CURRENT STATE OF PERFORMANCE-BASED DESIGN OF FILL DAMS

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Ⅰ PERFORMANCE BASED DESIGN FOR FILL DAMS

1 INTRODUCTION

Regarding damage of soil structures in Japan, the Niigata-Chuetsu earthquake of October 2004 caused a heavy damage to Kan-etsu Expressway, and the Noto Hanto earthquake of March 2007 also caused it to the Noto toll road. Considering a great deal of earthquake damage and restoration, soil structures seem to be easy to be restored within a relatively short time even if they are damaged, which are very different from concrete structures, and are expected to have ductility capacity. Limit equilibrium methods of stability analysis such as Circular arc method were adopted as aseismic design of soil structures. The deformation analyses during earthquakes for level 2earthquake motions, however, have increased since the 1995 Hyogoken-Nanbu earthquake. In addition, the performance target has also been clearly specified in aseismic design standard of railway structures in Japan [(1) (2)], the guideline for aseismic checking of dams and its commentary [(3)], and so on. And the deformation checking whose evaluation index is settlement has been adopted since then, too.

On the other hand, the design method of infrastructure facilities in Japan aims to change from the existing system of specifications design to the performance code in which reliability-based design is fundamental, according to the 1995 Technical Barriers to Trade (TBT) of World Trade Organization (WTO). In this stream, it must be impetus that there is necessity to ensure consistency with the international design standard such as ISO23469 (Seismic Actions for Designing Geotechnical Works) [(4)]. In this paper, the author mentions about the fundamental idea of the performance based design of the soil structures and also that design of a fill dam, which is a soil structure, as an illustration.

2 PERFORMANCE BASED DESIGN

Although there are many interpretations and expressions about performance based design according to publications, the details are not written in this paper, therefore I hope that you will consult with those publications. Under the condition that design guides and design standards are being shifted from specifications design to performance code, the related academic societies have organized the committees and made standards for performance based design and its checking. And Japanese Geotechnical Society (JGS) also enforced the design principle for fundamental structures based on the concept of performance based design [5] as the standard of this society, in March 2006. In these reports and standards, as the effect of the introduction of the performance based design, it is notionally mentioned that this shifting from specifications design to performance based design is expected to make room for the freedom of design, which gives many opportunities to new technologies and engineering methods, and to contribute to the cost-cutting. There, however, are some differences in concept of those reports and standards. Honjo reported "what the performance based design is?" from the viewpoint of historical background and in relation to the limit state design method [6]. However, according to a document [7], it is said that the performance based design and checking is the design which enterprising bodies clearly specify the purpose of structures, their applicable scope, and the performance required in every margin state, furthermore the design must be based on the design method proved to satisfy the performance code of engineers with proper reliability, to the structures without limitations of the way of checking their performance. In a simple term, the performance based design is the way of design that satisfies the required performance target or results regardless of the methods of design and detailed checking.

In the design system of the type of performance code, the purpose of structures, their applicable scope (involving the loading condition), and the clear performance required in every margin state must be clearly specified, in order to design so that structures satisfy the performance required by enterprising bodies without the detailed rules on the design and checking. The design method to satisfy the required performance needs the ground survey proved to satisfy the requirement with proper reliability, the way of setting up soil parameter based on the geotechnical test, the method of checking of performance (analysis method) with proper reliability, and the validation of performance based design which is based on the high design method. And there are also problems of ensuring the human resources of engineers who have the expertise of this area, and the establishment of evaluation system of engineers.

In some existing reports and standards about it, there are some cases where showing of the qualification methodology and deemed regulation are required in the design and checking of the performance code. In the case where the qualification methodology and deemed regulation are offered for the design standards, even if design is done based on more reliable qualification methodology, actually second look is often demanded by qualification methodology which is pointed out in the deemed regulation and the standards. Besides, the original purpose of increasing in freedom of design, for example, adoption of new technologies and new construction methods, and cutting down the cost of construction might not be satisfied.

There are various problems in the performance based design of soil structures, and it might be inevitable that the design methods which I mentioned above are remaining as transitional measures. However, hereafter it is necessary we accomplish aggressively the design based on the conception "of the design method that is proved to satisfy the performance code stipulated by a designer with proper reliability.

3 PERFORMANCE BASED DESIGN FOR SOIL STRUCTURES

According to the definition of ISO23469, soil structures are the structures in the ground such as Box-Culverts and Pipelines, foundation, earth retaining walls, fill-up structures, and reclaimed land, which are structures of interaction with foundation ground in a broad sense. But here fill-up structures involving foundation ground are dealt with as a subject. As to performance based design, items are about as follows.

3.1 *Design condition*

(1) The required performance of soil structures

This is the constructive and social performance required by enterprising bodies or users. In general ways, it is the required performance of usability, repairability and safety. It, however, differs depending on the degree of importance and considerable external force term. As an example of the requirement of safety, notionally it becomes such expressions as the function must not be lost even in the case of damages caused by earthquakes, and on the other hand in concrete terms it becomes such expressions as the maximum settlement is under 1.0 m.

(2) Design load and its magnitude

These are kinds and magnitude of considerable loadings during design period. These also differ in each structure depending on the importance of structures, but generally speaking, earthquakes, rainfalls and loadings are referred as acting load. In earthquakes, level 1and 2 earthquake motions are commonly used. And in rainfall, 100-year or 200-year probable rainfall is expected to be used for the case of water storage facility, such as fill dams and riverbanks.

(3) The soil constants of ground and banking material

This means setting up parameters of soil proved to satisfy the requirement with proper reliability based on the geotechnical tests and ground surveys. For the way of evaluation of data spread is different between structures which are built on a narrow area like fill dams and structures which are built on a large area like road embankment.

3.2 *Design method and evaluation method*

(1) The checking (or the analysis method) of the performance based design with proper reliability, and the validity of the performance based design using the advanced design method

The designers must have the responsibility of proving and explaining the validity on the method of the performance based design and the result. And the enterprising bodies should also have the charge and ability to check the validity of the design method. Because advanced judgment and sophistication are required, in the designers' side the technique must advance, and in the enterprising bodies' side the third party should be set up as a performance evaluation organization, as needed.

4 THE EXAMPLES OF THE PERFORMANCE BASED DESIGN (A HYPOTHETICAL MODEL DAM)

In the several cases, as prerequisites for the performance based design, there are the data spread of soil constants, and as considerable performance there are landslide stability caused by precipitation, erosion resistance and quake resistance. Here, however, the

Table 1. ISO23469 Compliance Check List

Item			Application	Explanation of contents and items	Related Chapter/Section of ISO23469
Stipulation of seismic performance earthquake motion	Stipulation of seismic performance criteria	Usability	○	Seismic performance and soundness not reduced by level 1 earthquake motion	5.1.2
		Safety	○	Seismic performance and only limited damage by level 2 earthquake motion	5.1.2
	Reference earthquake motion	Verifying usability	○	*Design seismic intensity for strong earthquake region of 0.15 set as level 1 earthquake motion	5.1.3 6.2
		Verifying safety	○	*Wave form suited to verification use lower limit acceleration response spectrum and Miyagi Prefecture Offshore Earthquake hypothetical wave form used as level 2 earthquake motion	5.1.3 6.2
	Indication of seismic performance provisions	Usability	○	Stipulated by slip safety factor	5.1.4
		Safety	○	Stipulated by maximum settlement of dam body crest	5.1.4
Evaluation of "ground response, fault displacement, etc."	Ground response analysis	Empirical analysis	—	Application of amplification factor according to ground category under design seismic intensity	6.3.2
		Simple dynamic analysis	—	Stipulating acceleration time history of hypothetical soil mass	6.3.4
		Detailed dynamic analysis	○	Stipulating acceleration time history of foundation surface	6.3.5
	Evaluating liquefaction		—	No prediction of liquefaction	6.3
	Evaluating spatial fluctuation of earthquake motion		—	Considering horizontal wave propagation	6.4
	Evaluating fault displacement etc.		—	Considering fault displacement/ground failure etc.	6.5
Evaluation of earthquake action	Evaluating earthquake action	Simple equivalent static analysis	—	Static analysis by a ground – structure non-integrated model using circular slip analysis	7.2.1
		Detailed equivalent static analysis	—	Static analysis by a ground – structure integrated model	7.2.2
		Simple dynamic analysis	—	Dynamic analysis by a ground – structure non-integrated model using the Newmark Method	7.3.1
		Detailed dynamic analysis	○	Dynamic analysis by a ground – structure integrated model using FEM	7.3.2

author has earthquake as acting force and fill dams as event under consideration. For first of all, since the density of banking materials and strength control are regulated in fill dams, according to the execution management standard, the data do not spread so much.

Second, for rainfall, the useful facilities of the drop inlet spillways are made. Then, the performance based design is to be made only for the safety at level 2 earthquake motion. In table 1, some conditions of performance based design are shown, which is made connection with ISO23469. In this report, the usability at level 1 earthquake motion is not explained. But the factor of safety is more than or equal to 1.20 for the example of analysis object. The validations of the safety at level 2 earthquake motion are discussed below.

4.1 *Setting up of earthquake motion for analyzing*

As to aseismic design, it is necessary to set up design earthquake motion. According to a document of the River Bureau of the Ministry of Land, Infrastructure, Transport and Tourism (MLIT 2005),[(3)] aseismic design depends on the earthquake motion, scenario earthquake, which is assumed to occur in that area. But if the largest earthquake motion which has been measured in the location of a dam as well as around it or the minimum value of acceleration response spectrum for checking were to effect very heavily, that value would be used.

4.2 *Required performance*

Considering from the view point of the functions and safety of dams, settlement at crest must be set so that the function of reservoir can be kept at level 2 earthquake motions. Then it is necessary to design as the crest is not below the reservoir water level. In other words, at least it is required to make the settlement at crest less than the free board. Since the “free board” should secure at least 1.0m in fill dams according to the design standards, the settlement is almost 1m in the performance target. However, considering from the data spread of the soil constants and errors for analyses, the author thinks in this case that as a rough guide around50cm is the allowable settlement, which is 50% of 1m.

4.3 *The way of checking and the evaluation of the checked results.*

Figure1 is the typical cross-section used for this analysis. (the hatching part shows the reinforced zone.) As to the analysis, the author did the Dynamic Response Analysis using Elasto-Plastic Model. Table 2 shows two maximum settlements at crest obtained when the author entered the ground motion of the scenario earthquake and the seismic waves which were taken from the lower limit of acceleration response spectra for checking. As you see, we understand that the settlement of the scenario earthquake is under 50cm, which satisfies the performance target, 0.5m.

On the other hand, in the case of the seismic waves of the lower limit of acceleration response spectra for checking, the settlement of the performance target cannot be satisfied. In this latter case aseismic reinforcement is to be considered necessary, as the author mentioned in the former part (4)-1) — if the largest earthquake motion which has been measured in the location of a dam as well as around it or the minimum value of acceleration response spectrum for checking were to effect more heavily than that of scenario earthquake, that value would be used.

Figure1 shows the cross-sectional view of the aseismic reinforcement, and the hatching part illustrates the one which was reinforced with a cement-stabilized improved soil. As to the settlement, the maximum settlement was around 25cm in the both cases, which satisfies the performance target. Then even in new fill dams performance based design can be done in the same process. And in the other soil structures such as road embankments, there are some problems in performance targets such as safety in rainfall, long duration settlement, differential settlement, on the other hand in the soil constants such as the evaluation of data spread. Then you have to consider as needed if necessary.

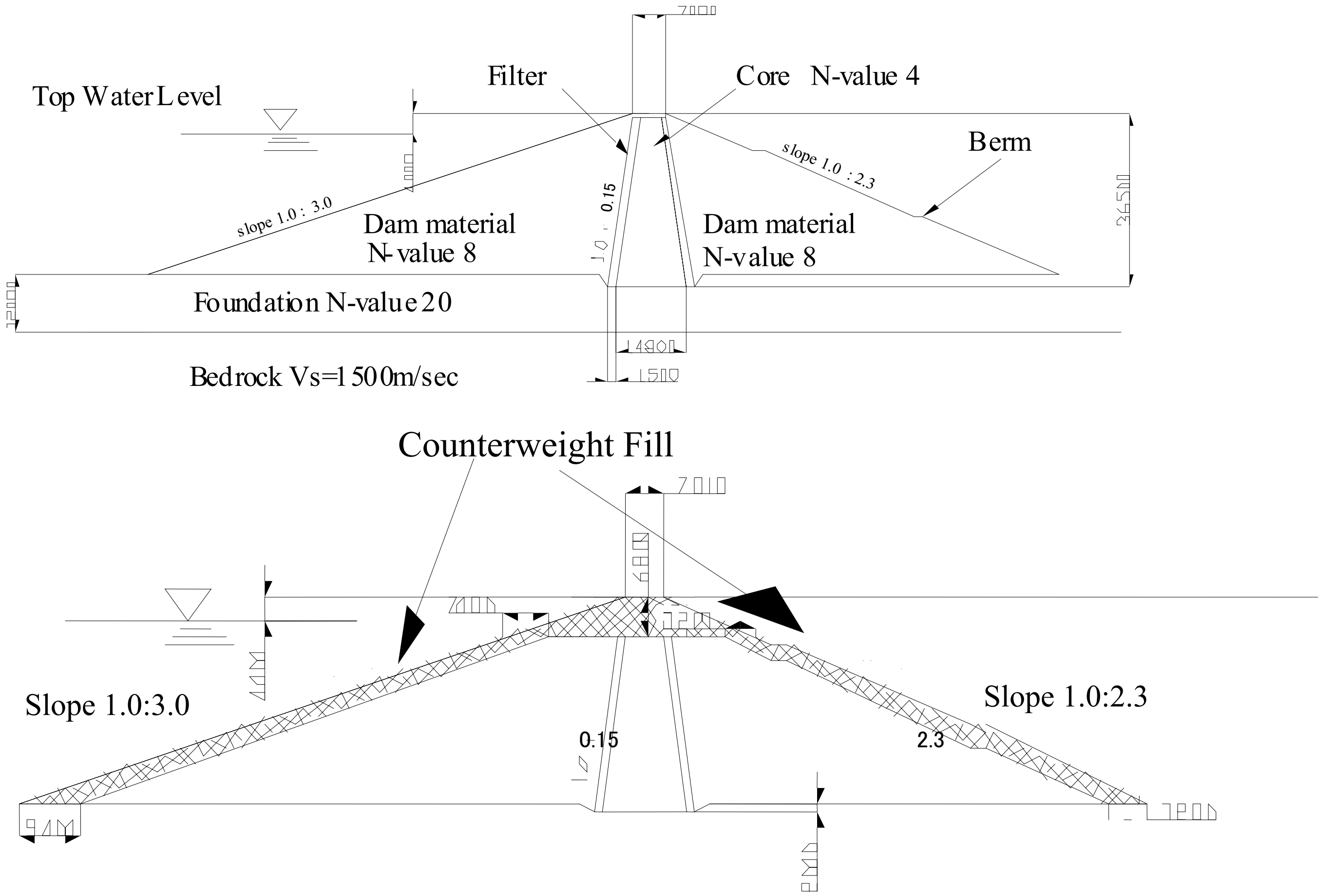

Figure 1. Original Sections and Aseismic Reinforcement Sections

Table 2. Maximum Settlement of Embankment Calculated by FEM Dynamic

	Maximum settlement of dam body crest (m)	
Earthquake wave	Kawanishi Wave	Miyagi Wave
No countermeasure	0.676	0.268
Reinforcement section	0.244	0.102

5 CONCLUSIONS

In this paper the author mentions the performance based design, especially the basic idea, and the example of a fill dam under earthquake loading as a practical case. Whatever design can satisfy the performance target expands the possibility of design such as introduction of new technology and construction methods, and probably brings the cost down and fulfills the appropriate accountability. That is the performance based design. Some people say that there are some problems such as the evaluation of the data spread of the soil constants and the checking methods of deformation, and that at this time the application is difficult. In my opinion, it is important to understand the effectiveness and necessity of performance based design and to introduce it at a practical level positively. Although it is very practically important, there are a large number of discussions in various fields. The author mentions one of those ideas in this paper. I must apologize that I omitted some items and expressions in each section due to limitation of space, if it leads to a misunderstanding.

Right now as the standard of JGS, 'the design principles on the basic structures and any related ones based on the performance based design' (tentative name) is being discussed, and will be standardized, and then the idea as JGS will be expressed there. In addition, one of the parts in this paper is based on the final report of "research committee for damage–tolerant design and performance based design of soil structures during earthquakes" (Japanese edition) (Tani and al., 2006).

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Ⅱ SAFETY OF ASSESSMENT OF OLD FILL-DAMS CONSIDERING THE RISK OF STRONG EARTHQUAKE MOTION-

1 INTRODUCTION

During the Iwate-Miyagi inland earthquake that occurred in July 2008, earthquake motions over an area of 1,000 (cm/s^2) were recorded at the foundations of the Aratozawa Dam, located in Kurikoma, Miyagi prefecture, Japan. The safety of fill-dams at the time of large-scale earthquakes has since become an important issue. Recently, there has been a demand for an explanation concerning the need for measures to improve earthquake resistance and the feasibility of such measures in the evaluation of the resistance of degraded fill-dams against large-scale earthquakes. Conventionally, measures to improve earthquake resistance have been taken to ensure a given level of safety against targeted earthquakes.

Recently, associated with the shift of fill-dam designs from specification designs to performance designs, even for earthquake-resistance reinforcement, optimization of measures to improve earthquake resistance based on the LCC concept that considers the degree of such measures and the risks from earthquakes were in demand. This study examines the safety of fill-dams under level 2 earthquake motions, using dynamic application analysis on a virtual fill-dam. Considering the influence of dam material variation on dam deformation volume, earthquake motions and earthquake hazard settings and a certain reinforcement method were presumed to calculate the failure rate of banking where earthquake resistance reinforcement measures were in place. According to the above, the life cycle cost (LCC) of degraded fill-dams that considers the risk of large-scale earthquake motions was assessed. The flow of the LCC assessment is illustrated in Figure1.

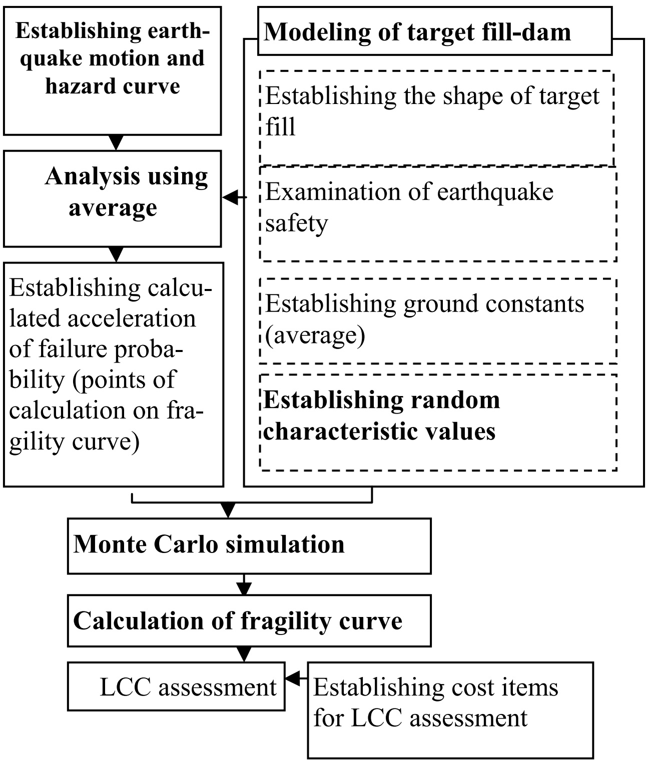
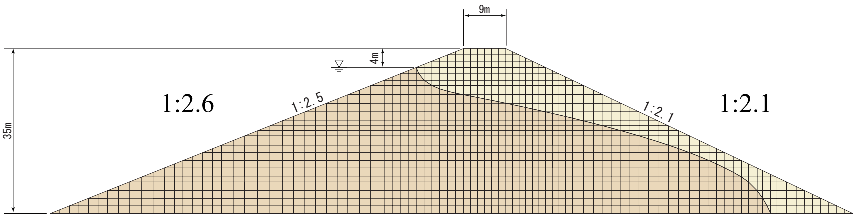


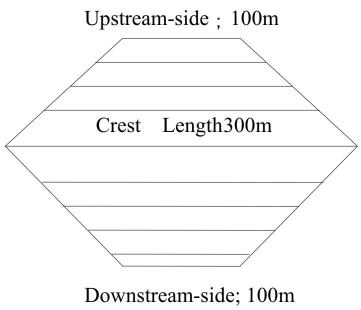
Figure 1. Flow of LCC assessment

2 CONDITIONS OF EXAMINATION

2.1 *model fill-dam*

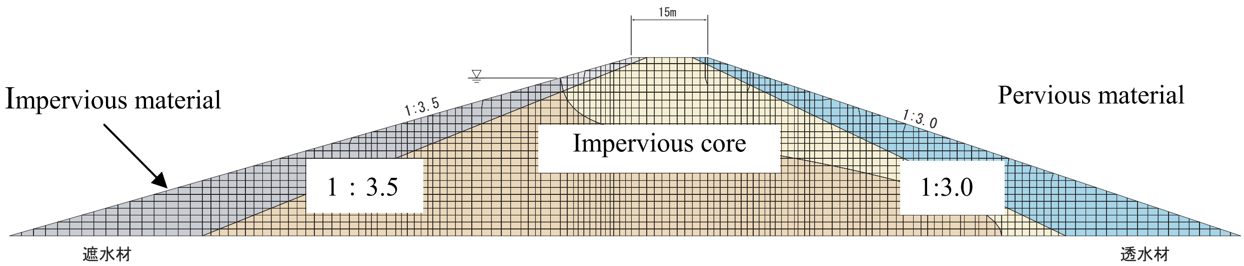
The type of fill-dam is a homogenous embankment dam with a dam height of 35 m and a crest length of 300 m. The gradients and such are set in reference to common fill-dam cases. Figure.2 shows the typical cross-section of the model fill-dam. As a measure to Improve earthquake resistance, counterweight fill is placed on the upstream and downstream slopes. The slopes are 1:3.5 gradient upstream and 1:2.5 gradient downstream. Furthermore, the target fill-dam is presumed to be a degraded fill-dam used before the earthquake resistance design standard was established, and the slide safety factor before earthquake resistance reinforcement is presumed to be 1.0. the ground constants are set so that the post-reinforcement safety factor would be 1.20 or over, satisfying the present dam standard in japan, when the ground design seismic coefficient is 0.18. the ground constants used in the analysis are shown in Table 1.





(a)No reinforcement

Top View of Model Dam without Earthquake-proof Reinforcement



(b)With reinforcement

Figure 2. Cross-section of model dam used for analysis (Solid foundation is assumed for the bottom end of dam. Reinforcement cost is 2.3 billion yen.)

Table 1. Ground constants used in analysis

Material Category		N value	Vs* (m/s)	Modulus** (kN/m²)	μ Poisson ratio	Density (t/m³)	ϕ (°)	C (kN/m²)	Shear modulus (kN/m²)
Dam material	Above phreatic line		188	152,000	0.41	1.47	40	3	51,007
	Under phreatic line	8	170		0.49	1.86			53,734
Core	Above phreatic line		149	87,000	0.41	1.32	28	20	29,195
	Under phreatic line	4	133		0.49	1.74			30,778
Filter	Above phreatic line		312	573,000	0.41	2.00	40	3	192,282
	Under phreatic line	—	(306)*		0.49	2.17			203,200

* $V_s = \sqrt{\dfrac{G}{\rho}}$ $\quad$ $**E = 2(1+\mu)G$

2.2 *Random Characteristic of Ground Constants*

2.2.1 *Random Variable and Random Characteris-tic Values*

The ground constants, with variability considered, are: Angle of shear resistance φ for sandy soil dam material and filter, angle of shear resistance φ of cohesive soil core and its cohesion *c*. The random characteristic values of each ground constant are shown in Table 2. The coefficient of variation of the angle of shear resistance of the dam and filter is calculated as 10%, referencing the values of peak strength observed in railway embankments in Japan (7-12%). The coefficient of variation of the cohesion is calculated as 30%, referencing the values listed in the literature (Nishimura 2007) and the probability density function is presumed to have a normal distribution. Φ and *c* of the core are presumed to be independent variables.

Table 2. Random characteristic values of ground constants

Material classification		Average	Coefficient of variation	Probability density function
Dam	Angle of shear resistance φ	40°	10%	Normal distribution
Filter				
Core		28°		
	Cohesion *C*	20 kN/m²	30%	

2.2.2 *Random Variables*

The random variables to be used in the Monte Carlo simulation are created with the random characteristic values of each ground constant set in 2.2.1, using random numbers. Figure.3 shows the variation of ground constants (average; coefficient of variation) with sample sizes (number of Monte Carlo simulations) of 50, 100 and 200, in which approximation to set values is observed to be proportional to sample size. In light of the purpose of this study, which is to demonstrate the LCC difference between before and after earthquake resistance reinforcement, and the calculation time required for Monte Carlo simulation, the sample size used is 50. Figure.4 shows the variation (angle of shear resistance of the dam) of ground constants.

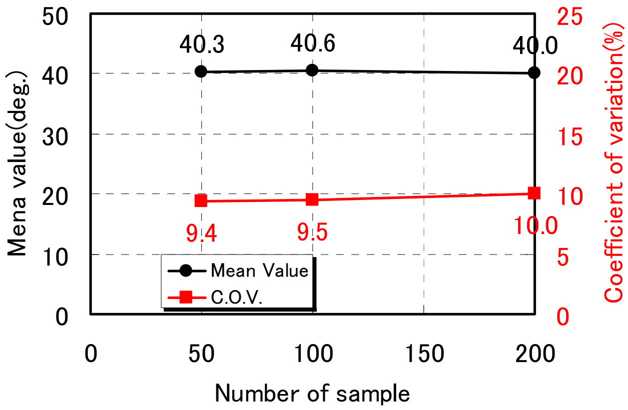


Figure 3. Relationship between Sample Numbers and Probabilistic Characteristics of Internal Friction Angle.

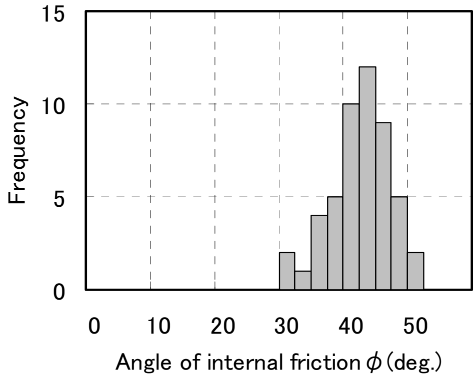


Figure 4. Example of Internal Friction Angle Samples Applied for MSC.

2.2.3 *Earthquake Motions and Hazard Curve*

The following 3 earthquake motions are set as input level 2 earthquake motions (maximum earthquake motion that may possibly occur at the location of fill-dam is presumed) to be used for dynamic analysis. Figure.5 shows the seismic waveform.

1) Miyagi earthquake – scenario earthquake motions (earthquake motions created by statistical Green's function method using the fault model of the 1978 Miyagi earthquake)

2) Hachinohe seismic wave – earthquake motions corresponding to the lower spectrum (simulated earthquake motions created using the Hachinohe seismic wave (NS) of the 1968 Tokachi earthquake)

3) Kawanishi seismic wave-earthquake motions corresponding to the lower spectrum (simulated earthquake motions created using the Kawanishi Dam waves up and down stream direction of the 2004 Chuetsu earthquake)

The fill-dam under examination is presumed to exist in Miyagi prefecture, and the earthquake hazard curve is set from the relevant existing data. The earthquake hazard curve plots the probability of the maximum acceleration exceeding a certain value during one year at a target location point. Literature (JNES 2006). estimated the Miyagi earthquake and assessed the earthquake hazard curve at the nuclear power plant location point in the prefecture as shown in Figure.6 using the distance decay method. This study utilizes that earthquake hazard curve.

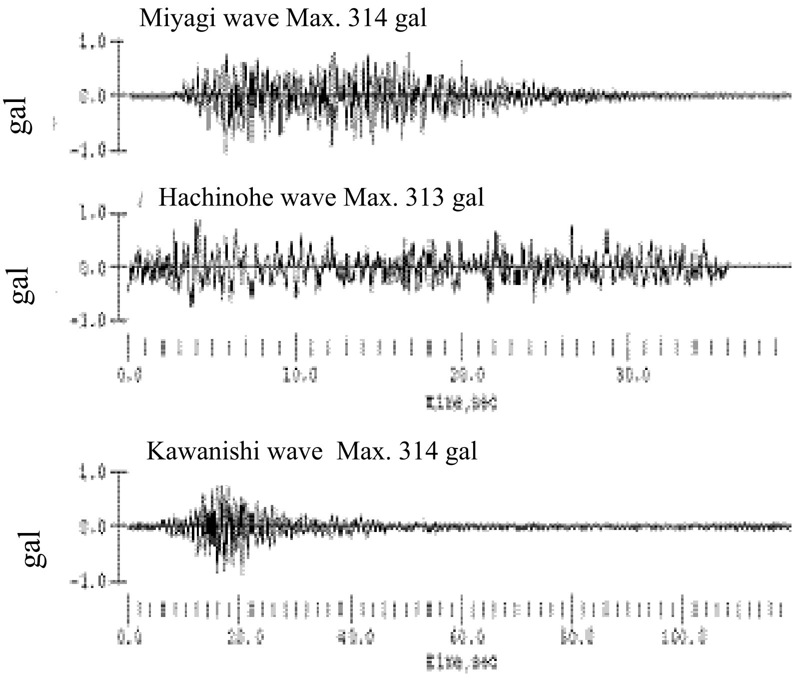


Figure 5. Input Ground Acceleration.

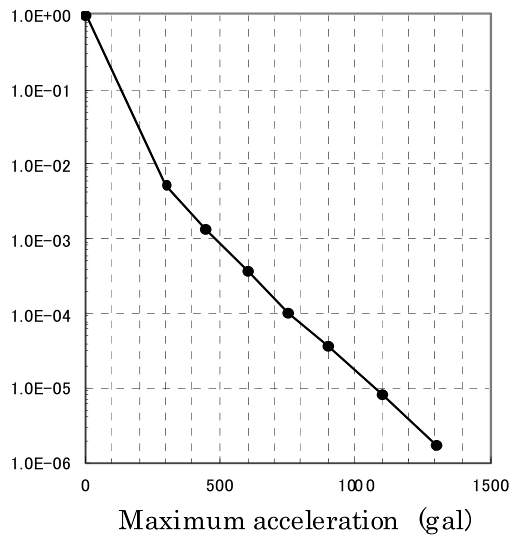


Figure 6. Applied Seismic Hazard Curve (JNES 2006)

2.2.4 *Performance Goal of the Fill-Dam at the Time of an Earthquake*

The relation between function retention and the degree of damage varies with the structure and thus is difficult to determine uniformly in reality. Earthen structures are generally assessed for safety by limit analysis, but in considering level 2 earthquake motions, taking banking for example, the ‘safety factor’ goes below 1.0 implying slope failure and thus cannot describe the level of danger or whether it is safe. A safety factor under 1.0 does not indicate an immediate slope failure. Therefore it is necessary to assess the level of danger and whether it is safe, from displacement. Particularly in a reservoir structure fill- dam, settlement is important in relation to the reservoir water level. In order to find the displacement in this case, it will require analysis by the finite element method or other relevant methods. Supposing the ‘precise residual displacement’ could be calculated, it is still necessary to determine the ‘performance goal’, the allowable extent of residual displacement. The performance goal shall be determined in combination with earthquake motions. The following explains the performance goal of the fill-dam.

As an indicator of the performance required of a reservoir structure such as a fill-dam, the crest settlement can be considered. Considering the functional aspect of the dam, the allowable settlement will be up to the maximum settlement that can sustain water storage function under a level 2 earthquake motion. Specifically, the crest elevation after an earthquake should not be lower than the water level, in other words, it is the minimum requirement to design the dam so that the settlement is within the freeboard of banking. In order to sustain minimal storage function, the water should not flow over the dam, and be within the freeboard of the dam under the wave height conditions caused by the earthquake. The formula for freeboard is specified in the dam design standard. The value calculable from this formula is the allowable settlement and in the case of fill-dams, a minimum of 1.0 m is ensured regardless of the dam height.

The Public Works Research Institute (Technical note, 2005) states that piping destruction should not occur in cases where sliding is not envisaged on the downstream slope. Furthermore, there are hardly any cases reported of fill-dam earthquake damage over 0.5 m unless liquefaction of the foundation was involved. Taking these observations into account, the performance goal is approximately 1.0 m regardless of the dam height, as a determinant of safety. Furthermore, in consideration of analysis errors, to be on the safe side, the performance goal value is set to 50% of the approximate 1.0 m, i.e. a 50 cm settlement or thereabouts as the goal of allowable settlement.

3 CALCULATION OF FRAGILITY CURVE FROM AN EARTHQUAKE

3.1 *Results of analysis using average ground constant*

The analysis model is shown in Figure.2 and the seismic waves are 1) Miyagi earthquake, 2) Hachinohe seismic wave and 3) Kawanishi seismic wave. The Mohr-Coulomb model was utilized as the constitutive law for soil. Earthquake response of the analysis model before and after reinforcement was analyzed by selecting the Tools menu, Options and then by changing the maximum acceleration from 200 to 1000 gal in 5 steps. A significant dcrease of residual settlement due to earthquake resistance reinforcement is observed for all seismic waves. The relation between the settlement in the dam upstream end and the input acceleration of all analysis cases is shown in Figure.7. The settlement becomes greater as the input acceleration becomes greater. At the same acceleration level, settlement icreases in the following order: Miyagi seismic wave, Kawanishi seismic wave and

Hachinohe seismic wave. In the comparison of before and after reinforcement, the settlement after reinforcement is significantly reduced, but the rate of decrease varies with the level of acceleration and input earthquake motion. This is considered to derive from the differences in the spectrum property of input earthquake motions.

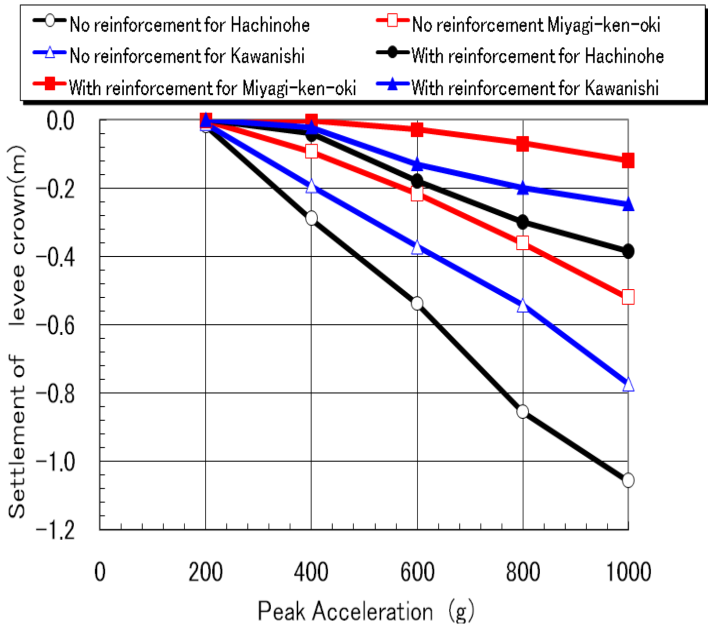
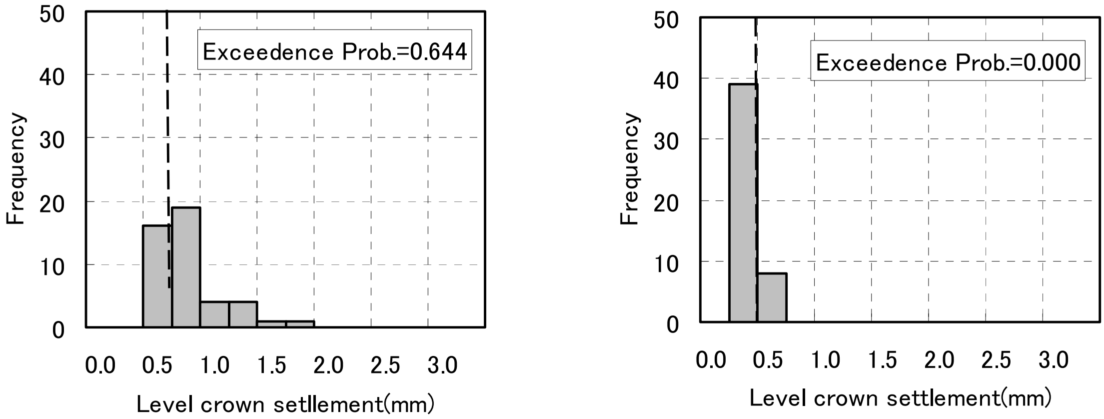


Figure7. Relation between input maximum acceleration and crest settlement.

3.2 *Monte Carlo Simulation*

Using the analysis model of the target fill-dam, an MS of the two-dimensional dynamic FEM analysis is performed with the elasto-plastic body model (Mohr-Coulomb model) applied to the constitutive law for soil. For LCC assessment in this study, the variation of MS crest settlement is calculated with the critical stage as overflow from crest settlement. MS is conducted 50 times each on accelerations 300, 600 and 900 gal. The variation of crest settlement for the Hachinohe seismic wave obtained by MS for existing dam and post reinforcement is shown in Figure.8. Based on the allowable settlement (performance goal under level 2 earthquake motion), 50 cm, the probability of exceeding this at each maximum acceleration is the ratio of the number of times exceeding the allowable settlement to the number of trials (50) for each acceleration.



(a) No reinforcement (b)With reinforcement

Figure 8. Example of MCS result for peak acceleration 600 (cm/s^2)

3.3 *Calculation of the Fragility curve*

The fragility curve is calculated based on the variation of crest settlement for each maximum acceleration obtained from the above Monte Carlo simulation. As shown in Figure.9, for the calculation of the fragility curve, the relation of maximum acceleration and failure probability is regressed by cumulative distribution function of log-normal probability to set the average value and standard deviation, the parameters of cumulative distribution function, with the minimum of residual errors. The failure probability p_{f1}, p_{f2}, p_{f3} for each maximum acceleration α_1, α_2, α_3 in Figure.9 is the aforementioned failure probability for each maximum acceleration. Figure.10 shows the calculated fragility curve.

Due to the lower limit inclination of the calculated acceleration in the fragility curve for the post-reinforcement fragility curve, it was deemed difficult to apply these 3 points to the cumulative distribution function of log-normal probability. Therefore, the coefficient of variation (=standard deviation/average) is calculated using the average and standard deviation obtained from the application of pre-reinforcement cumulative distribution function, and presuming that the coefficient of variation remains the same after reinforcement, only the average was used as a parameter for application.

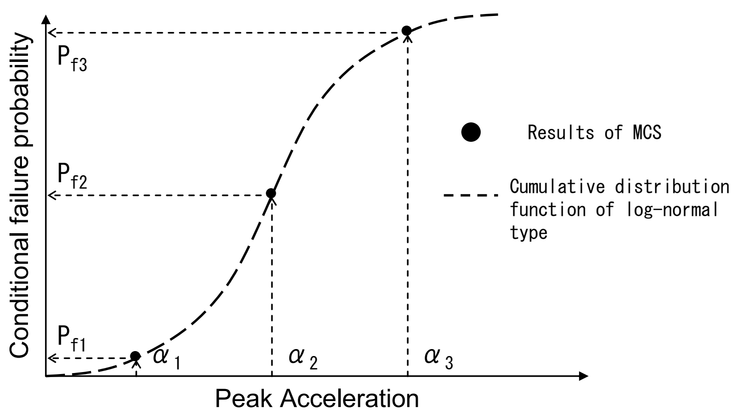


Figure 9. General of fragility curve evaluation

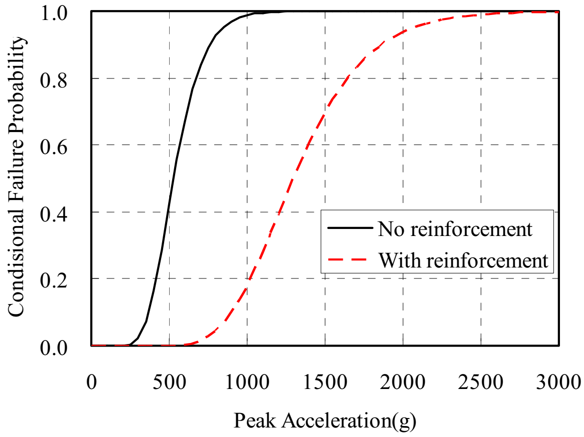


Figure 10. Comparison of fragility curve between no and with reinforcement

4 LCC ASSESSMENT

4.1 *Lcc assessment method*

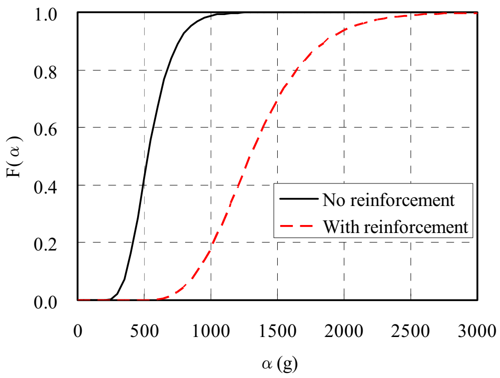
LCC is assessed as the total cost, TC, over a period of time from present to an arbitrary future time. This total cost signifies the cost expectation over a certain period of time. TC is calculated using the formula (4.1) from the cost of reinforcement on an existing fill-dam C_0, failure probability of fill-dam P_f and cost of damage (including flood damage of peripheral areas from overflow and costs relating to restructuring) C_f.

$$TC = C_0 + p_f C_f \tag{1}$$

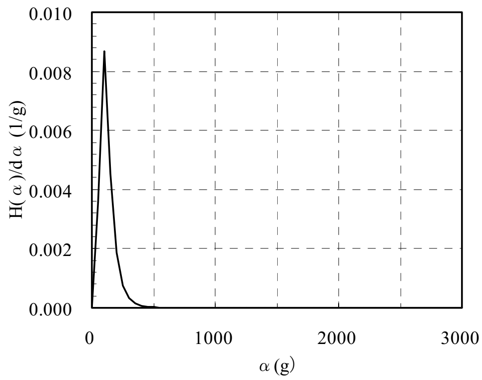
In the formula above, the failure probability p_f is the annual failure probability calculated using the hazard curve and fragility curve. The failure probability of the fill-dam here considers all earthquakes (maximum acceleration) that may occur at the target location. Because the occurrence probabilities of all maximum acceleration for earthquakes are calculated on a yearly basis for the hazard curve, the failure probability of a fill-dam will also be considered on a yearly basis. The failure probability is calculated for two cases: no reinforcement and post-reinforcement.

$$p = \int_0^\infty F(\alpha)\frac{dH(\alpha)}{d\alpha} \tag{2}$$

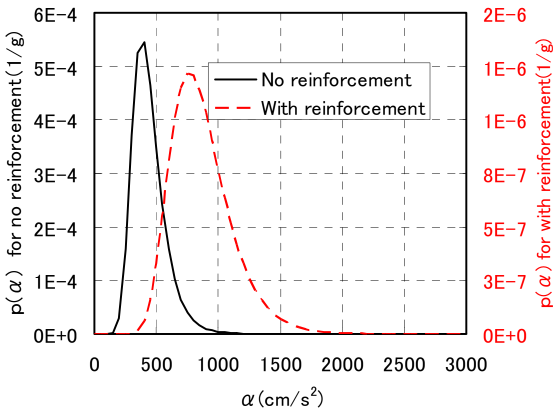
Each factor represents the following: p: annual failure probability; $F(\alpha)$: fragility curve; $H(\alpha)$: earthquake hazard curve; α: maximum acceleration.



(a) Fragility curve



(b) PDF of Peak Acceleration H (α)/d α



(c)PDF of Failure Probability per Year F (α) × d H(α) / d α

Figure 11. Process of Annual Failure Probability Evaluation.

4.2 *LCC Assessment Before and After Earthquake Resistance Reinforcement*

LCC is assessed using the results of the calculation of the annual failure probability that utilizes the hazard curve and post-reinforcement fragility curve. The calculation results of the annual failure probability are shown in Table 3. The reinforcement has decreased the failure probability by 2 orders of magnitude.

Table 3. Calculation results of annual failure probability.

Pre-reinforcement	**2.90×10^{-3}**
Post-reinforcement	**1.15×10^{-5}**

The costs considered in the LCC assessment are listed in Table 4. The volume of counterweight fill for earthquake resistance is calculated from the shape of the target fill-dam. As a result, the cost of earthquake resistance measures by counterweight fill for the model used in this study was 2.3 billion yen. When damages of 1 billion yen, lower than the cost of earthquake resistance measures, are presumed, for the next 500 years the LCC will be greater for undertaking earthquake countermeasures. However, when damage costs of 5 or 10 billion yen are presumed, while the initial cost (TC of 0 elapsed years) is higher when earthquake countermeasure is taken, the LCC will decrease with elapsed years, thus increasing the difference with the non-measure option. This indicates that taking earthquake resistance measures for important fill-dams where large-scale damage is presumed will reduce its LCC.

Figure.12 shows the LCC assessment results (LCC assessment results with a reinforcement cost of 2.3 billion yen and damage costs of 5 billion yen). With a reinforcement cost of 2.3 billion yen, the LCC will be higher for executing reinforcement until approximately 210 years later, but thereafter the LCC will become lower. Furthermore, if the reinforcement cost is presumed as 0.8 billion yen (no calculation), the LCC will be lower for carrying out the reinforcement than no reinforcement at an earlier stage, as shown in Figure.12.

Table 4. Costs considered in LCC assessment.

Cost item	Value	Note
Cost of reinforcement C_0	10,000 yen/m^3	Counterweight fill
Cost of damage C_f	**10,23 billion yen**	**Parameter**

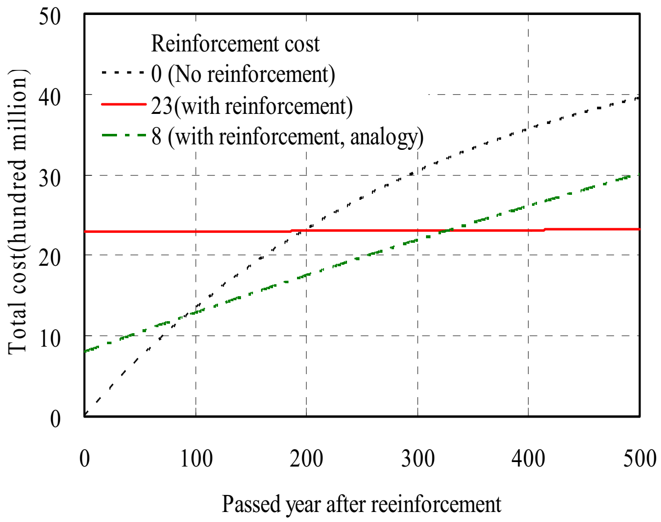


Figure12. Results of LCC estimation in case of damage amount of 50 hundred millions.

Taking this result and elongating the target period (transverse axis) infinitely, the total costs will be asymptotic without limit to the presumed cost of damage. This indicates that as the target period lengthens, the probability of fill-dam failure during that period would become 1 and consequently would require the presumed cost of damage. Furthermore, when the scale of reinforcement is smaller than what has been examined in this study, the intercept (reinforcement cost) will decrease and the gradient becomes greater (approximates to non-reinforcement), and thus would intersect with the non-reinforcement function at an earlier period, which can be interpreted as a more reasonable measure. As observed above, it can be deemed possible to examine the need of reinforcement and the reasonable scale of reinforcement from the LCC assessment that combines the presumed fill-dam characteristic (fragility curve) and earthquake environment of the target location point (earthquake hazard curve).

5 CONCLUSION

This study examines the safety of fill dam under level 2 earthquake motions through dynamic response analyses, using a virtual fill-dam. Considering the influence of the deformation volume of dams owing to the variation of dam materials, the failure probability of banking is calculated for a reinforcement presuming certain earthquake motions, earthquake hazards and a single reinforcement method. The LCC assessment for a degraded fill-dam considering the risks of large-scale earthquake motions was conducted through the above-mentioned procedures.

This study treats the cost of damages for the model fill-dam as a provisional damage cost, but in subsequent studies the actual fill-dam and region should be assumed to actually calculate the flood damages and examine the LCC. The authors hope to assess LCC according to reinforcement costs by changing the reinforcement method in future studies.

REFERENCES

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