Vulnerability of volcanic loose soils having cementation and crushable particles

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Abstract. In Japan, a multitude of slope disasters occur annually, resulting in often severe loss of human life. On occasion, disasters that occur at very gentle slopes and disasters that flow long distance, associated with destructive energy, are observed. In such cases, the trigger layer often consists of volcanic soils with extremely high void ratios and external triggers, including earthquake or heavy rainfall, increase the risk of slope disasters. These volcanic soils are often characterised by an extremely loose structure and are either retained mainly by weak cementation or are composed of crushable particles having intra voids. Although the cause of destructive long-distance flow has been explained by liquefaction, there are some observed disasters in which the trigger layer was not fully saturated, i.e. liquefaction should not be considered as a major cause. Focusing on the fact that long-distance flow or gentle slope flow, disasters are triggered by volcanic soil layers of extremely loose structure, artificial loose soil samples were prepared in the laboratory. These samples were prepared with cementation between particles and with crushable particles. In this study, deformation and strength characteristics of the extremely loose soils were investigated by conducting CD triaxial tests. It is revealed that such extremely loose soils can be vulnerable after the weakly cemented structure collapses as a result of external factors such as earthquakes.

1 Introduction

Volcanic soils, characterised by high void structure, are found widely at various geographic locations in Japan (Table 1), and caused several historic slope disasters. Prominent examples include large scale slope disasters such as the Ontake land slide, the Takanodai land slide in Minamiaso village, the Izu Oshima slope disaster, the 2008 Iwate-Miyagi Nairiku earthquake slope failure and the slope disasters that occurred in Hokkaido in 2018. In the case of the Ontake land slide, long distance flows occurred at loose unsaturated volcanic layers in response to the 1984 Western Nagano Prefecture earthquake. In 2016, Kumamoto earthquake triggered a gentle slope failure of the slack surface of a loose pumice stone layer of Minamiaso village, and killed 5 people. The slope disaster of Izu Oshima was notable as the slope that had been evaluated as safe in the hazard map. The collapse was caused by torrential rain due to a typhoon. In the case of the 2008 Iwate-Miyagi Nairiku earthquake, sediment flowed over a long distance on a gentle slope with a gradient of 1° to 2° due to deposition of loose non-plastic volcanic ash soil with cementation. The landslides caused by pumice layers in Hokkaido in 2018 took 41 lives away, some of which located at gentle slopes (Fig. 1).

This study focuses on the fact that slope disasters, such as muddy flow and long-distance flow, are triggered by volcanic pumice layers with a high void ratio. In some cases, the void ratio of such volcanic soils, observed at real sites, was too low to reproduce in the laboratory.

Table 1. Typical Japanese volcanic soils [1-5].

|                | Kanto loam | Shirasu | Masado | Scoria |
|----------------|------------|---------|--------|--------|
| Cementation    | Yes        | Yes     | No     | Yes    |
| Porous Particles| No         | Yes     | Yes    | Yes    |
| Dry density    | Small      | Small   | Small  | Small  |
| Max. e         | 3.38       | 1.68    | 1.11   | 1.23   |
| Viscosity      | Large      | Small   | Small  | No     |

Fig. 1. Landslide at Horonai of Atsuma in Hokkaido Eastern Iburi earthquake (2018).

Therefore, such extremely loose soils, with volcanic cementation and crushable particles, were reproduced artificially in the laboratory by adding Portland cement to silty DL clay. The deformation and strength
characteristics of the loose soils were investigated by performing consolidated drained tests and unconsolidated drained tests at different confining pressure levels.

2 Experimental procedure

In general, the micro-structure and fabric of naturally deposited volcanic soils are neither homogeneous nor isotropic; consequently, it is hard to collect a sufficient number of undisturbed samples repeatedly with an equivalent quality. Thus, artificial samples of extremely loose soils were produced in the laboratory using non-plastic clay (DL Clay) and Portland cement, in order to systematically understand the fundamental characteristics of such specific soils. The non-plastic clay was used to consider the fact that volcanic trigger layers of many landslides consisted of non-plastic volcanic soils.

In natural conditions, high void ratios of extremely loose volcanic soils are maintained by cementation effect between particles or by a composition of crushable particles, which have voids within their own particle body. In general, most of extremely loose volcanic soils have crushable particles, and some of them have a cementation effect between particles. In order to examine how cementation or crushable particles affect the overall behavior of shear deformation, artificial loose specimens of both types were prepared (cementation type and crushable particle type). The cementation type soil consists of crushable particles and a cementation effect (Fig. 2(a)), and the crushable type soil consists of uncedmented crushable particles (Fig. 2(b)). It should be noted again that cementation type also has crushable particles.

2.1 Specimen preparation

2.1.1 Cementation type specimen

In this study three kinds of cementation-type specimens (specimens A, B, C) were prepared as follows and the mixing ratio and the designed void ratio were summarized in Table 2.

1. DL clay, a non-plastic fine, was mixed with ordinary portland cement at specific mixture ratios (Table 2).
2. Normal water was added and thoroughly mixed with the material for 10 minutes.
3. The mixture was sieved by using 4.76 mm opening sieve until all the material has passed through the sieve.
4. By applying suction, the mixture was poured loosely into a mould with 5 cm in diameter and 10 cm in height to achieve the designed void ratio.
5. The mould was kept in a moist curing box for 7 consecutive days.
6. Subsequently the mould was kept in a drying oven for 24 hours.
7. Experiments were started immediately after removing the specimens from the drying oven.

By following this procedure, a lot of specimens were produced, having almost identical cementation effect, crushable particle compositions, and density. The prepared samples were used to conduct experiments systematically. Hereafter, specimen A will be considered as the standard (reference) specimen. In comparison, B has less cementation due to smaller amount of cement, and C was characterised by a higher void ratio.

2.1.2 Crushable particles type specimen

In this study two kinds of crushable particle type specimens, Particle A and Particle B were prepared as follows:

1. DL clay was mixed with ordinary portland cement at specific mixture ratios (Table 2).
2. Normal water was added and thoroughly mixed with the material for 10 minutes.
3. The mixture was sieved using 4.76 mm opening sieve until all the material has passed through the sieve.
4. The mould was kept in a moist curing box for 7 consecutive days.
5. The weak cementation between particles under a diameter of 4.76 mm was broken by using hands and 4.76 mm opening sieve.
6. The mould was kept in a drying oven for 24 hours.
7. Specimens (H=10cm, φ=5cm) were prepared by 20 cm air pluviation, and experiments were started immediately after taking samples from the drying oven.

By following above procedure, multiple specimens were produced, having almost identical crushable particle composition and density. The prepared samples were used to conduct experiments systematically. Particle A specimens are considered as the standard (reference) specimen hereafter. In comparison, Particle B specimens have less cementation due to smaller quantities of cement.

![Fig. 2. (a) Cementation between particles (b) Assembly of crushable particles.](image-url)
2.1.3 Physical properties of the specimen

Table 3 shows mean values of the particle densities \( \rho_s \), dry densities \( \rho_d \) and void ratios \( e \) of each specimen types as prepared in the laboratory. The loose DL clay specimen was also prepared to enable a better comparison with the artificial specimens. Although DL clay specimen was made as loose as possible, its void ratio was smaller than that of other artificial specimens. This substantiates that artificial specimens reproduced extremely loose volcanic soil well.

Table 3. Physical properties of the prepared specimens.

| Type          | Specimen | \( \rho_s \) (g/cm\(^3\)) | \( \rho_d \) (g/cm\(^3\)) | \( e \) |
|---------------|----------|--------------------------|--------------------------|--------|
| Cementation   | A        | 2.69                     | 0.89                     | 2.02   |
| Cementation   | B        | 2.66                     | 0.89                     | 1.99   |
| Cementation   | C        | 2.69                     | 0.75                     | 2.61   |
| Crushable particle | Particle A | 2.69   | 0.86–0.87                 | 2.09–2.14 |
| Crushable particle | Particle B | 2.66   | 0.84                     | 2.14–2.16 |
| -             | Loose DL clay | 2.65   | 1.03                     | 1.58   |

2.2 Test programme

Table 4. Experimental cases.

| Test | Specimen | Type | SR(%) | Consolidation Pressure(kPa) | Reconsolidation Pressure(kPa) | B   |
|------|----------|------|-------|-----------------------------|------------------------------|-----|
| A-50 | A        | CD   | 100   | 50                          | 30                           | 0.98 |
| A-100| A        | CD   | 100   | 100                         | 30                           | 0.99 |
| A-200| A        | CD   | 100   | 200                         | 30                           | 0.99 |
| A-250| A        | CD   | 100   | 250                         | 30                           | 0.99 |
| A-300| A        | CD   | 100   | 300                         | 30                           | 0.95 |
| A-400| A        | CD   | 100   | 400                         | 30                           | 0.97 |
| A-CU50| A-CU    | CU   | 100   | 50                          | 30                           | 0.96 |
| A-CU150| A-CU    | CU   | 100   | 100                         | 30                           | 0.99 |
| A-CU250| A-CU    | CU   | 100   | 250                         | 30                           | 0.95 |
| A-CU300| A-CU    | CU   | 100   | 300                         | 30                           | 0.95 |
| B-50 | B        | CD   | 100   | 50                          | 30                           | 0.99 |
| B-100| B        | CD   | 100   | 100                         | 30                           | 0.96 |
| B-200| B        | CD   | 100   | 200                         | 30                           | 0.95 |
| C-15 | C        | CD   | 100   | 15                          | 10                           | 0.99 |
| C-30 | C        | CD   | 100   | 30                          | 10                           | 0.99 |
| C-100| A        | CD   | 100   | 100                         | 30                           | 0.95 |
| pA-20| Particle A| CD   | 100   | 20                          | 10                           | 0.99 |
| pA-200| Particle A| CD   | 100   | 200                         | 10                           | 0.98 |
| pA-CU50| Particle A| CU   | 100   | 50                          | 20                           | 0.96 |
| pA-CU200| Particle A| CU   | 100   | 300                         | 20                           | 0.96 |
| pB-20| Particle B| CD   | 100   | 20                          | 10                           | 0.99 |
| pB-200| Particle B| CD   | 100   | 200                         | 30                           | 0.99 |
| D-100| Loose DL clay | CD | 100 | 100                         | 30                           | 0.97 |
| D-CU100| Loose DL clay| CU | 100 | 100                         | 30                           | 0.98 |

A series of the laboratory tests were carried out using the fully automated triaxial compression apparatus (Fig. 3). The strength characteristics of each artificial extremely high void ratio soil were investigated by performing CD tests and CU tests at several confining pressures. In the consolidation process of the CD tests, a double vacuum method was applied to obtain the Skempton’s pore water pressure parameter B exceeding 0.95. The rate of confining pressure increment was set to 2.5 kPa/min in the consolidation process, and the rate of axial strain increment was set to 0.29 %/min in the shearing process. A summary of the conditions in all experiments are shown in Table 4.

Fig. 3. Outline of triaxial compression test apparatus.

3 Results and discussion

3.1. Results of consolidation process

3.1.1 Consolidation results of Cementation type

The volume change of cementation type specimens during the initial consolidation stage is illustrated in Fig. 4. Specimen A shows low volumetric strain at low confining pressure levels, but its compressibility changes suddenly at around 150 kPa and increases after a sudden jump. It is hypothesized that cementation between particles was lost at this pressure level. A larger compressibility is observed after the loss of cementation, compared with the loose DL clay. This implies that an extremely loose structure was ascribed to this high compressibility. Since cementation between particles were already lost when the high compressibility was observed, it is thought that crushable particles contributed to maintain such a loose structure. Specimen B also exhibits a sudden jump due to loss of cementation at around 70 kPa, followed by a similar compressibility compared with specimen A, with a similar
resultant void ratio. In the case of specimen C, loss of cementation was not observed. This might be due to the high void ratio in specimen C and cementation being too weak to retain against the initial back pressure.

![Fig. 4. Volumetric strain during consolidation stage.](image)

### 3.1.2 Consolidation results of crushable particle type

Fig. 5 shows pseudo consolidation graph obtained by monitoring volume changes between pre-consolidation pressure and consolidation pressure. Although data points obtained for particle B specimens and loose DL clay specimens were limited, 5 points were identified for the particle A specimens. Compressibility of particle A continued to increase without any obvious consolidation yield point. This appeared to be caused by particle crushing.

![Fig. 5. Pseudo consolidation of crushable particle specimens.](image)

### 3.2. Results of CD tests

#### 3.2.1 CD test results of cementation type

CD test results of specimen A are shown in Fig. 6. At confining pressures of 50 and 100 kPa, prior to the loss of cementation, a peak strength was observed at a small strain (Fig. 7). After cementation was lost, specimen A increased its strength gradually as loose soil did. At all confining pressures, significant negative dilatancy was observed. This compressibility is attributed to the loose structure kept by crushable particles.

![Fig. 6. Relationship between (a) stress and strain (b) volumetric strain and axial strain in CD tests of specimen A.](image)

CD test results of specimen B and C are shown in Fig. 8. At a confining pressure of 50 kPa, specimen B showed a peak strength at a low strain because of the cementation effect. In other cases, the cementation effect did not have any visible impact because it was lost in the consolidation process. Comparing with the loose DL clay, high negative dilatancy and compressibility was observed in specimens B and C, as well as specimen A. There is no considerable difference between specimen particle A and specimen particle B, which displays different strengths against particle crushing because of the difference in the amount of ordinary portland cement.
3.2.2 CD test results of crushable particle type

Fig. 9 shows CD test results of crushable particle type specimens, particle A and particle B. High negative dilatancy and compressibility due to crushable particles were observed. Crushable particle type specimens did not show peak strengths at low strain even when the applied confining pressure was very low, because it did not have cementation effect as with cementation type specimen. Particle distribution tests were conducted before and after test pA-200 and results are shown in Fig. 10. It was considered that particle crushing occurred during consolidation or compression process, as fine content was increased after CD test.

3.3 Results of CU tests

3.3.1 CU test results of cementation type

Referring to Fig. 11 for CU test results of specimen A, in the case of low confining pressures of 50 kPa and 100 kPa, the specimen shown a peak strength at a low axial strain, as was observed in CD tests. In all the cases, the deviator stress reaches a residual (steady) state when the axial strain reaches 10%. This behaviour is ascribed to its extremely loose structure maintained by crushable particles, as the cementation effect was already lost in these states. It is hypothesized that once the cementation effect is lost in extremely loose soils, high strains can be developed dramatically within a short time in the field.

3.3.2 CU test results of crushable particle type

CU test results of crushable particle type are shown in Fig. 12. Artificial specimens which were composed of crushable particle shown extremely higher compressive behaviour, in comparison to the loose DL clay. In all the
cases, the deviator stress reaches a residual (steady) state when the axial strain reaches 10%. In the case of pA-CU300, which was conducted at 300 kPa consolidation pressure, peak strength was observed around an axial strain of 2%. This indicates that extreme loose soils, consisting of crushable particles, can show high strengths at small strains, without any cementation effect between particles. This vulnerable behaviour, in which the soil shows a high initial strength, but loses this after subsequent strain increase (showing high compressibility), is considered one of the key factors of landslides.

Fig. 12. Relationship between (a) stress & strain (b) excess pore water pressure & strain in CU tests of crushable particle type.

3.3.3 Effective stress path of all CU tests

In both cementation type case and crushable particle case, the specimen reached to a steady state (Fig. 13). It is presumed that the cementation effect between particles is only worth considering at small strains, even if cementation was not lost during consolidation process. This is because each case converges to almost the same point in their effective stress paths.

Fig. 13. Effective stress path of all CU tests.

4 Conclusions

Crushable particles and cementation effects between particles were investigated, as two characteristic features of extremely loose volcanic soils. Extremely loose volcanic soil kept by cementation exhibits a high shear strength and a low compressibility when cementation is maintained. However, once cementation is lost, a high compressibility and a significant negative dilatancy were observed. This compressibility and negative dilatancy was ascribed to the extremely loose structure due to a composition of crushable particles. In undrained compression tests, after the shear strength reached its peak and the cementation was lost, a brittle behaviour was observed in which the shear strength converges to a residual state, called steady state. This complex behaviour was caused by the extremely loose structure maintained by crushable particles.

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