IMPULSIVE FAILURE EXPERIMENTS ON V-SHAPED SLOPING EMBANKMENT MODELS

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ABSTRACT

Despite numerous experiences of serious earthquake damages, little knowledge has been obtained about dynamic failure mechanism for sloping embankments constructed in narrow valleys. A fundamental study by means of a series of model tests followed by several analyses was performed to clarify dynamic failure characteristics of V-shaped sloping embankments subjected to impulsive loadings. A simple test equipment system has established to provide the embankment model made of wet sand with well-controlled impact loads. The system is capable of producing an impulsive inertia force with a triangular waveform, and giving rise to a three dimensional failure mass in the embankment. Accelerograms observed on the failure mass are found to give a truncated waveform with an abrupt drop at the end, as well as concavity associated with three dimensional geometry of the failure mass. Yield acceleration read off the truncated accelerograms shows a remarkable increase with a decrease in a width to depth ratio of the embankment, demonstrating restraining effects of the valley walls. Stability and relative displacement are calculated for imaginary tetrahedral failure blocks to compare with the yield acceleration and observed settlement, respectively.

Key words: dynamic, earthfill, failure, impact load, model test, slope stability, test equipment (IGC : E 6/H 4/E 8)

INTRODUCTION

Sloping embankments constructed in narrow valleys were seriously damaged by the 1978 Miyagiken-oki Earthquake. Extensive explorations after the earthquake revealed the cause and effect of the disaster, suggesting a countermeasure against existing poor embankments (Asada et al, 1980). Due to scarcity of ideal housing sites around densely populated urban area, construction of such sloping embankments will keep on increasing on a larger scale than ever with natural valleys filled with soil material.

It has been usual that earthquake resistant analysis of any embankment is performed

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by means of two-dimensional methods in which earthquake loading is treated as static inertia force acting mostly in the horizontal direction. From a viewpoint of earthquake engineering, this kind of highly simplified analysis is very practical, but involves a strong suspicion that it may lead us to misconception on real characteristics of dynamic failure of sloping embankments subjected to disastrous earthquake motion. These also apply to earthquake resistant design of earth and rockfill dams.

First of all, it has been pointed out that not only such vibration characteristics as natural period and vibration shape, but also static stability itself are likely to depend on geometry of an embankment, especially when it locates in a narrow valley (Hatanaka, 1952; Baligh et al, 1955; Lefebre et al, 1973; Hovland, 1977; Ohmachi et al, 1982; Ohmachi et al, 1983). This simply requires the embankment built in a narrow valley to be analyzed as a three dimensional earth structure.

Next, as for the direction of the inertia force concerned in the stability analysis, it should be related to a vibration mode in which total failure of the embankment can be easily triggered. It seems reasonable to suppose that the fundamental mode of vibration is most influential in the total failure of the embankment. Through experimental and numerical studies, it has been well known that the fundamental vibration mode of an embankment is, whether it is sloping or not, much associated with shear distortion in the direction parallel to the natural ground surface on which the embankment is founded.

In addition to these, characteristics of failure caused by dynamic loading which varies with time will be different from those of failure caused by static loading as suggested by Newmark (Newmark, 1965). Anyway, regarding sloping embankments locating in narrow valleys, only a limited knowledge on the dynamic failure characteristics has been obtained in spite of a great importance for geotechnical earthquake engineering. For this reason, impulsive failure experiments were performed, intending to make an elementary contribution to clarify dynamic failure mechanism with respect to V-shaped sloping embankments put into vibration in the fundamental mode.

**EXPERIMENTAL PROCEDURE**

**Similitude**

In order to simulate realistic dynamic failure on a small-scale model of an embankment, similitude was taken into account in selection of model material as well as application of impulsive loading (Okamoto, 1973). The subscripts \( m \) and \( p \) will be used to refer to the model and the prototype, respectively.

The ratio of own weights is given by

\[
\frac{W_m}{W_p} = \frac{\rho_m L_m \sqrt{g}}{\rho_p L_p \sqrt{g}} = \frac{\rho_m}{\rho_p} L_m^2 \text{ (1)}
\]

where \( \rho, L, g \) and \( \lambda \) represent mass density, any linear dimension, acceleration of gravity and the ratio of the linear dimensions, respectively. Regarding inertia forces, the same ratio must be maintained. Accordingly, acceleration \( a \) associated with the impulsive loading should be given by

\[
\frac{a_m}{a_p} = \sqrt{\frac{L_m^2/T_m^2}{L_p^2/T_p^2}} = \lambda \tau = 1 \text{ (2)}
\]

where \( \tau \) represents the ratio of the time dimensions. This leads to

\[
\tau = \sqrt{\lambda} \text{ (3)}
\]

The ratio of stresses which are induced by the body forces such as the own weight and inertia force is given by

\[
\frac{\sigma_m}{\sigma_p} = \frac{W_m/L_m^2}{W_p/L_p^2} = \frac{\rho_m}{\rho_p} L_m^2 \text{ (4)}
\]

Regarding shear strength of the embankment material, the ratio of cohesive components is the same as that of the stresses, giving

\[
\frac{c_m}{c_p} = \frac{\rho_m}{\rho_p} \lambda \text{ (5)}
\]

while the frictional component is not affected.
Each embankment model was prepared by pouring Toyoura sand with one percent of moisture content from almost constant height of 20 cm into the V-shaped sliding table. Deposited sand then was leveled to form a uniform tetrahedron with a horizontal top surface. Toyoura sand with about one percent of moisture content was chosen for the embankment material in order to satisfy the similitudes in Eqs. (5)–(6). The fundamental period of vibration of the embankments were supposedly around 10 ms.

Application of Impulsive Loads
When preparation of an embankment model and installation of instruments are finished, steel–wire strings stretched to support the sliding table at the upper end is released to let the table slide down smoothly on the rollers along the rigid table until it collides with the rigid wall, which causes impulsive loading to the sliding table as well as to the embankment. Intensity and duration of the impulsive load can simply be controlled by adjusting sliding distance and selecting interface material of the rigid block. A 2 cm thick rubber mat was added to the surface of the rigid wall for this purpose.

Test Program and Instrumentation
Geometry of the embankment model depends on inclination of the rigid table and angle between the V-shaped surfaces of the sliding table. Those parameters used in the experiments are:

1. Inclination of the rigid table \((\tan \alpha') = 1:3.0\) and \(1:5.5\)
2. Width to depth ratio of embankment \((W/D=2\cot \theta')=2, 4\) and \(6\)
3. Angle of embankment slope \((\beta')=30^\circ, 35^\circ, 40^\circ\) and \(45^\circ\)

In each case, the maximum base length of the embankment was kept to 160 cm.

To detect acceleration time history, accelerometers were installed at observation points shown by numbers from 1 through 8 in Fig. 1. For the points 1 and 2 locating on the sliding table, accelerometers of a
strain-gage type were used, while those of a piezoelectric type with a diameter of 6 mm, length of 10 mm and weight of 1.2 gf were used for the points 3-8 locating on the embankment model. Directions of the installed accelerometers were parallel to the sliding table for the points 1 and 2, and horizontal for the rest. Settlement developed over the top surface of a failed soil-mass were measured by a eddy-current pickup the measuring range of which is 0-5 mm.

**EXPERIMENTAL RESULTS**

*Acceleration Waveform*

Examples of accelerograms obtained in the experiments are shown in Fig. 2, from which one can see that;

1. At the moment when the wire strings are released, every accelerogram shows a shift to lower position corresponding to \(-g\cos \alpha'\), in which \(\alpha'\) denotes an angle of inclination of the rigid table.

2. The accelerograms of an impulsive loading input to the sliding table are of a symmetrically triangular form with the duration about 70 ms. At the time when the main rising is observed in the input accelerograms, the rising is also observable without a time delay in the rest of the accelerograms.

3. During the acceleration of the sliding table exceeds a certain level, most accelerograms of the embankment show a truncated form. This is largely associated with sliding failure developed in the embankment. If any force is applied to the embankment during sliding along the rollers, the acceleration level will change from the shifted position. Thus, the input acceleration measured from the shift axis is, hereafter, defined as the yield acceleration of the embankment.

4. As for the truncated form, concavity is perceptible especially for the embankment model with a small \(W/D\) ratio. The truncated accelerogram finally shows an abrupt drop to coincide with the accelerograms of the sliding table.

5. The time when the abrupt drop occurs in the accelerogram is not always the same for the various location within a sliding mass.

**Yield Acceleration**

In Fig. 3 are shown the previously defined
yield accelerations read from the accelerograms. It is obvious that the yield acceleration tends to increase with a decrease in the width to depth ratio $W/D$, indicating that the restraint of valley walls has a significant effect to increase the yield acceleration of the embankment. It should be noticed, however, that the increase in the yield acceleration is dependent not only on the $W/D$ ratio but on inclination of the sloping rigid table. For example, for the cases where $\tan \alpha'$ is equal to 1:3, the acceleration increase is almost negligible for $W/D$ larger than 4, while for the cases where $\tan \alpha'$ is equal to 1:5.5, the acceleration increase is remarkable over a wider range of $W/D$.

**Fig. 3. Yield acceleration of V-shaped sloping embankments**

**Shape of Failure Surface**

Outlines of cracks and failure surfaces observed on the top surfaces of the embankments are shown in Fig.4. Although the shape of the failure surface seems sensitive enough to localized conditions to vary among embankments, it essentially gives a shallow U- or V-shaped line on the top surface and extends orthogonally to the side boundaries of the lower surface.

In case of extremely large amplitude of pulse acceleration, say over $1g$, total sliding of embankments along bedding planes was observed with a similar truncation in all the accelerograms of the embankments. Judging from visual observation and waveforms of the accelerograms, such total sliding hardly took place in the experiments described in this paper.

It was difficult to distinguish toe lines of the failure surfaces at the front of the embankments. In Fig.2, the accelerograms
shown in Fig. 5. The x axis is parallel to the embankment shoulder, and the z-axis is parallel to the bottom line of the failure mass. In Fig. 5, \( \theta \) and \( \alpha \) denote angles between the x-axis and the failure surface, and between the z-axis and a horizontal plane, respectively.

The own weight of the failure mass is given by

\[
W = \frac{1}{3} H^3 \left( \cot \alpha - \cot \beta \right)^2 \cot \theta \sin \alpha \quad (7)
\]

where \( \gamma \) is the unit weight of the material and \( H \) is the maximum height of the embankment slope which makes an angle \( \beta \) against the horizontal plane, as shown in Fig.1. If sliding motion in the z direction is only permitted for the failure mass, equilibrium of forces in the y direction gives

\[
2 N \cos \theta = W \left[ \cos \alpha - k(t) \sin(\alpha - \alpha') \right] \quad (8)
\]

where \( k(t)W \) denotes the inertia force directing parallel to the slope of the table which makes the angle \( \alpha' \) against the horizontal plane. From the equation of motion in the z direction, a pair of sliding resistances acting along the failure surfaces are given by

\[
2T = W \sin \alpha - (k(t)W \cos(\alpha - \alpha')) \quad (9)
\]

In a limit equilibrium state, this becomes equal to the maximum shear force associated with the soil strength. Thus,

\[
T = \mu N + c A \quad (10)
\]

where \( A \) denotes a half area of the sliding surface given by

\[
A = \frac{1}{2} H^2 \left( \cot \alpha - \cot \beta \right) \cosec \theta \quad (11)
\]

Equating Eqs. (9) and (10), and substituting Eqs. (8) and (11) provide the yield acceleration as

\[
k_y \theta = \frac{\sin \alpha \cos \theta \mu \cos \alpha - \frac{3c \sin \alpha}{\gamma H \sin(\alpha - \alpha')}}{\mu \sin(\alpha - \alpha') + \cos \theta \sin(\alpha - \alpha')} \quad (12)
\]

The yield acceleration of the embankment is determined by calculating \( k_y \) values for a lot of tentatively supposed geometric parameters and detecting the smallest one.
among them. To compare with the observation, the analytical results are shown by broken lines in Fig.3, for which strength parameters of the Toyoura sand were inversely determined to give the best fit to the measured yield acceleration. Though the analytical yield acceleration tends to increase with a decrease in the $W/D$ ratio, the increase is not so remarkable as that of the observation. The disagreement between the analysis and observation could be attributed to rough assumptions employed in the analysis with respect to the failure surface and mobilized shear strength.

**Acceleration Waveform**

Regarding the previously assumed failure mass, the equation of motion in the $z$ direction is written by

$$\frac{W}{g}(\ddot{u}_o+\ddot{u})=W \sin \alpha - 2T \quad (13)$$

where $u_o$ denotes displacement of the sliding table, $u$ sliding displacement of the failure mass relative to the rest of the embankment, and a superposed dot derivative with respect to time.

In the above mentioned experiments, the impulsive acceleration applied to the table varies with time and is directed in parallel with the slope of the rigid table. Thus, one obtain the following expression.

$$\ddot{u}_o=k(t)g \cos(\alpha - \alpha') \quad (14)$$

As the forces acting in the $y$ direction are in equilibrium during the sliding takes place, the equilibrium equation provides

$$k(t)W \sin(\alpha - \alpha') = 2N \cos \theta - W \cos \alpha \quad (15)$$

Finally, the relative acceleration in Eq. (13) can be given by a simple expression as

$$\ddot{u}=[\cos(\alpha - \alpha') + \sin(\alpha - \alpha') \sec \theta] \times [k_y - k(t)]g = -B[k(t) - k_y]g \quad (16)$$

For simplicity, let $k(t)$ be approximated by an alternate triangular impulse with the maximum $F$ larger than $k_y$ and the period 2$T$. Taking the time axis $t=0$ at the time when $k(t)$ reaches $k_y$, and letting the duration in which $k(t)$ exceeds $k_y$ be expressed by $t_i$, the accelerogram of the failure mass can be analytically determined, with a result shown in Fig.5. In Table 1 are explicitly shown the acceleration values corresponding to the time 1 to 6 numbered in Fig.5. It is worth noticing that the analytically determined waveform is well representing the characteristics of the observed accelerograms such as the truncation, the concavity and the abrupt drop.

As for the concavity, it appears to be inherent to a three-dimensional failure mass with a small $W/D$ ratio. This is because the concavity results when the value $B'$ defined by

$$B' \equiv B \sec(\alpha - \alpha') = 1 + \mu \tan(\alpha - \alpha') \sec \theta \quad (17)$$

is significantly larger than unity. In Eq. (17), the term $\sec \theta$ takes its minimum for a two-dimensional failure mass for which the angle $\theta$ vanishes, and it will become larger as the $W/D$ ratio of a three-dimensional failure mass becomes smaller, making the concavity in the truncated accelerogram more remarkable. This seems to be consistent with the observation of the experiments in which the value $B'$ was 1$<B' \leq 1.3$

**Settlements due to Sliding**

Integrating twice the relative acceleration in Eq. (16) and taking into account appropriate initial conditions provide relative displacement between the failure surface. At the time 6 shown in Fig.5, the relative displacement $u(t)$ and vertical settlement $v(t)$ associated with the relative displacement

<table>
<thead>
<tr>
<th>No.</th>
<th>Time $t$</th>
<th>Input Acc. $k(t)g$</th>
<th>Acc. of sliding Block $k(t)g + \ddot{u} \sec(\alpha - \alpha')$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$t_1/2$</td>
<td>$F_g$</td>
<td>—</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>$k_yg$</td>
<td>$k_yg$</td>
</tr>
<tr>
<td>3</td>
<td>$t_1/2$</td>
<td>$F_g$</td>
<td>$[(F-k_y)(1-B')-k_y]g$</td>
</tr>
<tr>
<td>4</td>
<td>$t_1$</td>
<td>$k_yg$</td>
<td>$k_yg$</td>
</tr>
<tr>
<td>5</td>
<td>$(2F-k_y)t_1$</td>
<td>0</td>
<td>$Bk_yg$</td>
</tr>
<tr>
<td>6</td>
<td>$(1+\sqrt{2}/2)t_1$</td>
<td>—</td>
<td>$[k_y-2(B'-1)(k_y-F)]g$</td>
</tr>
</tbody>
</table>
result in

\[ u(t_0) = \frac{1}{8} Bg T^2 (F - k_0)^{3/2}/F^2 \]  
\[ v(t_0) = u(t_0) \sin \alpha \]

According to these, the settlements are presumably proportional to \((F - k_0)^{3/2}/F^2\). On this basis, using the peak and yield accelerations read off the accelerograms, the observed settlements were plotted against \((F - k_0)^{3/2}/F^2\), with a result shown in Fig. 6 in which ○ and • are for the slope of the rigid table 1:3.0 and 1:5.5, respectively. The broken lines in Fig. 6 indicate rough trends among the plotted data from which the linear relationship presumed above can hardly be found. The scattering of the plotted data could be attributed to the above mentioned rough assumptions regarding the failure surface and mobilized shear strength.

CONCLUSION

Dynamic failure characteristics of V-shaped sloping embankments subject to impulsive ground acceleration were studied by means of laboratory failure experiments on models followed by several analyses. Conclusions drawn from this study are:

1. The yield acceleration which gives rise to the onset of sliding failure in a three-dimensional embankment is much affected by geometry of the embankment, in other words, by restraint of the valley walls. That is, the yield acceleration shows a significant increase with a decrease in the width to depth ratio \(W/D\).

2. The restraining effect of valley walls on the yield acceleration appears dependent on the slope angle of the embankment.

3. Stability analysis which assumes an a priori wedge-shaped failure surface does not provide the yield acceleration with a satisfactory level of accuracy. The restraining effect is not adequately accounted for by the analysis, either.

4. During the sliding failure takes place in the embankment subjected to the impulsive loading, acceleration of the failure mass shows a truncated form with an abrupt drop at the end.

5. Concavity which can be found in the truncated acceleration waveform is associated with the three-dimensional geometry of the failure mass.

6. The observed settlements of the failure mass relative to the rest of the embankment does not show good agreement with the estimation which was formulated in terms of the yield acceleration, peak acceleration of the input loading and the duration. Reliable prediction of the failure surface and the mobilized strength is needed to improve the estimation.

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REFERENCES

219-226.