Soil investigations and soil tests required for seismic inspection of abandoned tailings dams in Japan

S. Yasuda
Tokyo Denki University, Saitama, Japan, yasuda@g.dendai.ac.jp

ABSTRACT: Japan has mined many metals for centuries. Most of the mines has been closed due to the low total amount of ore deposits, leaving many abandoned tailings dams. Japan compiled its first design code for tailings dams in 1954. A tailings dam damaged due to the liquefaction of tailings materials in the 1978 Izuoshima-kinkai Earthquake. After the earthquake, many cyclic triaxial tests were conducted to study the liquefaction strength of the tailings. Based on the results, the code was revised in 1980. The existing tailings dams were examined and strengthened if needed, in accordance with the reviced code. Despite these inspections, three tailings dams failed during the 2011 Great East Japan Earthquake. The main reason for the failures was strong shaking. Based on detailed investigations, the seismic design code was revised again in 2012 to ensure the stability of tailings dams during strong earthquake motion. Failures of tailings dams during past earthquakes, the history of the revisions of seismic design code and examples of soil investigations, soil tests and analyses are introduced in this paper.

Keywords: soil investigation, soil test, tailings dam, earthquake

1. Introduction

In the process producing gold, copper, lead and other metals, the mother ore is broken by mills into fine grains, such as fine sands or silts. Metals are then separated by special techniques. As the metals contents are very low, such as 0.1% or less of the mother rocks, many remaining grains have to be disposed into tailings dams. In the process of disposal, the grains are separated into coarse grains and fine grains, mixed with water, and then transported to tailings dams. The coarse grains, named "sand", are used to construct dam bodies by filling them on a starter dam, as shown in Fig. 1. Fine grains, named "slime", are discharged into ponds without compaction, as shown in Fig. 2. Therefore, in general, the slime is highly liquefiable during operation of the tailings dams because it is loose, cohesion-less and saturated. After a tailings pond and dam cease operation, the liquefaction resistance of the slime slightly increases because the water table drops below the ground surface and liquefaction strength increases due to aging. Despite this, the abandoned tailings dams can fail due to liquefaction of the slime.

Figure 1. Construction of a tailings dam by filling "sand" at Veta del Agua in Chile

Figure 2. A pond filled with discharged "slime" at Veta del Agua in Chile

There are mainly three methods to construct tailings dams as shown in Fig. 3. Among them, the tailings dams constructed by inner filling method are usually weaker during the earthquake than the dams constructed by outer filling method.

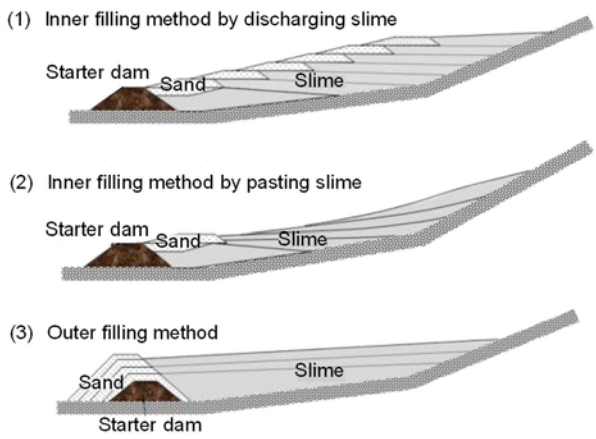

Figure 3. Three methods to construct tailings dams

In this paper, the failures of tailings dams during past earthquakes in Japan and the history of the developement of a design code for tailings dam in Japan are introduced. Japan has mined gold, silver, copper, lead, and other metals for centuries. In the 13th century, Marco Polo named Japan "Zipangu (Cipangu)", which became "Japan" in English. He mentioned in his book, *La Description du Monde*, that a lot of gold was produced in Zipangu, and the palaces and houses there were covered with gold. However, Japanese mines usually contained small amounut of overall ore deposits. Most of them had been closed by the end of 20th century, leaving many abandoned tailings reservoirs in mountainous areas. Now, the Japanese Ministry of Economy, Trade and Industry (METI) lists 388 major tailings dams in Japan. About one-third of these dams were constructed by the inner filling method.

2. The 1978 Izuoshima-kinkai Earthquake

The 1978 Izuoshima-kinkai Earthquake, with a magnitude of Mj=7.0, caused serious damages to many natural slopes and a tailings dam at Mochikoshi Mine. The author and his colleagues visited the Mochikoshi Mine one week after the earthquake. Many sand volcanos were observed in the pond of the Houzukizawa tailings dam, as shown in Fig. 4. Fig. 5 shows the temporarily repaired Houzukizawa tailings dam. The tailings dam was constructed by the inner filling method, as shown in Fig. 6. It was considered that the dam failed due to the liquefaction of slime. About 80,000 m^3 of slime and coarse tailings flowed out of the dam and down to a river and futher into a bay, as shown in Fig. 7. The river and the bay were contaminated by sodium cyanide, that killed many fishes.

Figure 4. Sand volcanos observed at the pond of the Houzukizawa tailings dam

3. The history of seismic design code before 2011 Great East Japan Earthquake

Table 1 shows the years and sites of tailings dams in Japan suffered major damages since 1936, and the history of the development of a design code for tailings reservoirs and their inspection in Japan. Mosy of the damages were triggered by earthquakes and leakages. Japan compiled its first code for the construction of reservoirs for waste soils and tailings in 1954 and revised the code in 1973 to include associated drainage facilities. Based on the revised code, all licensed tailing dams were inspected from 1974.

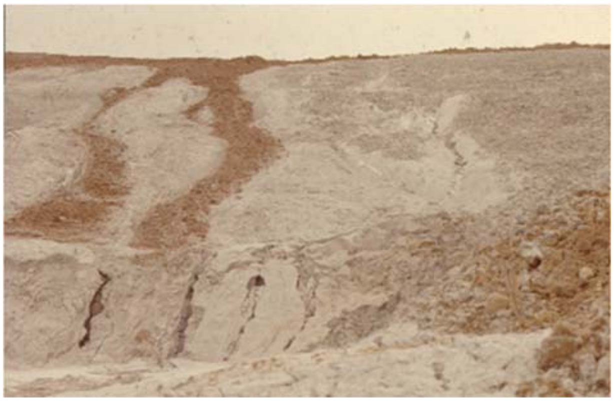

Figure 5. Temporarily repaired Houzukizawa tailings dam at the Mochikoshi Mine

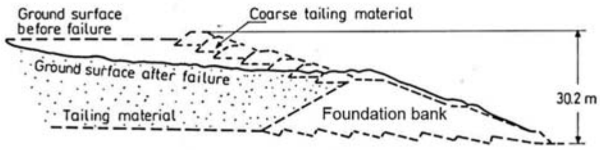

Figure 6. Cross section of the Hoizukizawa tailings dam

Figure 7. Flowing slime in the Kano River

The 1978 Izuoshima-kinkai Earthquake seriously damaged the Houzukizawa tailings dam due to the liquefaction of tailings materials as mentioned above. After this disaster, Japan started to study the liquefaction strength of tailings. Many cyclic triaxial tests and seismic response analyses were conducted (Ishihara et al. [1]). Based on the results of these tests and analyses, the design code was revised in 1980 to include the effect of liquefaction on the relevant structures (Japanese Mining Industry Association [2]).

The revised code of 1980 required the calculation of the in-situ cyclic shear resistance ratio, R, and the shear stress ratio during an earthquake, L, at every 2 m of depth in layers that are considered to influence the stability of the structure if liquefied. Using these values, the safety factor against liquefaction, F_L, is obtained by the following equation:

$$F_L=R/L \tag{1}$$

wher R is caluculated from the cyclic shear resistance ratio, R_L, obtained by cyclic triaxial tests, through the formula:

$$R = 1.2R_L \tag{2}$$

Table 1. Years and sites of tailings dams in Japan significantly damaged since 1936, and the history of the development of a design code for tailings reservoirs in Japan and their inspection

Tailings dams suffering major damage		History of the development of design code and inspection
Year and name of mine	Trigger of the damage	
1936, Osarizawa	Leakage	1954, Established a design code for the construction of tailings reservoirs
1968, Toyoha	Earthquake	1973, Revised the design code
1978, Mochikoshi	Earthquake	1974, Inspected existing tailings dams
1980, Tokyo-sekkai	Leakage	1980, Revised the design code to consider the effect of liquefaction
2011, Ohya, Takatama and Ashio	Earthquake	1982, Revised the design code
2016, Taio	Earthquake	1986, Inspected existing tailings dams
		2012, Revised the design code to ensure the stability of tailings dams during Level 2 earthquake motion

Table 2. Rough estimation of cyclic shear resistance ratio, R_L, from SPT N-value

Fill material or deposits	Cyclic shear resistance ratio, R_L
(1) Waste soils and tailings from black ore ("Kuro-kou") deposits	$R_L = 0.088\sqrt{\dfrac{N}{0.1\sigma_V' + 0.7}} + 0.20$
(2) Waste soils or tailings other than those from black ore deposits	$R_L = 0.088\sqrt{\dfrac{N}{0.1\sigma_V' + 0.7}} + 0.085\log_{10}\dfrac{0.50}{D_{50}}$ The minimum value of R_L is set at 0.15.
(3) Deposits containing sediments	$R_L = 0.088\sqrt{\dfrac{N}{0.1\sigma_V' + 0.7}} + 0.10$ When N is less than 1, R_L is estimated to be 0.

where N = measured SPT N-value, σ_v' = effective overburden pressure (in tf/m^2),
D_{50} = mean particle diameter of the soil (in mm)

If cyclic triaxial tests are not conducted, R_L can be roughly estimated from SPT N-value using the equations shown in Table 2. L is obtained by a seismic response analysis when the dam could adversely affect public safety and is located in an area with high seismicity. Otherwise it can be roughly evaluated by the formula:

$$L = \frac{4}{3}K_h\frac{\sigma_v}{\sigma_v'}(1 - 0.025Z) \quad (3)$$

where K_h is the horizontal seismic coefficient used in the seismic design (0.10 to 0.15, as shown in Table 3) , σ_v is the overburden pressure (in tf/ m^2), σ_v' is the effective overburden pressure (in tf/ m^2), and Z is the depth from the surface of the dam (in m).

Table 3. Seismic coefficients K_h used in the design of structures

Type of foundation bank	Zone of seismic activity	
	High	Low
Concrete	0.12	0.10
Others	0.15	0.12

The excess pore water pressure, U_L, generated during liquefaction is evaluated from the F_L value using the following relationship:

$$U_L = \begin{cases} 0 & (F_L > 1.25) \\ 0.3\sigma_V' & (1.00 \le F_L \le 1.25) \\ \sigma_V' & (F_L < 1.0) \end{cases} \quad (4)$$

The code requires that a stability analysis of each dam be conducted assuming a circular slip surface as shown in Fig. 8. The pressure U_L is applied to the sliding surface of each slice according to the following formula:

$$F_S = \frac{\Sigma r[c's + \{(W - U_L b)\cos\alpha - K_h W\sin\alpha\}\tan\phi']}{\Sigma(rW\sin\alpha + K_h Wy)} \quad (5)$$

where c' is the cohesion and ϕ' is the angle of shear resistance.

Existing tailings dams were examined for seismic safety in accordance with the revised code and were strengthened to increase their safety as needed.

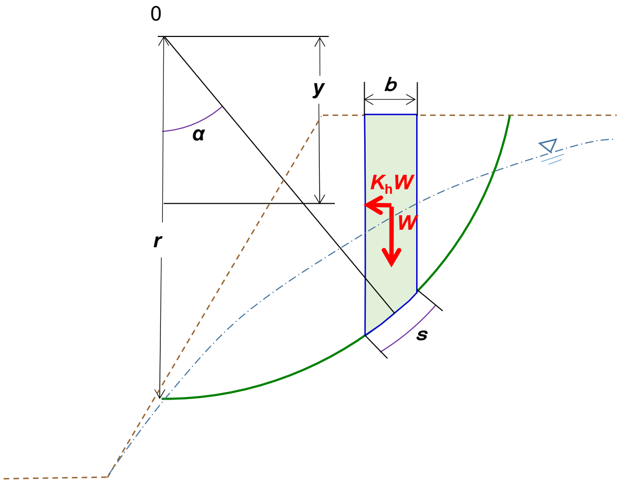

Figure 8. Stability analysis method intrduced in the code

4. *The 2011 Great East Japan Earthquake*

The 2011 Great East Japan Earthquake (Tohoku Earthquake), with a magnitude of M_W=9.0, occurred in the Pacific Ocean about 130 km off the northeast coast of Japan's main island on March 11, 2011. The hypocentral region of this quake was about 500 km in length and 200 km in width. Many structures were severely damaged in the Tohoku district of northeastern Japan and in the Kanto district surrounding Tokyo. Seismic intensity in the affected area, as measured by the Japanese Meteorological Agency (JMA) scale, was 5 to 7, which corresponds to an intensity of 7 to 11 according to the Modified Mercalli (MM) scale. In the geotechnical field, three tailings dams (Kayakari, Zenigami and Gengorou tailings dams, shown in Fig.9) failed, many houses and lifelines were damaged by soil liquefaction, landslides occurred, agricultural dams failed, and river dikes settled.

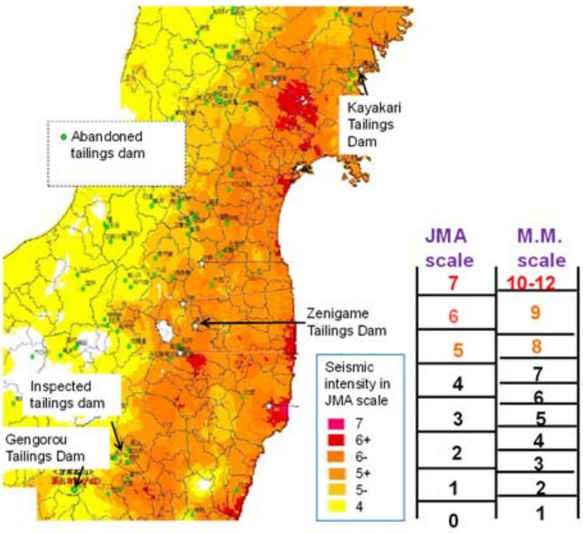

Figure 9. Locations of the three failed tailings dams and of tailings dams that did not fail (quoted from Technical Committee for Administration of Reservoir of Waste Soils and Tailings [3])

Detailed soil investigations, laboratory tests, seismic response analyses, slope stability analyses and deformation analyses were carried out on the failed tailings dams shortly after the earthquake to ascertain the mechanism of failure and to select appropriate methods to restore them. The locations of the three failed tailings dams and those did not fail are shown in Fig. 9. The dimensions of the failed dams are shown in Table 2 (Technical Committee for Administration of Reservoir of Waste Soils and Tailings [3]).

Table 4. Evaluation in the code of 1980 and dimensions of three failed tailings dams (partially quoted from Technical Committee for Administration of Reservoir of Waste Soils and Tailings [3])

		Name of tailings dam		
		Kayakari	Zenigami	Gengorou
Metals and minerals mined		Gold, silver, arsenic, copper, iron sulfide	Gold	Gold, silver, copper, lead, zinc
Period of operation		1951 to 1966	1939 to 1972	1943 to 1959
Area (m^2)		55,200	68,800	7,263
Volume (m^3)		400,000	673,600	161,995
Foundation bank	Type	Soil with RC wall	Soil	Rock
	Height (m)	14.7	10	7
	Length (m)	206	552	370
	Angle (deg.)	13	18	34
Height of tailings dam (m)		36.3	95	16.4
Ground water table		Shallow	Shallow	Tailings are saturated
Results of inspections in 1974 to 1981	Seismic Fs	1.38 (1974), 1.28 (1981)	1.47 (1974)	1.5 (1974)
	F_L	1.8 to 3.4	No liquefaction	No liquefaction
Existing structures downstream		Houses and fields 500 m downstream	Road and river just downstream	Railway and river just downstream

Figs. 10 and 11 show the failed slope and its cross section, respectively. The dam was constructed by the inner filling method by discharging slime. Sand volcanoes were observed on the ground surface of the pond. Based on detailed soil tests and analyses, it was concluded that slime above the foundation bank slid due to liquefaction (Ishihara et al. [4]), as shown in Fig. 11, and flowed 1 km downstream into houses and fields. The river downstream was contaminated with arsenic. In the restoration work, the slime behind the starter dam was treated by mixing it with cement to prevent liquefaction and the starter dam was covered by stone mass make it higher and stronger as shown in Fig.12, based on the analysis results of stability of dam slope under Level 2 earthquake motion as specified mentined later.

Figure 10. Failed upper slope of the Kayakari tailings dam

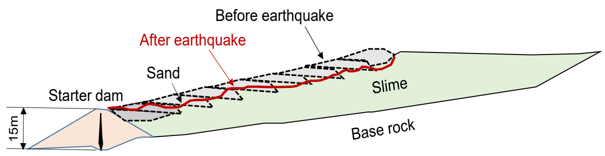

Figure 11. Cross section of the Kayakri tailings dam

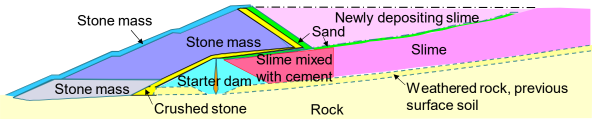

Figure 12. Restored cross section of the Kayakri tailings dam

Figs. 13 and 14 show the failed slope and its cross section of the Zenigami tailings dam at the Takatama Mine, respectively. The dam was constructed by the inner filling method by pasting slime. No sand volcano was observed on the failed slope. Based on detailed soil tests and analyses, it was concluded that the tailings in the upper slope of the dam failed, as shown in Fig. 13, and flowed over a road and into a river. The road was temporarily closed. In the restoration work, a new starter dam was constructed slightly higher than the existing starter dam. The failed tailings were then filled behind the new starter dam at a gentle slope to prevent sliding failure in the future earthquakes. A counterweight fill was heaped on the downstream slope of the existing starter dam to prevent sliding during earthquakes.

Figure 13. Failed upper slope of the Zenigami tailings dam

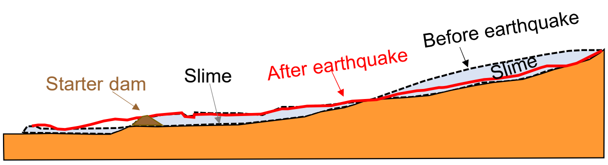

Figure 14. Cross section of the Zenigami tailings dam

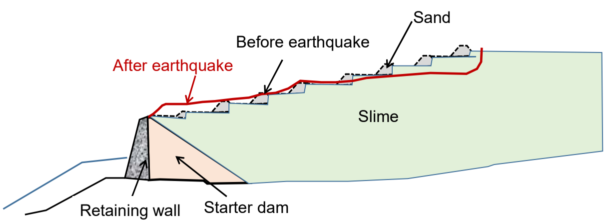

Figure 15. Cross section of the Gengorou tailings dam

Fig. 15 shows a cross section of the Gengorou tailings dam at the Ashio Mine. The dam was constructed by the inner filling method by discharging slime. Slime above the foundation bank slid and flowed over a railway and into a river. The railway was temporarily closed. The river was contaminated by lead.

The failed dams shared some common seismic and ground conditions (Technical Committee for Administration of Reservoir of Waste Soils and Tailings [3]):

1) They were subjected to intense shaking (more than 5 on the JMA scale) for a very long duration more than 2 minutes.
2) Only the upper slopes of the dams, constructed of tailings, failed. The lower foundation banks did not fail.
3) They were constructed by the inner filling method. The dams were high and with steep slopes. The ground water tables were shallow or the tailings were saturated.
4) Strong, long shaking lowered the shear strength of the tailings and caused the failures. Liquefaction occurred in sandy slime of the Kayakari tailings dam.

5. Revised application of seismic design code after the Great East Japan Earthquake (Partially quoted from Yasuda et al. [5])

As shown in Table 4, the three tailings dams that failed during the Great East Japan Earthquake had been investigated once or twice before the earthquake and judged to be stable against potential earthquake motion because their safety factor against sliding, F_S, was greater than 1.0. Moreover, soil investigations conducted after the earthquake showed that the equations in Table 2 were effective to estimate R_L roughly. Therefore, a revision of the seismic design method was considered unnecessary, but it was considered necessary to inspect the following types of tailings dams deemed susceptible to damage by very strong Level 2 earthquake motion.

1) Tailings dams constructed by the inner filling method with a reservoir surface higher than the foundation bank and with a slope steeper than 15 degrees.
2) Dams with a ground water table shallower than GL-10 m.

Very intense shaking of 0.6 g to 0.8 g occurred during the 1995 Kobe Earthquake, causing many buildings, bridges, and houses collapsed. After this earthquake, the Japan Society of Civil Engineers organized a technical committee to investigate new design concepts that could withstand very strong shaking. This committee suggested to base the earthquake-resistant design on two types of ground motion: Level 1 earthquake motion, which is likely to strike a structure once or twice while it is in service, and Level 2 earthquake motion, which is extremely strong but very unlikely to strike a structure during its lifetime. In the design of embankments to withstand Level 1 earthquake motion, it is not always necessary to estimate the degree of damage because it is easy to improve the embankment slopes to prevent failure under this level of shaking. Under Level 2 earthquake motion, on the contrary, the slight sliding or deformation of embankments cannot be prevented by current countermeasures. Therefore, it was considered necessary to introduce a new design concept based not on the occurrence of sliding but on the likely degree of damage to structures from sliding.

This new design concept, a so called performance-based design, is rational. In performance-based design, two items must be decided: 1) the allowable deformation or displacement of embankments, and 2) the relevant method to estimate the deformation. After the Kobe Earthquake, three methods of estimating deformation were developed for water reservoir dams, river dikes, road embankments and railway embankments: empirical methods, static (residual deformation) analyses and dynamic (seismic response) analyses. It was recommended that these new techniques be applied in the inspection of tailings dams deemed susceptible to failure. The allowable deformation of these dams must be judged as unlikely to cause serious damage to structures, such as houses, if the dams were deformed. The recommended measures to counter dam deformation are shown in Fig. 16.

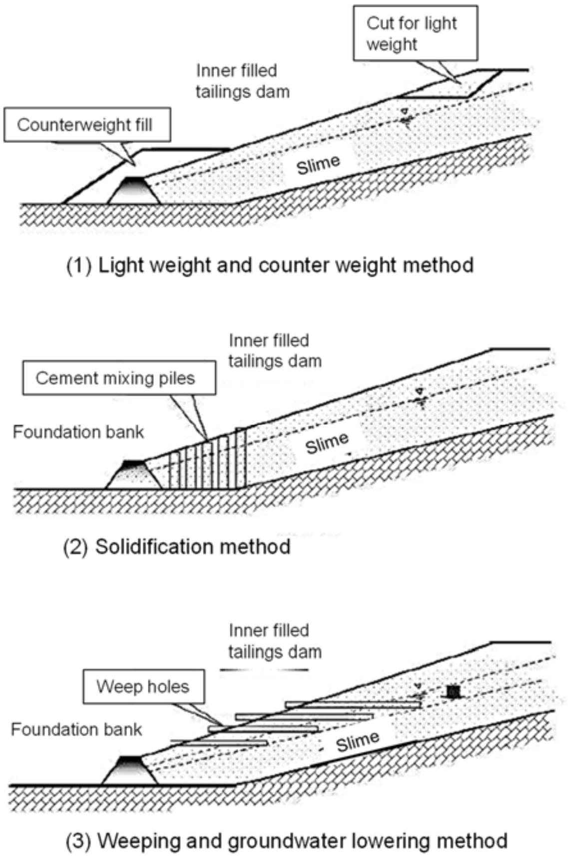

Figure 16. Appropriate measures to counter the deformation of tailings dams

6. *Inspection by the application of the new design code (Partially quoted from Yasuda et al. [5])*

The Japanese Ministry of Economy, Trade and Industry (METI) lists 388 major tailings dams in Japan. About one-third of these dams were constructed by the inner filling method as mentioned before. Several tailings dams susceptible to failure have been inspected since 2012. The results of the inspections of one of the dams are discussed below.

The location of the inspected tailings dam is shown in Fig. 9. The dam was not damaged during the 2011 Great East Japan Earthquake, although it was subjected to a slightly higher seismic intensity of 5 in JMA scale. Detailed soil investigations, including borings, standard penetration tests, PS loggings, measurement of the ground water table, undisturbed samplings, triaxial tests, and cyclic triaxial tests, were conducted. The estimated soil cross section of the dam and the tailings is shown in Fig. 17. The stability of the slope of the dam under Level 1 earthquake motion was analyzed first based on the seismic design code of 1980. The results showed that the safety factor against liquefaction, F_L, was greater than 1.0 and the safety factor against sliding, F_s, was 1.2 under a seismic coefficient of K_h = 0.15. Therefore, it was confirmed that this tailings dam was stable according to the current design code. Then, the stability of the slope of the dam under Level 2 earthquake motion was analyzed in the following four steps:

(1) Step 1: The static stress distribution in the cross section was analyzed by the static finite element method.
(2) Step 2: The time history of the dynamic stress distribution in the cross section was analyzed by seismic response analysis.
(3) Step 3: The time history of the distribution of the safety factor against liquefaction, F_L in the section was calculated based on the static and dynamic stresses and on soil strength. Then the time history of the distribution of the excess pore-water pressure was calculated. The relationship between F_L and excess pore-water pressure ratio used in this analysis was not from Eq. (4) but a newly proposed one as shown in Fig.18, derived based on many laboratory tests for liquefaction.
(4) Step 4: The slip surfaces were assumed and the sliding displacements along the slip surfaces were estimated by Newmark's method considering increase of excess pore-water pressure.

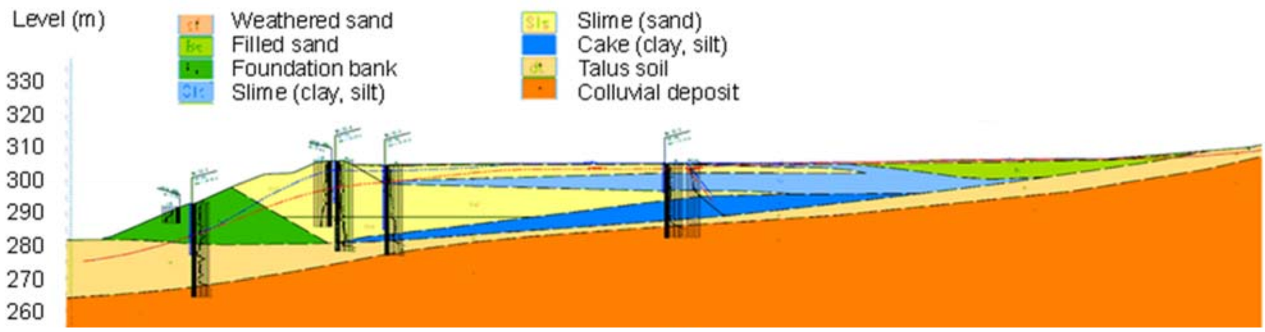

Figure 17. Estimated soil cross section at a recently inspected tailings dam

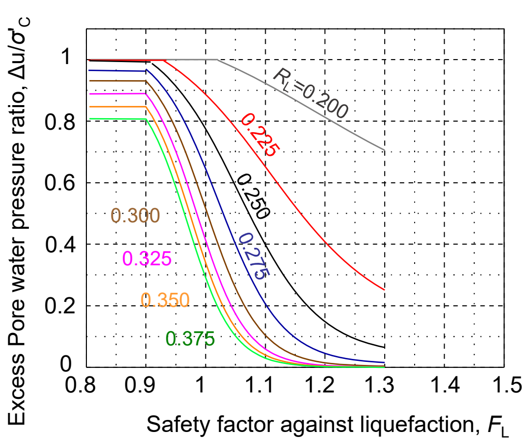

Figure 18. Relationship between F_L and excess pore-water pressure ratio used in this analysis

In the second step, the "FLUSH" computer program for seismic response analysis was used. The following three seismic waves were selected as the input motions:

i. Seismic wave 1: A seismic wave induced by near fault was estimated by the technique introduced in the code for reservoir dams. The maximum surface acceleration of the estimated wave was 300.7 gals (cm/s^2).

ii. Seismic wave 2: A seismic wave recorded near the site during the 2011 Great East Japan Earthquake was used. The peak ground-acceleration of the estimated wave was 295.8 gals (cm/s^2).

iii. Seismic wave 3: Wave 2 with its amplitude adjusted based on the attenuation curve during the earthquake. The peak ground-acceleration of the estimated wave was 122.0 gals (cm/s^2).

Figure 19 shows the distribution of peak acceleration under seismic wave 1. The peak acceleration increased gradually from the bottom of the slope to about 500 gals (cm/s^2) at the surface of the slope. Figs. 20 and 21 show the distributions of F_L and the excess pore-water pressure ratio, respectively, under seismic wave 1. The shallow parts of tailings slime in upper zone liquefy and the excess pore-water pressure ratio U_L/σ_v' reaches 1.0. The liquefied zone is almost the same under the seismic waves 2 and 3. Fig. 22 shows assumed several slip surfaces and sliding displacements along these surfaces under seismic wave 1. The maximum sliding displacement which occurs in upper zone is 16 cm. Fig. 23 shows the time history of the input seismic wave 1, the excess pore-water pressure, the safety factor against sliding, F_S, and the sliding displacement along this sliding surface. The estimated sliding displacement is not large enough to cause flow of the liquefied slime. Therefore it was judged that the tailings dam is stable under Level 2 earthquake motion.

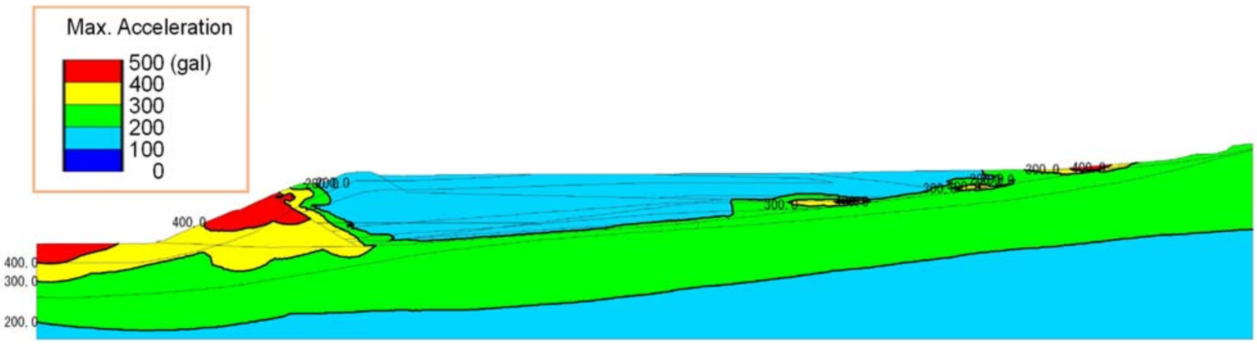

Figure 19. Distribution of peak acceleration under seismic wave 1

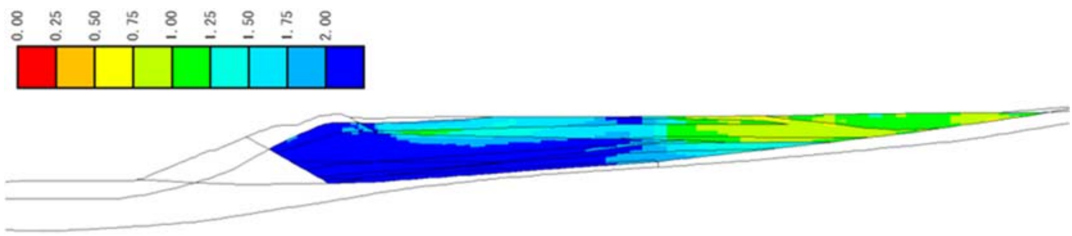

Figure 20. Distributions of the minimum F_L under seismic wave 1

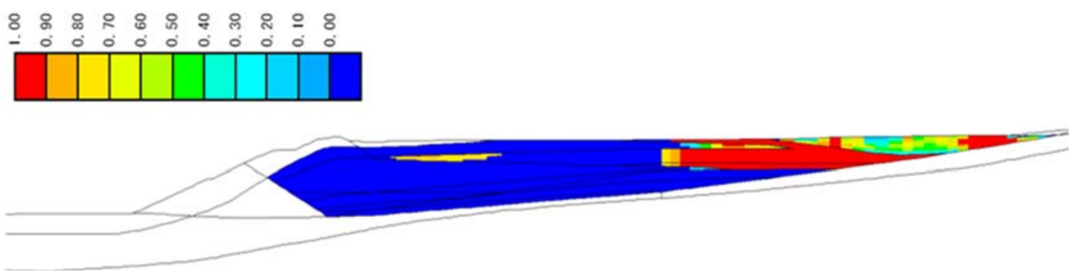

Figure 21. Distribution of the maximum excess pore-water pressure ratio under seismic wave 1

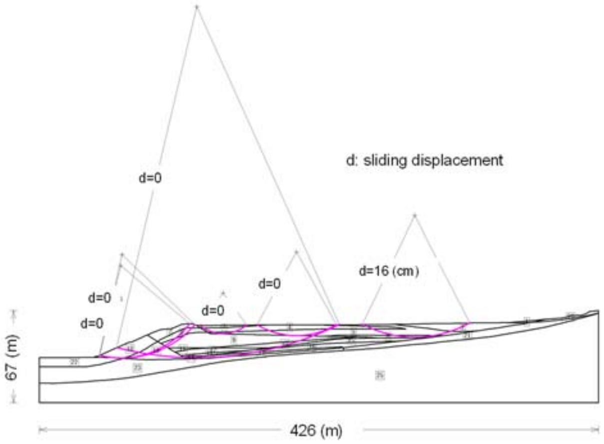

Figure 22. Sliding displacements of the input seismic wave No.1

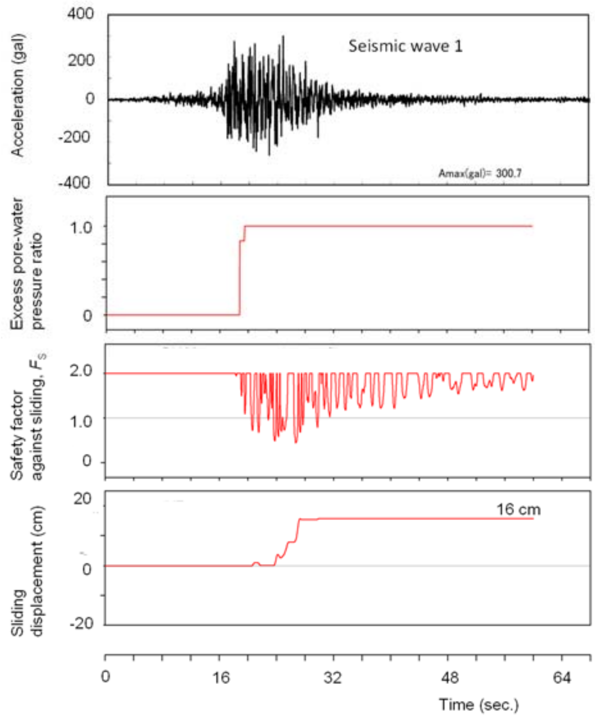

Figure 23. Time history of the input seismic wave 1, the excess pore-water pressure ratio, the safety factor against sliding F_S, and the sliding displacement

7. The 2016 Kumamoto Earthquake

A strong foreshock of the 2016 Kumamoto Earthquake hit Kumamoto in Japan on April 14. The main shock, with a magnitude of Mj=7.3, hit 28 hours later. Many houses and buildings collapsed due to the strong shaking. An abandoned tailings dam at the Taio Mine failed during the main shock, as shown in Fig. 24. Soil investigation, laboratory tests and a stability analysis were conducted to restore the failed slope with appropriate countermeasure. Fig. 25 shows a soil cross section established by the investigation. Slime was very loose, with an SPT N-value of 0 to 1, and the liquefaction strength ratio, R_L (CSR), obtained by a cyclic triaxial test, was very small, about 0.2. Based on slope stability analysis, it was concluded that the sliding failure occurred due to an increase in pore water pressure in the slime. The lattice type cement mixing method with grids of 4.4 m square and 10 m deep was applied in the restoration to prevent sliding during future earthquakes, as shown in Figs. 25 and 26.

Figure 24. Failed slope of a tailings dam at the Taio Mine

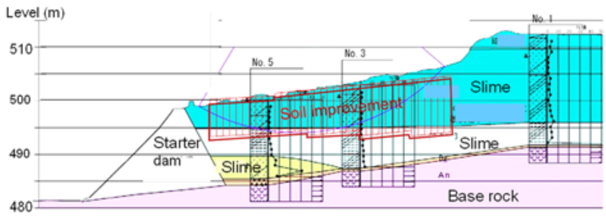

Figure 25. Cross section of the tailings dam

Figure 26. Restoration work at the tailings dam

8. Conclusions

The damages of the tailings dams during past earthquakes, the history of the tailings dam's seismic design code and and examples of soil investigations in Japan are introduced. A huge number of tailings dams, operational or abandoned, exist in all over the world. In general, the tailings dams are unstable to intense shaking during earthquakes because the slime is liquefiable. Abandoned dams must be inspected. If a dam is judged as unstable, an appropriate measure to counter the failure of the dam must be applied.

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