Simulation of Sliding Failure Occurred at Highway Embankment in 2011 Great East Japan Earthquake

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Abstract: In the 2011 Great East Japan Earthquake, the highway embankments were almost less damaged comparing with the past earthquakes in Japan. But the only one embankment close to the Naka Interchange at Joban Highway was damaged a little severely and remarkably because of two interesting phenomena. One phenomenon is the toe-sliding failure observed at the shallow soft base ground and the other one is one-side slope sliding failure. It can be seen that the increase in the degree of saturation at embankment body or the direction of the ground motion or the interaction between the strength of the base ground and the embankment body are involved in the stability of the embankment by modifying the phenomenon by analytical approach such as circular sliding method and dynamic response analysis. Through this research, some important lessons can be obtained for future seismic countermeasure of embankments.

Key words: Highway embankment, circular sliding method, dynamic response analysis.

1. Introduction

In the 2011 off the Pacific Coast of Tohoku Earthquake on March 11 in Japan which will be named the 2011 Great East Japan Earthquake in the followings, the highway embankments were almost less damaged comparing with the ones occurring in the past earthquakes in Japan. The only one embankment close to Naka Interchange at Joban Expressway was damaged a little severely and remarkable because of the following two interesting phenomena from the engineering view points. One point can be indicated that the embankment constructed on the base ground with the soft layer of 3.6 m thick, the sliding failure form of the embankment was the toe sliding one. Second one can be indicated that the whole form of the embankment was almost symmetrical, one-sided slope of the embankment was slid and the opposite side slope was not slid.

In order to simulate the toe sliding failure of the embankment on the soft ground occurring in this

earthquake, the circular sliding method with the design seismic coefficient of 0.0, 0.1, 0.2 and 0.3 is applied to the embankment whose base ground model is estimated with use of the limited data obtained just after the earthquake. Furthermore, to verify the reasons why the slopes were slid or not at the same cross section of the embankment, both the circular sliding method and the finite element method using the seismic ground motion estimated from the observed ground motion in this earthquake are applied.

Through this study, the possibility of the toe sliding failure on the soft base ground can be clarified firstly, comparing the soil properties of the embankment body with that of the base ground. Secondly, the reason why the embankment slopes in case of the symmetrical cross section of the embankment was slid or can not be clarified only with use of the circular sliding method. Then this study focuses on the relation between the soil strength such as cohesion and/or internal friction angle and the water condition expressed by degree of saturation which was indicated by Uchida et al. [1] and Athapaththu et al. [2]. Referring to research results by Athapaththu et al. [2], the possibility and

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importance to consider the reduction of the soil strength of the unsaturated embankment body induced by the increase of the degree of saturation is discussed and indicated to investigate the sliding occurrence of the embankment slope in detail. Furthermore, the direction of the seismic ground motion can be clarified to be very important to estimate the sliding failure with use of the dynamic response analysis.

As the results of this study, very important lessons on the sliding failure of the embankment can be obtained for the future.

2. Outline of Damaged Expressway Embankment

2.1 Location of Objective Site

Although the 2011 Great East Japan Earthquake was the plate boundary type with the magnitude of 9.0, the highway embankments were almost less damaged comparing with the ones in the past earthquakes in Japan. Because the seismic source area expanded widely from Tohoku Region to Kanto Region, very huge ground motion was observed widely at East Japan.

Fig. 1a shows the location of the embankment at Joban Expressway investigated in this study which is about 100 km far from Tokyo [3]. Fig. 1b shows the satellite photograph [4] where the damaged embankment is covered with blue sheets about 1 km away from Naka Interchange which locates in the right direction of the figure. It is necessary to note that

the highway lanes from the right side to the left one in Fig. 1b is named “Up Lane” to Tokyo where the embankment was slid and the reverse direction is named “Down Lane” from Tokyo in this paper.

2.2 Outline of Damaged Embankment

Fig. 1c reported by the East Nippon Expressway Co. Ltd. [3] just after the earthquake shows the sliding failure at the embankment crest with the maximum displacement of 1.5 m along the distance of about 150 m together with a slight sliding occurring along the boundary of the two traffic lanes. Because the one lane of the Up Lane and two lanes of the Down Lane were not damaged, they could be used for the recovery works after the earthquake. So the damage level at this site can be indicated to be medium considering the performance-based design. Fig. 1c shows the toe sliding failure of the embankment on March 13 because the side road and the base ground neighboring the embankment are not deformed. Furthermore, the sand boils induced by soil liquefaction could not be observed at the base ground surface.

2.3 Conditions of Base Ground and Embankment

The analytical modeling of the cross section of the damaged embankment and the base ground is conducted based on the data shown by the East Nippon Expressway Co. Ltd. [3] as shown in Fig. 2. The conditions of the base ground are assumed based on the soil classification and N-value estimated by the

Fig. 1 Objective site and embankment investigated.

Fig. 2 Fundamental model on cross section of embankment and base ground.

boring data which were measured before the earthquake about 60 m away from the slid embankment as shown in Fig. 3 [3]. Based on Fig. 3, the depth of the stiff diluvium ground can be estimated to be 9.45 m deep and the above alluvium ground are divided into three layers such as layer 1 of 5.8 m thick, layer 2 of 1.15 m thick and layer 3 of 2.05 m thick as shown in Fig. 2. Especially, the soft ground at this site is assumed to be 3.6 m deep from the ground surface.

The embankment body is modeled with four layers such as layer 4, layer 5, layer 6 and layer 7 shown in Fig. 2 where layers 5, 6 and 7 are the pavements except for the asphalt and modeled as the sandy gravel in this

study. The thickness of layer 7 of 0.6 m is measured from Fig. 4 and that of layers 5 and 6 is assumed to be half of that of the layer 7.

The soil properties of the embankment body are assumed based on the results of the unconfined compression tests of the slid soils which were conducted on March 13 by the East Nippon Expressway Co. Ltd. [3].

Based on the mentioned above, the saturated unit weight, wet unit weight, internal friction angle and cohesion of each layer is summarized in Table 1 referring to the Guideline of Retaining Wall [5], and Fig. 2 and Table 1 is assumed to be the fundamental model for the circular sliding method in this study.

Feature	Thickness (m)	Depth (m)	S-wave vel. (m/s)	N-value
topsoil	0.45	0.45	86	0.64
cohesive soil	2.05	2.50	86	0.64
cohesive soil	1.15	3.65	123	1.88
sand gravel	5.80	9.45	246	29.0
rock surface			330	

Fig. 3 Soil profile of base ground applied to damaged embankment.

Fig. 4 View of pavement and gravelly fine sand layer.

Table 1 Contents of embankment, pavement and base ground in Fig. 2.

Number	Saturated unit weight (kN/m ³)	Wet unit weight (kN/m ³)	Internal friction angle (°)	Cohesion (kN/m ²)
1	20.00	20.00	35.86	0.00
2	20.00	16.00	0.00	18.75
3	20.00	16.00	0.00	6.40
4	20.00	19.00	33.70	0.00
5	20.00	20.00	30.00	0.00
6	20.00	20.00	35.00	0.00
7	20.00	20.00	40.00	0.00

Table 2 Soil properties of modified model.

Number	Saturated unit weight (kN/m ³)	Wet unit weight (kN/m ³)	Internal friction angle (°)	Cohesion (kN/m ²)
1	20.00	20.00	35.86	0.00
2	20.00	16.00	20.00	18.75
3	20.00	16.00	15.00	13.00
4	20.00	19.00	20.00	5.00
5	20.00	20.00	30.00	0.00
6	20.00	20.00	35.00	0.00
7	20.00	20.00	40.00	0.00
8	20.00	19.00	20.00	10.00

3. Simulation of Toe Sliding Failure with Use of Circular Sliding Method

3.1 The Method to Conduct Simulation

In order to clarify the reason why the sliding failure form takes on the toe sliding failure, the simulation with use of the circular sliding method is conducted on the fundamental model and then the fundamental one is modified by trial and error to simulate the toe sliding failure, changing the soil conditions of each layer at the embankment body and the soft layer of the base ground.

3.2 Simulation in Case of Fundamental Model

The minimum safety factor with the circular sliding method in case of the fundamental model shown in Fig. 2 can be calculated to be 0.56 which is smaller than 1.0 in the general condition as shown in Fig. 5. Because the embankment in general condition was stable before the earthquake, the minimum safety factor of 0.56 is necessary to be more than 1.0. Furthermore, the sliding failure form calculated takes on the base sliding failure which means that the sliding plane goes through the soft alluvium layer 3 and is different from the actual phenomena which

occurred.

The above two results indicate that the fundamental model estimated firstly is different from the actual conditions before the earthquake. As for the reason, it can be considered that because the fundamental model assumed in Fig. 2 and Table 1 is based on the data obtained at the site of about 60 m away from the slid embankment, the conditions of the soil properties at the slid embankment are different from the fundamental model.

3.3 Modification and Simulation in Case of Modified Fundamental Model

The modification of the fundamental model is conducted by trial and error to secure the minimum safety factor in general condition larger than 1.0 and simulate the toe sliding failure form at the slid embankment as follows.

As for the soil layers of the embankment body considering the collapsed profile in detail, it can be clarified that the upper part of the embankment body whose thickness is 5.3 m seems to be composed of the gravely fine sand and the lower part of that whose thickness is 3.3 m seems to be composed of the loam.

Then the embankment body is divided into two layers such as layer 4 and layer 8 shown in Fig. 6.

Additionally, the ground water level which is 0.59 m below the bottom of layer 4 or the surface of layer 6 is assumed as shown in Fig. 6. The unit weight of water is 10.0 kN/m^3 .

The trial and error to modify the soil properties of each layer in Fig. 6 satisfy the two conditions such as the minimum safety factor in general condition and the toe sliding failure in both general condition and seismic one applied with seismic coefficients of $k = 0.1, 0.2$ and 0.3 . Table 2 shows the final results obtained by trial and error and Fig. 7a shows the example where the minimum safety factor of 1.085 is larger than 1.0 and the toe sliding failure form is performed in general condition.

Furthermore, the sliding failure forms applied with the above seismic coefficients are also performed based on Table 2 as shown in Fig. 7b which is the example of $k = 0.2$. Comparing Table 2 of the modified model with Table 1 of the fundamental model, generally speaking, the toe sliding failure can

be assumed to occur when the strength of the base ground is relatively larger than that of the embankment body.

As mentioned above, the possibility and the conditions to perform the toe sliding failure at the embankment occurring in this earthquake can be confirmed comparing the soil properties of the embankment body and the soft base ground. The detail process of the trial and error can be referred to Murakami et al. [6].

4. Investigation on Difference between Damaged Slope and Non-damaged Slope

4.1 Investigation on Symmetrical Cross Section with Use of Circular Sliding Method

Because the slid embankment is almost symmetry, the sliding failure can be estimated to occur at both side slopes during earthquakes in general. However only one side slope at the Up Lane to Tokyo was slid, the opposite side slope at the Down Lane was not slid. The reason why the difference between the slid embankment and non-slid one occurred is very

Fig. 5 Minimum safety factor and sliding failure form in general condition.

Fig. 6 Modified model considering soil layers of embankment body and ground water level.

Fig. 7 Minimum safety factor and sliding failure form by modified model.

interesting from the engineering view point and can be applied for the future design of the highway or other embankments.

The above reason is investigated and discussed with use of the circular sliding method by the seismic coefficient method firstly. The embankment model identified as the modified model shown in Fig. 6 and Table 2 is applied to investigate the difference of minimum safety factors and sliding plane positions calculated at both side slopes applying the seismic coefficients of $k = 0.1, 0.2$ and 0.3 .

The example of results on the slope at the Down Lane in case of $k = 0.1$ can be shown in Fig. 8a. As shown in Fig. 8a, the sliding failure form takes on the toe sliding failure one. Furthermore, the summaries of results in all cases of $k = 0.1, 0.2$ and 0.3 are shown in Fig. 8b and the sliding failure forms in all cases take on the toe sliding failure ones. As shown in Fig. 8b, although the minimum safety factors at the slid slope at the Up Lane are a little larger than those at the non-slid slope, the differences between them seem to be very small from the engineering view point.

As mentioned above, the reason why only one-side slope was slid at the same cross section can not be clarified by the circular sliding method on the modified model shown in Fig. 6. The result is clear and expected because the both slopes at the almost symmetrical embankment with the same soil materials are estimated to be same as far as the circular sliding method is applied, in other words, it indicates the limitation to apply the circular sliding one.

4.2 Investigation with Use of Seismic Response Analysis

In order to investigate the reason why only one-side slope at the symmetrical embankment slid, the seismic response analysis with the FEM (finite element method) has been conducted on the modified model established in Fig. 6 and Table 2 to know the difference between the slid slope and non-slid one in detail.

4.3 Model of Base Ground and Embankment

Fig. 9a shows the analytical FEM model of the objective embankment and base foundation. The left-side slope did not slide and the right-side one slid in this earthquake. The static soil properties are same with Table 2 applied for the circular sliding method. The dynamic soil properties of both shear modulus ratio G/G_0 where G is the shear modulus at a certain shear strain γ , and G_0 is the initial one at the very

(a) Critical slip circle at Down Lane in case of $k = 0.1$

Condition	Fs	
	Down Lane	Up Lane
In General	1.094	1.085
$k = 0.1$	0.902	0.896
$k = 0.2$	0.758	0.754
$k = 0.3$	0.645	0.642

(b) Safety factor at both lanes

Fig. 8 Stability at Down Lane and safety factors between Down Lane and Up Lane.

(a) Analytical FEM model for finite element method

(b) Dynamic properties of each layer

Fig. 9 FEM model and dynamic deformation properties.

small shear strain and damping content h according to the shear strain shown in Fig. 9b, which are estimated referring to Yasuda and Yamaguchi [7] for the layer 1 to layer 4 of the base ground, and Seismic Design Code of Railway Structure [8] for the layer 5 to layer 9 of the embankment body, respectively are applied for the equivalent linear method. The layers 7, 8 and 9 are the pavements and modeled as the gravel, and the layer 10 is assumed to be the engineering base ground.

4.4 Analytical Case and Inputted Seismic Ground Motion

Two analytical cases shown in Table 3 are considered to investigate the effects by the input earthquake ground motion in this study. The earthquake ground motion inputted at the lower plane of the layer 10 of 10 m deep in Fig. 6 is estimated by Hata et al. [9] from the earthquake ground motion recorded at Naka City Office which is about 4.5 km far away from the slid embankment. The time history of the acceleration estimated in the direction of the cross section of the slid embankment is shown in Fig. 10 where the applied time history is 100 s including the main large ground motion. It can be known that the maximum acceleration amplitude is +651 gal and -533 gal.

The plus (+) direction of the acceleration in Fig. 10 means to be the right side direction in Fig. 9a and then the seismic force induced by the plus acceleration tried to drive the body at right side slope to slide toward the right side in Fig. 9a. So the direction of the inputted acceleration becomes important. In this study, two cases on the inputted direction of the base ground motion are considered, i.e., the direction of the plus acceleration is set to the right-side which is named “Real Direction” and the left-side which is named “Reversal Direction” to investigate the effects of the

direction of the irregular ground motion on the dynamic behavior of the embankment.

4.5 Effect of Direction of Seismic Ground Motion

Fig. 11 shows the distribution of the maximum acceleration at both embankment body and base ground for all duration time comparing the Real Direction with the Reversal one, and the maximum values only at the embankment body are picked up in Table 4. It can be known that both the distribution of the maximum acceleration and the maximum values are almost same at the right-side body and the left-side one in the Real Direction and the Reversal Direction. Therefore, the difference induced by the direction of the inputted earthquake ground motion acting on the maximum acceleration at the embankment body can not be clarified from the view of the maximum acceleration. Fig. 12 shows the distribution of the maximum shear strain at both embankment body and base ground for all duration time comparing the Real Direction with the Reversal one, and the maximum values only at the embankment body are picked up in Table 4. However both maximum accelerations are same, they are checked. It can be known that both the distribution of the maximum acceleration and the maximum values are almost same between the right-side body in the Real Direction and the left-side body in the Reversal Direction. This indicates that referring the maximum shear strain, the slope at the Down Lane has the possibility to be slid when the earthquake ground motion in Fig. 10 was inputted in the Reversal Direction.

Table 5 is another discussion based on the averages of absolute maximum acceleration, shear strain and horizontal displacement occurring in the embankment body. From this Table 5, the influence by the direction of the earthquake ground motion can be assumed.

Table 3 Analytical cases.

Case	Input direction	Max. Acc. to right direction (gal)	Max. Acc. to left direction (gal)
1	Real	651	-533
2	Reversal	-533	651

Fig. 10 Inputted earthquake ground motion at engineering base ground of 10 m deep.

Fig. 11 Distribution of maximum acceleration.

Fig. 12 Distribution of maximum shear strain.

Table 4 Maximum of acceleration, shear strain and horizontal displacement at embankment body above base ground.

Case	Ave. of Max. Acc. (gal)		Ave. of Max. shear strain		Ave. of Max. Disp. (m)	
	Left-sided body	Right-sided body	Left-sided body	Right-sided body	Left-sided body	Right-sided body
1	5.42	5.10	5.43E-03	5.76E-03	0.051	0.069
2	5.01	5.53	5.71E-03	5.39E-03	0.066	0.053

Table 5 Average of absolute maximum of acceleration, shear strain and horizontal displacement at embankment body above base ground.

Case	Max. Acc. (gal)		Max. shear strain		Max. Disp. (m)	
	Left-sided body	Right-sided body	Left-sided body	Right-sided body	Left-sided body	Right-sided body
1	-1,649	1,674	1.31E-02	-1.58E-02	-0.077	0.100
2	-1,649	1,674	1.53E-02	1.28E-02	-0.097	0.081

The above results indicate that the direction of the inputted base ground motion seems to relate to the sliding failure, which seems to occur at the slope which is forced in the direction of the absolute maximum acceleration amplitude which is +651 gal in this earthquake.

4.6 Investigation on Effects of Degree of Saturation with Use of Circular Sliding Method

Because the sliding stability of the embankment is generally affected by the water contents in the embankment body, the conditions of water contents are

paid attention in this chapter to discuss the reason why the slid slope and non-slid one occurred at the same cross section in this earthquake.

4.6.1 Possibility of Increase of Degree of Saturation in Embankment Body

Although the slid embankment with the symmetry shape locates on the flat alluvium plane, the plane linearity of the highway route around the slid embankment can be known to be curved a little toward the lower direction in Fig. 1b. Then, according to the layout of the drainage system around the slid embankment shown in Fig. 13 shown by the East

Nippon Expressway Co. Ltd. [10], it can be assumed that the rain fall and the water drained from the pavement are leaded to the lower direction in Fig. 13.

Furthermore, the detail field survey [10] has reported that the water content is observed to be high just after the rainfall at the inside of the right-side body at the Up Lane.

Based on the above conditions, the water oriented by the rainfall and/or drainage may be possible to concentrate and/or store in the embankment body at the Up Lane in Fig. 1b where the sliding failure occurred in this earthquake. On the contrary, the embankment body at the Down Lane where the sliding failure did not occur may be difficult to be concentrated and/or stored with the above water.

In other words, it can be indicated that the degree of saturation of the embankment body before the earthquake is different between the slid body and the non-slid body. Then the circular sliding method is applied to discuss the above estimation to clarify the reason why the slid and non-slid slope at the same cross section of the embankment as follows.

4.6.2 Estimation of Effect by Change of Degree of Saturation

As shown in Fig. 6, the groundwater level is lower than the base ground surface. This means that the whole embankment body is located above the groundwater level and is not saturated generally. However, estimated the body water conditions concentrated and/or stored as mentioned above, it can be assumed that the water contents above the groundwater level may be distributed irregularly and then the strength of body soils may be different at each position.

For investigating the above estimation, it is necessary to know the factors affected by the change of degree of saturation S_r which are considered to be the shear strength and the wet density in this study. As for the above shear strength, Athapaththu et al. [2] indicate that the internal friction angle is not related to the S_r and on the contrary, the apparent cohesion is related to that as shown in Fig. 14a through the consolidated drain tests. Based on Fig. 14a, the cohesion of soils can be assumed to decrease according to the increase of the

Fig. 13 Drainage system of rainfall on pavement surface.

Fig. 14 Relation between cohesion c_d , degree of saturation S_r and void ratio e .

void ratio e of 0.7 to 1.0 depending on the S_r of 40% to 80%. Modifying Fig. 14a, Fig. 14b can be obtained as the relation between the S_r and cohesion in case of the e of 0.95.

$$c = -0.112S_r (\%) + 12.93 \text{ (kPa)} \quad (1)$$

As shown in Table 2 which shows the soil properties on the layers 4 and 8 measured by the East Nippon Expressway Co. Ltd. [3] just after the earthquake, the layer 4 is the loam with the void ratio e of 1.81 and then the cohesion is assumed not to be affected by the S_r in this study. On the other hand, because the layer 8 is the gravelly fine sand with the e of 0.95, the change of the cohesion is assumed according to Eq. (1) shown in Fig. 14b.

Based on the Eq. (1), it can be known that the cohesion of 0.112 kPa is decreased according to the increase of S_r of 1%. Although the S_r of the layer 8 measured just after the earthquake is 44.2%, the S_r of the layer 8 before the earthquake can not be known shown in Table 6. In order to investigate the difference of the slid slope and non-slid one, the layer 8 in the original cross section of Fig. 15a is divided into two layers: the half layer at the right side where the sliding

failure occurred and that at the left one where the sliding one did not occur shown in Fig. 15b. Then, the degree of saturation of the layer 8 in Fig. 15b is estimated to be S_{r0} before the earthquake and the one of the layer 9 is estimated to be higher than S_{r0} before the earthquake. Then the sliding safety factor is calculated applying the increase of S_r from 10% to 30% with the step of 10% from the S_{r0} .

On the other hand, the relation between the wet density and S_r is estimated based on that estimated by two soil samples shown in Fig. 16a, and Fig. 16b shows the analytical cases where the degree of saturation of layer 9 is increased every +10%.

The calculated minimum safety factors with the circular sliding method in case of the seismic coefficient of $k = 0.1$ can be obtained in Table 7. Table 7 indicates that according to the increase of the S_r , the minimum sliding safety factor of the layer 9 at the right-side slope decreases compared with that of the layer 8 at the left-side slope, and the decrease of the cohesion affects a little more largely than the increase of the wet density on the decrease of the sliding safety factor of the embankment.

Table 6 Soil properties of layer 4, layer 8 and layer 9.

Layer No.	Soil classification	Water content (%)	Wet density (g/cm ³)	Void ratio (e)	Degree of saturation (%)
4	Loam	58.4	16.36	1.81	89.9
8 and 9	Gravelly fine sand	15.7	16.58	0.95	44.2

(a) Original cross section

(b) Modified cross section to consider S_r

Fig. 15 Modification layer 8 at original model to consider effects of S_r .

(a) Relation between S_r and wet density

Increase of S_r at right side of Layer 9 (%)	Left-sided Slope	Right-sided Slope
+0	1.102	1.089
+10	1.102	1.057
+20	1.102	1.025
+30	1.102	0.994

(b) Conditions of analytical cases

Fig. 16 Relation between cohesion and wet density considering S_r .

Table 7 Minimum sliding safety factors affected by S_r in case of $k = 0.1$.

Increase of S_r at layer 9 (%)	Analytical case	
	Cohesion (kN/m ²)	Wet density (kN/m ³)
+0	10	19
+10	8.88	19.49
+20	7.76	19.99
+30	6.64	20.48

As mentioned above, the possibility to explain the difference between the slid slope and the non-slid one can be indicated when the strength, i.e., the cohesion and the wet density of the body layer is possible to decrease according to the increase of the degree of saturation induced by the water concentrated and/or stored in the embankment body.

5. Conclusions

The typical sliding failure of the highway embankment interested in the 2011 Great East Japan Earthquake is investigated to simulate the toe sliding failure on the soft base ground and to clarify the reason why only one side slope at the symmetry formed embankment during the earthquake. The following lessons can be obtained in this study:

(1) The toe sliding failure at the soft base ground with the thickness of 3.6 m which was observed in this earthquake can be simulated with use of the circular sliding method considering the seismic coefficient of 0.0, 0.1, 0.2 and 0.3, comparing the soil strength of the embankment body and that of the soft base ground;

(2) The reasons why the slid slope and non-slid one at the same cross section of the symmetry shaped embankment with the same soil properties can not be clarified with use of the circular sliding method considering the seismic coefficient. This indicates the limitation to investigate the detail reasons of the sliding failure with the circular sliding method in case of same body conditions;

(3) One of the above reasons can be indicated to be the direction of the maximum acceleration amplitude of the earthquake motion inputted at the base ground with use of the dynamic response analysis of FEM.

This indicates that the direction of the earthquake ground motion should be paid attention when the dynamic response analyses are applied;

(4) Another reason mentioned above can be explained by the different conditions of the degree of saturation in the embankment body before the earthquake. In other words, when the degree of saturation in the embankment body is distributed un-uniformly even if the shape of the embankment body is symmetry, the sliding failure can be indicated to be influenced by the decrease of the cohesion and/or the increase of the wet density in the body soils relating to the increase of the degree of saturation before the earthquake. This indicates that both cohesion and wet density of soils above the ground water level should be considered to estimate the sliding failure of the embankments.

Further study on the decrease of the strength of the embankment body induced by the water seepage should be investigated.

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