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REVISED SEISMIC DESIGN CODE FOR TAILINGS DAMS AND ITS APPLICATION TO EXISTING DAMS IN JAPAN

S. Yasuda(1), K. Fukui(2), S. Kunimatsu(3), M. Okamura(4), T. Kiyota(5), K. Koguchi(6)

(1) *Professor, Tokyo Denki University, yasuda@g.dendai.ac.jp*
(2) *Professor, University of Tokyo, fukui@sys.t.u-tokyo.ac.jp*
(3) *Visiting researcher, National Institute of Advanced Industrial Science and Technology, s.kunimatsu@aist.go.jp*
(4) *Professor, Ehime University, okamura@cee.ehime-u.ac.jp*
(5) *Associate Professor, University of Tokyo, kiyota@iis.u-tokyo.ac.jp*
(6) *Engineer, Kiso-jiban Consultants Co., Ltd., koguchi.kazuaki@kiso.co.jp*

Abstract

The 2011 Great East Japan Earthquake, with a magnitude of M_W=9.0, caused severe damage to many structures not only in the Tohoku district of northeastern Japan, but also in the Kanto district, surrounding Tokyo. Seismic intensity in the affected area, as measured by 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. The earthquake caused the failure of three abandoned tailings dams that had been examined about 30 years previously and had been judged to be safe against earthquakes. Detailed study conducted after the earthquake demonstrated that the main reason for these failures was the intense shaking generated by the earthquake, which exceeded the shaking level applied in the examinations.

Following this earthquake, the Japanese Ministry of Economy, Trade and Industry (METI) revised the application of its seismic design code for tailings dams to insure safety against Level 2 earthquake motion, , which is extremely strong but very unlikely to strike a structure during its lifetime. Then a new design concept based not on the occurrence of sliding but on the likely degree of damage to structures from sliding was introduced. The METI lists 388 major tailings dams in Japan. About one-third of these dams were constructed by inner filling method. It was considered necessary to inspect following types of tailings dams deemed susceptible to damage by very strong Level 2 earthquake motion: i) constructed by inner filling method, ii) with a slope steeper than 15 degrees, and iii) with a ground water table shallower than GL-10 m. These tailings dams have been inspected since 2012. In the inspection, 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. Then, the stability of the slope of the dam under Level 2 earthquake motion was analyzed in the following four steps: i) Step 1; the static stress distribution in the cross section was analyzed by static finite element method, ii) Step 2; the dynamic stress distribution in the cross section was analyzed by seismic response analysis, iii) Step 3; the distribution of the safety factor in the section was calculated based on the static and dynamic stresses and on soil strength, and iv) Step 4; the slip surfaces were assumed and the sliding displacements along the slip surfaces were estimated by Newmark's method with consideration of an increase in excess pore water pressure. Then an inspected result for a tailings dam was introduced.

Keywords: Seismic design, Tailings dam, Liquefaction, Great East Japan Earthquake

1. Introduction

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 much gold was produced in Zipangu, and palaces and houses there were covered in gold. However, Japanese mines contained few deposits, and most of them had been closed by the end of 20th century, leaving many abandoned tailings reservoirs in mountainous areas. Japan established a design code for the construction of tailings reservoirs in 1954 and revised it in 1973 and 1980. Existing tailings dams were examined for seismic safety according to the revised codes and, if necessary, were strengthened to increase their safety.

Despite these inspections, three tailings dams failed during the 2011 Great East Japan Earthquake and their tailings flowed out of the dams. The tailings contaminated water in some valleys and forced the closure of a railway and a road. The main reason for the failures was strong and long shaking as the earthquake had a magnitude of M_W=9.0, the biggest earthquake in Japan since 1900. Based on detailed investigations of the failed tailings dams, the seismic design code was revised to ensure the stability of tailings dams during Level 2 earthquake motion in 2012. This paper presents a history of Japan's seismic design code for tailings dams and the methods used recently to inspect these dams.

2. History of seismic design code before the 2011 Great East Japan Earthquake

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 company drainage facilities. Based on the revised code, all licensed tailing dams were inspected from 1974. The 1978 Izuoshima-kinkai Earthquake seriously damaged the Houzukizawa tailings dam due to the liquefaction of tailings materials. Fig. 1 shows a cross section of this dam. About 80,000 m^3 of slime and coarse tailings flowed out of the dam and down a river into a bay. 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., 1981 [1]). Based on the results of these tests and analyses, the code was revised in 1980 to include a consideration of the effect of liquefaction on the relevant structures (Japanese Mining Industry Association, 1980 [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 believed 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:

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

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

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

If cyclic triaxial tests are not conducted, R_L can be roughly estimated by SPT *N*-value using the equations shown

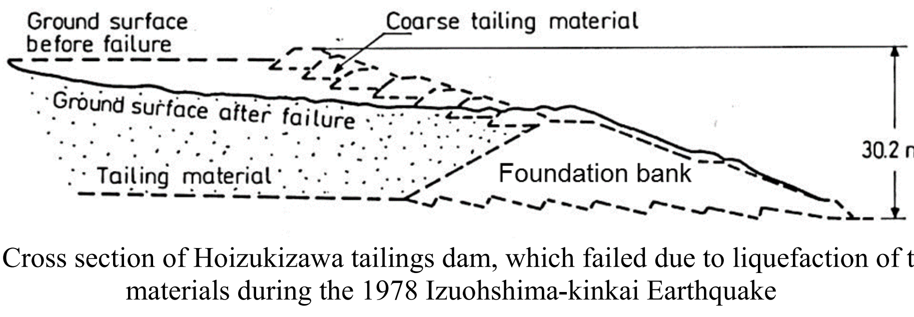

Fig. 1 - Cross section of Hoizukizawa tailings dam, which failed due to liquefaction of tailings materials during the 1978 Izuohshima-kinkai Earthquake

Table - 1 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)

Table - 2 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

in Table 1. 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:

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

where K_h is the horizontal seismic coefficient used in the seismic design (0.10 to 0.15, as shown in Table 2) , 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). The excess pore water pressure, U_L, generated during liquefaction is evaluated from the F_L value and the following relationship:

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

The code require that a stability analysis of each dam be conducted assuming a circular slip surface as shown in Fig. 2 where the pressure U_L is applied to the sliding surface of each slice according to the following formula:

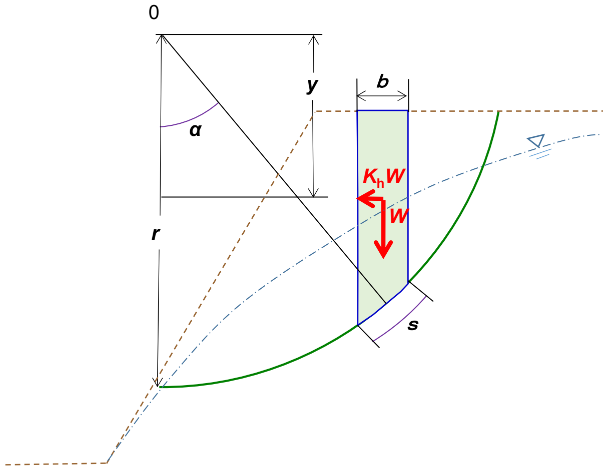

Fig. 2 – Stability analysis method intrduced in the code

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

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

Based on the revised code of 1980, all licensed tailings dams were inspected for seismic stability.

3. Dimensions of the three failed dams and description of the damage caused by the failures during the 2011 Great East Japan Earthquake

The 2011 Great East Japan 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. The quake was followed by a huge tsunami that destroyed many cities and killed and injured many people along the Pacific coast. Severe damage to many structures occurred not only in the Tohoku district of northeastern Japan, but also in the Kanto district, surrounding Tokyo. Seismic intensity in the affected area, as measured by 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, Zenigame and Gengorou tailings dams) failed, many houses and lifelines were damaged by soil liquefaction, landslides occurred, agricultural dams failed, and river dikes settled.

Detailed soil investigations, laboratory tests, seismic response analyses, slope stability analyses and deformation analyses were carried out on the failed tailings dams soon 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 of tailings dams that did not fail are shown in Fig. 3. The dimensions of the failed dams and an outline of the damage they caused are shown in Tables 3 and 4 (Technical Committee for Administration of Reservoir of Waste Soils and Tailings, 2012 [3]). The failed dams shared some common seismic and ground conditions:

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

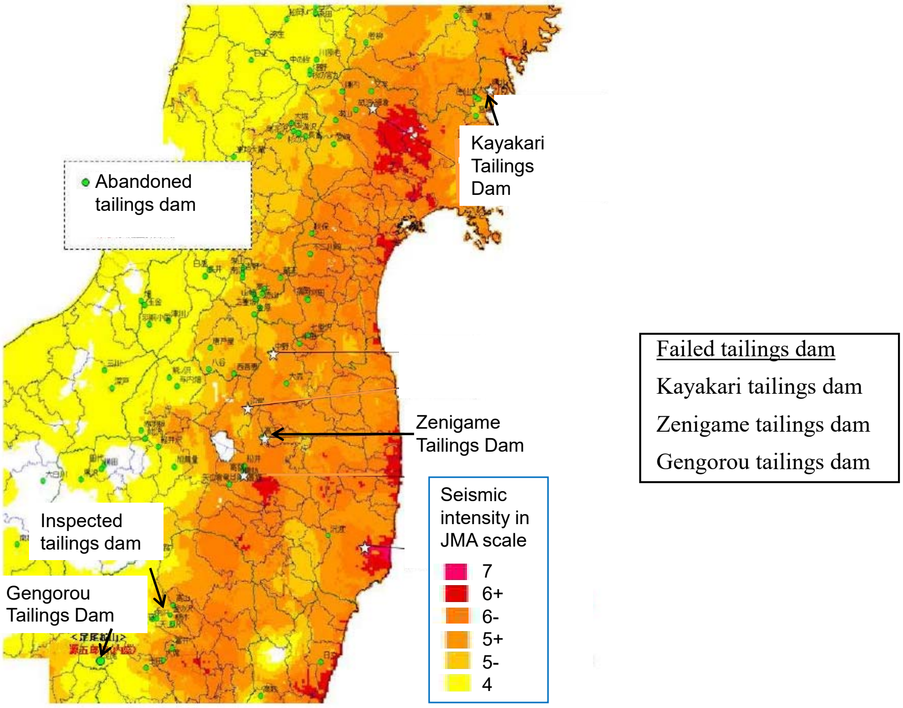

Fig. - 3 Locations of the three failed tailings dams and of tailings dams that did not fail (quoted from Technical Committee for Administration of Reservoir of Waste Soils and Tailings, 2012 [3])

Table - 3 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, 2012 [3])

		Name of tailings dam		
		Kayakari	Zenigame	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

Table - 4 Damage caused by the failure of three tailings dams during the 2011 Great East Japan Earthquake (partially quoted from Technical Committee for Administration of Reservoir of Waste Soils and Tailings, 2012 [3])

	Name of tailings dam		
	Kayakari	Zenigame	Gengorou
Seismic intensity	5+ in JMA scale	5+ in JMA scale	5- in JMA scale
Failure and flow of tailings	Slime above the foundation bank slid, as shown in Fig, 4, and flowed 1 km downstream into houses and fields. A river was contaminated by arsenic.	Tailings in the upper slope of the dam failed, as shown in Fig. 5, and flowed onto a road and into a river. The road was temporarily closed.	Slime above the foundation bank slid and flowed onto a railway and into a river, as shown in Fig. 6. The railway was temporarily closed. A river was contaminated by lead.

Fig. - 4 Failure of the upper slope of the Kayakari tailings dam)

Fig. - 5 Failure of the upper slope of the Zenigame tailings dam

Fig. - 6 Tailings materials from the Gengorou tailngs dam flowed into a river (quoted from Technical Committee for Administration of Reservoir of Waste Soils and Tailings, 2012 [3])

4. Revised application of seismic design code after the Great East Japan Earthquake

As shown in Table 3, the three tailings dams that failed during the Great East Japan Earthquake had been examined 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 1 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.

a) 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.
b) 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 to collapse. 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 basing 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 judge the degree of damage because it is easy to improve embankment slopes to prevent failure under this level of shaking. Under Level 2 earthquake motion,

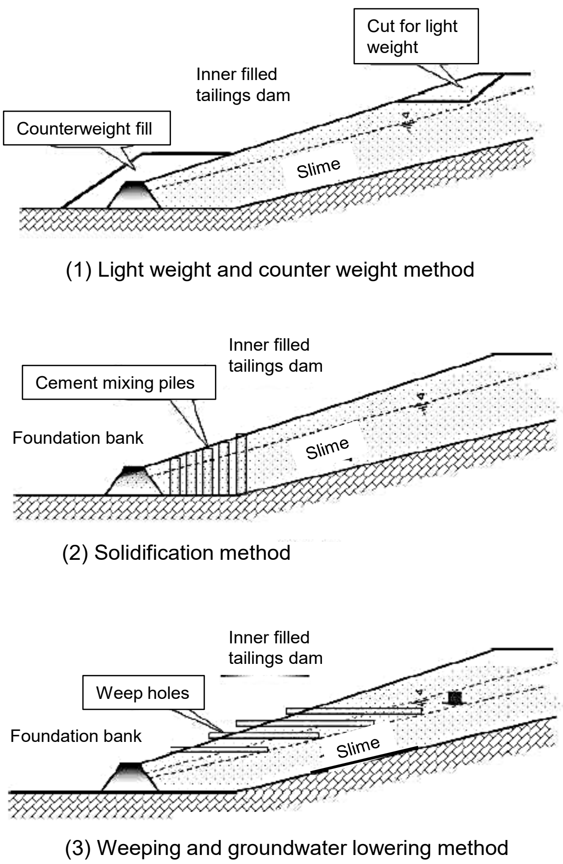

Fig. - 7 Appropriate measures to counter the deformation of tailings dams on the contrary, the slight sliding or deformation of embankments cannot be prevented by current countermeasures. Therefore, it was considered necessary to introduce a new design concept based not on the occurrence of sliding but 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: the allowable deformation or displacement of embankments, and 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 to deform. The recommended measures to counter dam deformation are shown in Fig. 7.

5. An example of inspection by the application of the new design code

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. Several tailings dams deemed susceptible to failure have been inspected since 2012. The results of inspection of one dam are discussed below.

The location of the inspected tailings dam is shown in Fig. 3. The dam was not damaged during the 2011 Great East Japan Earthquake, although it was subjected to a slightly high 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 at the dam is shown in Fig. 8. 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:

a) Step 1: The static stress distribution in the cross section was analyzed by the static finite element method.

b) Step 2: The time history of the dynamic stress distribution in the cross section was analyzed by seismic response analysis.

c) 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.9, derived from many laboratory tests for liquefaction.

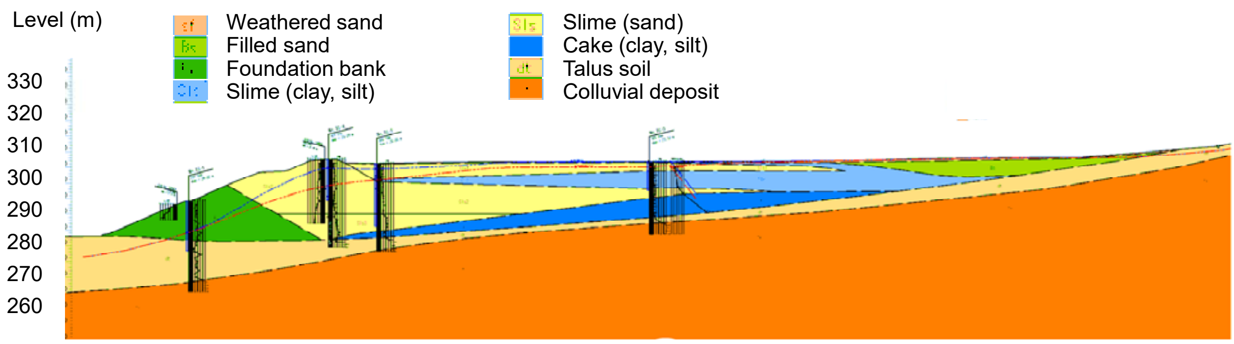

Fig. - 8 Estimated soil cross section at a recently inspected tailings dam

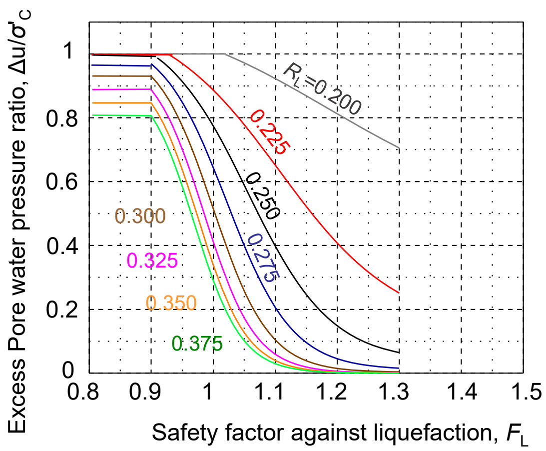

Fig. – 9 Relationship between F_L and excess pore-water pressure ratio used in this analysis

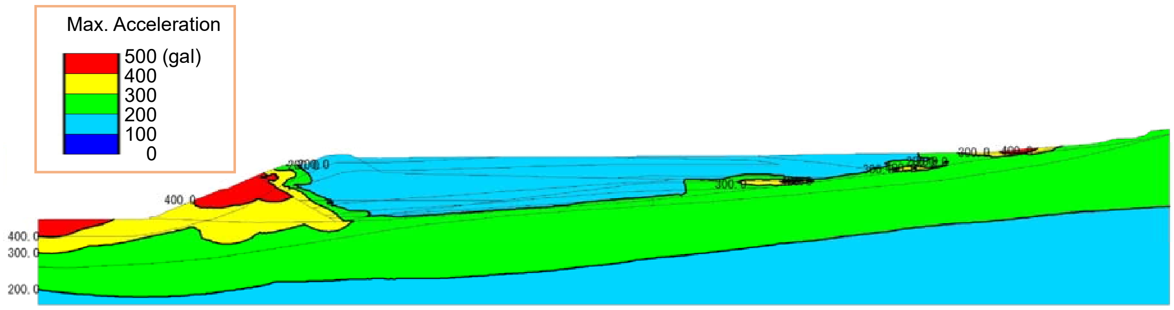

Fig. - 10 Distribution of peak acceleration under seismic wave 1

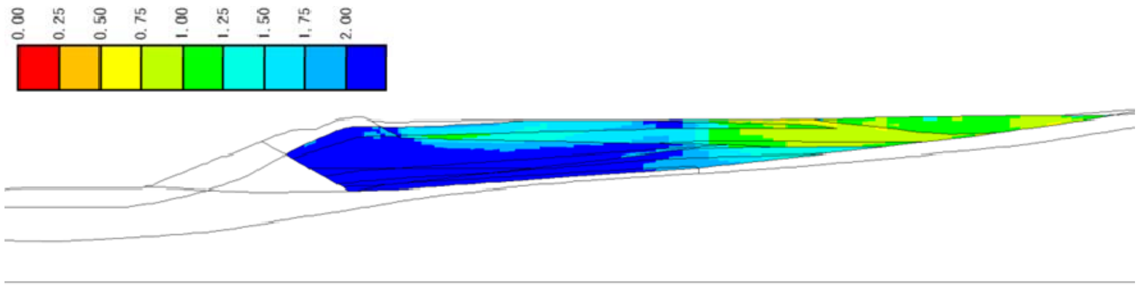

Fig. - 11 Distributions of the minimum F_L under seismic wave 1

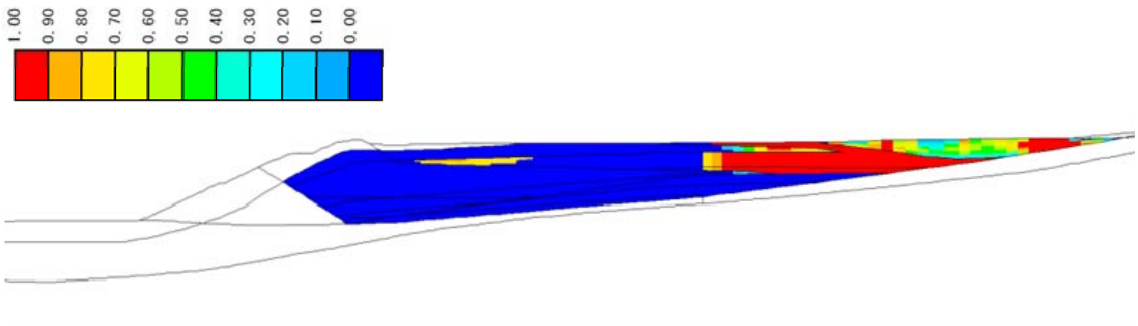

Fig. - 12 Distribution of the maximum excess pore-water pressure ratio under seismic wave 1

Step 4: The slip surfaces were assumed and the sliding displacements along the slip surfaces were estimated by Newmark's method with consideration of an increase in excess pore-water pressure.

In the second step, the "FLUSH" software program for seismic response analysis was used. The following three seismic waves were selected for 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 maximum surface 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 maximum surface acceleration of the estimated wave was 122.0 gals (cm/s^2).

Fig. 10 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 slope. Fig.s 11 and 12 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 liquefy zone is almost same under the seismic waves 2 and 3. Figure 13 shows assumed several slip surfaces and sliding displacements along their surfaces under seismic wave 1. The maximum sliding displacement which occurs in upper zone is 16 cm. Figure 14 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 slip surface. The estimated sliding displacement is not large to cause flow of the liquefied slime. Therefore it was judged that the tailings dam is stable under Level 2 earthquake motion.

6. Conclusions

The history of Japan's seismic design code for tailings dams is reviewed, the mechanism of the damage to three tailings dams during the 2011 Great East Japan Earthquake is demonstreted, and the new code for Level 2 earthquake motion developed after the 2011 Great East Japan Earthquake is introduced. The following conclusions are derived through these studies:

(1) Three tailings dams failed during the 2011 Great East Japan Eartquake. They were costructed by inner filling method. The dams were high, with steep slopes, and the ground water tables were shallow or the tailings were saturated.

(2) They had been examined once or twice before the earthquake and judged to be stable against potential earthquake motion. However, strong, long shaking during the 2011 Great East Japan Earthquake lowered the shear strength of the tailings and caused the failures.

(3) It was considered necessary to inspect following types of tailings dams deemed susceptible to damage by very strong Level 2 earthquake motion: i) constructed by inner filling method, ii) with a slope steeper than 15 degrees, and iii) with a ground water table shallower than GL-10 m.

(4) In the inspection, it is 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.

(5) Existing tailings dams are now being inspected for Level 2 earthquake motion based on the new design concept.

7. Acknowledgements

Some of the results cited in this paper are quoted from reports by Technical Committee for Administration of Reservoir of Waste Soils and Tailings organized by Ministry of Economy, Trade and Industry.

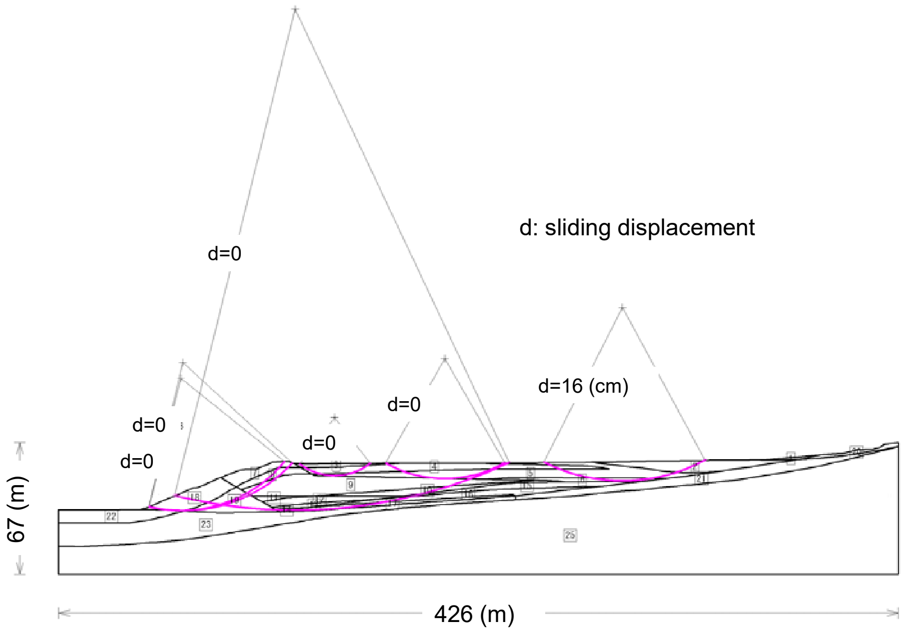

Fig. - 13 Sliding displacements of the input seismic wave No.1

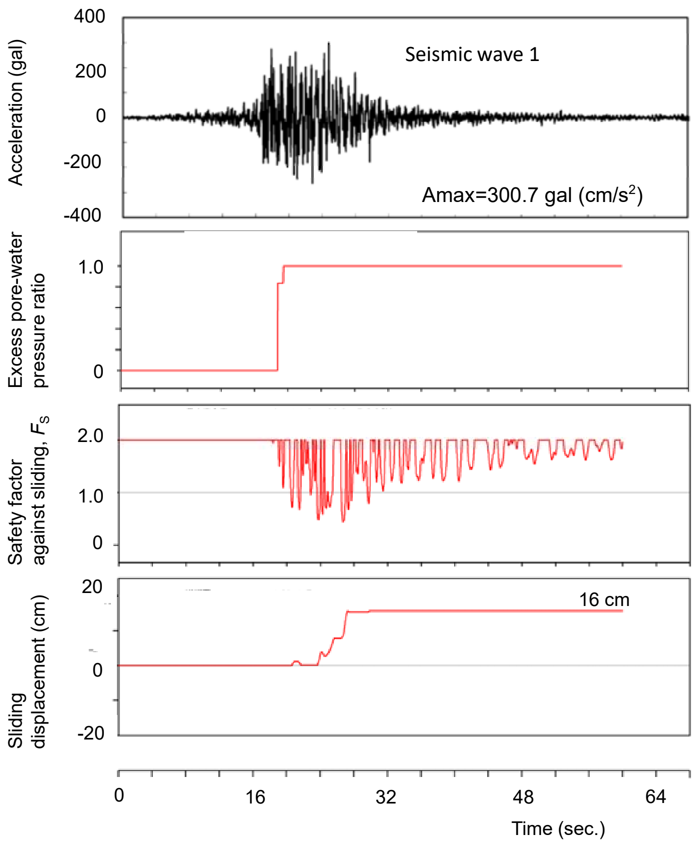

Fig. - 14 Time history of the input seismic wave 1, the excess pore-water pressure ratio, the safety factor against sliding F_s, and the sliding displacement

8. References

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