Effects of Earthquakes on Ground (I)

— Ground Cracking, Soil Liquefaction, and Sliding of Slopes —

By

Yoshimasa KOBAYASHI

Railway Technical Research Institute

Abstract

An empirical formula for the hypocentral distance of damaged railroad subgrade in earthquakes is given. Ground cracks caused by earthquakes are classified into those of tensile type, normal-fault type, and strike-slip type. After description of soil-liquefaction phenomena in earthquakes, the mechanism of spontaneous liquefaction of soils is reviewed critically. The solid-mass sliding and flow slide of soil slopes, and the fragment failure and rock-mass sliding of rock slopes are described. The effect of pore-water pressure on the stability of slopes in cases of the solid-mass sliding and the flow slide is discussed.

§ 1. Introduction

Among various types of earthquake damage to ground, a slight deformation of ground or earth structures is distributed most widely. The cause of the deformation must substantially vary according to the locality or occasions: densification of unconsolidated layers resulting in settlement of the ground surface; shear deformation of soils without any volume change leading to a subsidence at a part of the ground surface and an upheaval at the adjacent part of it; soil liquefaction developing into flow of soils adapting to the pressure gradient, etc. It is noted, however, that the tectonic movement of the earth crust is not included in the deformation of ground throughout in this paper (RETAMAL and KAUSEL, 1969).

The damage classified into the deformation in this paper is, therefore, the initial stage of various types of damage as cracking, sliding, or liquefaction and it will be included in other types of damage if its mechanism should be known. Moreover, it will always accompany any other type of damage at least partly.

The hypocentral distance of damaged railroad subgrade in earthquakes on which relatively much data are available has been considered to correspond to the reduced radius of the crustal deformation in the earthquakes (DAMBARA, 1966), i.e. $\log_{10} r = 0.51 M - 2.27$ (km) (KUBOTERA and KOBAYASHI, 1970). Further

investigation on this phenomenon revealed that the farthest hypocentral distance where deformation of railroad subgrade exceeding 50 mm either in the lateral or the vertical direction may occur is about 2.5 times as large as the radius of the crustal deformation and is approximated by

$$\log_{10} r = 0.51 M - 1.86 \text{ (km)}, \quad (\text{Fig. 1})$$

Fig. 1. Farthest hypocentral distance of railroad subgrade deformed more than 50 mm either in the lateral or the vertical direction and the earthquake magnitude.

where M is the earthquake magnitude.

The earthquake damage to ground is so common and remarkable as described above, but its phenomena are of variety and many of them have not yet been quantitatively treated in success nor qualitatively well understood. A phenomenological approach, therefore, is as important as the theoretical one in the investigation of the problem in order to supplement the latter by observation of actual phenomena.

From the viewpoint above, this paper deals with several frequent phenomena of earthquake damage to ground and earth structures. First, ground cracks caused by earthquakes are classified and probability of each type is considered in relation to stress states in which every type of cracking may occur. Second, after various phenomena of soil liquefaction in earthquakes are described, the mechanism

of spontaneous liquefaction of soil is reviewed critically. Third, failures of soil and rock slopes are classified into four groups according as materials of the slopes and states of sliding mass. Lastly, the effect of pore-water pressure on the stability of slopes in cases of the solid-mass sliding and the flow slide is discussed on the basis of some observations in the Tokachi-oki earthquake, 1968 and of an analysis on a simple idealized slope model.

§ 2. Morphology of Ground Cracks in Earthquakes

Ground cracks caused by earthquakes are classified into those of tensile type, normal-fault type, and strike-slip type.

A crack of tensile type runs in most cases vertically and in parallel to contour lines of the ground surface (Fig. 2). It is caused by a lateral tension at a part of ground as at a

Fig. 2. Crack of tensile type in the Tokachioki earthquake, May 16, 1968.

slope shoulder prior to apparent sliding or on the top of an embankment. The above situation is most typically realized on a sliding mass in flow slide, where cracks perpendicular to the direction of flow are predominant (see Fig. 8). It is reported that many cracks of this type occurred on the ground surface in the Alaska earthquake and they were attributed to "landspreading" caused by liquefaction of

underlying sediments (McCULLOCH and BONILLA, 1970).

A crack of normal-fault type is produced along a nearly vertical sliding surface and a discontinuous vertical difference in level across it results. It occurs at the upper end of a sliding surface or at the boundary of subsidence of ground due to liquefaction or other causes (Fig. 3). It is natural that the ones

Fig. 3. Crack of normal-fault type in the Tokachioki earthquake, May 16, 1968.

relating to sliding are produced very often nearly in parallel to contour lines of topography.

While the above two types are very frequent, cracks of strike-slip type, which are characterized by a horizontal offset of ground across them, are found only in reports by NAGUMO (1964) and McCULLOCH and BONILLA (1970). The former writer observed two sets of this type of crack on the bank of Hachirogata lagoon running diagonally to the axis of the bank in the Oga-Oki earthquake, 1964. The first set is assumed to have been caused by an axial compression of the bank while the other, which is noted to have occurred at a curving point of the bank, by an axial tension (Fig. 4).

Each type of ground cracking occurs in a corresponding stress state. The stress state near the ground surface is influenced by the presence of the ground surface (SCHEIDEGGER, 1964). If the ground surface is assumed to be horizontal, the directions of three principal stresses are vertical (z) and two horizontal directions (x and y) perpendicular to each other. Relations between the order of magnitudes of the principal stresses and the type of ground cracks which may be produced in each stress state are summarized in Table 1, where the compressive stress is defined as positive.

The probability of occurrence of each type

of cracking is considered as follows: The case (1) $\sigma_x > \sigma_y > \sigma_z$ corresponds to a state where

Fig. 4. Cracks of strike-slip type on the bank of Hachirogata lagoon in the Oga-Oki earthquake, May 7, 1964.

Table 1. Types of Ground Cracking, Stress States, and Propability of Their Occurrence

Case	Stress States	With or Without Tension*	Tensile Strength	Type of Cracks	Probability		Notation
					Static	Dynamic	
(1)	$\sigma_x > \sigma_y > \sigma_z$			Thrust	none	none	(1)
(2)	$\sigma_x > \sigma_z > \sigma_y$	$\sigma_y \geq 0$		Strike Slip	none	low	(2.1)
		$\sigma_y < 0$	low	Tensile	none	high	(2.2.1)
		$\sigma_y < 0$	high	Strike Slip	none	low	(2.2.2)
(3)	$\sigma_z > \sigma_x > \sigma_y$	$\sigma_y \geq 0$		Normal Fault	high	high	(3.1)
		$\sigma_y < 0$	low	Tensile	high	high	(3.2.1)
		$\sigma_y < 0$	high	Normal Fault	low	low	(3.2.2)

* Compressive stresses are defined as positive

the magnitudes of two horizontal principal stresses are larger than that of the vertical one and is hardly realized by a static displacement of soil. Dynamically, the state described above can be realized when a strong compressional wave is propagated, however the fact that no ground cracking of thrust type has been observed in earthquakes suggests that such a strong compressional wave does not exist in nature; The case (2) $\sigma_x > \sigma_z > \sigma_y$ corresponds to a state that one of the horizontal principal stresses is larger than the vertical one and the type of cracking also hardly occurs by a static displacement of soil. Accordingly, the two sets of cracking of strike-slip type observed by Nagumo (2.1) and (2.2.2) might have been produced dynamically. The tensile crack (2.2.1) is easier to be produced under the existence of tensile stresses, because of generally a slight tensile strength of soil. Consequently, the case of strike-slip type (2.2.2) reported by Nagumo is considered to be very rare; The case (3) $\sigma_z > \sigma_x > \sigma_y$ corresponds to a state where both magnitudes of two horizontal principal stresses are smaller than that of the vertical one and is analogous to the natural state of stress in ground. The state is, therefore, most easily realizable, statically as well as dynamically. Such a stress state is realized in the neighborhood of a shoulder of slopes or in a similar situation. However, it is noted in this case also, that, if the smaller horizontal principal stress becomes tension, a tensile crack will be produced easier than that of any other type.

All the cracks cited above except those of strike-slip type may be caused by static displacements of soil during earthquakes. In contrast to these, cracking presumably under direct influence of dynamic ground motion is reported by KAWASUMI (1950) in the Fukui earthquake, 1948. According to KAWASUMI, a peasant wife was crushed to death in an about 100 m long "opening and closing crack" between which her body was held up to her neck. In the same earthquake it is also reported that a mother with her child who was lost after running out of a house at the first shock of the earthquake was dug out of a crack in front of the house three days after the earthquake. The type of cracking must be possible on embankments as well as on the plain ground.

The absolute value of strain $|\epsilon|$ in ground due to a wave motion with a particle velocity v may be approximated by v/C , where C is the velocity of wave propagation in the ground. Assuming $v=50$ cm/sec and $C=100\sim500$ m/sec in the epicentral region (KOBAYASHI, 1969), one obtains the strain $|\epsilon|=1\sim5\times10^{-3}$, which coincides with the order of the ultimate strain of concrete or other brittle materials (HATANO, 1968).

§ 3. Soil Liquefaction in Earthquakes

Liquefaction of soils during earthquakes had been experienced since long (SEED, 1968) and it was reconfirmed through the Alaska earthquake and the Niigata earthquake, both in 1964, that it may be quite destructive for ground or earth structures. In the Niigata

earthquake, liquefaction occurred in the area consisting of loose deposited sands and silts along the Shinanogawa and Aganogawa rivers with N value of standard penetration test lower than 10 down to the depth of 10 to 15 m. The water table in the area was 1 to 2 m deep from the ground surface.

In the Niigata earthquake, liquefied sands were ejected frequently along weak lines in the ground as well as at isolated holes of crater shape, where the surface was covered by the deposited sands (Fig. 5). It was especially

Fig. 5. Ejecting holes in a row on the liquefied ground in the Niigata earthquake, June 16, 1964.

frequent in the neighborhood of buildings suggesting a certain effect of a mechanical discontinuity on the occurrence of ejecting holes. A borer who was working on a reclaimed land at Nanaehama beach, Hakodate and experienced the Tokachioki earthquake, 1968 told, "Ejections of sand water occurred 5 to 10 minutes after the initiation of the

earthquake and lasted for about an hour. The ground was so unstable as a floating island during the earthquake. The rate of ejection from a large hole seemed to be about 2 tons per minute. The ground surface subsided about 10 cm from the level prior to the earthquake at the point of thickest deposit." (Committee of the Japanese Society for Soil Mechanics., 1968).

In the liquefied state a sand converts into a liquid of a density equal to that of the corresponding saturated sand (TSUCHIDA, 1965). In the area where the ground is liquefied on the whole, objects of larger bulk density than that of the liquefied sand sink and that of smaller density are lifted (Fig. 6). In the Tokachioki earthquake, a tensile crack is reported to have occurred above a buried pipeline due to an upheaval of the pipe which was caused by the difference in bulk densities of the empty pipe and the liquefied sand in the reclaimed ground at Nanaehama beach. It takes rather long until a liquefied sand restores the strength and in the case of Nanaehama beach the sand was so soft that it could not support the weight of a person without a large consolidation even after several days from the earthquake (Committee of the Japanese Society for Soil Mechanics., 1968).

When liquefied, a sand is deprived of the shearing resistance almost perfectly and flows like a liquid under an extremely gentle inclination as was the case in a flow slide at a railway-embankment slope 11 m high in the Tokachioki earthquake, in which the fill material flowed about 70 m farther than the slope end. According to McCULLCH and BONILLA (1970), many ground cracks have resulted from the stress generated in the surface materials by lateral displacement and spreading of the presumably liquefied underlying sediments in the Alaska earthquake.

The spontaneous liquefaction of sand is defined by TERZAGHI and PECK (1948) as "the sudden decrease of the shearing resistance of a quick sand from its normal value to almost zero without the aid of seepage pressure" and is considered to be "caused by a collapse of the structure of the sand, associated with a

Fig. 6. Upheaval of a river bed due to liquefaction of ground in the Niigata earthquake, June 16, 1964.

sudden but temporary increase of the pore-water pressure." As a mechanism of the spontaneous liquefaction, it has widely been accepted that a saturated sand mass with a void ratio greater than the critical value corresponding to an overburden pressure contracts prior to failure and that as a consequence the pore-water pressure increases because of the decrease in voids in the sand mass to deprive of the shearing resistance entirely when the pressure attained the initial effective stress in the sand (TSCHEBOTAROFF, 1951).

Being opposed to the above concept of the critical void ratio, MASLOV (1957) presented a "Filtration theory" and claimed the presence of a critical acceleration. The theory is to conform with the fact that foundations of sand with a void ratio smaller than the critical value were observed to fail in USSR. He considered that a saturated sand is consolidated when subjected to an acceleration stronger than a value (critical acceleration) corresponding to its original density, and the rate of development of pore-water pressure is determined by the rate of densification of the saturated sand due to vibration and the rate of dissipation of excess pore-water pressure from decreasing voids.

The concept of the critical void ratio, therefore, may be more perfect if it should be

improved to be a function not only of confining pressure but of externally applied acceleration as well as of boundary conditions to which the soil is submitted. Thus, the liquefaction of soil is concerned with an irreversible consolidation of soils, which is recognized also experimentally. According to TSUCHIDA (1965), for instance, a saturated sand settles in a more compact state than the initial one after liquefaction and it can no more be liquefied by vibration of an intensity equal to that in the first time.

The types of soil which may be liquefied in earthquakes can be determined only inductively through experiments or field experiences. According to past experiences (TERZAGHI and PECK, 1948; SEED, 1968; MOGAMI *et al.*, 1966; YAMADA *et al.*, 1968), variety of soils can be liquefied, whose content of fine particles of diameter $D \leq 0.074$ mm is less than 20%.

§4. Classification of Failure of Slopes in Earthquakes

The failure of slopes is one of the most frequent phenomena among effects of earthquakes and the size of it may vary ranging from a slight falling off of weathered materials from hillsides to a disastrous failure as the one at Nebukawa in the Great Kanto earthquake,

1923, which swept a train with 111 passengers and clues into the sea by a stroke (Japanese National Railways, 1923).

The failures of slopes are classified according as materials composing the slopes and the state of sliding masses as follows:

soil slope	{ solid-mass sliding flow slide	{ slope failure base failure
rock slope	{ fragment failure rock-mass sliding	

The solid-mass sliding of soil slopes occurs in most cases rotating along a circular or other

curved surface. The type of sliding is further divided into slope failure and base failure. The sliding surface of the former passes through the toe of a slope and that of the latter touches the underlying firm base. This sliding occurs most frequently in slopes of cohesive soil which cannot sustain the stability of the sliding mass and the original shape of the mass is maintained relatively well as seen in Fig. 7.

The flow slide is characterized by that the material does not slide as a solid mass but fluidlike (Fig. 8). This type of failure occurs frequently in slopes of loose sandy soils satu-

Fig. 7. Rotating slide in the Tokachioki earthquake, May 16, 1968.

Fig. 8. Flow slide in the Tokachioki earthquake, May 16, 1968.

rated by water through rainfall or other causes, as was the case in the Tokachioki earthquake, 1968. The flowed material reaches extremely long from the slope end and is deposited as flat as almost horizontal. The soil which has covered the slope surface prior to sliding is torn into pieces and scattered over the deposited material. This type of failure is considered to be caused by the increase in pore-water pressure in soils either during vibration or after a sliding is initiated. The stability as a solid mass obviously does not apply to a liquefied or nearly liquefied state and the failure is initiated from weakened portions of the slope surface. The mode of

displacement of sliding material, accordingly, is different from that of the rotating slide and the amount of displacement decreases with the depth from the surface of the slope.

The fragment failure is falling off of weathered residual materials on a rock slope and the presence of a deteriorated part at the surface of slope is essential. The failure is caused when the strength of surface part of a rock slope is ultimately lost under the effect of seismic acceleration and the failed mass is deposited nearly equally inclined to the macroscopic angle of internal friction of the material (Fig. 9). In this case too, the stability as a solid mass is not applicable.

Fig. 9. Fragment failure of a weathered rock slope in the Tottori earthquake, Sept. 1, 1943.

In contrast to this, the rock-mass sliding is possible in case a rock mass is deteriorated along joints or other discontinuities without appreciable decrease in strength of rock piece itself. In the Nankai earthquake, 1946, a rock mass, 4 m thick, 10 m high, and 20 m long, consisting of mudstone was pushed out 45 cm horizontally along the bedding plane at a rock cutting of the National Railways.

§ 5. Difference in Mechanisms Between Solid-Mass Sliding and Flow Slide

The difference in appearance of slides is caused by the difference in mechanisms of them. It may be demonstrated through an

analysis on a plane-surface sliding model. If the shearing resistance of a soil composing a slope is assumed to be expressed by the following Coulomb-Mohr's equation

$$s = c + \gamma H \tan \phi,$$

the critical horizontal seismic intensity k_h for sliding with which an infinitely long soil layer of height H consisting of the above homogeneous soil is given by

$$k_h = \tan(\phi - \theta) + \frac{c}{\gamma H (\cos \theta + \sin \theta \tan \phi)},$$

(SEED and GOODMAN, 1964), where θ and γ are the inclination of the slope and the specific density of the soil, respectively. According

to the relation, k_h increases infinitely as H decreases and it tends to $\tan(\phi - \theta)$, as H increases. Accordingly, under the condition where the above relation holds, the critical acceleration decreases with the depth of the sliding surface.

In reality, however, the sliding surface is realized at a certain finite depth implying the existence of a resistance which increases with the depth and is ignored in the above approximate relation. We will now examine, as an

example, how the slope-end resistance which increases with the depth may affect the depth of the sliding surface. However, the problem will be treated only approximately, since if it might be done rigorously, the assumption of the plane-surface sliding itself must be replaced by a more general one which leads to introduction of many parameters and sacrifices the comprehensibility.

Assuming the slope-end resistance in Fig. 10,

Fig. 10. Plane-surface sliding model with slope-end resistance dQ under horizontal seismic intensity k_h .

$$Q = \frac{\gamma' H^2}{2} \tan^2 (45^\circ + \phi/2) + 2cH \tan (45^\circ + \phi/2),$$

one obtains the critical horizontal seismic intensity

$$k_h = (cL + \gamma' HL (\mu \cos \theta - \sin \theta) + \frac{\gamma' H^2}{2} N_\phi) / \gamma' HL (\cos \theta + \mu \sin \theta),$$

and the minimum critical seismic intensity occurs at the depth

$$H = \sqrt{2cL / \gamma' N_\phi},$$

where γ , γ' , μ , and N_ϕ are the saturated and submerged densities of soil, $\tan \phi$ and $\tan^2 (45^\circ + \phi/2)$, respectively.

The variation of the critical intensity as a function of H/L is shown in Fig. 11. In the figure it is recognized that the minimum is not a strong one, as was pointed out by SEED and GOOGMAN (1964), implying that the depth H may easily be influenced by other factors such as a slight inhomogeneity of strength of soil, not included in the analysis. The indetermi-

Fig. 11. Critical horizontal seismic intensity as a function of the ratio between the height H and the length L of a sliding mass.

nate character is especially remarkable in the side of larger H/L , while in the side of smaller H/L the critical intensity increases rapidly as H/L approaches to zero. From the above it might be suggested that the actual H/L may fluctuate widely on the both sides of the minimum without being excessively small.

The tendency described above will be examined on actual cases observed in the Tokachioki earthquake, 1968 in the following. In the damage survey immediately after the earthquake, slides were classified provisionally into slope-surface flow slide (Type I) and ordinary slope failure (Type II) as shown in Table 2. In Fig. 12 the value of cohesion and

Table 2. Dimensions of Sliding Masses in the Tokachioki Earthquake, 1968

Site	Measured Height H	Measured Length L	Type of Sliding
1	1.5 m	24 m	I
2	3.0	12	II
3	0.5	8	I
4	3.0	9	II
5	5.0	19	II

friction of soils and the corresponding critical seismic intensities are given which are deduced on the assumption of the plane-surface sliding described above, i.e., the depth of sliding

Fig. 12. Cohesion and friction of soil, and critical horizontal seismic intensity satisfying $H = \sqrt{2cL/\gamma'N\phi}$, on the basis of observed H and L of slided masses in the Tokachioki earthquake, May 16, 1968.

surface at $H = \sqrt{2cL/\gamma'N\phi}$. It is noted that the ranges of values of c and k_h differ markedly between the cases of the slope-surface flow slide and those of the slope failure, and that the strength and the critical seismic intensity are abnormally low in case of the former type of sliding. It suggests that at sites 1 and 3 the decrease in strength of soil presumably due to an increase in pore-water pressure is more important than the direct effect of the seismic acceleration, while at sites 2, 4, and 5, the acceleration is very important as a factor influencing the stability of slopes.

The effect of pore-water pressure may be examined by introducing the pressure u into the expression for the shearing resistance. In

this case the normal stress n , shear stress t , and resistance per unit area of a sliding surface r are

$$n = (\gamma_t H - \gamma_w h) \cos \theta - u,$$

$$t = (\gamma_t H - \gamma_w h) \sin \theta,$$

and

$$r = (\gamma_t H - \gamma_w h) \cos \theta \tan \phi - u \tan \phi - c$$

where h is the static ground-water level.

The liquefaction is defined as a state in which the normal stress n becomes zero. For the case the non-dimensional pore-water pressure is given by

$$\left(\frac{u}{\gamma_w H} \right)_l = \left(\frac{\gamma_t}{\gamma_w} - \frac{h}{H} \right) \cos \theta + A,$$

where A is a certain value of pressure neces-

sary for overcoming the effect of cohesion and is supposed to be between zero and $c/\gamma_w H$. On the other hand, the pore-water pressure corresponding to the critical state for sliding is

$$\left(\frac{u}{\gamma_w H}\right)_s = -\left(\frac{\gamma_t}{\gamma_w} - \frac{h}{H}\right) \frac{\sqrt{1+\mu^2}}{\mu} \times \sin(\theta - \phi) + \frac{c}{\gamma_w H \mu}.$$

The critical pore-water pressures for liquefaction and for sliding for the cases of $c/\gamma_w H=0$ and $c/\gamma_w H=0.5$ ($\gamma_t/\gamma_w - h/H$) are shown in Fig. 13. In the figure, it is recognized that when

Fig. 13. Critical pore-water pressure for liquefaction and for sliding.

- a. — Case without cohesion —
b. — Case with cohesion —

$c=0$, the liquefaction cannot occur, since a sliding will occur by a smaller increase in pore-water pressure than necessitated for liquefaction, but when $c \neq 0$, the liquefaction may occur in the range of a gentle inclination corresponding to a value of ϕ .

The tendency described above is a paradox, since the soil liquefaction in slopes can occur only when a cohesion does exist, while the soil is hardly liquefied when the cohesion is too high. The flow slide, therefore, might be considered to be either a sliding of slopes of cohesionless sand under a pore-water pressure nearly equal to that for liquefaction, which results in a perfect liquefaction after the initiation of the sliding, or a liquefaction of a slightly cohesive soils at the original position in slopes. In every case, the permissible range of inclination is limited only within a certain small value. It can be confirmed also by the fact that flow slides of natural slopes occurred mainly at slopes of gentle inclination in the Tokachioki earthquake, 1968 (SAITO *et al.*, 1968).

§ 6. Conclusions

The hypocentral distance of deformation of ground exceeding 50 mm either in lateral or in the vertical direction is approximated by

$$\log_{10} r = 0.51 M - 1.86 \text{ (km)},$$

where M is the earthquake magnitude.

Ground cracks in earthquakes are classified into those of tensile type, normal-fault type, and strike-slip type. From the stress state corresponding to each type of cracking, those of tensile type and of normal-fault type are considered to be more probable than those of strike-slip type. Cracks of tensile type can be caused also dynamically in strong earthquakes.

With liquefaction of soil, an ejection of sand water, a subsidence or upheaval of objects on ground, and flow slides of slopes may occur. The liquefaction of soil is realized when the excess pore-water pressure attains the initial overburden pressure and the rate of increase in the pressure is determined by the rate of dynamic densification of soil which leads to a decrease in voids in the soil mass and that of dissipation of pore water. The process of

liquefaction is, thus, concerned with an irreversible deformation of soil suggesting it to be a historical process. Soils which may be liquefied in nature seem to be the ones whose content of fine particles of diameter smaller than 0.074 mm is less than 20%.

Failures of slopes in earthquakes are classified into solid-mass sliding and flow slide of soil slopes and fragment failure and rock-mass sliding of rock slopes.

On the basis of analysis on a plane-surface sliding model and observation of failures of slopes in the Tokachioki earthquake, 1968, it may be concluded that the dynamic effect of seismic accelerations is important as a factor influencing the stability of slopes in solid-mass sliding, while in case of flow slides the effect of pore-water pressure is more important than the direct effect of the acceleration. The flow slide of slopes may be either a sliding of cohesionless soil with pore-water pressure nearly equal to that for liquefaction which results in a perfect liquefaction after the initiation of sliding, or a liquefaction of a slightly cohesive soil at the original position in the slopes. In every case the permissible range of inclination for flow slides is restricted within not a large value corresponding to the angle of internal friction of soil composing the slopes.

§ 7. Acknowledgments

The author is much indebted to members of the Committee for the Study of the Earthquake Damage to Railway Embankments of the Japanese National Railways in the Tokachioki earthquake, 1968 for many stimulating discussions on effects of the earthquake. He is also grateful to Professors I. Ozawa and S. Yoshikawa of Kyoto University for their suggestions and encouragement during the course of the study.

References

- Committee of the Japanese Society of Soil Mechanics and Foundation Engineering, A brief report on the earthquake damage to ground in the Tokachioki earthquake, 1968 (in Japanese), *Soil Mechanics and Foundation Engineering*, **16**(9), 5-17, 1968.
- Dambara, T., Vertical movements of the earth's crust in relation to the Matsushiro earthquake (in Japanese), *Journal of the Geodetic Society of Japan*, **12**(1), 18-45, 1966.
- Hatano, T., Theory of failure of concrete and similar brittle solid on the basis of strain (in Japanese), *Transactions of the Japan Society of Civil Engineers*, **153**, 31-39, 1968.
- Japanese National Railways, Damage to railways in the earthquake in 1923 (in Japanese), 1923.
- Kawasumi, H., General Description; Opening and closing of fissures; *Report of the special committee for the study of the Fukui earthquake*, 1950.
- Kobayashi, Y., Mechanism of earthquake damage to embankments and slopes, *Proceedings of the fourth world conference on earthquake engineering*, **3**(A5), 74-87, 1969.
- Kubotera, A., and Y. Kobayashi, Relation between damaged area of the earthquakes and magnitude (in Japanese), *Journal of the Seismological Society of Japan*, **23**, 82-84, 1970.
- Maslov, N. N. Questions of seismic stability of submerged sandy foundations and structures, *Proceeding of the fourth international conference of soil mechanics and foundation engineering*, **1**, 368-372, 1957.
- McCulloch, D. S., and M. G. Bonilla, Effects of the earthquake of March 27, 1964, on the Alaska railroad, *Geological Survey Professional Paper*, 345 D, 1970.
- Mogami, T., K. Kubo, M. Sawaguchi, A. Nakase, Soils-Geology-Ground movements (in Japanese), Report on the damage in the Niigata earthquake in 1964, *Japan Society of Civil Engineers*, 1966.
- Nagumo, S., Field studies of the Oga-Oki earthquake of May 7, 1964 (in Japanese), *Bull. Earthq. Res. Inst.*, **42**, 597-608, 1964.
- Retamal, E. and E. Kausel, Vibratory compaction of the soil and tectonic subsidence during the 1960 earthquake in Valdivia, Chile, *Proceedings of the fourth world conference on earthquake engineering*, **3**(A5), 17-28, 1969.
- Saito, M., S. Mishima, H. Kimura, H. Tsuchiya, and H. Shimizu, Features of the damage due to the Tokachioki earthquake, 1968 from the geotechnical point of view (in Japanese), *Soil Mechanics and Foundation Engineering*, **16**(9), 19-29, 1968.
- Scheidegger, A. E., Tektonische Spannungen und deren Einfluss auf geologische Verschiebungen, *Rock Mechanics and Engineering Geology*, **2**(2), 100-105, 1964.
- Seed, H. B. and R. E. Goodmam, Earthquake stability of slopes of cohesionless soils, *Journal of the Soil Mechanics and Foundation Division, Proc. ASCE*, **90**(SM 6) 43-73, 1964.

- Seed, H.B., Landslides during earthquakes due to soil liquefaction, *Journal of the Soil Mechanics and Foundation Division, Proc. ASCE*, **94** (SM5), 1054-1122, 1968.
- Terzaghi, K. and R.B. Peck, Soil mechanics in engineering practice, *Wiley Tuttle*, 1948.
- Tschebotarioff, G.P., Soil mechanics, foundations, and earth structures, *McGraw-Hill*, 1951.
- Tsuchida, H., An experimental study of liquefaction of sandy soils (in Japanese), *Proceedings of the 8th earthquake engineering symposium, JSCE*, 1965.
- Yamada, G., T. Takayama, T. Muromachi, T. Fujiwara, Y. Sato, and Y. Kobayashi, Report on the Tokachioki earthquake on May 16, 1968, (in Japanese), *Railway Technical Research Report*, 650 1968.

(Received May 20, 1971)