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<th>Journal:</th>
<th>Canadian Geotechnical Journal</th>
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<tr>
<td>Manuscript ID</td>
<td>cgj-2017-0385.R1</td>
</tr>
<tr>
<td>Manuscript Type:</td>
<td>Article</td>
</tr>
<tr>
<td>Date Submitted by the Author:</td>
<td>23-Jan-2018</td>
</tr>
<tr>
<td>Complete List of Authors:</td>
<td>Murao, Hidehiko; Nagoya University, Civil Engineering Nakai, Kentaro; Nagoya University, Civil Engineering Noda, Toshihiro; Nagoya University, Disaster Mitigation Research Center Yoshikawa, Takahiro; Nagoya University, Civil Engineering</td>
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<tr>
<td>Is the invited manuscript for consideration in a Special Issue? :</td>
<td>N/A</td>
</tr>
<tr>
<td>Keyword:</td>
<td>1G shaking-table test, fill slope, resonance, natural frequency, dominant frequency</td>
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Deformation/failure mechanism of saturated fill slopes due to resonance phenomena based on 1G shaking-table tests

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Abstract

Earthquake motions include waves of various frequency bands ranging from low to high frequency. Notably, the dominant frequency affects the seismic stability of fill slopes. Hence, to analyze the deformation/failure mechanism during an earthquake, 1G shaking-table tests were conducted in this study on saturated fill slopes considering the resonance phenomenon. The results show that the deformation/failure mechanism of a fill slope cannot be defined by only one sliding surface. Multiple sliding surfaces are progressively formed depending on the ground and external conditions. Moreover, evaluating the seismic stability of fill slopes depends not only on the magnitude of the acceleration but also on the relationship between the natural frequency of the fill slope and the input frequency. These phenomena cannot be considered in the seismic-intensity method or in the Newmark’s method used in the conventional design. When implementing seismic countermeasures, it is important to determine the frequency characteristics of fill slopes and design the countermeasures considering the predominant frequency of the seismic wave anticipated at the site.

Keywords

1G shaking-table test, fill slope, resonance, natural frequency, dominant frequency
1. Introduction

In Japan, approximately 73% of the national land is mountainous with limited flat terrain. Hence, the hilly areas are cut, and the soil generated are used to form embankments in nearby valleys or low-lying areas. The embankments constructed via this method are largely found in residential areas on the outskirts of cities, which have contributed to urban expansion. However, major earthquakes that have occurred in recent years, such as the Miyagi Earthquake in 1978 (Tohoku branch of the JSCE 1980), the southern Hyogo prefecture earthquake in 1995 (Kamai 1995), Mid Niigata prefecture earthquake in 2004 (Mid Niigata Prefecture Earthquake research committee 2007), Noto Peninsula earthquake in 2007 (Tameshige et al. 2009), off the Pacific coast of Tohoku earthquake in 2011 (Kamai et al. 2013), and the Kumamoto Earthquake in 2016 (Mukunoki et al. 2016), caused widespread damage associated with the deformation of fill slopes. It has been found that the damages were due to both interplate and inland earthquakes with a large magnitude and shallow hypocenters. Hence, fill slopes are clearly vulnerable to major earthquakes and are considered a geotechnical risk factor.

Conventionally, for the seismic-stability assessment of fill slopes, a seismic-intensity method is used as the standard (Ministry of Land, Infrastructure, Transportation, and Tourism, 2006). This method is an aseismic design method wherein the dynamic excitation phenomenon occurring at the time of an earthquake is replaced with an equivalent horizontal static force. Although this method is simple and widely used in the design, the aseismic performance is evaluated by converting the maximum acceleration into seismic intensity; thus, the frequency characteristics of the seismic motions are inappropriately considered. In addition, the method proposed by Newmark (Newmark 1965; Makdisi and Seed 1978) is often used to calculate the residual displacement of the fill
slopes in the event of an earthquake. This method is a deformation-analysis method wherein the limit equilibrium method is applied, and the residual displacement occurring on only one slip surface assumed in advance is analyzed. This method is widely used in the actual design because the displacement can be quickly calculated. However, it cannot be used to accurately grasp the complex deformation/failure behavior of actual fill slopes and explain its mechanism because only one linear or circular slip surface is assumed beforehand despite the complicated shape of the actual slip surface and its progression as shown in Figure 1. Moreover, the reduction in the embankment strength due to earthquake motions is generally neglected. In fact, in the 2011 off the Pacific coast of Tohoku earthquake, there was a significant difference in the extent of seismic damage on fill slopes depending on the location, even if there were no difference in the fill material, construction period, and construction method. Furthermore, in some places, despite the fact that the countermeasures were properly taken, fill slopes where earthquake damage occurred have been reported (Kamai et al. 2013; Murao et al. 2013). These facts suggest that in the event of a massive earthquake, it is difficult to adequately explain the stability of fill slopes using only the conventional seismic intensity method or the limit equilibrium method.

To understand the complicated deformation/failure behavior of fill slopes in the event of an earthquake, several experimental studies have been conducted using 1G shaking-table tests and centrifugal-model tests. However, most of the studies were systematically performed by changing the ground conditions such as soil material, density (water content), inclination angle of the slope, and ground water level (Hoe et al. 2009; Higo et al. 2015), and only few studies focused on the characteristics of input motions. Even in the studies related to the input-motion characteristics, the focus was on the maximum acceleration, i.e., the peak ground acceleration (PGA),
wherein the aforementioned seismic intensity method and Newmark’s method were applied; only a few studies focused on the frequency characteristics. Under such circumstances, Wartman et al. (2003) examined the relationship between the dominant frequency of the input ground motion and the natural frequency of the slope by conducting simple 1G model tests. They showed that the amount of deformation of the slope varies significantly depending on the natural frequency of the input ground motions. However, they focused on the applicability of the Newmark’s method for a frequency ratio of 0.2 or more and found the Newmark’s method to be conservative, whereas it was not conservative in the range of 0.2 to 1.3. Wartman et al. (2005) showed that the acceleration amplification rate increases in the natural frequency band of the model slope when a random wave is inputted to the slope in the centrifugal-model tests. Brennan and Madabhushi (2009) showed that the acceleration amplification rate is different between the center and edge of the crest using the centrifugal model tests. However, both the aforementioned studies focused on the amplification characteristics of the acceleration and neglected the influence of the external frequency characteristics on the deformation/failure of the model slope. Earthquake motions include waves of various frequency bands ranging from low to high frequency. Notably, the dominant frequency affects the seismic stability of fill slopes. In the seismic design of concrete structures, steel structures, and buildings, the influence of the frequency characteristics or resonance phenomena have been considered. However, for soil structures such as fill slopes, the seismic-intensity method and the Newmark’s method are still used in the current seismic design, wherein the aforementioned frequency characteristics are hardly taken into consideration. Therefore, to understand the influence of frequency characteristics on the deformation/failure mechanism of fill slopes in the event of an earthquake, 1G shaking-table tests were conducted in this study on saturated fill slopes, focusing on the
resonance phenomenon. Moreover, plastic deformation is easily generated in a soil material, compared to concrete or steel materials. In particular, when the earthquake is strong and the duration is long, there is a concern that the soil strength would decrease significantly during the earthquake. If the state change in the soil due to the progression of plastic deformation during an earthquake is not considered, the earthquake resistance may be overestimated. Therefore, to understand the deformation/failure behavior of a soil structure, the elasticity and rigid plasticity are insufficient, and it is necessary to interpret based on the elastoplasticity. Therefore, a detailed observation of the previous experimental results show that the deformation/failure mechanism of the model slope should be described based on the elastoplastic theory.

To stabilize the existing fill slopes, implementation of drainage works are fundamental. This is to prevent the decrease in the effective constraint pressure due to the generation of excess pore water pressure by desaturating the inside of the embankment. However, some studies have reported the deterioration of drainage function because of the large displacement inside the embankment; breakage of drainage components; and clogging. Drainage maintenance is an important issue. In this study, we focus on the embankment slope in the saturation state. This assumption is based on the survey of fill slopes that were actually damaged. In addition to the aforementioned decline in the drainage function, the underground water level in a damaged fill slope steadily increases near the ground surface because of the supply of groundwater from the rear slope (Kamai et al. 2013; Murao et al. 2013). Even for the fill slope which drainage works are effective, saturation state assumes the safety side evaluation for design like the situation when an earthquake occurs immediately after a heavy rainfall.
2. Overview of Model Tests

A model slope with a 1/50 scale of a fill slope, comprising a bedrock part and an embankment part, was constructed within a stainless steel soil tank, as shown in Figure 2. The shapes of the bedrock and embankment are assumed the same as that of a typical fill slope, which is formed by the combination of cutting and banking. This shape helps in preventing the slipping at the boundary between the basement and the embankment. Hereafter, the term “model slope” indicates the overall model slope comprising the bedrock and embankment parts, as shown in Figure 3, and the term “fill slope” indicates the model slope scaled up to the full size. Moreover, the “bedrock part” indicates the bedrock part of the model slope or the fill slope, and the “embankment part” indicates the embankment parts of both. Seepage and saturation were induced in the model slope using a water tank at the front and rear of the soil tank. At the boundary between the water tanks and the foundation, a nonwoven fabric was provided to prevent the soil particles from flowing out with the water. An acrylic plate was provided on the side of the soil tank to observe the deformation. In addition, 326 thin aluminum plates with a diameter of 10 mm (thickness of 0.3 mm and a specific gravity of 2.7) were arranged in the form of squares with a side of 20 mm as markers. To reduce the friction of the markers, a Teflon sheet was applied to the contact surface along with an acrylic sheet. The shapes of the slip surfaces were largely perpendicular to the vibration direction and do not form a circular arc; thus, the friction between the model slope and the soil tank can be ignored. Moreover, a plane-strain condition can be obtained. As shown in Figure 3, seven accelerometers (ch.1 to ch.6 and ch.8) and seven pore-water pressure meters (ch.9 to ch.15) were embedded into the embankment while it was being constructed, and one accelerometer (ch.7) was installed at the bottom of the soil tank to measure the input excitation.
3. Preparation of the Model Slope

Figure 4 shows the results of the unconfined compression test conducted on the bedrock part. The bedrock material was Mikawa silica sand (Nos. 4, 5, 5.6, 6, and 7) mixed in proportions of 1:1:1:1:1 by mass, to which a cement-solidification material was added at a rate of 150 kg/m$^3$. The test specimen was compacted with a water content of $w = 15\%$. After curing for 28 days, it was formed into a stair shape. Soil cement was used as the bedrock material because the bedrock of the fill slopes that have deformed in an earthquake frequently follow the Neogene Series or Pleistocene Series and are relatively harder than the embankment part (Kamai et al. 2013). Thus, the aim was to reproduce the conditions wherein the difference in the strengths between the embankment and bedrocks part was considerable. Moreover, by increasing the frictional resistance between the bedrock and embankment parts via forming the boundary in the form of a stair shape, the occurrence of slip at the boundary was prevented.

Table 1 lists the physical properties. Figure 5 shows the grain-size distribution. Figure 6 shows the compaction curve. Figure 7 shows the undrained triaxial compression tests conducted on the saturated embankment material. In addition, Figure 5 shows the cumulative grain-size curve of the bedrock material prior to adding the cement. The embankment material was formed by mixing Mikawa silica sand (Nos. 4, 5, 6, and 7), DL clay, and Fujinomori clay in proportions of 3:3:3:4:4 by mass. This was because the embankment material of the fill slopes damaged in the earthquakes in many cases was made up of soil arising from cutting nearby hills and intermediate soil that included fine fractions (Kamai et al. 2013). The embankment part was prepared using compaction within 15 layers (1 layer = 2 cm) by controlling the water content at $w = 12\%$ with a dry density of $\rho_d = 1.72$ g/cm$^3$, which
is equivalent to a degree of compaction $D_C = 90\%$. The seepage surface was then raised to the ground surface in stages over a period of 30 h while maintaining the horizontal stratification. After leaving the model slope in the seepage condition for 10 h, the embankment was excavated and formed into a specific slope shape. The model slope was infiltrated until it could be regarded as a saturated condition. Note that when allowing the water to seep into the model slope, the hydrostatic pressure component due to the increase in the pore water pressure was checked, and there was no change after forming the slope. Thus, the model slope was considered to be in the saturated condition.

The test specimens, shown in Figure 7, were produced by controlling the density so that the degree of compaction was $D_C = 90\%$, which is the same as that of the embankment part of the model slope. The overburden pressure of the model slope was considered lower than 10 kPa even at the deepest part; however, for reasons associated with control, the triaxial compression tests were conducted with confining pressures of 30, 50, and 100 kPa. A positive dilatancy was observed after exhibiting a negative dilatancy behavior. In the model slope where the confining pressure was lower, it is considered that the positive dilatancy trend would have been even greater if the strain was significantly increased. Moreover, considering the effective stress path, a softening behavior (softening associated with plastic compression) was observed below the critical state line, which is a typical characteristic behavior of structured soils. Therefore, the shear strength might reduce because of the cyclic loading under undrained conditions.

4. Excitation Conditions
The input acceleration was given in the form of a sine wave with a constant frequency, and the excitation was carried out in stages with a gradual increment of 0.5 m/s², as presented in Table 2. The excitation duration of each stage was 60 s. An interval of 240 s was provided between the stages to prepare for the next excitation with no fluctuation in the pore-water pressure during the intervals; thus, the excitation was considered continuous. In addition, at each stage, the excitation was controlled so that the prescribed acceleration was reached within approximately 1.5 s from the start. The data from the accelerometers and pore-water pressure meters installed within the model slope were acquired by sampling at a frequency of 1000 Hz. The method of determining the natural frequency of the model slope is described later. The following three cases were investigated to clarify the deformation/failure mechanisms of the fill slope, focusing on the relationship between the natural frequency of the model slope and the input frequency.

Case 1: input frequency of 50 Hz (equal to the natural frequency of the model slope)

Case 2: input frequency of 20 Hz (lower than the natural frequency of the model slope)

Case 3: input frequency of 80 Hz (higher than the natural frequency of the model slope)

Note that in stage 15 for Case 3, the input acceleration significantly increased by 1.0 m/s² from the previous stage.

5. Determination of the Natural Frequency of the Model Slope

To conduct the study focusing on the frequency, a sweep test was performed to determine the natural frequency of the model slope. The sweep frequency was given in the form of a sine wave with a constant acceleration amplitude of 0.2 m/s²; the frequency was increased from 10 to 60 Hz. The acceleration amplitude was
measured using an accelerometer installed on the crest of the embankment part (ch.8). Figure 8(a) shows the input acceleration (ch.7) and the acceleration measured at the crest (ch.8). Figure 8(b) shows an enlargement at a frequency of approximately 40 Hz. Figure 8(c) shows an enlargement at a frequency of approximately 53 Hz, which is close to the peak-measured acceleration at the crest. Figure 8(d) shows an enlargement at a frequency of approximately 55 Hz. In Figure 8(a), the measured acceleration of the crest amplifies as the input frequency is changed up to approximately 390 s, and subsequently, it is attenuated. Thus, the input frequency at the time at which the amplitude is the highest is considered the natural frequency of the model slope. Notably, Figure 8(b) shows that there is largely no amplification at an input frequency of approximately 40 Hz. However, Figures 8(c) and 8(d) show that at input frequencies of approximately 53 and 55 Hz, an amplification is observed by factors of approximately 4 and 3, respectively. These results show that the natural frequency of the model slope is between 50 and 60 Hz, but closer to 50 Hz. Hence, in the shaking-table tests, the excitations were induced with an input frequency of 50 Hz, which is close to the natural frequency.

6. Test Results and Discussion

In processing the experimental results, the acceleration amplification factor (hereafter referred to as the amplification factor) was defined as the ratio of the amplified acceleration (observed acceleration at the crest) to the input acceleration, as shown in Equation (1).

$$\text{Amplification factor} = \frac{\text{Amplified acceleration}}{\text{Input acceleration}}$$  \hspace{1cm} (1)

To determine the amplification factor, if no amplification or attenuation is produced during the excitation, as shown
in Figure 9(a), a constant maximum amplitude is considered the amplified acceleration. In addition, if the amplitude increases during the excitation, as shown in Figure 9(b), the maximum amplified amplitude is considered the amplified acceleration. If the amplitude is attenuated, as shown in Figure 9(c), the amplitude when the attenuation converged is considered the amplified acceleration.

Based on the assumption that the stress–strain relationships between the grounds with different confining pressures are similar, the similitude obtained using the ratio of forces is represented as shown in Equation (2).

\[
\frac{f_m}{f_p} = \left(\frac{1}{n}\right)^{\frac{3}{4}}
\]  

(2)

The following is the similitude obtained using the dominant equation for a two-phase saturated ground.

\[
\frac{f_m}{f_p} = \left(\frac{1}{n}\right)^{\frac{1}{2}}
\]  

(3)

In the above equations, \( f_m \) is the frequency used in the model test, \( f_p \) is the frequency of the full-size object, and \( 1/n \) is the geometric scale of the model slope (in this case \( n = 50 \)). In the current 1G shaking-table tests, three different input frequencies were considered, namely 50, 20, and 80 Hz. The results obtained at these frequencies are converted to a full-size scale using Equations (2) and (3), as listed in Table 3.

6.1 Case 1: Input frequency of 50 Hz (equal to the natural frequency of the model slope)

Figure 10 shows the measured acceleration and corresponding change in the amplification factor. Figure 11 shows the change in the excess pore-water pressure. Figure 12 shows the side view of the model prior to excitation and at stage 11 (input acceleration of 5.2 m/s\(^2\)). The two slip surfaces, shown in Figure 12, were determined from
the displacements of the markers installed on the side surface and from the change in the amplification factor and the pore-water pressure behavior described later.

Stage 1 to stage 2: “resonance process”

By comparing the amplification factors in the depth direction, as shown in Figure 10, it can be seen that in the initial stage, the amplification is higher at positions close to the ground surface with the following relationship.

Slope side: ch.1 (lower) < ch.2 (middle) < ch.3 (upper)

Crest side: ch.4 (lower) < ch.5 (middle) < ch.6 (upper)

At each depth, the maximum amplification factor is observed in stage 2 (input acceleration of $0.7 \text{ m/s}^2$). This is because the input frequency is close to the natural frequency of the model slope, thereby causing resonance. The oscillations are higher in the shallower parts compared to the deeper parts. Resonance is observed at this stage. Thus, although the input acceleration is low, large oscillations are produced in the embankment part.

Stage 3 to stage 8: “process of reduction in the natural frequency as a system”

As the input acceleration is further increased (stages 3 to 8), the relationship of the magnitude of the amplification factor in the depth direction is maintained as the acceleration increases at each measurement point. However, the amplification factor in the shallow-to-middle parts of the embankment reduces. The amplification factor reduces from stage 3 (input acceleration of $1.2 \text{ m/s}^2$) onward because of the absence of resonance with the accumulation of small plastic deformations associated with the excitation. In other words, from the viewpoint of the elasto-plasticity theory, the following can be considered.

1. At the beginning of this stage, the input excitation was still small, but the amplification factor was 3 or higher.
Thus, the plastic deformation gradually accumulated on the model slope. The stress ratio during the repetition is not sufficiently high to reach the critical state (failure); thus, the plastic deformation acts as a plastic compression.

2. The conditions during the test can be considered as undrained; hence, an elastic expansion is produced because of the plastic compression. Therefore, the mean effective stress is reduced, and the pore-water pressure increases.

3. The rigidity reduces with the decrease in the mean effective stress. Thus, the natural frequency of the model slope as a system reduces.

Although no clear deformation can be seen in the model slope, the pore water discharges from the ground surface from excitation stage 3 onward. The reduction in the amplification factor from stage 3 onward is greater in the shallower parts; hence, it is considered that the accumulation of the plastic deformation increases closer to the surface of the slope where the confining pressure is lower. This implies that the discharge of the pore water is due to the increase in the pore-water pressure within the model slope as described above. The pore-water pressure does not increase even in the shallowest of parts; however, it is considered that the increase in the pore-water pressure in the shallower parts is more than that where the pore water pressure meters were installed.

Stage 9 to stage 10: “process of slip formation in the shallow parts

At stage 9 (input acceleration of 4.2 m/s²), an open crack is generated in the ground surface, as shown in Figure 13, and a fluid shallow slip can be clearly seen. At this stage, the amplification factor (ch.3) in the shallow part of the slope significantly reduces, and the magnitude of the amplification factor in the depth direction is
reversed as follows.

Slope side: ch.3 (upper) < ch.1 (lower) < ch.2 (middle)

Crest side: ch.5 (middle) < ch.4 (lower) < ch.6 (upper)

Figure 14 shows the excess pore-water pressure ratio and the acceleration waveform of the slope side in the case of excitation stage 9. At the end of the excitation, the excess pore-water pressure ratio at the shallower part (ch.11) gradually increases and exceeds 1 during the excitation, and the rigidity and shear strength reduce. The acceleration (ch.3) is attenuated after reaching a peak of 10 m/s². The generation of the slip surface and reversal of the magnitude of the amplification factor can be considered as follows.

1. Because of the increase in the input acceleration, the plastic deformation proceeds further, and the mean effective stress further reduces.

2. The reduction in the mean effective stress is clear in the shallower part where the confining pressure is low. Therefore, the excess pore-water pressure ratio exceeds 1 at the shallower part.

3. The rigidity is further reduced locally, and because of the repeated loading, the soil is disturbed (the degree of the structure is decreased) and the shear strength reduces.

4. The acceleration around the slip surface is attenuated by internal dissipation, and the amplification factor is reversed.

A shallow slip failure occurred at this stage because of the decrease in the rigidity and shear strength accompanying the accumulation of the plastic deformation due to the increase in the inertia force.

Stage 11 onward: “process of slip occurrence in the deep parts”
At stage 11 (input acceleration of 5.2 m/s\(^2\)), a deep slip occurred from the crest of the embankment toward the toe of the slope. At this stage, the amplification factor in the middle of the slope (ch.2) significantly reduces, and the relationship of the magnitude in the depth direction is reversed as follows.

Slope side: ch.3 (upper) < ch.2 (middle) < ch.1 (lower)

Figure 15 shows the excess pore-water pressure ratio and the acceleration waveform on the slope side in the case of excitation stage 11. Similar to the shallow parts in excitation stage 9, the excess pore-water pressure ratio exceeds 1 at the middle of the slope (ch.10), and the acceleration in the middle (ch.2) gradually attenuates after reaching a peak of 7.5 m/s\(^2\). Apart from the fluid slip near the surface that occurred in stage 9, a deeper slip occurred in stage 11. It is considered that this slip occurred via the same mechanism as that in the shallow part, i.e., reduction in the rigidity and shear strength associated with the accumulation of plastic deformation due to the increase in the inertia force (in the soil mass above the slip surface). Moreover, the excess pore-water pressure ratio at the shallow part of the slope side (ch.11), which tended to increase until this stage, starts to reduce after 50 s and ultimately becomes negative (occurrence of negative pore-water pressure). The occurrence of negative pressure is considered to be because of the occurrence of a positive dilatancy effect near the slip surface due to the large plastic deformation that occurred at the slip of the shallow part of the embankment in stage 9, resulting in a local recovery of the rigidity. Therefore, the acceleration at this part amplifies along with the recovery of the rigidity. The embankment material exhibits positive dilatancy properties; in other words, a plastic expansion is observed when a large shear deformation is imposed, as shown in Figure 7.

In summary, when the input frequency is approximately equal to the natural frequency, the resonance occurs
from the initial stage of the excitation, and the acceleration is amplified. With the accumulation of plastic
deformation, the mean effective stress reduces. Eventually, the rigidity and shear strength reduce locally at the
shallow part of the embankment where the confining pressure is low. At the same time, although the amplification
factor is reduced, the magnitude of the acceleration at the shallow part would have already increased, causing a slip
failure at the shallower part. Furthermore, when the input acceleration increases, slip occurs in the same manner
even in the deep part.

6.2 Case 2: Input frequency of 20 Hz (lower than the natural frequency of the model slope)

Figure 16 shows the measured acceleration and the change in the amplification factor. Figure 17 shows the
change in the excess pore-water pressure ratio. Figure 18 shows the side view of the model prior to the excitation
and at stage 12 (input acceleration of 5.7 m/s²).

Stages 1 to 8: “non-resonant process”

Figure 16 shows that the amplification factor is the same as that in Case 1 wherein it increases as it
approaches the surface. However, the value is lower than that in Case 1. This implies that the model slope does not
resonate and largely oscillates uniformly from the shallow part to the deep part.

Stages 9 to 11: “process of reducing the natural frequency as a system”

In stage 9 (input acceleration of 4.7 m/s²), the amplification factor starts to increase at the shallow-to-middle
parts of the embankment while maintaining the relationship in the depth direction. Simultaneously, the excess
pore-water pressure ratio increases, and the pore water from the surface starts to discharge because of the
accumulation of the plastic deformation, as explained in Case 1. The rigidity decreases as the mean effective stress reduces, thereby reducing the natural frequency of the system. The natural frequency in the initial stage of the excitation is considerably higher than the input frequency (20 Hz); thus, resonance does not occur, and the oscillations are small. However, the natural frequency gradually reduces and approaches the input frequency, thereby causing resonance, and the amplification factor increases. Figures 16 and 17 show that the increase in the pore-water pressure reaches its peak in stage 11 (input acceleration of 5.2 m/s²).

Stage 12: “process of simultaneous occurrence of slip in the upper and middle parts”

In stage 12, the amplification factor reduces in the shallow and middle parts of the embankment, and the relationship of the magnitude of the amplification factor in the depth direction is reversed as follows.

Slope side: ch.3 (upper) < ch.2 (middle) < ch.1 (lower)

Crest side: ch.6 (upper) < ch.5 (middle) < ch.4 (lower)

In this stage, the measured accelerations in the top and middle parts increase; the inertial forces increase; and the excess pore-water pressure ratio exceeds 1 in the shallow and middle parts of the embankment. Hence, the rigidity and shear strength reduce, and a slip suddenly occurs passing through the middle of the embankment. A visual observation from the side showed the occurrence of cracks on the ground surface and a clear deformation. Because the input acceleration is high at the time of resonance, the rigidity in the shallow-to-middle parts of the embankment simultaneously reduces. Therefore, the slip does not gradually progress from the surface to the deep parts as in Case 1, but occurs suddenly from the center of the embankment.

In summary, when the input frequency is lower than the initial natural frequency of the embankment slope in
the initial excitation stage, resonance does not occur, and the stability is maintained. However, as the acceleration increases with the progression of the excitation stages, the natural frequency of the model slope gradually reduces because of the accumulation of the plastic deformation, and eventually, resonance occurs when the natural frequency approaches the input frequency. Moreover, the oscillations of the embankment part significantly increase.

In the shallow and middle parts of the embankment, the mean effective stress reduces and a slip passing through the center of the embankment is generated via the same mechanism as that in Case 1; in other words, an increase in the inertial force of the slope surface is observed. At the time of resonance, the input acceleration is high; thus, a slip is generated suddenly from the middle of the embankment, unlike in Case 1, wherein the slip extended in stages from the surface toward the deep part. In addition to the fact that the input acceleration is high at the time of resonance, the natural frequency of the model slope at the time of resonance is low, causing a large oscillation. Therefore, the slip does not gradually progress from the surface to the deep part as in Case 1, but the slip fails at once from the center of the embankment.

6.3 Case 3: Input frequency of 80 Hz (higher than the natural frequency of the model slope)

Figure 19 shows the change in the excess pore-water pressure ratio. Figure 20 shows the side view of the model prior to the excitation and at stage 15 (input acceleration of 7.7 m/s²). In Case 3, unlike Cases 1 and 2, no visible deformation occurred even with an excitation up to stage 15. The input acceleration exceeded the allowable response frequency of the accelerometers, making it impossible to discuss the amplification factors. However, regarding the pore-water pressure behavior, the excess pore-water pressure ratio does not increase for excitations up
to stage 15. The plastic deformation accumulated as the input acceleration increases and the natural frequency of the system as a whole decreases, similar to that observed in Cases 1 and 2. However, the change in the natural frequency is considerably lower than that in the input frequency, and there was no acceleration amplification because of the resonance, thus maintaining the stability. In the excitation from stage 11 (input acceleration of 5.2 m/s$^2$) onward, water was gradually discharged from the slope surface and the crest. However, even in stage 15, slip did not occur. Higher modes different from those that allowed slip to occur in Cases 1 and 2 are observed. When the input acceleration is increased further, resonance may occur in the case of a higher mode. It is necessary to verify this in the future by conducting a numerical analysis.

In summary, when the input frequency is higher than the initial natural frequency of the embankment slope, the natural frequency of the model slope changes (reduces) because of the accumulation of the plastic deformation. However, because the change in the natural frequency is considerably lower than that in the input frequency, resonance does not occur and stability can be maintained.

### 6.4 Analysis of seismic countermeasures and seismic evaluation from the test results

The experimental results show that the seismic-stability evaluation of the fill slopes not only depends on the magnitude of the acceleration of the seismic wave but also on the relationship between the natural frequency of the fill slope and the predominant frequency of the input seismic motion. In particular, in the case of a seismic motion with a long duration and a long period component (low-frequency component) such as that in a trench-type earthquake, the embankment part loosens because of the long shaking period; moreover, the shear strength reduces,
and the natural frequency decreases. In this case, as the resonance phenomenon occurs in the low-frequency region, there is a risk that the oscillation of the fill embankment becomes considerable in addition to the rapid progression of the deformation. Following are the conclusions regarding the seismic countermeasures and seismic evaluations obtained from the experimental results.

1. It is possible to make the embankment more robust by increasing the degree of compaction for newly constructed slopes and for improving soil or reinforcing embankment for existing construction slopes. However, if the soil loosens because of repetitive loading, resonance will eventually occur in the low-frequency region. Therefore, the seismic resistance can be significantly improved by using soil materials or conducting soil improvement that are not likely to cause plastic deformation.

2. Various seismic countermeasures can be taken for embankments; however, strengthening and countermeasures alone are insufficient. If the frequency properties of the considered fill slope are not determined and if the predominant frequency of the seismic motion expected at the site is neglected, the results suggest that the risk of damage may increase.

3. Because of the plastic deformation of fill slopes due to earthquakes, the natural frequency of the embankment will gradually change. This does not imply that the fill slope that exhibited good stability in previous earthquakes is semi-permanently excellent in terms of seismic resistance, particularly in areas where earthquakes occur repeatedly. It is required to measure the natural frequency of the fill slope at present or before and after the earthquake using microtremor array observation or PS logging.

4. In this research, assuming the safety side, as reflected in the event of heavy rainfall or after a heavy rainfall,
and clogging of the drainage material, we focused on the saturated condition under which the effective constraint pressure (mean effective stress) tends to decrease with the increase in the excess pore water pressure by constraining the volumetric change during an earthquake. However, in the future, the frequency characteristics under an unsaturated condition should be studied. In other words, it is necessary to understand (1) how an unsaturated fill slope would deform under repetitive loading, and (2) how the natural frequency of the fill slope changes with respect to its state change.

7. Conclusions

The objective of this study was to clarify the deformation mechanisms of fill slopes in the event of an earthquake. To this end, 1G shaking-table tests were conducted on saturated model slopes by replicating a typical fill slope. The focus was on the frequency properties. The saturated fill slope was under such a condition that the effective constraint pressure (mean effective stress) tended to decrease with the increase in the excess pore water pressure, leading to a safety-side evaluation in terms of the design.

First, a sweep test was conducted with a sine wave of constant acceleration amplitude (0.2 m/s²). The results show that the natural frequency of the model slope was 50 Hz, which was considered the lowest prior to excitation. Tests were then conducted by changing the input frequency. Input frequencies of 50, 20, 80 Hz were considered in Cases 1, 2, and 3, respectively. The excitation was induced by applying a sine wave with a constant frequency, and the acceleration amplitude was increased in stages.

1. In Case 1, wherein the input frequency is approximately equal to the natural frequency, resonance occurred
from the initial stage of excitation, and the acceleration was amplified. As the plastic deformation accumulated, the mean effective stress reduced. Eventually, the rigidity and shear strength reduced locally at the shallow part of the embankment where the confining pressure was low. Moreover, although the amplification factor was reduced, the magnitude of the acceleration at the shallow part was already increased, thus causing a slip failure at the shallower part. Furthermore, when the input acceleration increased, slip occurs in the same manner even in the deeper parts.

2. In Case 2, wherein the input frequency is lower than the initial natural frequency of the embankment slope, resonance did not occur in the initial excitation stage, and the stability was maintained. However, as the acceleration increases with the progression of the excitation stages, the natural frequency of the model slope gradually reduced because of the accumulation of plastic deformation, and eventually, resonance occurred when the natural frequency approached the input frequency; the oscillations of the embankment part increased significantly. With the same mechanism as that in Case 1, the mean effective stress reduced in the shallow and middle parts of the embankment as the inertial forces increased, thereby reducing the rigidity and strength, and a slip suddenly occurred passing through the middle of the embankment.

3. In Case 3, wherein the input frequency is higher than the initial natural frequency of the embankment slope, the natural frequency of the model slope changes (reduces) because of the accumulation of plastic deformation. However, because the change in the natural frequency was considerably lower than that in the input frequency, resonance did not occur and the stability was maintained.
The deformation/failure of the fill slope cannot be defined by only one sliding surface. Multiple sliding surfaces are progressively formed depending on the ground and external conditions. Moreover, evaluating the seismic stability of the fill slopes depends not only on the magnitude of the acceleration of the input seismic wave but also on the relationship between the natural frequency of the fill slope and the input frequency. These phenomena cannot be considered in the seismic-intensity method or in the Newmark’s method used in the conventional design. Following are some of the studies planned for the future:

1. To improve the seismic countermeasures and seismic evaluations of fill slopes, it is important to grasp the frequency characteristics of unsaturated fill slopes, i.e., to investigate the natural frequency of the actual fill slope using microtremor array observation or PS logging, and to develop investigation techniques for the same.

2. We intend to conduct soil–water coupled finite deformation analyses to verify the deformation mechanisms of the fill slopes under a saturated condition. In addition, we intend to numerically investigate the effect of material properties (type of soil and density) and the shape of the slope (dimensions and slope angle) on the behavior of the slope in the event of an earthquake. Furthermore, the study will be extended to actual scale analysis for an actual fill slope for the purpose of predicting the damage of assumed earthquake.

3. A shaking-table test on an unsaturated fill slope should be carried out focusing on the natural frequency change during an earthquake to confirm the deformation/failure behavior and to examine the effect of seismic resistance by desaturation. We intend to conduct soil–water-air coupled finite deformation analyses.

4. In Case 1, a negative pore-water pressure occurred at the time of occurrence of the slip in the deep part. This negative pressure subsequently dissipated in conjunction with the absorption of water with the passage of time,
and a large deformation was produced in this process, which could cause the so-called delayed fracture. Some studies observed the occurrence of a delayed fracture of fill slopes from previous seismic damages, and this negative pressure is reproduced in the soil–water coupled analysis. However, in the tests, the excitation continued after measuring the negative pressure, thus failing to experimentally confirm this phenomenon. This will be investigated experimentally in the future.

Acknowledgments

The authors wish to acknowledge Associate Professor Tadashi Kawai of Tohoku University and Assistant Professor Toshiyuki Takahara of Kanazawa University for their assistance and beneficial discussion in conducting the 1G shaking-table tests. Shigeru Hotta, who is a master student in Geotechnical engineering laboratory of Nagoya University, helped in conducting the model tests. The authors would like to thank him for his enthusiastic effort. This work was supported by the JSPS KAKENHI (Grant Number 25249064)
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off the pacific coast of Tohoku earthquake' as an example. Journal of the Japan Society of Engineering


earthquake damage to Noto toll road and its restoration - Embankment damage and countermeasure work.


Figure captions

Figure 1  Classified deformation state due to the earthquakes (Murao et al. 2013)

Figure 2  Stainless steel soil tank

Figure 3  Cross section of the model slope

Figure 4  Results of unconfined compression test conducted on the bedrock part

Figure 5  Particle-size distributions for embankment and bedrock material

Figure 6  Moisture-density curve for the embankment material

Figure 7  Results of saturated triaxial compression test with constant lateral pressure for the embankment material

Figure 8  Sweep-test results: (a) Input acceleration and the acceleration measured at the crest, (b) Enlargement at a frequency of approximately 40 Hz, (c) Enlargement at a frequency of approximately 53 Hz, and (d) enlargement at a frequency of approximately 55 Hz

Figure 9  Amplification factor: (a) When maximum amplitude is constant, (b) when maximum amplitude increases, and (c) when maximum amplitude decreases

Figure 10  Measured acceleration and corresponding change in the amplification factor (input frequency of 50 Hz)

Figure 11  Change in the excess pore-water pressure (input frequency of 50 Hz)

Figure 12  Side view of the model prior to excitation and at stage 11 (input acceleration of 5.2 m/s²)

Figure 13  Slip in the shallow fluid part and an open crack in the surface layer at stage 9 (input acceleration of
4.2 m/s$^2$)

Figure 14 Excess pore-water pressure ratio and the acceleration waveform of the slope surface side under stage 9 of excitation (input acceleration of 4.2 m/s$^2$)

Figure 15 Excess pore-water pressure ratio and the acceleration waveform on the slope surface side in the case of excitation stage 11 (input acceleration of 5.2 m/s$^2$)

Figure 16 Measured acceleration and the change in the amplification factor (input frequency of 20 Hz)

Figure 17 Change in the excess pore-water pressure ratio (input frequency of 20 Hz)

Figure 18 State of the side surface of the model prior to excitation and during stage 12 (input acceleration of 5.7 m/s$^2$)

Figure 19 Change in the excess pore-water pressure ratio (input frequency of 80 Hz)

Figure 20 State of the model slope prior to excitation and during stage 15 (input acceleration of 7.7 m/s$^2$)
Table 1. Physical properties of embankment material.

<table>
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<td>Particle density</td>
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<td>Silt fraction (%)</td>
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<td>Medium sand (%)</td>
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<td>Coarse sand (%)</td>
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Table 2. Excitation conditions.

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<th>6</th>
<th>7</th>
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<td>—</td>
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<td>Input frequency 20 (Hz)</td>
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<tr>
<td>Input frequency 80 (Hz)</td>
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<td>5.7</td>
<td>6.2</td>
</tr>
</tbody>
</table>
Classified deformation state due to the earthquakes (Murao et al. 2013)
Stainless steel soil tank

364x191mm (300 x 300 DPI)
Results of unconfined compression test conducted on the bedrock part

218x206mm (300 x 300 DPI)
Particle-size distributions for embankment and bedrock material

212x205mm (300 x 300 DPI)
Moisture-density curve for the embankment material

Dry density $\rho_d$ (g/cm$^3$)

Water content $w$ (%)
Results of saturated triaxial compression test with constant lateral pressure for the embankment material
Sweep-test results: (a) Input acceleration and the acceleration measured at the crest, (b) Enlargement at a frequency of approximately 40 Hz, (c) Enlargement at a frequency of approximately 53 Hz, and (d) enlargement at a frequency of approximately 55 Hz
Measured acceleration and corresponding change in the amplification factor (input frequency of 50 Hz)

581x199mm (300 x 300 DPI)
Change in the excess pore-water pressure (input frequency of 50 Hz)

410x141mm (300 x 300 DPI)
Side view of the model prior to excitation and at stage 11 (input acceleration of 5.2 m/s²)
Slip in the shallow fluid part and an open crack in the surface layer at stage 9 (input acceleration of 4.2 m/s²)
Excess pore-water pressure ratio and the acceleration wave form of the slope surface side under stage 9 of excitation (input acceleration of 4.2 m/s²)
Measured acceleration and the change in the amplification factor (input frequency of 20 Hz)
Change in the excess pore-water pressure ratio (input frequency of 20 Hz)
State of the side surface of the model prior to excitation and during stage 12 (input acceleration of 5.7 m/s²)
Change in the excess pore-water pressure ratio (input frequency of 80 Hz)
State of the model slope prior to excitation and during stage 15 (input acceleration of 7.7 m/s²)