Cyclic soil tests to propose appropriate seismic design methods for sliding type and slump type failures of road embankments

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ABSTRACT :

On October 23 in 2004, the Niigataken-chuetsu earthquake, of Magnitude 6.8, occurred in Japan. The maximum surface acceleration recorded at Kawaguchi town was 1722 Gals. Many railways including Shinkansen, roads, houses, pipelines and other structures were severely damaged. Similarly highway embankments were damaged. Especially, the damages in two areas: 1) between Horinouchi and Kawaguchi and 2) near Ojiya IC, were serious. In the first area, the highway road was constructed by cut and fill method on the slope of hill. Sliding of filled embankments occurred at several sites. On the contrary, in the second area, highway embankment was constructed by filling soils on lowland plain. Surface soil of the plane is gravel, sand or clay. The reason why the sliding of the embankment in the first area occurred may be due to inertia force and/or excess pore water pressure.
Then the authors conducted many dynamic soil tests to study the detailed mechanism of the failures. Cyclic torsional shear tests were carried out to demonstrate the following items: a) cyclic strength of saturated soil, b) cyclic strength of unsaturated soil, and c) shear modulus of the soil after cyclic loading. Test results showed that cyclic strength and shear modulus of saturated soil was comparatively low. And, under high cyclic shear stress, failure occurred even though the sample was unsaturated. These test results contribute to appropriate seismic design methods of road embankments proposed for sliding type failure and slump type failure.

KEYWORDS: Cyclic strength of soils, Road embankment, Laboratory soil test

1. INTRODUCTION

In Japan the first expressway was opened for traffic in 1963. After that many expressways have been constructed. Total length of the constructed expressways reached 8,273 km in 2007. Several earthquakes, such as the 1978 Miyagiken-oki, the 1987 Chibaken-toho-oki and the 1995 Kobe earthquakes hit the opened expressways. During the 1995 Kobe earthquake, many bridges and elevated bridges were seriously damaged. However, expressway embankments were unsuffered severe damage. The 2004 Niigtaken-chuetu earthquake was the first event that earthquake caused severe damage to expressway embankments. Slope failures and large slump failures of embankments occurred at many sites of Kan-etsu Expressway. Then, detailed soil investigation was carried out to demonstrate the mechanism of the damages to embankments.

2. OUTLINE AND CLASSIFICATION OF THE DAMAGE TO KAN-ETSU EXPRESSWAY EMBANKMENTS

2.1 Outline of the damage to expressways

On October 23 in 2004, the Niigataken-chuetsu earthquake, of Magnitude 6.8, occurred and caused serious damage to many structures and slopes in Japan. Six expressways were closed due to the earthquake. Total length of the closed expressways was 580 km. Emergency treatments were applied to the damaged expressway embankments by filling, placing and spreading. Then all expressways were able to opened for emergency vehicles about 19 hours after the earthquake because no serious damage induced for expressway bridges and tunnels. About 13 days after the earthquake all expressways were opened for every vehicle. Among the inflicted six expressways, most serious damage occurred in the following sections:

a) Between Horinouchi IC and Echigokawaguchi IC (8.8 km), and between Yamamotoyama Tunnel and

Yamaya PA (5.5 km) of Kan-etsu Expressway, and

b) Between Ohzumi PA and Ngaoka JCT (6.0 km) of Hokuriku Expressway as shown in Figure 1

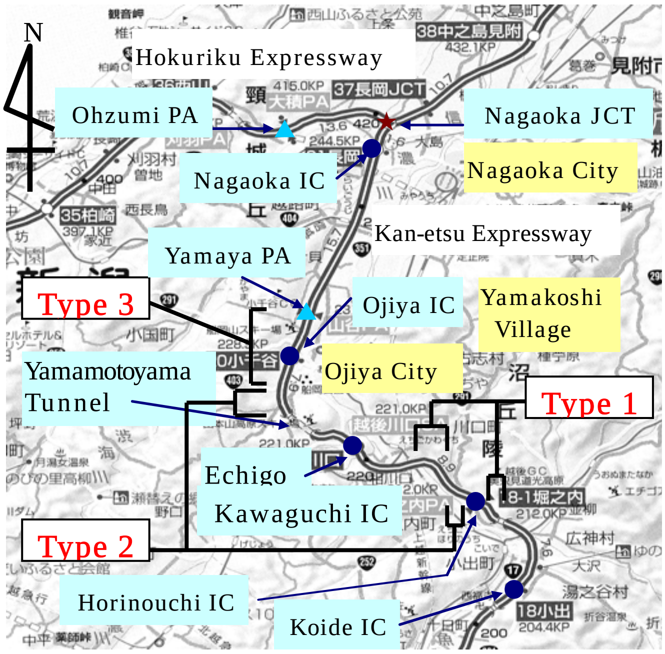


Figure 1 Route map of Kan-etsu Expressway and Hokuriku Expressway

2.2 Topographical condition and seismic motion in severely damaged sections of Kan-etsu Expressway

The section between Horinouchi IC and Echigokawaguchi IC of Kan-etsu Expressway was con-structed on gentle slopes of hills. On the other hands the section between Yamamotoyama Tunnel and Yamaya PA was constructed on flat grounds. In the former section, embankments were constructed mainly by half-bank and half-cut method on the slope of hill. Sliding of the filled embankments occurred at several sites during the 2004 Niigataken-chuetsu earthquake. On the contrary embankments were constructed by filling soils on level grounds in the latter section. Large settlement of the embankments occurred in this section.

Several seismic records were obtained in these severely damaged zones. The recorded maxi-mum surface accelerations were 489 cm/s^2 at Horinouchi Town, 1722 cm/s^2 at Kawaguchi Station, 1502 cm/s^2 at k-net Ojiya site and 1008 cm/s^2 at Ojiya Castle. Therefore it can be said that seismic motion in the severely damaged zones was very strong as the maximum surface acceleration was about 500 to 1700 cm/s^2.

2.3 Classification of the damage

A section between Koide IC and Ojiya IC of Kan-etsu Expressway was selected to study the influence of type of embankments upon the damage to expressway embankments. In this section, embankments were constructed by three methods: filling on a level ground, widening, and half-banking and half-cutting.

In Japan, damage to road embankment is classified in three levels as shown in Figure 2. Then the damage to the embankments of Kan-etsu expressway were classified into three levels and showed in Figure 3.

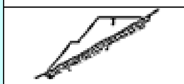
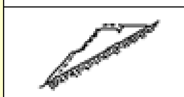
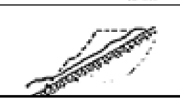
Level of damage	Schematic diagram	Definition of damage
Minor		Surface slide of embankment at the top of slope only
		Minor cracks on the surface of a road
Medium		Deep slide of embankment or slump involving traffic lines
		Medium cracks on the surface of a road and/or settlement of embankment
Serious		Serious slump of embankment
		Serious slide of embankment

Figure 2 A classification method of the level of damage to road embankments in Japan

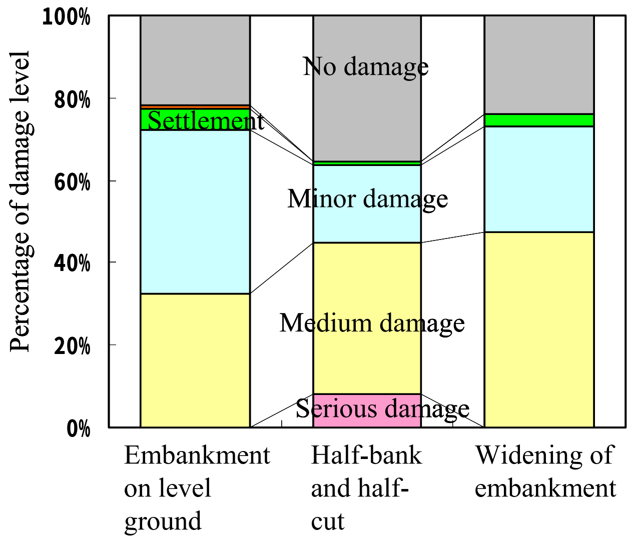
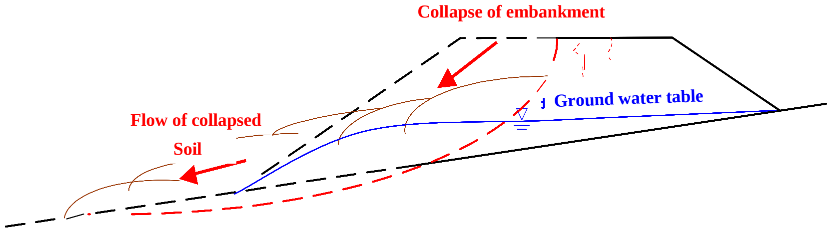
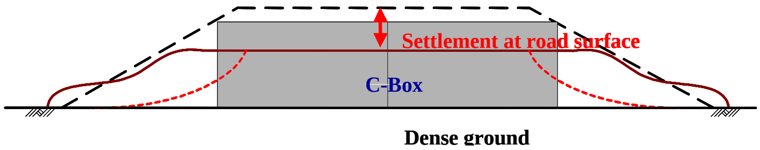


Figure 3 Classification of damage level of embankments

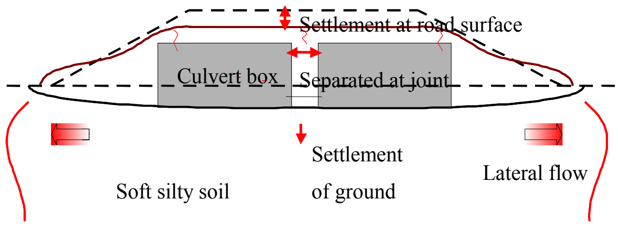
Serious damage occurred at half-bank and half-cut embankment only. In the embankments on level ground, medium or minor damages dominated.
According to the mechanism of failure, the damage of the Kan-etsu expressway embankments in the section between Koide IC and Ojiya IC, can be classified to three types as follows:
(1) Type 1: Serious slide of the embankment on the sloping ground as schematically shown in Figure 4 (a).
(2) Type 2: Settlement of the embankment on the level ground without the deformation of the ground as schematically shown in Figure 4 (b)
(3) Type3: Settlement of the embankment and the culvert on the level ground with the deformation of the ground as schematically shown in Figure 4 (c)
Locations where these types of failures occurred are shown on Figure 1.



(a) Type 1: Serious slide of the embankment on the sloping ground



(b) Type 2: Settlement of the embankment on the level ground without the deformation of the ground



(c) Type3: Settlement of the embankment and the culvert on the level ground with the deformation of the ground

Figure 4 Classification of the damage to the embankment of Kan-etsu Expressway according to the mechanism of failure

3. SOIL CONDITIONS AND ESTIMATED MECHANISM OF THE DAMAGED EMBANKMENTS

3.1 Type 1: Serious slide of the embankment on the sloping ground

Serious slide of embankment occurred between Koiede IC and Kawaguchi IC as shown in Figure 1. The expressway was constructed on gentle slopes of hills. As the expressway crosses several small valleys, embankments were constructed by filling soils on the valleys. Severe slide occurred at these sites as shown in Photo 1. Detailed soil investigation was conducted at 214.35 KP where the most severe damage occurred. Figure 5 shows the cross section at the site. Embankment was constructed on a gentle slope of about 5 degrees. A thin gravelly clay layer is deposited on the surface of the slope. Dense gravelly soils are underlaid. Filled material is clayey gravel with 20 to 30 % of fines. SPT *N*-values of the fill were 2 to 10. According to the measurement of ground water level conducted about two month after the earthquake, water level was about 2 m higher than the bottom of the fill. However, the measured water level is not accurate because retaining sheet piles had been constructed for emergency treatment before the measurement of water level. In addition to the soil investigation, laboratory tests and analyses were carried out to demonstrate the mechanism of failure. As shown in Figure 4 (a), lower part of the fill was saturated. The filled soil contains not so much fines and

Photo 1 Serious slope failure on the sloping ground at 214.35 KP

comparatively easy to induce excess pore water pressure due to shaking. Therefore it is estimated that some excess pore water pressure was generated in the saturated part of the fill and caused slide and subsequent flow towards downstream.

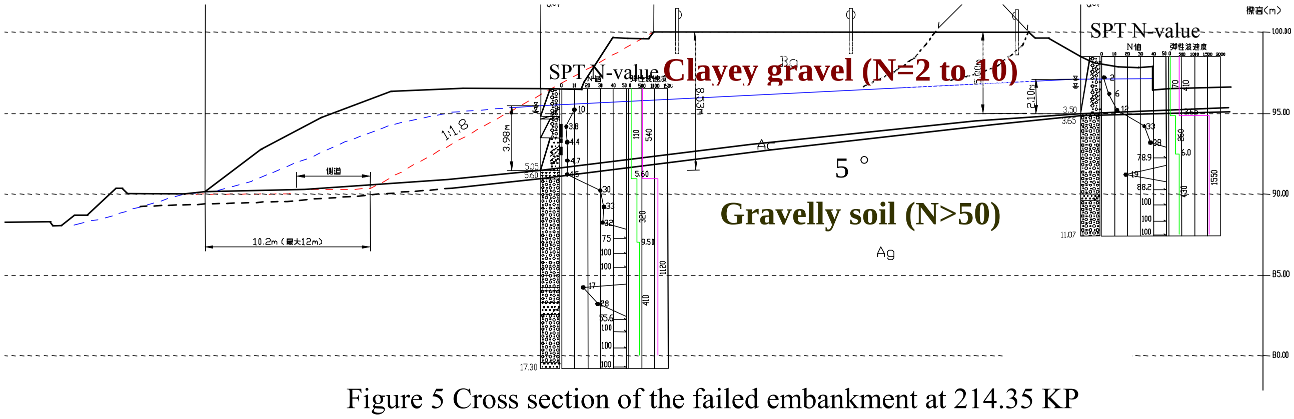


Figure 5 Cross section of the failed embankment at 214.35 KP

3.2 Type 2: Settlement of the embankment on the level ground without the deformation of the ground

Photo 2 shows C-Box Kawaguchi 11. Large differential settlement of 70 cm occurred between the embankment and the culvert box as shown in Photo 5. The culvert box consists of two concrete boxes. The culvert itself settled 10 cm only and joint of two concrete boxes opened 10 cm. Therefore it can be said culvert box settled and stretched slightly as schematically shown in Figure 4(b).

Figure 6 shows boring data and the estimated soil cross section of the embankment adjacent to C-box Kawaguchi 11. Subsurface soil of the foundation ground is dense gravel with SPT *N*-value of more than 50. Height of the embankment is about 10 m. Filled materials are sandy silt with gravel, gravel with cobbles and clayey silt with gravel. Fines content of these soils was 50 to 60 %. Measured water level was about 3 m higher than the bottom of the embankment. However, it is not clear whether the water was perched water or not.

Photo 2 C-Box Kawaguchi 11

Photo 3 Differential settlement between embankment and culvert at C-Box Kawaguchi 11

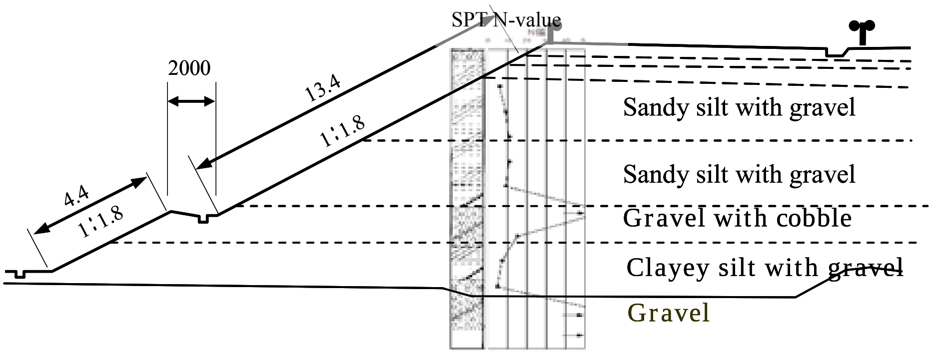


Figure 6 Soil cross section adjacent to the culvert at C-Box Kawaguchi 11

Based on the soil investigation and additional laboratory tests, it is estimated the settlement of the embankment occurred due to decrease of shear modulus of the filled materials.

3.3 Type3: Settlement of the embankment and the culvert on the level ground with the deformation of the ground

Around Ojiya IC, differential settlement of several ten cm occurred between embankments and culverts as shown in Photo 4. Moreover culverts settled several ten cm and stretched as shown in Photo 5. Embankment soil fell down through the opened joints. Embankments, culvert boxes and grounds deformed as schematically shown in Figure 4 (c).

Photo 4 Differential settlement at C-Box Kawaguchi 22

Detailed soil investigation was conducted at C-Box Kawaguchi 22 and C-Box Ojiya 2 to demonstrate the mechanism of the Type 3 failure (Inagaki et al., 2005). Figure 7 shows locations of two sites, together with K-net Ojiya Site, where accelerograph is installed. The maximum surface acceleration recorded at K-net Ojiya Site was 1314 cm/s^2 in EW direction. Surface soil conditions at C-Box Kawaguchi 22, investigated after the earthquake, are shown in Figures 8. Embankment soils at C-Box Kawaguchi 22 are clayey soils with 70 to 80 % of fines. SPT N-values of the embankment soils are 5 to 10. Heights of the embankments at two sites were 5.6 to 6.8 m, respectively. Water levels at two sites were about 3 m higher than the bottom of the embankments, though the embankments were constructed on level grounds.

Photo 5 Separation of joint at C-Box Ojiya 2

At C-Box Kawaguchi 22, thick soft silty layers, with about 5 of SPT *N*-values, are deposited from original ground surface to the depth of 16 m. A thin soft silt layer with 2 m thickness is deposited under the embankment at C-Box Ojiya 2. Then, silty sand, silt, sandy silt and silt lay ers, with 10 to 20 of SPT *N*-values, were underlaid to the depth of 24 m. Figure 18 shows estimated cross section at C-Box Kawaguchi 22.

As mentioned before, large settlements of embankments and culverts occurred at these sites. Differential settlements between embankments and culverts were 20 cm and 70 cm at C-Box Kawaguchi 22 and C-Box Ojiya 2, respectively. Settlements of culverts were 48 cm and 30 cm, respectively. Therefore, total settlements of embankments were 68 cm and 100 cm, respectively.

Cyclic torsional shear tests were carried out to obtain cyclic shear strength and shear modulus after cyclic loading. Then an analytical code “ALID (Yasuda et al., 2003)”, which is one of the residual deformation methods, was applied to evaluate deformation of embankments and grounds.

Analyzed settlements and horizontal displacements were fairly coincided with

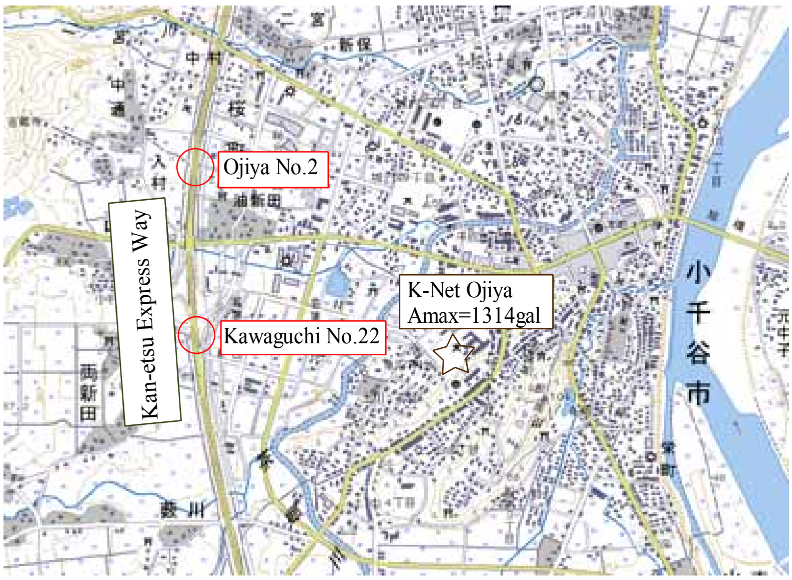


Figure 7 Location of investigated sites

the measured values, then it was concluded that the settlement of the embankments and culverts occurred due to the reduction of shear modulus of the filled soils and foundation soils (Inagaki et al., 2005).

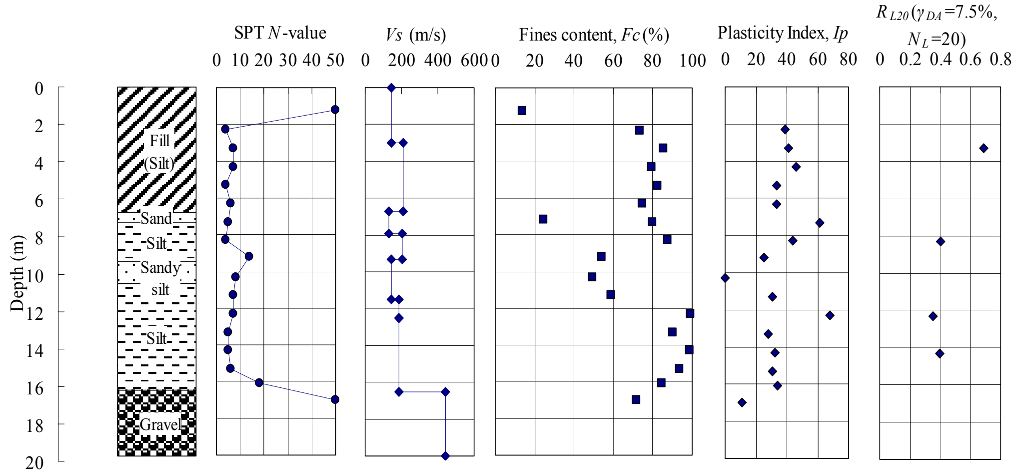


Figure 8 Soil profile and tests results at C-Box Kawaguchi 22

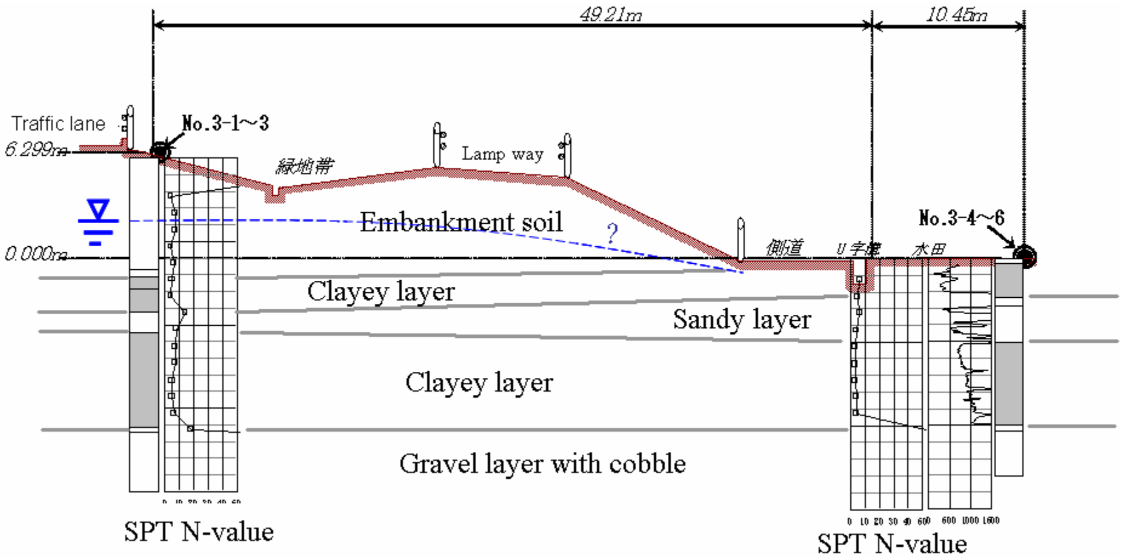
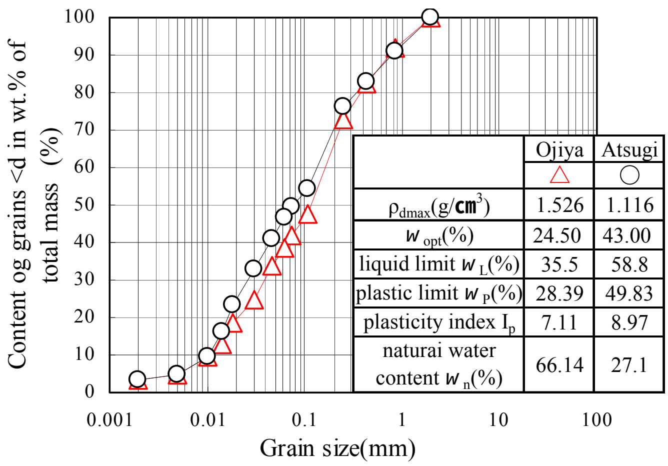


Figure 9 Cross section of the embankment adjacent to C-Box Kawaguchi 22

4. CHARACTERISTIC AT CYCLIC STRENGTH OF SOILS

4.1 The experiment sample which we used

The authors selected two sites for detailed soil investigation to demonstrate the mechanism of the failures. The one is Ojiya of the Kan-etsu highway. Other site is Atsugi of the Tomei expressway. In Ojiya site, sandstones and the mudstones weathered, and in Atsugi site volcanic cohesive soils called Kanto loam are used as embankment materials. Figure 10 shows grain-size distribution curves of Ojiya and Atsugisites.



	Ojiya △	Atsugi ○
ρ_{dmax}(g/cm³)	1.526	1.116
w_{opt}(%)	24.50	43.00
liquid limit w_L(%)	35.5	58.8
plastic limit w_P(%)	28.39	49.83
plasticity index I_p	7.11	8.97
naturai water content w_n(%)	66.14	27.1

Figure 10 Grain-size distribution curves of Ojiya and Atsugi sites

The grain distribution of both soil are almost the same, but, as for the materials of Atsugi, and the optimum moisture content of Ojiya soil are clay-like and considerably great value.

4.2 Cyclic soil torsional shear test method

Authors used a repeatedly cyclic torsional shear machine on hollow cylindrical specimens having an electric motor for monotonous loading. In the tests, samples were compacted to the degree of compaction 90% with the optimum moisture content. And trimmed to become hollow cylindrical specimens outer, inner diameter and height of the specimen were 10 cm, 6 cm and 10 cm, respectively. Cyclic soil tests carried out two cases, made the specimen Case 1 so that pore water pressure coefficient was more than 0.95, afterwards consolidated by effective confining pressure of 50kPa. In Case2, consolidated as keeping to optimum moisture content. After the consolidation, 20 cycles of cyclic loading with 0.1 cyclic/sec. was applied to the specimens as shown in Fig.11 under undrained condition. Then, a monotonic loading was applied under undrained condition with a speed of 10% of shear strain/minute.

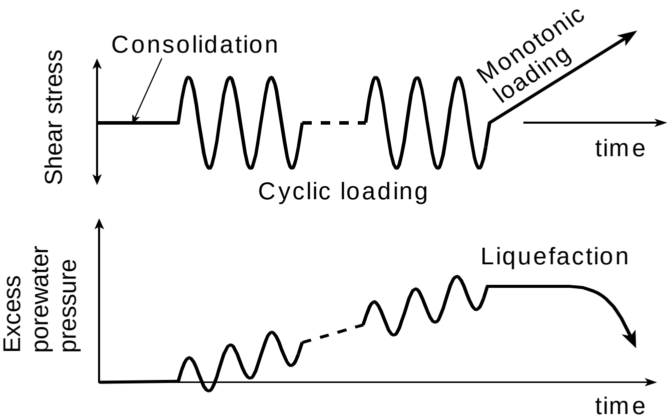


Figure 11 Procedure of cyclic and monotonic loading

4.3Result and consideration

Time histories of shear stress, Shear strain and pore water pressure during the monotonic loading were measured. About 4 to 8 specimens were used in one sample. And, different amplitude of cyclic loading was applied to each specimen to control safety factor against liquefaction, F_L, which implies severity of liquefaction or failure, mentioned later. In addition, static tests to apply monotonic loading only were carried out. These tests were called as "static" hereafter. As mentioned above, different amplitude of cyclic loading was applied to each specimen. Then, relationships between cyclic stress ratio, τ_d / σ_c'and double amplitude of shear strain at 20th cycle, γ_{DA} *(N=20)* were plotted. And, the stress ratio to cause 7.5% of shear strain by 20 cycles, R_L (γ_{DA}=7.5%, N_L=20) was estimated. This stress ratio is same as the stress ratio to cause liquefaction, R_L (ε_{DA}=5%,N_L=20) in cyclic triaxial tests for soils. Therefore, this stress ratio means liquefaction strength in soils.

Figure 12 shows relationships between cyclic shear resistance content, R and γ_{DA}*(N=20)* about Case 1. Soils at Atsugi were R_L=0.28 when read cyclic shear resistance content, R_L from figure 12. As a result, saturated soil of Atsugi is the materials which are hard to liquefy too much. Soil of Ojiya became R_L=0.178 and were easy to liquefy and it showed a fairly small value more than soil of Atsugi. Figure 13 shows R and γ_{DA}*(N=20)* about Case 2. Soils at Atsugi were R_L=1.059 when read R_L from figure 13, and the unsaturation soil of Atsugi shows hard to failure very much. Soil of Ojiya became R_L=0.67. Then in the case

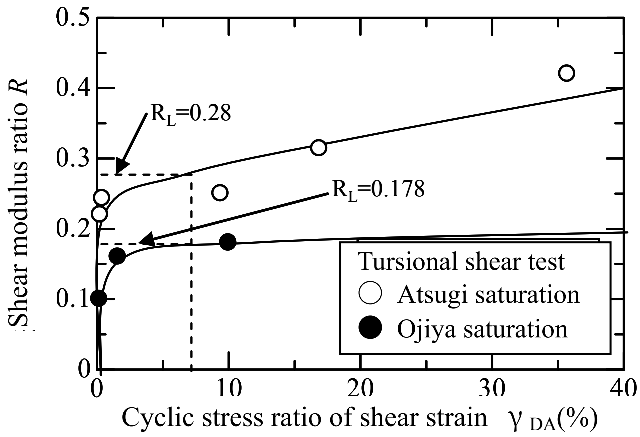


Figure 12 Relationship between R and γ_{DA}(Case 1)

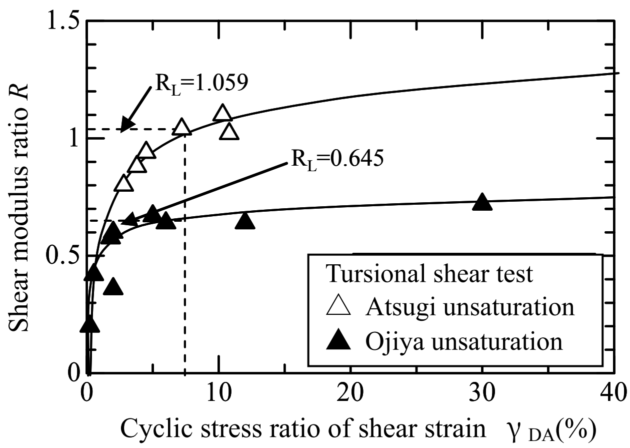


Figure 13 Relationship between R and γ_{DA}(Case 2)

of the unsaturation compared it with soil of Atsugi and showed a small value. Authors predicted unsaturated embankments at Atsugi don't collapse unless the shear power that is considerably strong increases.
Figure -14 shows relationships between shear modulus,G_1 and safety factor of fliquefaction resistance, F_L(In case of unsaturation, it shows safety factor of cyclic shear failure) in Case 1.
Stress-strain curves in case of the monotonous loading defined shear modulus of soil as the secant modulus at 1% of shear strain, G1 here (Yasuda et al., 2004).
Shear modulus declined significantly in the vicinity of F_L=1.0 both of Atsugi and Ojiya. In addition, Ojiya becomes the low value on the whole.
Figure -15 shows relationships between shear modulus, G_1 and safety factor of liquefaction resistance, F_L in Case 2. Shear modulus by unsaturated soil are more strong than saturated soil, in the vicinity of F_L=1. These results are thought related to the size of the plasticity index.

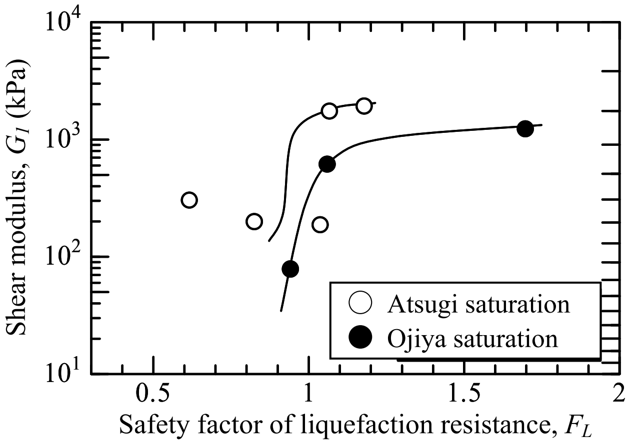


Figure 14 Relationship between G_1 and F_L(Case 1)

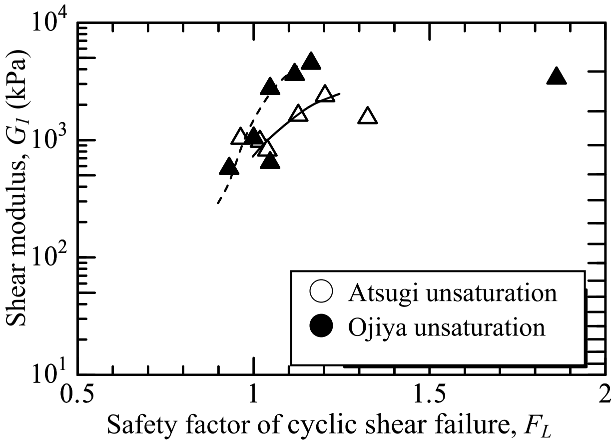


Figure 15 Relationship between G_1 and F_L(Case 2)

5.CONCLUSIONS

The 2004 Niigataken-chuetsu earthquake caused serious damage to highway embankments of Kan-etsu Expressway in Japan. The damage is divided into three types. In Type 1, some excess pore water pressure had to be generated in the saturated part of the fills and caused slide and subsequent flow towards downstream.
In Type 2 settlement of the embankment had to be induced due to the reduction of shear modulus of the filled materials. And the reduction of shear modulus of soils of the foundation grounds had to be added in Type 3.
The other hand, test results showed that cyclic strength and shear modulus of saturated soil was comparatively low. And, under high cyclic shear stress, failure occurred even though the sample was unsaturated.
It is thought that influences of the water are great as the cause why the damage becomes large.
Appropriate drainage processing and enough compaction are necessary to build stable embankments, and steer to satisfy demanded earthquake resistance, and we have to account on the basis of the characteristic of earth structure has exercise to the maximum such as the stability and retrivalty.

6. REFERRENCE

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