The next-generation risk assessment method about the effect of a slope and foundation ground on a facility in a nuclear power plant

Susumu Nakamura[a], Ikumasa Yoshida[b], Masahiro Shinoda[c], Tadasi Kawai[d], Hidetaka Nakamura[e] and Masaaki Murata[f]

[a] Dept. of Civil & Environmental Eng., College of Engineering, Nihon University, Koriyama, Japan
[b] Tokyo City University, Tokyo, Japan
[c] Railway technical research institute, Kunitachi, Japan
[d] Tohoku University, Sendai, Japan
[e] Japan nuclear regulation authority, Tokyo, Japan
[f] Mitsubishi heavy industry, Takasago, Japan

Abstract: From the background of the accident of the nuclear power plant caused by The 2011 off the Pacific coast of Tohoku Earthquake, the view about the effect of ground such as a slope and a foundation on the nuclear power plant in not only the regulatory guidance for seismic design but also the standard about seismic probabilistic safety assessment was also revised remarkably in JAPAN. A view of the limit state to evaluate the fragility curve about the effect of a slope failure on the facilities described in the latter standard was improved by geotechnical approach such as considering the dynamic behavior of geomaterials after collapse. The view should be called the next-generation assessment about slope stability. The limit state regarding on the slope failure on the facility was specified based on an experimental consideration. Here, the view is reported with experimental results obtained from shaking table tests and its numerical analysis. The experimental examples are also described to verify the effect of countermeasure against the seismic action exceeding the limit state. As a examples to evaluate the movement of rock block induced by slope collapse, the numerical method and its example of application were also described:

Keywords: Slope, Limit state, Shaking table test, Countermeasure, Numerical analysis

1. INTRODUCTION

The regulatory guidance for seismic design of nuclear power plant in JAPAN was revised in 2006. As a phenomenon accompanying an earthquake, the consideration about the effect of a slope failure around a reactor building on the safe performance of a nuclear power plant was newly specified. Then, the standard about seismic probabilistic safety assessment was published by Atomic Energy Society of Japan in 2007, and the revision was carried out in 2013. Although the standard in the 2007 was also considered about the effect of a slope failure on nuclear power plant, the view about the effect was revised remarkably based on the loss accident of the external power source by collapse of the power transmission steel tower by slope failure of the embanked ground which produced within the Fukushima Daiichi nuclear power plant by The 2011 off the Pacific coast of Tohoku Earthquake. Furthermore, the range of a slope for which consideration of AM is required was expanded greatly from the natural slopes around facilities to the slope of embanked ground around a passageway.

An important matter in the revision is a view of the limit state to evaluate the fragility curve about the effect of a slope failure on facilities in nuclear power plant. Based on the procedure described in the 2007 standard about the slope stability assessment around a reactor building in nuclear power plant, the effect of a slope failure on the facility has been estimated by considering a possibility to cause a slope failure as a limit state in the safe side. As for the revision, the state that rock mass reached at a facility after collapse occurred was considered as a limit state. The movement of collapsed rock and soil are used as index to evaluate the limit state. These limit states consider the ground behavior as either the structural damage or the functional damage, and is used to evaluate the damage probability of facilities indirectly. On the other hand, in order to evaluate directly either the structural damage or the functional damage to facilities after collapse of slope, it was specified that an action such as an impulse force at the time of a rock mass reaching to a facility was evaluated as a hazard which acted to a

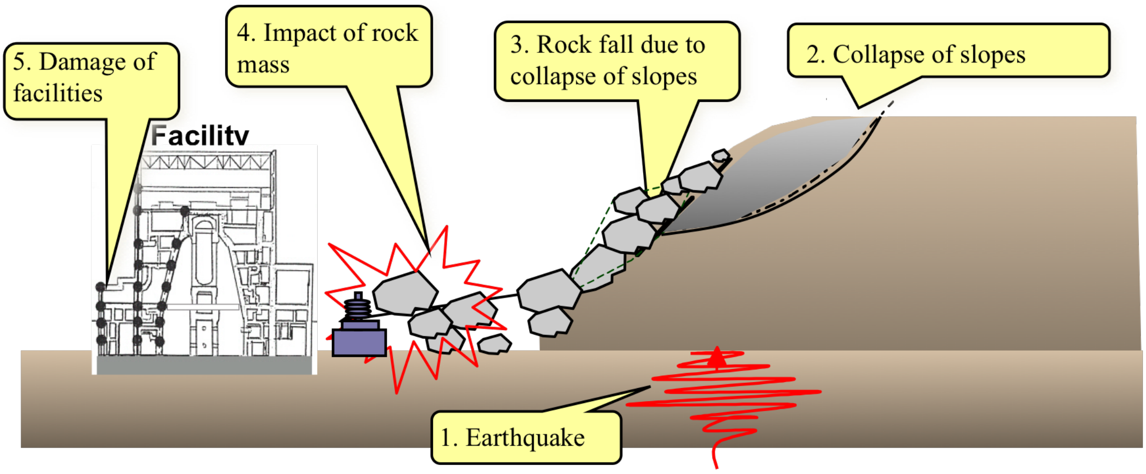


Figure 1 Image of behavior of rock block after collapse of slope

structure. While the previous limit state is indirect evaluation of the effect of a slope failure on a facility, it is the difference among both that the latter limit state is direct evaluation of the effect. The latter limit state was specified based on the experimental results described in Annex of the new standard

This revision associated with the effect of slope failure was carried out by the outcomes of not only experimental study about the slope failure mechanism but also numerical study about slope failure behavior ground response deformation in slope based on many shaking table tests of slope models. These studies had been conducted for the development of the slope stability assessment technology by Japan nuclear energy safety organization. The outcome of Japan nuclear energy safety organization was published as "a guideline of the design and risk evaluation against the seismic stability of the ground foundation and the slope, 2013". In this paper, the fundamental concept about the effect of slope failure on nuclear power plant is described by using not only some experimental examples but also some verification examples of numerical method to evaluate the collapsed behavior of slope based on the experimental results. The important feature of the guideline is to have used the numerical method to be able to evaluate seamlessly the behavior from a seismic response to collapse.

2. THE RISK SCENARIO AND LIMIT STATE CAUSED BY SLOPE FAILURE

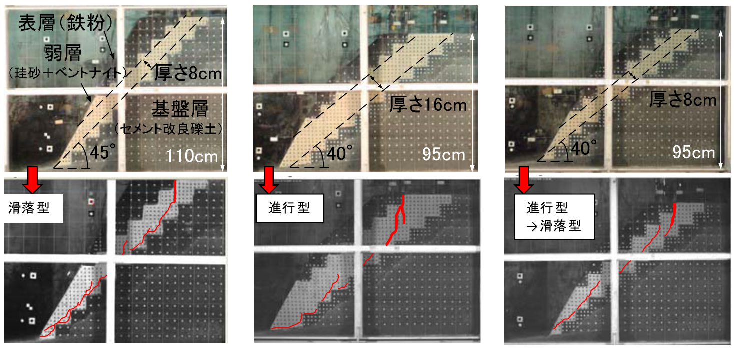
Many of the nuclear power plants in Japan are located in a coastal area because of the necessity to acquire a lot of cooling water. According to the geographical condition around the seashore, a natural slope may exist near some of the nuclear power plants. In the plant, we are afraid of about the secondary damage caused by not only the collision of the rock masses to facilities but also slope failure against strong earthquake ground motion, and it becomes important to take into consideration the accident scenario by slope collapse.

The outline of a accident scenario in a nuclear power plant caused by the slope failure against strong earthquake ground motion is shown in Figure 1. As one of accident scenarios, the following scenario is thought about. First of all, a slope failure occurs due to strong earthquake motion. Then, roll of some rocks in a slope occurs, and collides with a reactor building, outdoor important apparatus, or equipment. And it is assumed that apparatus in a building and outdoor apparatus are damaged and that the functional safety of facility in the plant is lost. Moreover, as another scenarios due to slope failure, a collapse of a power transmission steel tower, interruption of passageway for the AM, the effect on AM apparatus such as a water supply car and the power supply car on the slope are also assumed.

In the risk assessment to the effect of a slope in the plant, the fragility characteristics of each facility is evaluated by using the relationship between the limit value and response value. The both values are specified by the physically meaningful index associated with the limit state of some facilities having the important functional safety of the nuclear power plant, and are obtained by the realistic values modeled by characteristic value whose uncertainty is considered adequately.

The following two limit states can be considered for evaluating the effect of unstable behavior of slope on the facilities.

i) The state that a collapse of slope occurs.



(a) Slip-down type (b) Progressive collapse type (c) A type changed from (b) to (a)

Figure2 Collapse modes for three small-scale slope models by shaking table tests

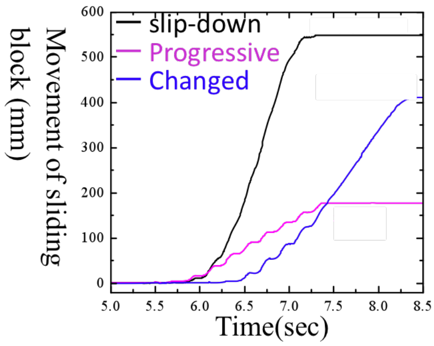


Figure 3 Time histories of slide block for each collapse mode

ii) The state that the damage of facilities in the plant due to the collapse of slope causes the loss of the functional safety.
Here, the limit state described in i) specifies the effect of slope unstability on the facilities in the plant indirectly by the possibility of collapse of the slope. The safety factor of slip (resistance / acting shear force on a slip surface) has so far been used as an evaluation index. However, even if the safety factor of slip is less than 1.0, the slope failure behavior during earthquake differs significantly depending on not only the strength and deformation characteristics but also the ground structure. The method to evaluate a possibility of occurrence of slope failure by using the movement of rock mass obtained numerically based on the results of the shaking table tests was proposed by authors in order to evaluate a actual failure behavior.

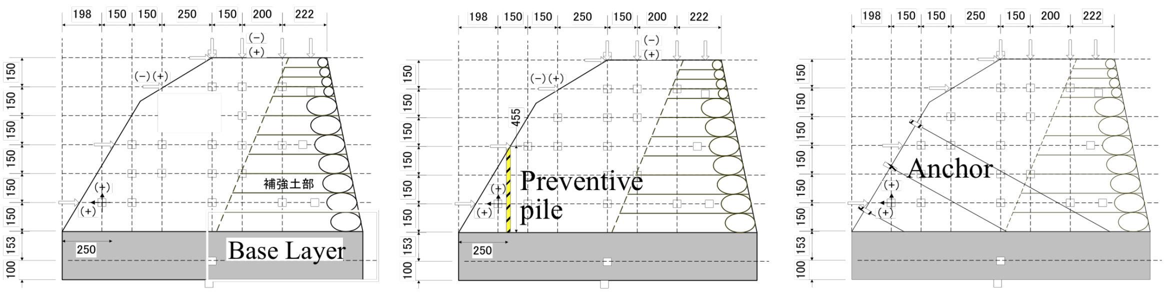
On the other hand, the limit state described in ii) is the state that the damage of the facilities due to the movement and impulse force of the collapsed ground which reached the reactor building, the switching station, and the condensate tank causes the loss of the functional safety directly. The state may estimate not only as a hazard about the action to an facility like impulse force but also as the movement of the rock mass which is indirectly equivalent to the damage of the facilities to cause the loss of the functional safety. These were evaluated by using the numerical methods which were verified based on the comparison of the experimental result about the collapse of slope as described in Chapter 3.

3. FAILURE MECHANISM OF A SLOPE AND EFFECT OF COUNTERMEASURE BY EXPERIMENT

3.1. Failure mode and the limit state of a slope

The collapse mode of a rock slope can be classified into collapse of slide down, slide collapse, toppling collapse, and buckling collapse. A slide type was selected as a target collapse, collapse which generates in the slope with the decreased strength by the surface weathering, Slide collapse which generates within the almost horizontal weak layer of a slope, collapse which generates in weak layers, such as seam were used as the experimental slope model. Based on the experiment, the behavior after the collapse was also evaluated in accordance with the relationship between the failure mode and the slope characteristics. Here, a failure mode corresponds to the slope state of changing to the unstability from stability. The characteristics are specified by the ground structure, strength and deformation.

In order to evaluate those characteristics, the shaking table tests by using some small-scale models of rock slope, the medium scale models and the large scale models have been carried out. Here, based on some results obtained by the shaking table tests [1] [2] of the small-scale models of rock slope, the relationship between failure mode and a limit state are described. The typical failure modes are shown in Figure 2. The slope models consisted of a base layer, a weak layer, and a surface layer. And inclination and thickness of the weak layer were changed as an experimental parameter. The base layer was made by using stability treated particle size adjustment rubble with the cement to regard the layer as the stable rock layer. Furthermore, the layer was completely fixed with container by an anchor to control slide during shaking. The weak layer was made by using the materials which mixed bentonite with quartz sand 6 at 1% of weight ratios. The surface layer was made by using the materials which mixed bentonite at 10% of weight ratios to iron powder for keeping sufficient inertia force. In addition, stepping was used as the structural model around the layer boundary to prevent the sliding in the layer boundary.



(a) Non-reinforced (b) Preventive pile (c)Anchor

Figure.4 Reinforced slope model and arrangement of measurement

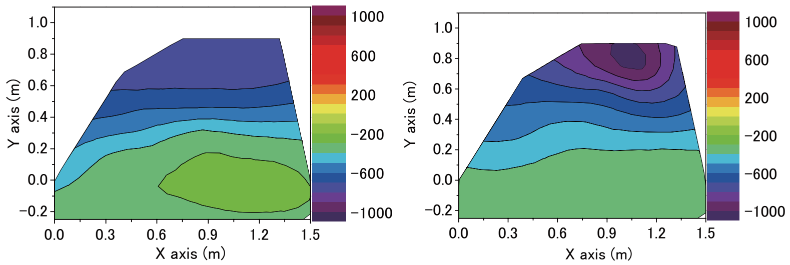
In order to measure the collapse behavior of the slope model during shaking every moment, marked points for image analysis were installed in the side face of the model. By using a high-speed camera, displacements in the two-dimensional plane were measured during shaking. The input waveform was assumed ten cycle of sine wave which have a period of 5Hz. The amplitude of the waveform increased gradually by 100Gal from 100Gal. And the shaking table test was finished at the stage which reached collapse. Comparison of time history of the movement of the surface layers after the failure was shown in Figure.3. After a tension crack occurred near the shoulder of slope, surface layer collapsed as a block along slip surface which occurred in a weak layer. At first, the behavior of the slip-down type that a surface layer on the slip surface suddenly slid down after collapse was shown in Figure.3a). The slip-down collapse occurred suddenly at the shaking at input acceleration 400gal. After failure occurred due to the formation of the slip surface, the behavior of the progressive collapse mode which surface layer on the slip surface moved gradually was shown in Figure.3b). The behavior occurred at the shaking at input acceleration 500gal. The behavior changed to the slipping down type after a progressive collapse occurred as shown in figure 3b) was shown in Figure.3c). Although the behavior according to a progressive collapse mode occurred during the shaking at input acceleration 500gal, slip-down collapse occurred suddenly after collapse at toe of slope.

As the behavior of the progressive collapse mode that rock block slides gradually during shaking, a sliding block stops when slide movement after shaking is not large. For this reason, the slope stability can be evaluated reasonably by specifying a standard value in a safe side. However, since collapse of either toe of slope or surface layer may be induced during the behavior of the progressive collapse mode, collapse mode may change to slip-down type from progressive collapse mode. Therefore, it is important to make sure of a collapse behavior. On the other hand, since collapse mode of slip-down type is a phenomenon that the whole slide block slips down to the toe of slope in an instant, the influence on the facility in a nuclear power plant near the slope is serious. Thus, by judging the collapse mode appropriately according to a ground structure, a strength characteristic and a deformation characteristic, it becomes possible to set up a suitable limit state.

3.2. Effect of countermeasure

When a collapse of a slope affects on the facilities in a nuclear power plant, it is necessary to take a adequate countermeasure. Although the anchor and preventive pile which have been used generally as countermeasure will be executed even in a nuclear power plant, the seismic design method to be able to consider the influences has not established so far. In order to establish the method to evaluate the stability of the slope with reinforcement against strong earthquake motion, not only the dynamic response characteristics of the slope with reinforcement but also the effect of reinforcement are made clear based on the experimental results obtained by the shaking table tests of small-scale reinforced slope model. The experimental results are described here.

There are three kinds of slope models , an unreinforced slope, the slope reinforced by anchor, and the slope reinforced by preventive pile. Shape of slope model and arrangement of measuring instruments are shown in Figure 4. The shapes of slope models are equal each other. The height and the width are 1.15 m and 1.5 m respectively. This model consists of a base part, a general part, and a reinforced part, The material of a base part is improved gravel mixed with cement and the general part is imitating the weathering layer. The improved gravel was made of a gravel, cement, and water which are 100 vs 7 vs 4 as ratio of weight. The general part and the reinforced part were considered to satisfy not only the condition that they are stable when making model but also the condition that they collapse due to the strong shaking. The general part was made by using silica sand 6 grade 100, bentonite 1 and water 10 as the ratio of weight. In reinforced part, geo-net with the low strength was laid every 10cm. The vibration condition of a shaking table is the same with that mentioned in 3.1.



(a)Non-reinforced slope (b) Preventive pile

Figure 5 Contour of Horizontal acceleration when the maximum acceleration generates at the shoulder of slope

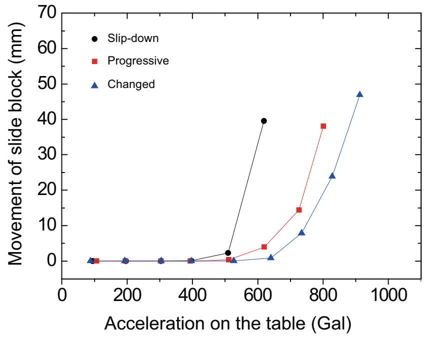
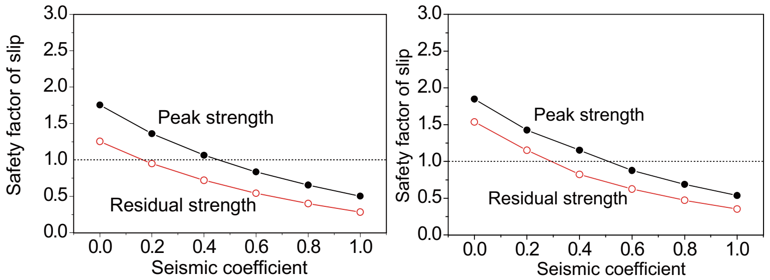
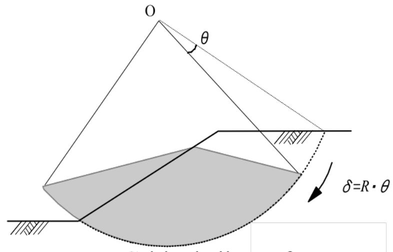


Figure 6 Relationship between acceleration at shaking table test and settlement at the shoulder of slope



(a) Slip-down b) Progressive collapse

Figure.7 Stability analysis results by a circle slip method



Critical slip surface

Figure.8 Image of New mark method

Contour lines of a horizontal acceleration at the time that the horizontal acceleration at the shoulder of slope becomes minimum are shown in Figure 5. A case of non-reinforced slope and the case of the slope reinforced with preventive pile are shown. This figure indicates at the state that the inertia force to the direction of the slope front becomes maximum. Accelerations on the table are 800gal for non-reinforced slope and 600gal for reinforced slope. It is found that the response at the shoulder of slope is amplified greatly against the bottom of slope. Especially, as for the reinforced slope, it is found that the amplification becomes larger due to the increase of the stability of the slope according to the effect of countermeasure. The relationship between the amount of shoulder subsidence and the acceleration on the table is shown in Figure 6. The relationship between the amount of shoulder subsidence and acceleration on the table is shown in Figure 6. Although the non-reinforced model collapsed at 600gal, the model reinforced by a preventive pile and the model reinforced by an anchor collapsed at 800gal and at 900gal, respectively. Based on these results, the effect of countermeasure was verified.

4. EVALUATION METHOD OF THE LIMIT STATE AND THE VERIFICATION

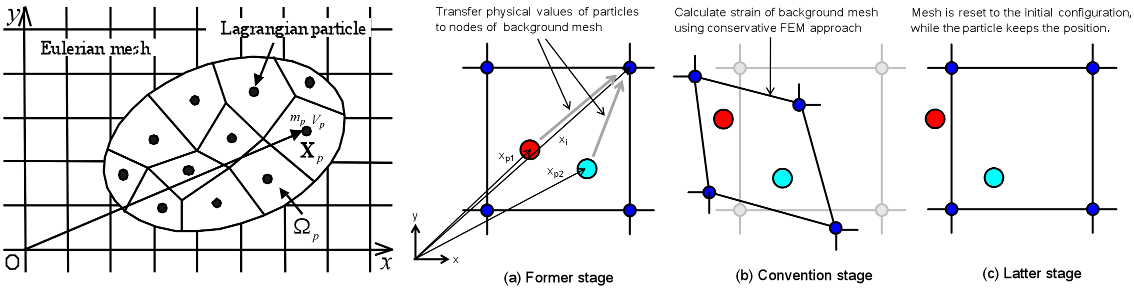
4.1. Examination of the limit state about the stability by the sliding safety factor

Stability analysis of the slope model in which different collapse mode occurred as shown in figure.2 was carried out by the circle slip method using seismic coefficient as a seismic action. As the different collapse modes, slipped-down type, progressive collapse type and type changed slipped-down type from progressive collapse type were selected. The analysis results of a slipped-down type and a progressive collapse type are shown in Figure 7 as a representative case. Using peak strength and residual strength as strength characteristics, The safety factors of those slope models were calculated against the horizontal seismic coefficient changed at every 0.2 from 1.0 to 0.0. First of all, as a result of shaking table test against each collapse model, accelerations which collapses generated, became slipped-down type 400Gal, progressive type 500Gal, and progressive/slipping-down type 500Gal. Next, the validity of stability analysis verifies by checking that the slide safety factor becomes about 1.0 when the horizontal seismic coefficient 0.4, 0.5, and 0.5 acted to each model. As a result of stability analysis, the safety factor in each collapse mode became slipped-down type 1.065, a progressive type 0.987, and a progressive/slipping-down type 1.065. Stability analysis was verified to be appropriate based on the results that the safety factor of slip for each collapse mode was almost 1.0.

4.2. Estimate of movement of rock block for progressive collapse type by New Mark method

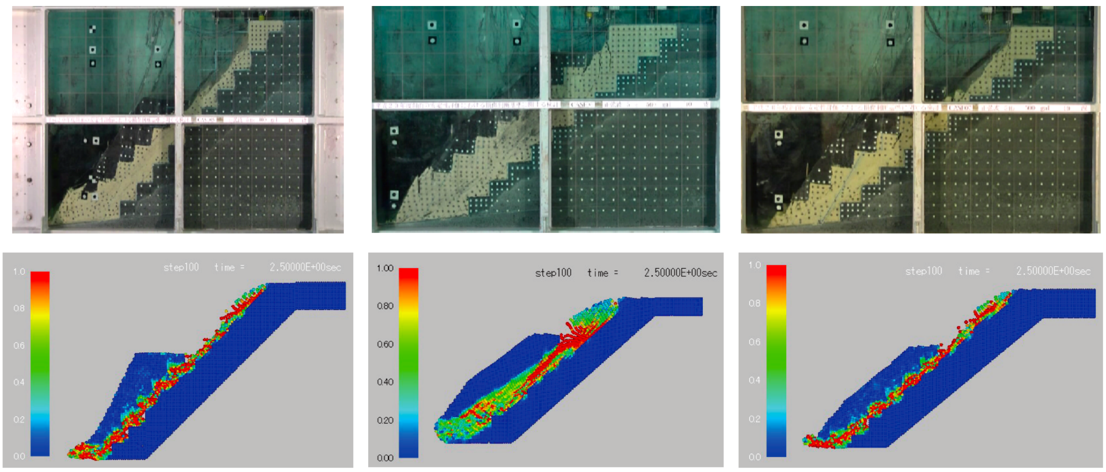
Table 1 Comparison of experimental results with numerical results

Collapse mode	Experimental results	Numerical results
Slip-down	560mm	Not applicable
Progressive	195mm	182mm
Changed type	70mm (progressive)	40mm (Progressive)



(a) MPM model (b) Analytical flow of the MPM

Figure 8 Schematic figures of the model and the analytical flow of the MPM



a) Slip-down b) Progressive c) Changed type from progressive to slip-down

Figure 9 Comparison of deformation obtained by MPM with experimental behaviors

In order to make sure of the deformation of the rock slope after collapse, deformation analysis was carried out by the Newmark method as shown in Figure.8 using the material properties obtained from the laboratory tests. Newmark method is the method to evaluate a sliding displacement on the sliding surface by integrating acceleration as a inertia force to sliding block after generating a sliding surface (critical slip surface) when the safety factor on the sliding surface calculated by the circle slide method becomes equal to 1.0. Furthermore, in the case that the safety factor is more than 1.0, the deformation analysis is carried out by using a cohesion and a internal frictional angle at a peak strength. And in the case that the safety factor is less than 1.0, the deformation analysis is carried out by using a cohesion and a internal frictional angle at a residual strength. Based on these processes, more realistic displacement is possible to be estimated.

An analysis result is shown in table 1. Calculation of a slipped-down type by Newmark method was not completed since a yield seismic coefficient was negative value at the time to calculate a displacement. The calculated value for slope model with an progressive collapse type is good agreement with the experimental value. Then it is found that Newmark method has a good applicability. Moreover, for a slope model with collapse mode which changes from a progressive collapse type to a slipped-down type, the experimental value was good agreement with the calculated value at the time when the collapse mode changed from progressive collapse type in early stages of shaking. As mentioned above, it is thought that the Newmark method has a good applicability to evaluate the movement of slide block in the mode which slope collapses gradually.

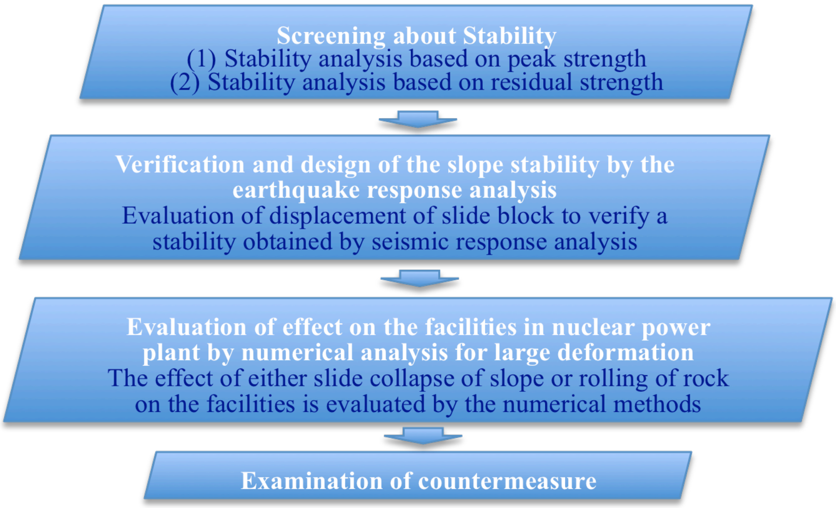


Figure 10 Design procedure about the slope stability

4.3. Estimation of the movement of a rock block to each collapse mode by a particle method

In order to establish the numerical method which can evaluate the behavior before and after collapse of the rock slope, a particle method among some methods about large deformation analysis was applied to evaluate the experimental results. The material properties obtained from the laboratory tests were used to make a slope model. The particle method is a method called MPM (Material Point Method), and is a kind of a particle method called PIC (Particle in Cell) which calculates an advective term by particles and calculates other clauses with a lattice. This method uses the technique to calculate an advection by the perfect Lagrange method using particles, therefore has following characteristics; One is hard to generate numerical diffusion. The other is that the boundary where particles can move is possible to be easily set up in a lattice. Moreover, since MPM performs the formation of a weak form type and discretization using the interpolation function to the lattice like FEM as shown in Figure 8, it can utilize the numerical-analysis technology of FEM accumulated until now.

As for modeling the slope model mentioned in 3.1, perfect-plasticity model is used as a constitutive relationship in weak layer. The base layer was modeled as an elastic body. Although material properties were obtained from the triaxial compression test, a cohesion at residual strength was set to 1.0kPa for the slope model except the progressive collapse type. The value was set as a strength at the time when a safety factor by a circle slip method becomes 1.0 at a horizontal seismic coefficient 0.0. With the slip-down type, the idea about setting this value is based on the evidence that horizontal acceleration in surface layer becomes near the zero, after a slip surface occurs in a weak layer.

Not only the contour lines about the maximum shear strain but also the collapse situation of the slope model by a shaking table test is shown in Figure 9. In all cases, it is found that the behavior of collapse obtained by MPM analysis is good agreement with an experimental result. Furthermore, different collapse behaviors according to the collapse modes in which a slip-down type collapses suddenly and a progressive collapse type collapses gradually have reappeared in MPM analysis. In addition, although generation of the tension crack in the collapse behavior was recognized clearly by a shaking table test, the behavior was not able to imitate completely in numerical analysis by MPM. Hereafter, it is necessary to develop in consideration of the tension characteristic of the ground under low confined pressure.

5. VIEW OF THE DESIGN ABOUT SLOPE STABILITY

Based on not only the knowledge about the collapse mode and limit state of a slope mentioned above but also the verification about the numerical method to evaluate the index associated with limit state, the design procedure about the slope stability which will be required for risk assessment is shown in Figure 10.
The outline is as follows.

First step : Screening about stability

Slope stability is evaluated based on the safety factor of slip using the dynamic response characteristic of slope obtained by the seismic response analysis against reference earthquake ground motion. Here, for the slope judged as a safety factor of slip becoming below a required value, stability analysis against a slope's own weight is carried out using a residual strength. For a case that a safety factor obtained by the stability analysis against a slope's own weight is below a required value, the slope is judged to have a high risk to generate the collapse of a slipped-down type. And the effect of rock movement such as sliding and falling is evaluated quantitatively based on the following procedures.

Second step : Verification and design of the slope stability by the earthquake response analysis
For the slope which satisfies with a required safety factor of slip by stability analysis against a slope's own weight as mentioned in previous procedure, a possibility that collapse of slipped-down type occurs is considered to be small. Therefore, verification and design of slope stability are performed based on a response displacement of slope calculated from the dynamic response of slope obtained by seismic response analysis.

Third step : Evaluation of effect on the facilities in nuclear power plant by numerical analysis for large deformation
The effect of either slide collapse of slope or rolling of rock on the facilities is evaluated by the numerical methods which can consider a collapse behavior of slope. First of all, it is evaluated whether a rock block generated by collapse of slope arrives at a facility. Next, impulse force is evaluated when judged with reaching to the facilities. Finally, an adequate countermeasure is installed when it is thought that the effect is serious.

Fourth step : Examination of countermeasure
As countermeasures installed on slope around the facilities in a nuclear power plant, there is an earth removal work, an anchor work, a preventive pile work. The design method of countermeasure is based on the design method of a non-reinforced slope. By considering two or more slip surfaces which have the high possibility about the loss of slope stability, slope stability is evaluated based on the safety factor of slip obtained by the dynamic response of the slope.

6. CONCLUSION

In order to establish the stability assessment procedure of a next generation to evaluate the effect of the collapse of the slope on the functional safety of a nuclear power plant, the developed view in a geotechnical aspect based on the experiment and numerical approach was described. The limit states which depend on the collapse modes were specified based on the experiment. Furthermore, the applicability of numerical method and the effect of countermeasure were made clear. The numerical method was used to evaluate movement of rock block after a collapse of slope, and is able to evaluate seamlessly from a dynamic response to a collapse behavior. The major results in this report are described as follows.

1) The following two limit states were specified for evaluating the effect of unstable behavior of slope on the facilities having the important functional safety of the nuclear power plant.
i) The state that a collapse of slope occurs.
ii) The state that the damage of facilities in the plant due to the collapse of slope causes the loss of the functional safety.

2) Based on the experimental results obtained by the shaking table tests using three scale models of slope, it was made clear that there are three modes of collapse. The first type was a slipped-down type. Second type was a progressive collapse type. The third type was a type which changed from a progressive collapse type to a slipped-down type. A progressive collapse mode is that slide block stands it still when the movement of the block is not large after shaking. For the mode, considering the failure induced at a toe of slope and a surface, it was showed that the slope stability is able to evaluate rationally by using the appropriate value in a safety side. Moreover, the acceleration when collapse occurred at the slope reinforced by not only a preventive pile work but also an anchor work was larger in comparison with the non-reinforced slope, and the effect of countermeasures was verified.

3) As a result of stability analysis by a circle slip surface method, stability analysis was verified to be appropriate based on the results that the safety factor of slip for each collapse mode was almost 1.0.

4) Newmark method has a good applicability for the mode which slope collapses gradually.

5) The collapse situation of the slope model by a shaking table test is good agreement with the collapse behavior obtained by MPM analysis. Furthermore, different collapse behaviors according to the collapse modes in which a slipped-down type collapses suddenly and a progressive collapse type collapses gradually was able to reappear in MPM analysis. In addition, generation of the tension crack in the collapse behavior was not able to imitate completely in numerical analysis by MPM.

6) Based on not only the knowledge about the collapse mode and limit state of a slope as mentioned above but also the verification about the numerical method to evaluate the index related with limit state, the design procedure about the slope stability which will be required for risk assessment was established.

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