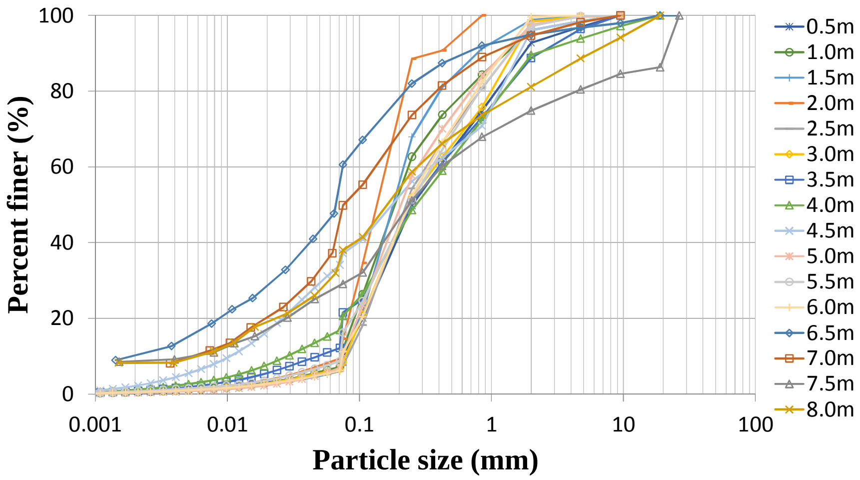
Fig. 9–Stratum structure of background



(a) Line A



(b) Line B



(c) Line C

Fig. 10– Grain size accumulation curve

Table 1 shows the physical properties for the studied specimens which sampled with thin-wall sampling. The surveyed specimens consist of a total of four samples: Samples A, B and C, which were subjected to consolidated undrained ($\overline{CU}$) triaxial compression testing, and Sample D, which was used for the unconfined compression test. According to the test results, the particle size composition of the test specimens varies to some extent, and they contain a considerable amount of fine-grained fractions, with the fine fravtion content, F_c, being 74.9 to 85.9. In particular, the F_c for Sample D was 85.9%, the largest among all the specimens, and its plasticity index, denoted by I_p, was 33, meaning that clay is predominant in the specimen.

Table 1 – Physical properties of samples

NAME			Sample A	Sample B	Sample C	Sample D
Soil particle density	ρs	(g/cm^3)	2.688	2.86	2.893	2.452
Wet density	ρt	(g/cm^3)	1.677	1.697	1.709	1.711
Dry density	ρd	(g/cm^3)	1.177	1.169	1.176	1.212
Moisture content	w	(%)	42.5	45.2	45.3	41.2
Fine fraction content	Fc	(%)	74.9	83.5	76.3	87.8
Mean grain size	D_{50}	(mm)	0.074	0.042	0.07	0.081
Plasticity index	Ip		4	7	19	33

3.3 Shear strength of back ground

Figure 12 shows the shear strength of the background using soil strength test rod. The shear strength, τ, estimated using a soil strength test rod, correlates with the penetration stress, σ, in some cases, while in other cases they do not, depending on the survey location. Thus it is estimated that the particle size composition of the backfill soil varies. The average of the shear strength, τ, was 32.8 kN/m^2.

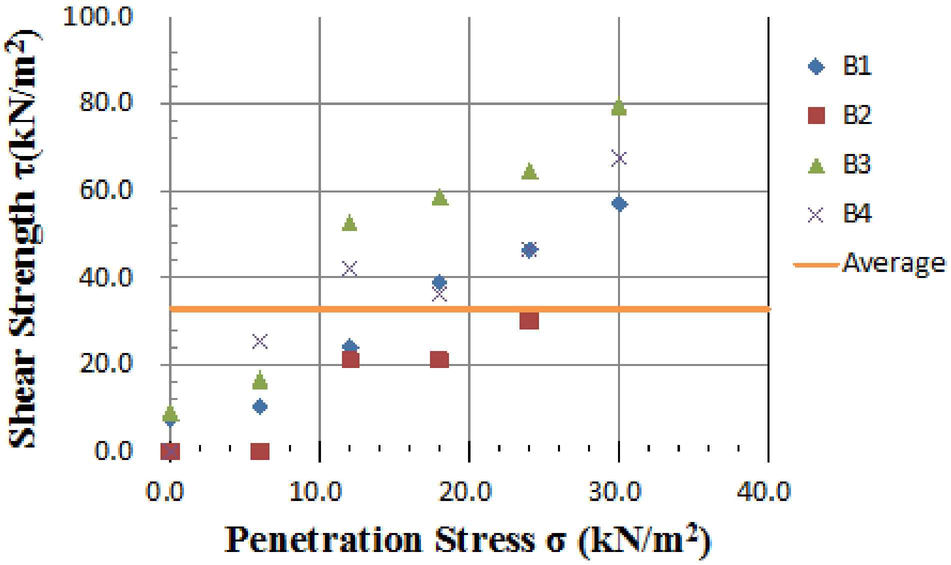


Fig. 12– Shear strength, τ, estimated with soil strength test rod

Figure 13 shows the relationship between stress and strain found through the unconfined compression test. While the unconfined compression strength, q_u, was 74.2 kN/m^2, the deformation modulus, E_{50}, was 1.06 MPa. Based on the premise that $q_u/2$ is equal to the shear strength, τ, the value is estimated to be approximately equal to the average of τ, 32.8 kN/m^2, which was calculated with a soil strength test rod.

Figure 14 shows the results of the triaxial test for each sample, referring to the relationships between the deviator stress, pore water pressure and axial strain. As for the relationship between deviator stress and axial strain, the deviator stress for Samples A, B and C rises, as the axial strain increases. The major principal stress for them does not reach its obvious peak until ε_a rises to 15%. The higher the peak strength becomes, the more the effective confining pressure, σ_c', rises, providing clear evidence of confining pressure dependency. As for the relation between the pore water pressure and axial strain, a positive pore water pressure is generated immediately after the start of loading, which indicates negative dilatancy characteristics. The pore water pressure reaches its peak at around 5% of ε_a and then begins to decline gradually, showing positive dilatancy.

Figure 15 shows the effective stress path based on the results of the triaxial test. Considering the relationship between the mean effective principal stress, denoted by p', and the deviator stress, q, the mechanical characteristics are consistent with those found in middle-density sandy soil, unlike the sample described in Figure 2, in which viscous material was predominant. The internal friction angle, φ', was 37.5°, while the viscosity, c', was estimated at 5.0 kN/m^2. Meanwhile, φ is estimated to be 35° with a method of *Meyerhof* [7]based on the *N*-value of 10 obtained through the portable dynamic cone penetration (PDCP) test conducted at a depth of 1.0 m in our earlier survey. The *N*-value obtained by the laboratory triaxial test and the value obtained by the PDCP test are almost consistent with each other.

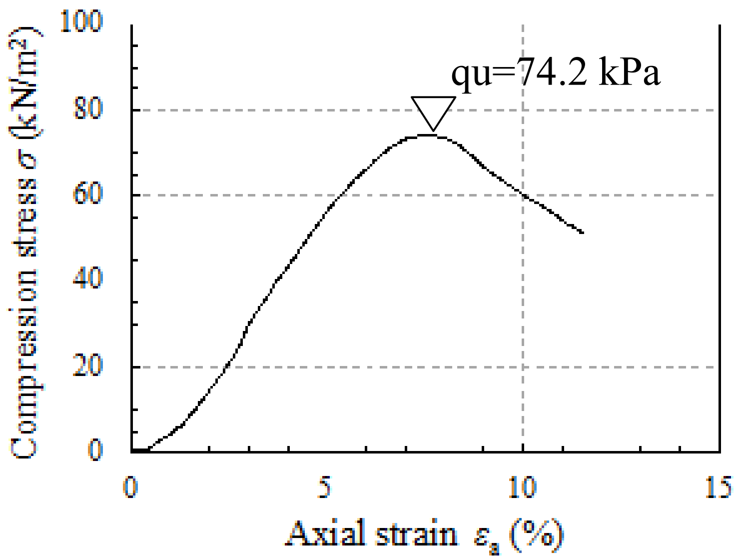


Fig. 13– Relationship between compression stress and axial strain (Sample D)

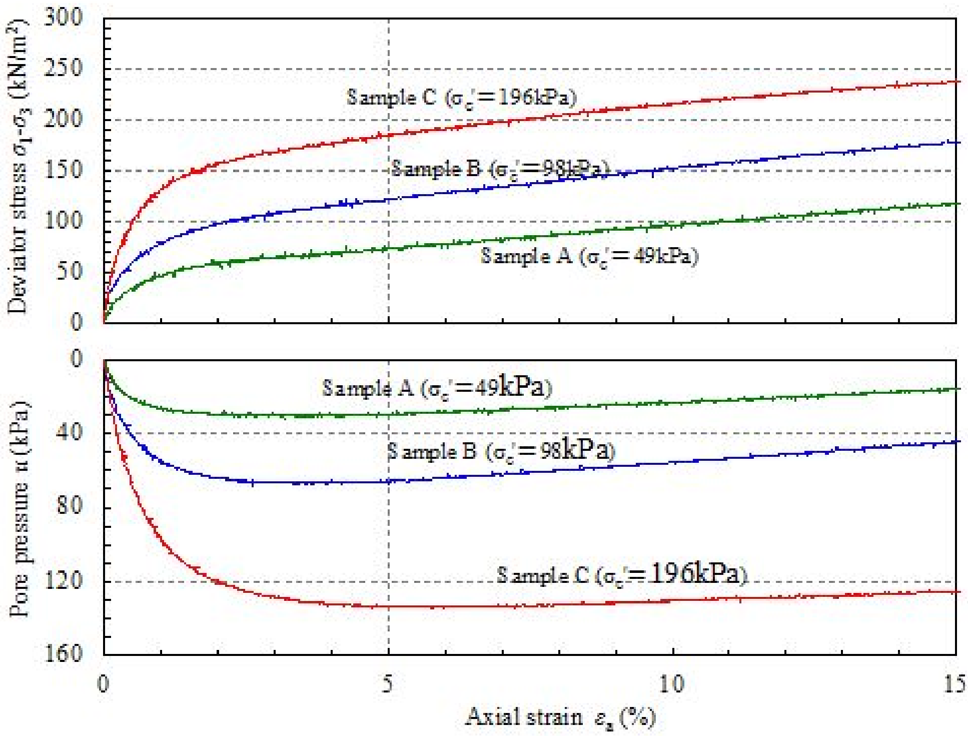


Fig. 14– Relationships between deviator stress, pore water pressure and axial strain (Sample A,B,C)

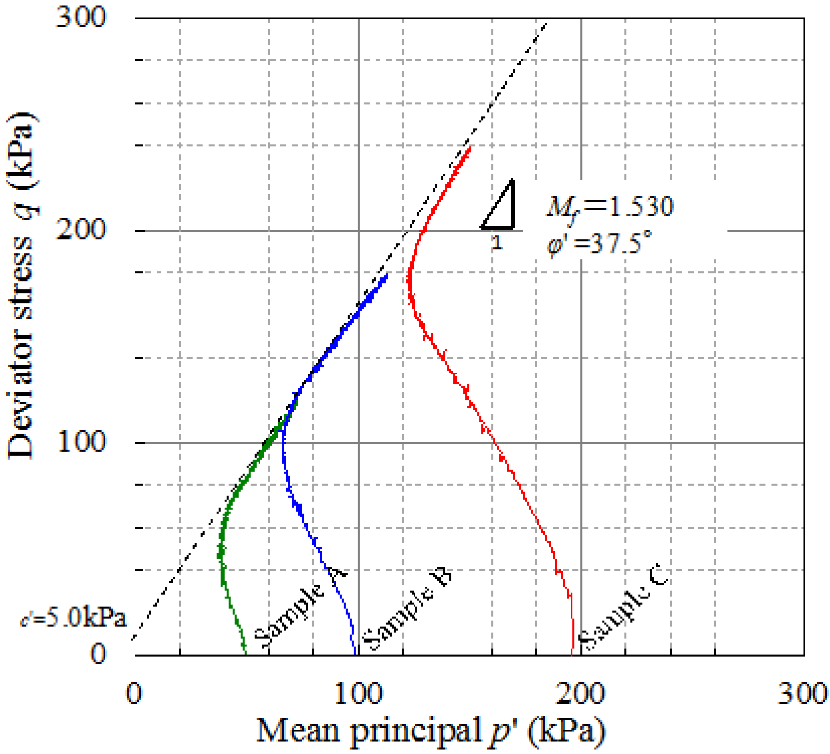


Fig. 15– Effective stress path (Sample A,B,C)

4. Conclusion

From a series of results of laboratory tests, we found that the soil behind the surveyed retaining wall contains sand, cohesive soil and other various substances. Furthermore, the results of the field investigation are almost consistent with those of the laboratory tests. The design standard for sandy backfill soil behind retaining walls[8] requires 30° of φ. So it can be said that the backfill soil at this site has sufficient strength from a broad perspective, though there are some variations of it.

Based on the results of our investigations and the fact that more buildings

in the Horinouchi district in Nagano Prefecture, including our surveying area, collapsed than in other regions, it can be estimated that the site of the retaining wall have suffered greater amplification of earthquake shaking than other locations.

We showed a mechanism of collapsing of the wall at Figure 15. We surmise the mechanism as follows,

1) A seismic wave amplificated due to the influence of the slope of the foundation ground(hard soil), a large internal force acted the top of the retaining wall.
2) Because the retaining wall is a top heavy structure, a horizontally force due to earthquake consentrated on the base of the concrete block retaining wall.
3) As a result, the concrete blocks were collapsed at first, and then the upper concrete wall fell on the blocks.

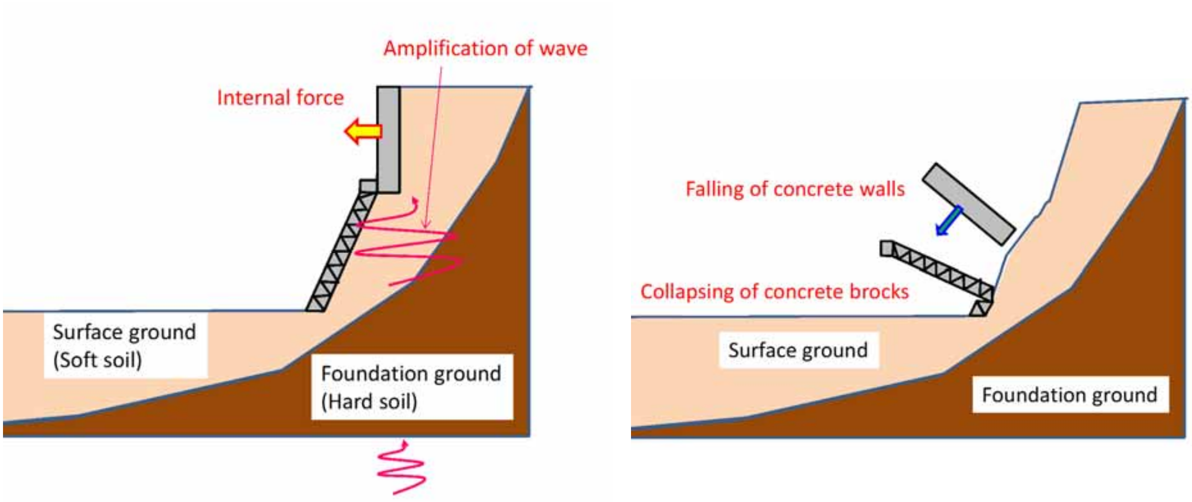


Fig. 16– A mechanism of collasping of the wall

5. Acknowledgements

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6. References

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