学位申請論文

小林芳正

# Effects of Earthquakes on Ground (I)

## - Ground Cracking, Soil Liquefaction, and Sliding of Slopes -

主論文

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 would be included in other types of damage if its mechanism should be known. Moreover, it will always be accompanied at least partly by any other type of damage.

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

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

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 viewpoit 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 Tokachioki 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 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 relating to slidings 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 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 neighbourhood 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 $|\varepsilon|$ 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 = 100~500 m/sec in the epicentral region(Kobayashi, 1969), one obtains the strain $|\varepsilon| = 1 \sim 5 \times 10^{-3}$, 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 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 Nanae-hama 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 McCulloch 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 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(Tschebotarioff, 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 theoty 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 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 $D \leqslant 0.074$ 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
    - slope failure
    - base failure
  - flow slide
- 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 devided 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 saturated 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.

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 $\theta$ and $\gamma$ 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 approaches 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

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

one obtains the critical horizontal seismic intensity

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

and the minimum critical seismic intensity occurs at the depth

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

where $\gamma_t$, $\gamma'$, $\mu$, and $N_\phi$ 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 Goodman(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 indeterminate 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 provisorily into slope-surface flow slide (Type I) and ordinary slope failure(Type II) as shown in Table 2. In Fig.12 the values of cohesion and 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 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

$$\left(\frac{u}{\gamma_w H}\right)_{\ell} = \left(\frac{\gamma_t}{\gamma_w} - \frac{h}{H}\right) \cos\theta + \Delta$$

where $\Delta$ is a certain value of pressure necessary 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

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

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 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 the value of $\phi$.

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 may 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

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

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 densificatio 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 retricted 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 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 forth 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, Proceedings 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 Proffessional 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(A 5), 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), 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(in Japanese), Report on the Tokachioki earthquake on May 16, 1968, Railway Technical Research Report, 650, 1968.

Table 1

Types of Ground Cracking, Stress States, and Probability of Their Occurrence

| Case | Stress States | With or Without Tension* | Tensile Strength | Type of Cracks | Probability Static | Probability Dynamic | Notation |
|---|---|---|---|---|---|---|---|
| (1) | $\sigma_x > \sigma_y > \sigma_z$ | | | Thrust | none | none | (1) |
| (2) | $\sigma_x > \sigma_z > \sigma_y$ | $\sigma_y \geqslant 0$ | | 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) | $\sigma_z > \sigma_x > \sigma_y$ | $\sigma_y \geqslant 0$ | | 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

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 |

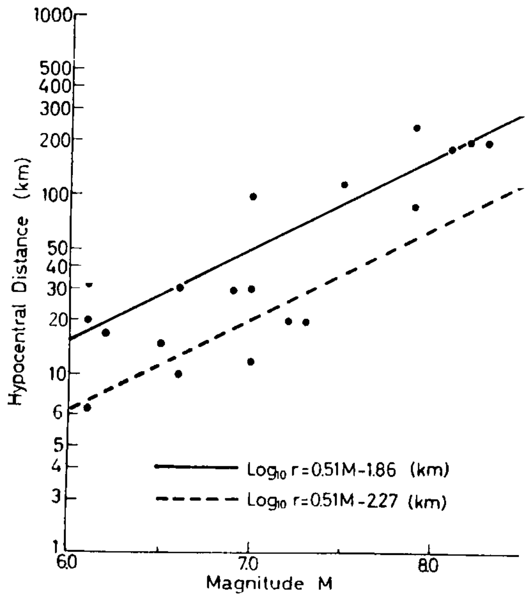



Fig.1 Farthest hypocentral distance of railroad subgrade deformed more than 50 mm either in the lateral or the vertical direction and the earthquake magnitude.

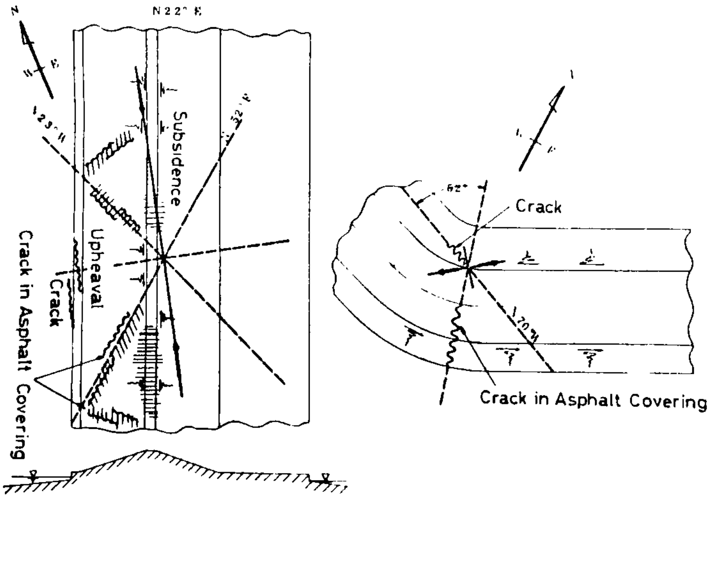



Fig.4 Cracks of strike-slip type on the bank of Hachiro-gata lagoon in the Oga-Oki earthquake, May 7, 1964.

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

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

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

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

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

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

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

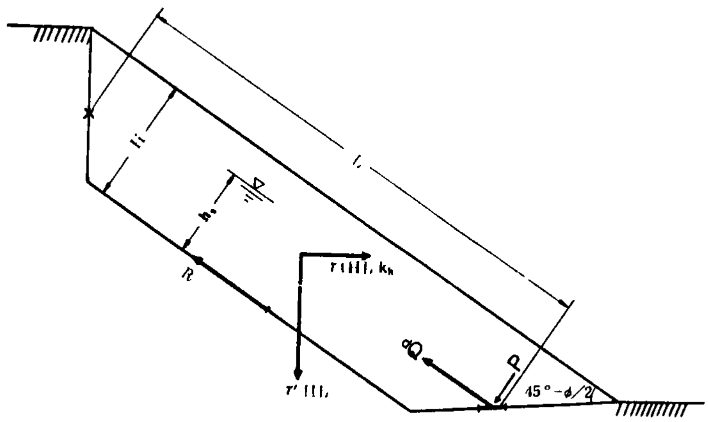



Fig.10 Plane-surface sliding model with slope-end resistance dQ under horizontal seismic intensity $k_h$.

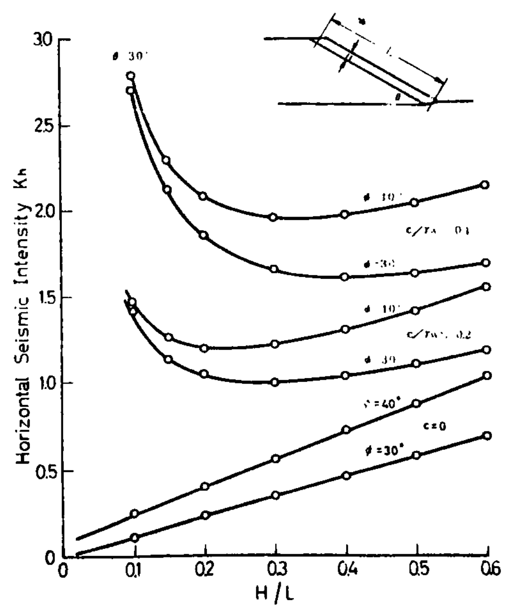



Fig.11 Critical horizontal seismic intensity as a function of the ratio between the height H and the length L of a sliding mass.

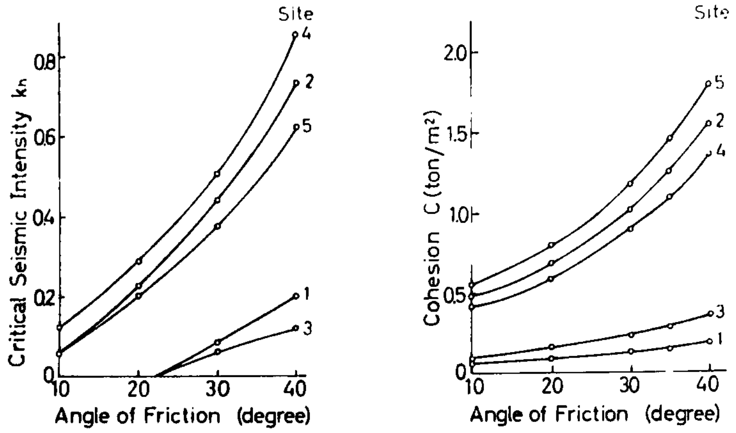
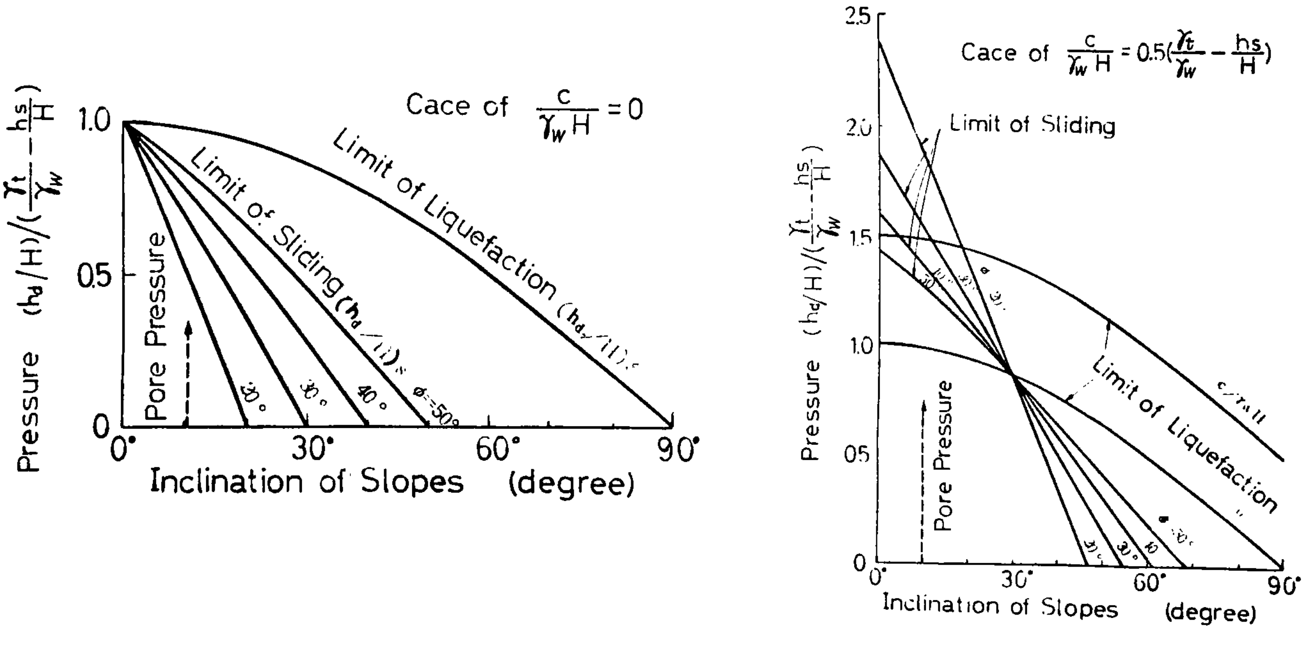



Fig.12 Cohesion and friction of soil, and critical horizontal seismic intensity, satisfying $H = \sqrt{2cL/\gamma N_\phi}$, on the basis of observed H and L of slided masses in the Tokachioki earthquake, May 16, 1968.




a. Case without cohesion

b. Case with cohesion

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

# Effects of Earthquakes on Ground (II)
## -Ground Motions and Pore-Water Pressure in Soils -

By

Yoshimasa KOBAYASHI

Railway Technical Research Institute

Abstract

The displacement and stress induced in surface layers by earthquake ground motions are correlated with the response spectrum of strong-motion records obtained on the ground surface. An empirical formula for the duration of ground motion in an earthquake during which the acceleration may exceed an arbitrary threshold value at stations near the epicenter is given. On the basis of experimental data of soil liquefaction a procedure for estimating the pore-water pressure in soils, when subjected to an arbitrary stress history, is proposed and applied to an observed value at a railway embankment in the Tokachioki earthquake, 1968 with due consideration for the spectral character and the duration of ground motion.

## §1. Introduction

The amount of sliding of soil blocks during an earthquake sometimes cannot be explained without consideration for the decrease in strength of soils during the process of failure(Kobayashi,1969). The flow slide of slopes can take place without any strong seismic acceleration only if the condition for the failure is prepared by the increase in pore-water pressure(Kobayashi, 1971). The pore-water pressure in soils builds up gradually when the soils are subjected to earthquake ground motions and it may be considered as one of the most important factors influencing the strength of soil, since it may deprive of the strength entirely by causing liquefaction.

In order to clarify the effect of dynamic loading on the development of pore-water pressure, a series of experimental studies has been carried out in recent years, but little of them deals with the relation between the experimental results and field observations except a study by Seed(1966). In the study Seed analysed the ground motion supposed in Niigata, where a severe liquefaction of loose alluvial sands occurred in the earthquake in 1964, to assess the liquefaction behavior and found a good agreement between the analytical results and the field performance. However, also in the study the procedure for assessment of the effect of irregular ground motions is ambiguous and it seems to be necessary to investigate the irregular character of the ground motions and to develop an appropriate procedure for estimating the pore-water pressure due to arbitrary stress histories.

In this paper, frequency characteristics of the displacement and stress induced in surface layers by earthquake ground motions are discussed first with the aid of the response spectrum of strong motion records. Second, the duration of the strong part of earthquake ground motion is investigated on several strong motion accelerograms and an empirical formula for the duration as a function of the earthquake magnitude is derived. On the basis of experimental data of soil liquefaction, a procedure for estimating the pore-water pressure due to earthquake ground motion is proposed and applied to an observed value in the Tokachioki earthquake, 1968.

## §2. Frequency Characteristics of Earthquake Ground Motions

The earthquake ground motion is highly irregular and transient and it is often convenient to utilize special procedures for understanding of their frequency charcteristics. The response spectrum developed by Housner et al.(1953) is one of such procedures and is composed of the maximum relative responses of one degree of freedom systems of various periods and dampings subjected to a time history of acceleration to be analysed. Adapting to various purposes, there are spectra of displacement, velocity, and acceleration, D, V, and A, which correspond to the maximum displacement, velocity, and acceleration of a time history of response to an input acceleration, respectively, but it is recognized that there are in most cases the following correlations between them:

$$V = 2\pi D/T \qquad \text{and} \qquad A = 2\pi V/T$$

where T is the period of the system concerned. In this case a tripartite logarithmic plot is especially convenient for representation of the frequency characteristics of the ground motion, since it represents with a single curve the spectra of displacement, velocity, and acceleration at the same time with reference to either coordinate axis. Examples of the displacement-response spectrum are shown in Fig.1.

According to the theory of seismograph, the acceleration value of the response spectrum in the range of short periods may be considered to give the maximum ground acceleration(principle of accelerograph):

$$A = \alpha_{max}$$

On the other hand, according to studies by Muramatu(1962, 1966), the envelope of the maximum displacements of ground of various periods in a tripartite logarithmic plot may be expressed schematically by three straight lines corresponding to a constant acceleration in $T \leq T_m$, a constant velocity in $T_m \leq T \leq T_M$, and a constant displacement in $T \geq T_M$ as shown in Fig.2, where

$$T_m(\mathrm{sec}) = 0.005\ r(\mathrm{km})$$

and $$T_M(\mathrm{sec}) = 0.2\ M + 0.27 \log_{10} r(\mathrm{km}) - 1.57$$

r and M being the hypocentral distance and the earthquake magnitude, respectively.

If one employs the maximum ground acceleration and $T_m$ after Muramatu, the spectrum of the ground displacement may be approximated in the range of $T \leq T_M$ by solid straight lines drawn in Fig.1 which are in many cases almost coincident with the response spectrum for the case of damping coefficient h = 0.4 except that the period $T_M$ after Muramatu giving the maximum velocity and displacement is not found in the response spectrum. The shape of the response spectrum in $T \leq T_M$ is recognized to be preserved in every section of the whole duration of the predominant motions of an earthquake(Kobayashi, 1971).Accordingly, the spectrum of the ground displacement at any moment may be expressed, as far as the coincidence of it with the response spectrum may be assumed, in terms of the ground acceleration and the period $T_m$ as

$$D(T) = (T/2\pi)^2 \alpha \qquad \text{for} \quad T \leq T_m$$
$$(T_m/4\pi^2) T \alpha \qquad \text{for} \quad T_m \leq T \leq T_M$$

It is known that the vertical component of ground motion is much smaller than the horizontal one as shown in Fig.3 during the predominant part of earthquakes and that the motion on the ground surface can approximately be correlated with SH waves incident and reflected vertically at the ground surface. In this case the equation of motion becomes particularly simple and is given by

$$\rho \frac{\partial^2 u}{\partial t^2} = \mu \frac{\partial^2 u}{\partial x^2}$$

where $\rho$, $\mu$, and $u$ are the specific mass and rigidity of soil and the horizontal displacement, respectively, and the x axis is taken vertically downward with its origin on the ground surface. The solution of the equation of motion satisfying the boundary condition $\tau = 0$ at the ground surface is

$$u(x,t) = \Lambda(T) \exp(i \frac{2\pi}{T} t) \cos(\frac{2\pi}{\lambda} x)$$

The corresponding stress is given by

$$\tau(x,t) = -\frac{2\pi\Lambda(T)}{\lambda} \exp(i \frac{2\pi}{T} t) \sin(\frac{2\pi}{\lambda} x)$$

where $\lambda = \sqrt{\mu/\rho}\, T = V_s T$.

The envelope of amplitudes $\Lambda(T)$ for various T may be approximated by two straight lines corresponding to a constant acceleration in $T \leq T_m$ and a constant velocity in $T_m \leq T \leq T_M$, which nearly coincide with the response spectrum D(T) for h = 0.4, as described before. Accordingly, the envelopes of amplitudes of $u$ and $\tau$, which will be denoted by U and S respectively, may be given by

$$U(x,T) = D(T) \cos(\frac{2\pi x}{V_s T})$$

and

$$S(x,T) = \frac{2\pi\rho V_s D(T)}{T} \sin(\frac{2\pi x}{V_s T})$$

If one introduces the ground acceleration $\alpha$ and the period $T_m$, these spectra can be written also in the form

$$U(x,T) = (\frac{T}{2\pi})^2 \alpha \cos(\frac{2\pi x}{V_s T}) \qquad \text{for } T \leq T_m$$

$$(\frac{T_m}{4\pi^2}) T \alpha \cos(\frac{2\pi x}{V_s T}) \qquad \text{for } T_m \leq T \leq T_M$$

and

$$S(x,T) = \frac{\rho V_s T}{2\pi} \alpha \sin(\frac{2\pi x}{V_s T}) \qquad \text{for } T \leq T_m$$

$$\frac{\rho V_s T_m}{2\pi} \alpha \sin(\frac{2\pi x}{V_s T}) \qquad \text{for } T_m \leq T \leq T_M$$

In natural ground, a layer which is soft as compared with underlying layers in most cases exists at the ground surface and the spectrum described above may be considered to be in effect an envelope of several peaks developing in the layer. The periods of waves predominating in the layer of height H and S-wave velocity $V_s$ are determined by

$$T_n = 4H/(2n - 1)V_s \qquad (n = 1,2,3, - - - - - )$$

The corresponding displacement and stress are obtained by substituting $T_n$ in T in the relations derived above and their modes are shown in Fig.4.The predominant periods $T_n$ versus $H/V_s$ and the amplitudes of U and S are shown in Fig.5. The amplitudes decrease rapidly with increasing order of normal mode n. It may be concluded, accordingly, that only some of lower order of waves will be significant from the viewpoint of the earthquake damage in grounds of $H/V_s \leqq 0.25$ sec as ordinarily is the case.

## §3. Duration of Strong Phases of Earthquake Ground Motions

The duration of strong shaking in an earthquake has not been studied much until recently except by Gutenberg and Richter(1942) having given a crude empirical formula

$$\log_{10} t_o = 0.25\ M - 0.7 \ (\text{sec})$$

where $t_o$ is the duration and M is the earthquake magnitude.

Accumulation of strong-motion records in recent years has made more thorough study of the problem possible and Housner(1965) gave the upper limit of the duration of strong shaking as

$$t_o = 11\ M - 52 \qquad (\text{sec})$$

In a similar study Ambraseys and Sarma(1967) defined the duration more quantitatively as the duration equal to or greater than 0.03 g at the epicentral region and presented the relation

$$t_o = 11.5\ M - 53.0 \qquad (\text{sec})$$

It is noted that the empirical formula is given in an algebraic form by Housner as well as by Ambraseys and Sarma, while it is in a logarithmic form by Gutenberg and Richter. As for the poit, a study of the duration of earthquakes from the beginning to the end by Tsumura(1967) suggests the logarithmic form, since it is based on a great number of records of micro earthquakes of magnitudes less than 6. Accordingly, it will be assumed hearafter that the duration is expressed in a logarithmic form.

It should further be noted that the area in which the above formulas are applicable is not clear and even in the study by Ambraseys and Sarma in which the range of application is specified to be the epicentral region, the hypocentral distances of stations of which the records are employed do not always seem to be small enough. The range of application should be restricted within a limit, since it is obvious that the duration is influenced by the epicentral distance strongly. On the other hand, however, the number of strong motion records is so far not satisfactorily large and it is unavoidable to utilize records at more or less large hypocentral distances. Thus, the author employed only the data at stations within the hypocentral distances less than five times of the reduced radii of the crustal deformation of the corresponding earthquakes after Dambara(1966) ($\log_{10}r = 0.51\ M - 2.27$ (km)) from that given by Ambraseys and Sarma and that from several SMAC records(Strong-Motion Earthquake Observation Committee, 1969) (Fig.6). The limit was introduced as a choice of a distance which makes relatively much existing data applicable between which dependence of the duration on the distance is not very significant. The duration must also be a function of ground conditions, but the tendency cannot be clarified so far within the limit because of insufficient number of available records.

From Fig.6 the duration of ground motion exceeding 30 gals within the above defined hypocentral distance is expressed very approximately by

$$\log_{10}t_{30} = 0.50\ M - 2.08\ \text{(sec)} \qquad (\text{for } \log_{10}r \leqslant 0.51\ M - 1.57,\ \text{(km)})$$

which applies only to the horizontal components of acceleration. In the figure the relations after Gutenberg and Richter, Housner, as well as Ambraseys and Sarma with rough estimations of the duration of the San Francisco earthquake, 1906 and Alaska earthquake, 1964 by Seed(1968) are also shown. The formula derived in the above seems to give a closer approximation than that by Housner or Ambraseys and Sarma in higher values of the magnitude.

The fluctuation of acceleration during an earthquake may be visualized by plotting the maximum accelerations in sections of an accelerogram of an equal length of time as shown in Fig.7. The duration exceeding an arbitrary value of acceleration can readily be read from Fig.7 and is shown in Fig.8.

On the basis of the result above, the empirical formula of the duration exceeding 30 gals can be extended to that for an arbitrary threshold value $\alpha$ gals given in the form

$$\log_{10}t_{\alpha} = -0.0088(\alpha - 30) + \log_{10}t_{30} = -0.0088\alpha + 0.50\ M - 1.82\ \text{(sec)}$$

which is applicable within the hypocentral distance less than five times of the radius of the crustal deformation and under an average ground condition where strong-motion accelerographs are installed.

## §4. Empirical Formulas for the Pore-Water Pressure due to Cyclic Loading

The mechanism of liquefaction of soils due to cyclic loading is very complex and it is for the moment difficult to establish quantitative relations purely theoretically. Therefore, it will be practical to introduce empirical fomulas on the basis of experimental data.

Seed and his cooperators have conducted a series of cyclic loading tests of saturated sands using either the dynamic triaxial or simple-shear appratus and clarified the effect of various factors on liquefaction of soils(e.g.Seed, 1968). Concerning the effect of the confining pressure $\sigma_0$ and the initial relative density of a sand $D_r$ , they obtained for Sacramento River sand an empirical formula for the cyclic stress $\tau_{\ell 10}$ required to cause liquefaction in 10 cycles as

$$\tau_{\ell 10} = \frac{\sigma_0 D_r}{200}$$

where $$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100 \ (\%)$$

As for the effect of the magnitude of cyclic stress on its required number to cause liquefaction, they found a tendency shown in Fig.9 which may be practically be expressed as

$$\tau_{\ell N} = \sigma_0 (a \log_{10} N + b) \qquad 5 \leqslant N \leqslant 100$$

where a and b are constants. According to Shibata(1970), the value of a seems to be from about -0.125 to -0.150. The author and Imamura(1970) conducted a provisory test of simple-shear type on soils taken from damaged sites in the Tokachioki earthquake, 1968 and on the assumption of the logarithmic relation of the number of cycles to cause liquefaction and the relation $\tau_{\ell 10} = \sigma_0 D_r / m$, found out that m varies from 110 to 270 according as the type of soils(Kobayashi, 1971).

In order to give a formula for pore-water pressure until liquefaction, the following two assumptions are introduced; (1) the liquefaction is a state where the mean pore-water pressure u becomes equal to the initial effective stress i.e. the confining pressure $\sigma_0$ and (2) other things being equivalent, the increase in pore-water pressure is equal in every cycle. In this case, the increment of pore-water pressure $\Delta u_N$ by a pulse of stress $\tau_{\ell N}$, the the cyclic shear stress required to cause liquefaction in N cycles, is given by

$$\Delta u_N = \sigma_0/N$$

The number N is determined by

$$\tau_{\ell N} = \sigma_0\,(a\,\log_{10} N + b) \qquad \text{or} \qquad \log_{10} N = (\,\tau_{\ell N}/\sigma_0 - b)/a$$

and the value of b by

$$\tau_{\ell 10} = \sigma_0(a + b) = \sigma_0 D_r/m$$

where a is chosen appropriately. Since the values of $\tau_{\ell N}$ is arbitrary, the increment of pressure $\Delta u_\tau$ by a pulse of stress $\tau$ is easily obtained.

It should be noted, however, the range of applicability of the above relations is limited within that of the logarithmic approximation on the effect of the magnitude of cyclic stress, say $N \leqq 100$. Accordingly, the stress smaller than $\tau_{\ell 100} = \sigma_0(2a + b) = \sigma_0(a + D_r/m)$ should be ignored in assessing the effect of cyclic loading on the increase in the pore-water pressure.

## §5. Interpretation of an Observed Pore-Water Pressure in the Tokachioki Earthquake, 1968

An increase in pore-water pressure was recorded by chance during the Tokachioki earthquake, 1968 by Uezawa(1970) who was observing the behavior of the pore-water pressure due to rainfalls in a railway embankment at 195 km from the epicenter. The pressure showed in the record an abrubt increase followed by a gradual decrease at the moment of the main shock(JMA scale V) and also at that of the largest aftershock(JMA scale IV) on the same day(Fig.10).

The amount of the increase in pressure during the main shock was about 2.5 ton/m$^2$ at the pickup No.4 which had been installed at about 9 m below the ground surface and gave a fine record. The earthquake was preceded by a heavy rainfall amounting to 150 mm until the evening of the preceding day and the embankment may be assumed to have been nearly perfectly saturated up to the top. Accordingly, on the assumption of the saturated density of 1.6, the effective stress due to the weight of soil at the pickup No.4 is estimated to have been

$$\sigma_0 = (\gamma_t - \gamma_w)h = 5.4 \text{ ton/m}^2$$

The initial relative density at the depth is assumed to have been 80 %, considering the results of survey in similar sites. The value corresponds to about 50 m/sec of S-wave velocity in experimental data(Kobayashi and Imamura, 1970).

The depth of the pickup No.4 was at the boundary of the embankment and the underlying firm base and the stress spectrum given in §2 is applicable. In this case, $H/V_s = 9.0/50 = 0.18$ sec and the waves of the periods, $T_1 = 0.72$ sec, $T_2 = 0.24$ sec, $T_3 = 0.15$ sec, etc. induce large stresses at the boundary. If one assumes the input acceleration to have been equal to the NS component with the maximum 230 gals at Hachinohe harbor, which is located at about 180 km from the epicenter, and refers to Fig.5, the largest displacements and stresses coresponding to each mode are given by

$$U(T_1) = 3.03 \text{ cm}, \qquad S(T_1) = 2.15 \text{ ton/m}^2$$

$$U(T_2) = 0.35 \text{ cm}, \qquad S(T_2) = 0.72 \text{ ton/m}^2$$

and $$U(T_3) = 0.12 \text{ cm}, \qquad S(T_3) = 0.42 \text{ ton/m}^2$$

The lower limit of stress which may influence the development of pore-water pressure is assumed to be

$$\tau_{cr} = \tau_{\ell 100} = \sigma_0 (D_r/m + a)$$

Substituting $D_r = 80$ % and $a = -0.150$, one obtains the values $\tau_{cr}$ as a function of m for example:

$$\tau_{cr} = 3.12 \text{ ton/m}^2 \qquad \text{for } m = 110 \text{(lower limit)}$$

$$\tau_{cr} = 2.15 \text{ ton/m}^2 \qquad \text{for } m = 150$$

and $$\tau_{cr} = 0.79 \text{ ton/m}^2 \qquad \text{for } m = 270 \text{(upper limit)}$$

Since the maximum stress expected at the depth, which is realized by the first mode of waves, is 2.15 ton/m$^2$, the cases of m smaller than 150 is unacceptable for the interpretation of the actually observed development of pore-water pressure. On the other hand, the stresses induced by the second or higher modes are smaller than 0.72 ton/m$^2$ and may be ignored in estimating the increase in the pore-water pressure, since the values are smaller than any $\tau_{cr}$ corresponding to m from 110 to 270; The number of stress pulses which is effective on the increase in pore-water pressure is determined only by that of the first mode of waves and can be derived by dividing the duration of the ground motion exceeding the critical acceleration,which corresponds to $\tau_{lr}$ , given in §2 by the period of the first mode, $T_1$ = 0.72 sec.

The amplitudes of waves distributing between the critical acceleration and the maximum can easily be determined from Fig 8 and the increment of the pore-water pressure due to every stress pulse is estimated by

$$\log_{10}\left(\frac{\Delta u}{\sigma_0}\right) = -\left(1 + \frac{\tau - \tau_{\ell 10}}{\sigma_0\, a}\right)$$

The sum of the pressure induced by every pulse corresponding to various m is shown in Fig.11. The observed pore-water pressure 2.5 ton/$m^2$ is concordant with the value of m = 230 in the Figure.

## § 6. Conclusions

The envelope of the maximum amplitudes of displacement at the ground surface is nearly coincident with the response spectrum for the damping coefficient h = 0.4.

The stress spectrum S(x,T) at a depth x in ground can be denoted in terms of the displacement spectrum at the ground surface D(T) as follows:

$$S(x,T) = \frac{2\pi\rho V_s}{T}\sin\left(\frac{2\pi}{V_s T}x\right)D(T)$$

where $\rho$, $V_s$, and T are the specific mass of soil, S-wave velocity, and the period of waves, respectively.

The duration of the ground motion exceeding $\alpha$ gals within the hypocentral distance five times as large as the reduced radius of the crustal deformation of an earthquake after Dambara(1966) is given by

$$\log_{10} t_\alpha = -0.0088\alpha + 0.50\ M - 1.82\ (\text{sec})\quad (\log_{10} r \leq 0.51\ M - 1.57\ (\text{km}))$$

The cyclic shear stress required to cause liquefaction in a saturated sand in 10 cycles may be expressed by

$$\tau_{\ell 10} = D_r \sigma_0 / m$$

where $\sigma_0$, $D_r$, and m are the overburden pressure, the initial relative density $(e_{max} - e)/(e_{max} - e_{min})$ x 100, and a constant varying from 110 to 270, respectively.

The relation between the magnitude of the cyclic stress and its required number to cause liquefaction is denoted approximately by

$$\tau_{\ell N} = \sigma_0 (a \log_{10} N + b)$$

where $\underline{a}$ is a constant between -0.125 and -0.150 and $b = D_r/m - a$.

The increase in pore-water pressure by a pulse of the magnitude $\tau$ may be estimated by

$$\log_{10}(\Delta u/\sigma_0) = -(1 + \frac{\tau - \tau_{\ell N}}{\sigma_0 a})$$

An increase in pore-water pressure due to the Tokachioki earthquake, 1968 observed in a railway embankment can be interpreted quantitatively with the empirical formulas given above.

## §7. Acknowledgments

The author is much indebted to members of the Committee for the Study of the Earthquake Damage to Railway Embankments of Japanese National Railways in the Tokachioki earthquake, 1968 for many helpful 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. Some of the strong-motion records used in computations were made available by Mr. H. Tsuchida of the Port and Harbour Technical Research Institute and Mr. T. Fujiwara of the Railway Technical Research Institute. Computations in this study were carried out on FACOM 230-60 in the Railway Technical Research Institute.

References

Ambraseys, N. N. and S. K. Sarma, The response of earth dams to strong earthquakes, Géotechnique, 17, 181-213, 1967.

Dambara, T., Vertical movements of earth's crust in relation to the Matsushiro earthquake(in Japanese), Journal of the Geodetic Society of Japan, 12(1), 18-45, 1966.

Gutenberg, B. and C. F. Richter, Earthquake magnitude, intensity, energy, and acceleration, Bulletin of the Seismological Society of America, 32(3), 163-191, 1942.

Housner, G. W., R. R. Martel, and J. L. Alford, Spectrum analysis of strong-motion earthquakes, Bulletin of the Seismological Society of America, 43(2), 97-119, 1953.

Housner, G., Intensity of earthquake ground shaking near the causative fault, Proceedings of the third world conference on earthquake engineering, 1, III 94-111, 1965.

Kobayashi, Y., Mechanism of earthquake damage to embankments and slopes, Proceedings of the fourth world conference on earthquake engneering, 3(A5), 74-87, 1969.

Kobayashi, Y., and H. Imamura, Test on an increase in porewater pressure due to vibration during earthquakes and application of the results to an in-situ case(in Japanese), Railway Technical Research Report, 728, 1970.

Kobayashi, Y., Phenomena and mechanisms of earthquake damage to ground(in Japanese), Railway Technical Research Report, 744, 1971.

Muramatu, I., Estimation of the earthquake danger with due consideration on the vibrational characteristics of earthquake motion, Science Report of the Faculty of Liberal Arts and Education, Gifu University, 3(1), 91-114, 1962.

Muramatu, I., Expectation of maximum velocity of earthquake motion within 50 years throughout Japan, Science Report of the Faculty of Liberal Arts and Education, Gifu University, 3(5), 470-481, 1966.

Seed, H. B., Soil liquefaction in the Niigata earthquake, Proceedings of Japan Earthquake Engineering Symposium, 97-104, 1966.

Seed, H. B., Landslides during earthquakes due to soil liquefaction, Journal of the Soil Mechanics and Foundations Division, Proc. ASCE, 94(SM 5), 1055-1122, 1968.

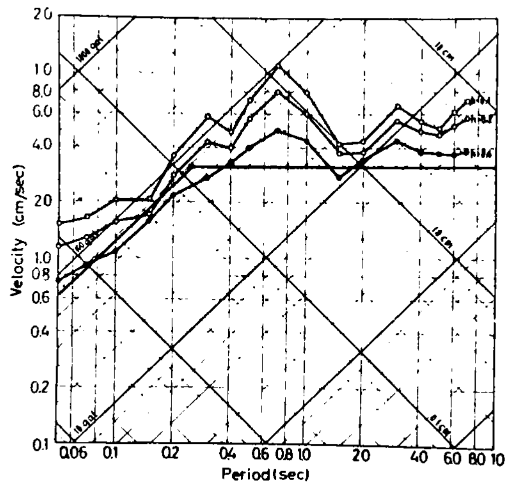
Shibata, T., Analysis of liquefaction of saturated sand during cyclic loading(in Japanese), Disaster Prevention Research Institute Annuals, Kyoto University, 13 B, 1-8, 1970.

Strong Earthquake Response Analysis Committee, Digital values used for analog computation by SERAC, SERAC Report, 6, 1964.

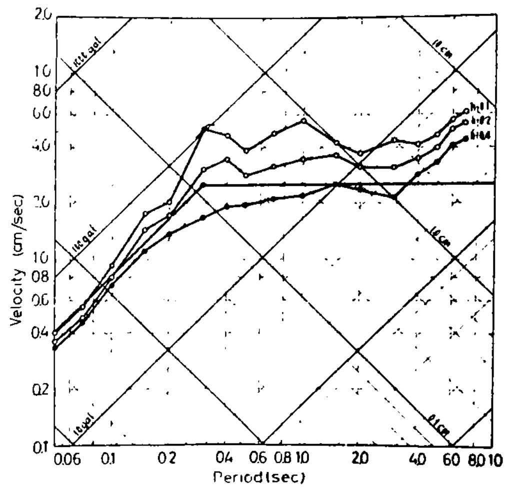
Strong-Motion Earthquake Observation Committee, Strong-motion earthquake records in Japan, National Research Center for Disaster Prevention, 8, 1969.

Tsumura, K., Determination of earthquake magnitude from duration of oscillation (in Japanese), Journal of the Seismological Society of Japan, 20(1), 30-40, 1967.

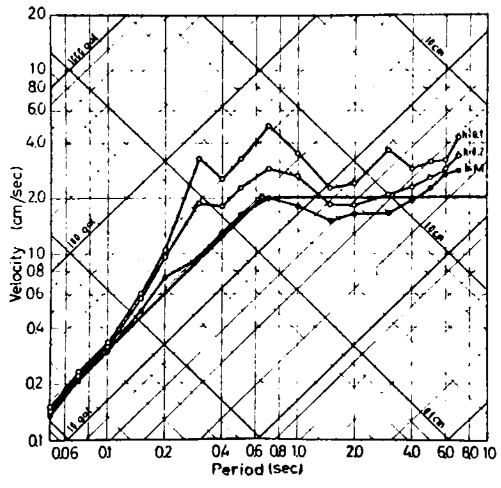
Uezawa, H., S. Menjo, Y. Yasuda, and J. Kumagai, Observation of porewater pressure at a disastered embankment and a cause of disaster(in Japanese), Railway Technical Research Report, 717, 1970.



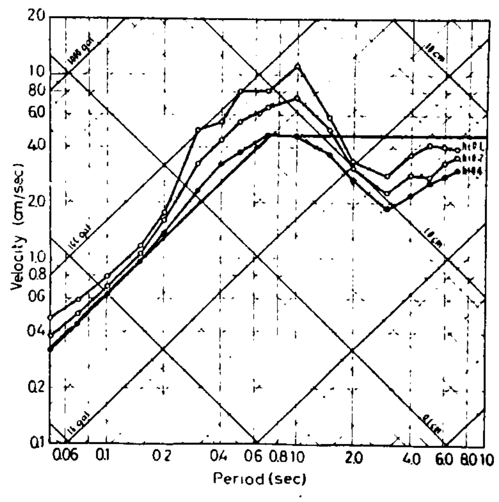

a. Western Chiba earthquake, Feb. 14, 1956, recorded at the Earthquake Research Institute, Univ. Tokyo, M=6.0, $\Delta$=50 km, Max. acc.=74 gals.




b. Northern Miyagi earthquake, Apr. 30, 1962, recorded at the Architect. Dept., Tohoku Univ. Sendai, M=6.5, $\Delta$= 60 km, Max. acc.=50 gals.




c. Echizen Cape earthquake, Mar. 27, 1963, recorded at Nippon Sheet Glass Co. in Osaka, M=6.9, $\Delta$=130 km, Max. acc.=19 gals.




d. Echizen Cape earthquake, Mar. 27, 1963, recorded at Akashi Planetarium, M=6.9, $\Delta$=150 km, Max.acc.=39 gals.

Fig.1 Examples of response spectrum of ground acceleration(SERAC,1964) and the envelope of peaks of ground displacement approximated by two straight lines.

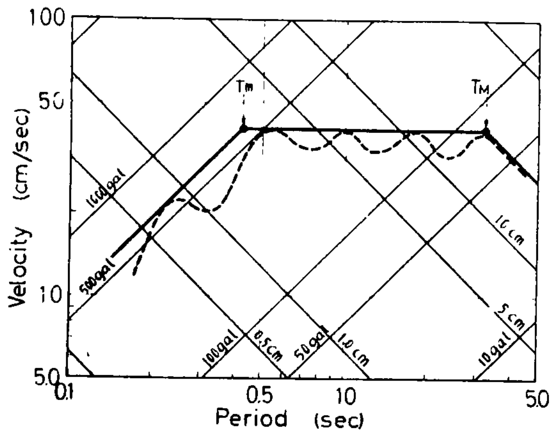



Fig.2 Ground-displacement spectrum as an envelope of displacement peaks of various periods after Muramatu(1962,1966).

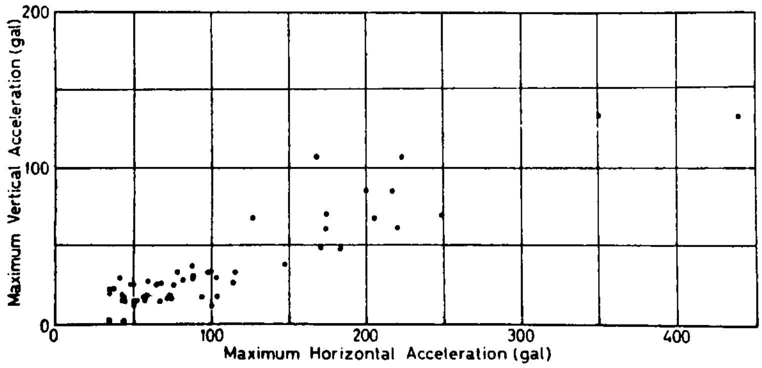



Fig.3 Predominance of horizontal motions in strong earthquakes read from SMAC records(Strong-Motion Earthquake Observation Committee, 1969).

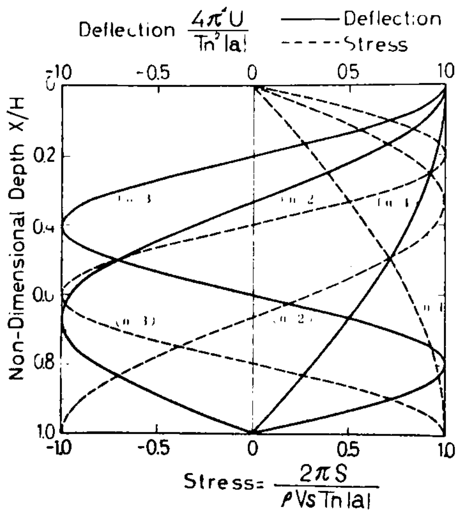



Fig.4 Normal modes of displacement U and stress S for a soft surface layer of height H and S-wave velocity $V_s$ (particular period $T_n = 4H/(2n - 1)V_s$), and their amplitudes for an input acceleration $|a|$ .

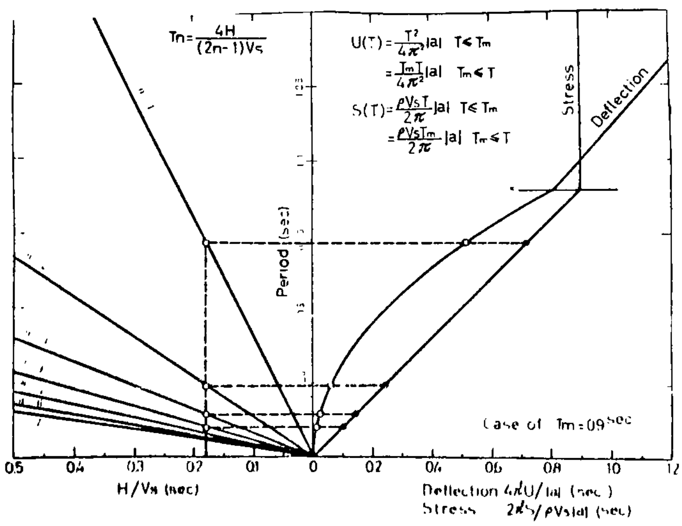



Fig.5 Particular periods of a soft surface layer $T_n$ of height H and S-wave velocity $V_s$, and amplitudes of displacement and stress spectra, U and S.

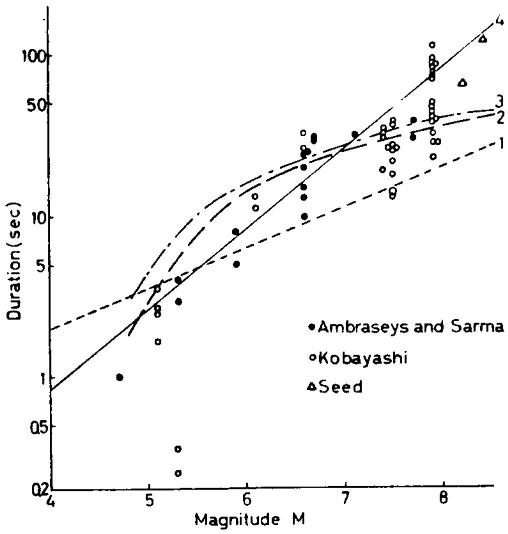



Fig.6 Duration of strong shaking and the earthquake magnitude. 1.Gutenberg and Richter; $\log_{10}t_0$=0.25M-0.7, 2.Housner; $t_0$=11M-52, 3.Ambraseys and Sarma; $t_0$=11.5M-53.0, 4.Kobayashi; $\log_{10}t_{30}$=0.50M-2.08.

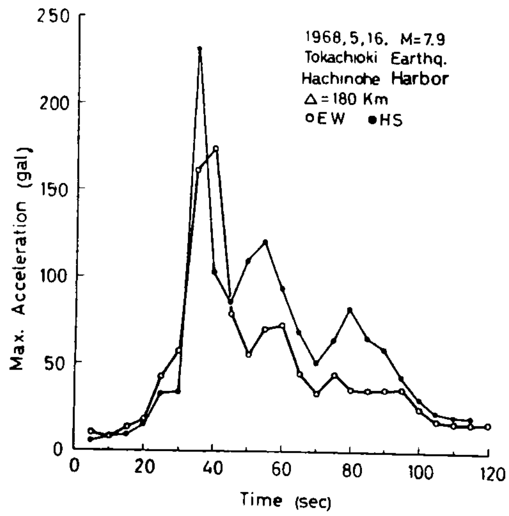



Fig.7 Fluctuation of acceleration during an earthquake; Tokachioki earthquake, May 16, 1968, M=7.9, recorded at Hachinohe harbor, $\Delta$=180 km.

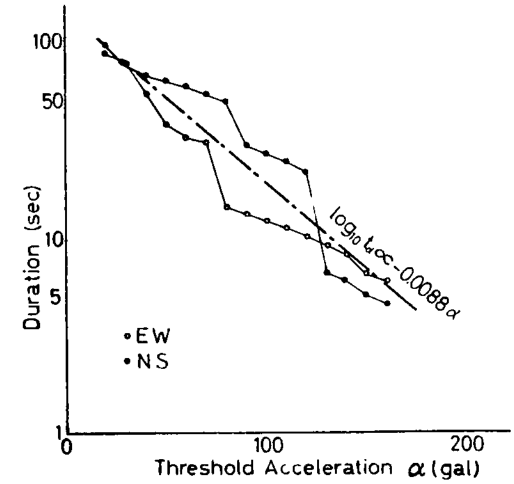



Fig.8 Duration of ground motion exceeding arbitrary threshold values of acceleration; Tokachioki earthquake, May 16, 1968, M = 7.9, recorded at Hachinohe harbor, $\Delta$= 180 km.

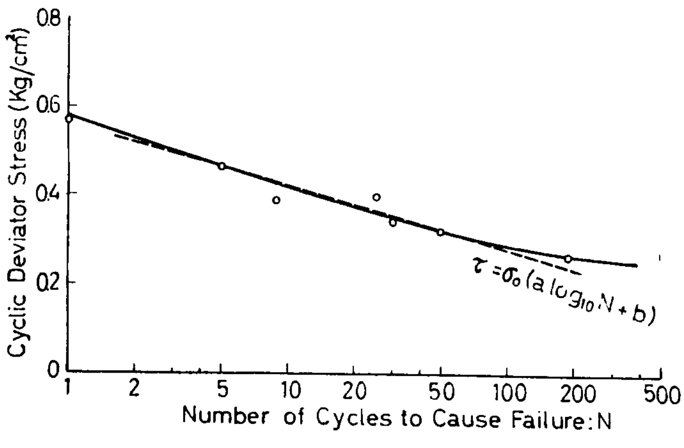



Fig.9 Effect of the magnitude of cyclic stress on its required number to cause liquefaction after Seed(1968) and its approximate formula.

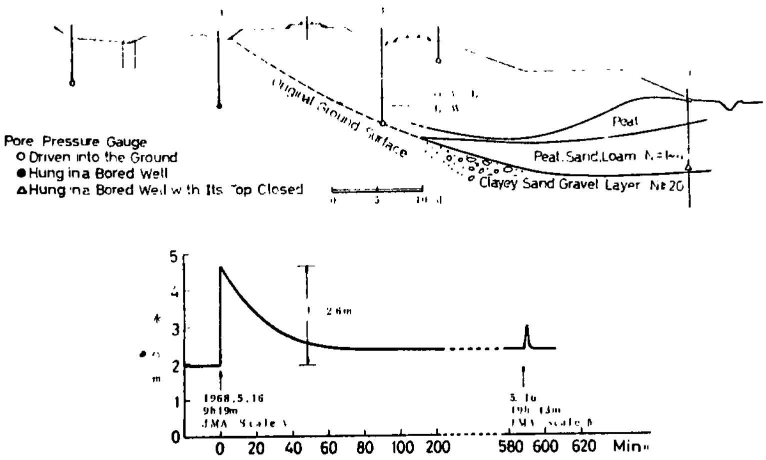



Fig.10 Record of pore-water pressure in a railway embankment, $\Delta$= 195 km in the Tokachioki earthquake, May 16, 1968.

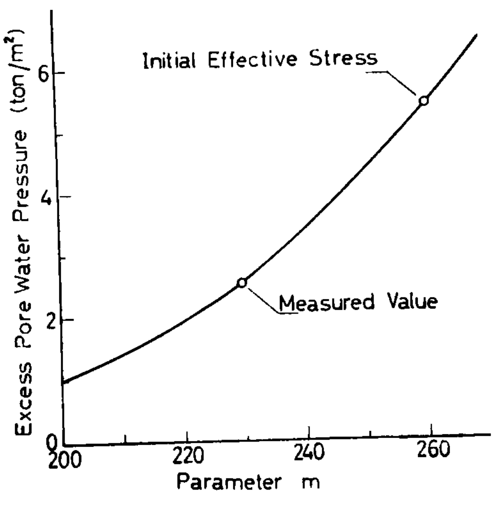



Fig.11 Pore-water pressure estimated an a railway embankment for parameter m.