STUDY ON SETTLEMENT ASSOCIATED WITH LIQUEFACTION OF SAND SUBJECTED TO IMPACT LOADS*

*By Kiichi Tanimoto*** and Masaru Nishi****

1. Introduction

It has been often observed that liquefaction of underlying saturated sand deposits during earthquakes resulted in severe damage to engineering works. Extensive laboratory investigations[1)~10)] on this liquefaction phenomenon have been conducted by many soil engineers for many years, especially since the Niigata earthquake of 1964 which provided much valuable information of this phenomenon. Some important conclusions have been obtained, on the necessary combination of soil conditions and vibratory deformations leading to liquefaction and on the liquefaction process, from these studies. Unfortunately only a few attempts, however, seem to have been made to study this phenomenon in a quantitative way. Ref. 9) may be one of such works of solving case problem in Niigata. The present paper will give the behavior of a saturated sand mass liquefied in a rigid sand bin and an approach to the quantitative analysis for the settlement associated with its liquefaction and the process from occurrence to dissipation of excess pore water pressure.

2. Test Apparatus and Procedure

The main test apparatus[11)] consists of a steel sand bin with interior dimensions of $60\times80\times130$ cm having a glass plate as a side wall, a shaking table with 14 steel plate springs, and impact hammer of 60 kg weight, and several measuring instruments as shown in Fig. 1. The soil specimen used in the experiment is a relatively uniform sand which has $D_{60}=0.78$ mm and $D_{10}=0.29$ mm, or $C_u=2.69$.

After the sand was loosely placed in water to the 110 cm height of the sand bin, the saturated sand mass was subjected to horizontal impact-vibrating motion produced by free falling of the hammer to the shaking table. The original void ratio, e_0, of

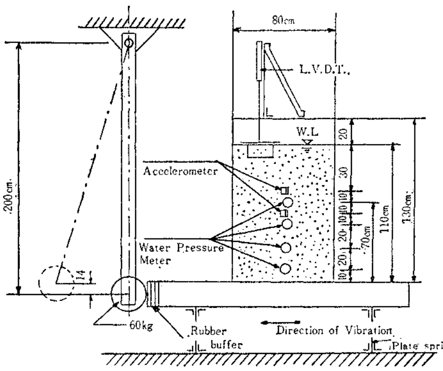

Fig. 1 Test apparatus.

the sand mass was in the range of 0.65 to 0.67, and the height to release the hammer was roughly controlled so that the maximum vibratory acceleration of about 200 to 300 gal may occur in the specimen. During the motion applied to the sand mass, the variations in vibratory acceleration, pore water pressures at various depths, and surface settlement were recorded on the oscillograph through the accelerometers, water pressure meters installed in the specimen, and a L.V.D.T. connected to a porous plate with two legs buried in the specimen, respectively.

In order to investigate the influence of a change in soil conditions and vibratory deformations on the liquefied state, the shaking table was vibrated by two ways : (1) single impact application, and (2) repeated impact application. In the first investigation, an impact load of constant magnitude was applied fifteen times, without changing each final soil condition, at the intervals of 6 hours or more which might bring about the perfect dissipation of excess water pressure induced by the last impact application. In the second investigation, the same impact load as in the first one was repeatedly at the intervals of about 3 seconds until the pore water pressure produced in the sand mass got back to the original one.

3. Test Results

(1) Results of the single impact test[12)]

Fig. 2, as a typical example, is obtained by replot-

* Presented at the 22nd Annual Meeting of J.S.C.E., May, 1967

** Dr. Eng., Professor of Civil Engineering, Kobe University

*** M.S.C.E., Assistant Professor of Civil Engineering, Kobe University

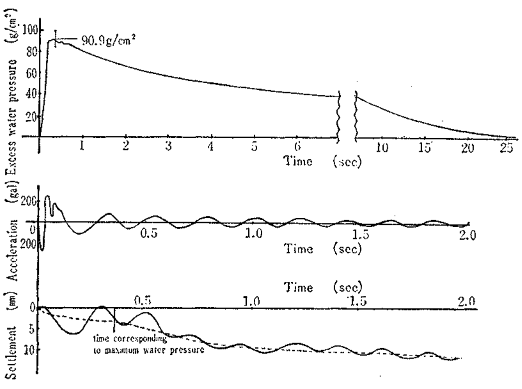

Fig. 2 Records of excess water pressure, vibratory acceleration and surface settlement by the first application of impact in single impact test.

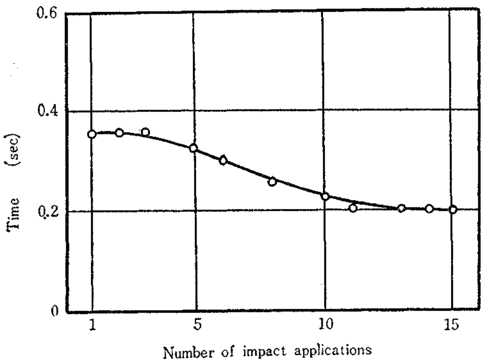

Fig. 3 Time required to reach the peak value of excess water pressure in single impact test.

ting the oscillograph records of the excess water pressure at the height of 10 cm from the bottom, the vibratory acceleration in the specimen and the surface settlement recorded during the first impact application. It may be seen from the top figure that, if the instant of impact is considered as zero of the elapsed time, t, the excess water pressure reaches the maximum value at about $t=0.36$ sec., and that after this peak it gradually decreases with increase in the elapsed time and almost dissipates at about $t=25$ sec. The time required for the peak point varies with the number of impact applications as shown in Fig. 3.

From the bottom figure in Fig. 2, it may be found that, although the variation in the surface settlement is remarkably undulatory immediately after impact application, it gradually approaches a smooth curve as the elapsed time increases. For simplifying analysis, the settlement curve was substituted by a broken line passing through the center of each amplitude as shown in the same figure.

A family of curves 1 to 4 in Fig. 4 presents the variations in the maximum excess water pressures at various depths in each impact application, which indicate that the maximum excess water pressures increase, in the same number of impact applications, with increasing depth. The curves 5 and 6 in the same figure show the variations in the surface settlement and average void ratio at the initial stage in each impact application, respectively. These results may give us a conclusion that the maximum excess water pressures and the initial void ratio (in each impact application) decrease as the number of impact applications increases, or that the denser the sand mass is, the more difficult it develops a liquefied state.

The initial void ratio at the n-th application of impact can be given by

$$e_n=\frac{H_0-S_{n-1}}{H_0}(1+e_0)-1 \quad \cdots\cdots\cdots\cdots(1)$$

in which

H_0 =original height of the specimen,

e_0 =original void ratio of the specimen, and

S_{n-1}=settlement before the n-th impact is applied.

(2) Results of the repeated impact test

Fig. 5 is obtained from the oscillograph records of the excess water pressures at various depths, the acceleration in the specimen, and the surface settlement recorded continuously during the repeated impact test. The curves of the excess water pressures and surface settlement, in this figure, are provided as the curves modified by the same means as in the single impact test.

These results show that the excess water pressure at any depth dissipates slowly in the early portion, but in a rapid rate in the late portion (from $t=40$

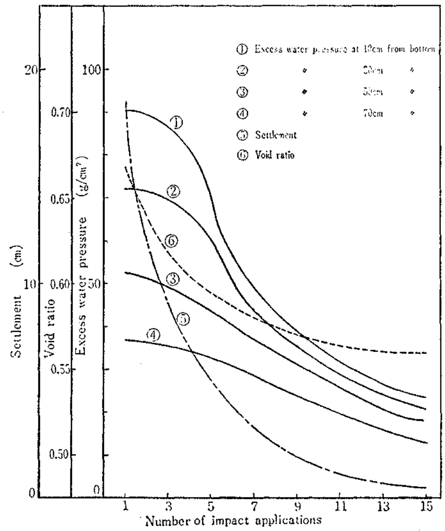

Fig. 4 Variations in excess water pressure, settlement and void ratio with number of impact applications in single impact test.

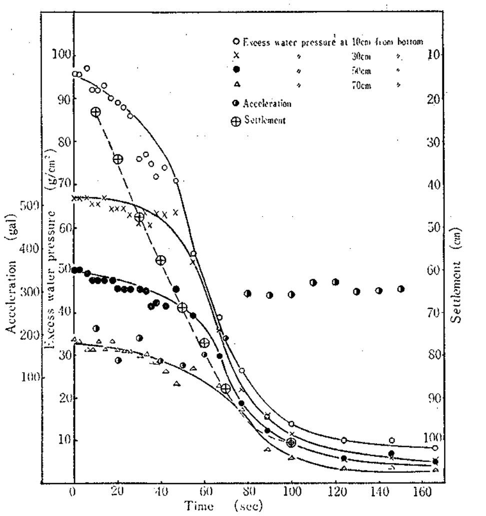

Fig. 5 Variations in excess water pressure, vibratory acceleration and settlement with elapsed time in repeated impact test

to 60 seconds), with increase in time, and that the surface settlement increases in a constant rate and approaches a constant value at the almost same time when the excess water pressures are approximately equal to the original ones. The values of acceleration plotted in the same figure were almost in the range of 200 to 300 gal as desired.

4. Experimental Consideration on Surface Settlements

By magnifying the modified settlement curve in Fig. 2 on a semilogarithmic scale, the S vs. $\log t$ curve as shown in Fig. 6 is obtained. From this figure, it is seen that the relationship between S and $\log t$ approximates a straight line in the range of the elapsed time of about 3 to 20 seconds. This may justify a simplified assumption that the general consolidation theory of clays is also valid for, at least, the late portion of this settlement process in which the linear relationship of S and $\log t$ is kept.

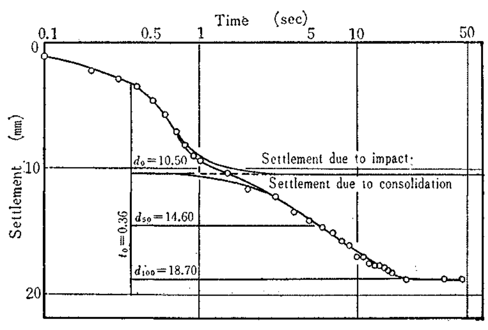

Fig. 6 Relationship between settlement and elapsed time (single impact test).

On the additional assumptions that the consolidation process may begin at $t=t_0$ when the maximum excess water pressure occurs in the mass, and that a parabolic relationships of S and t may exist in the early portion of the consolidation process, it will be inferred that the total surface settlement during the single impact application consists of two parts: one due to consolidation and the other due to the other cause.

The latter may be caused by temporary collapse of sand structure due to impact itself, since the process begins immediately after impact application, increases rapidly, and dissipates within a very short time. This tendency to decrease in volume caused by temporary collapse of sand structure is well understood to induce an increase in pore water pressure and sometimes to develop a liquefied state. The discussion presented above led to an idealized analysis of the settlement process as shown in Fig. 6.

Since the maximum excess water pressures at various depths induced by each impact application were recoreded in an approximately triangular distribution with depth, even for an incomplete liquefied state, the coefficient of consolidation, c_v, for the consolidation part can be calculated by the general theory. The idealized curve for the consolidation part in Fig. 6 gives

$$c_v=\frac{T_{50}H_0^2}{t_{50}}=\frac{0.294\times110^2}{5.54}=0.64\times10^3 \text{ cm}^2/\text{sec}$$

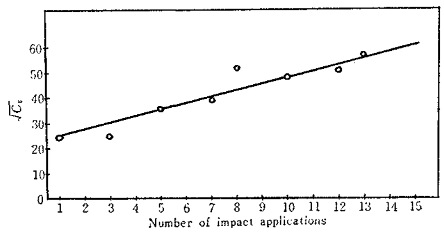

Fig. 7 Relationship between square root of coefficient of consolidation and number of impact application.

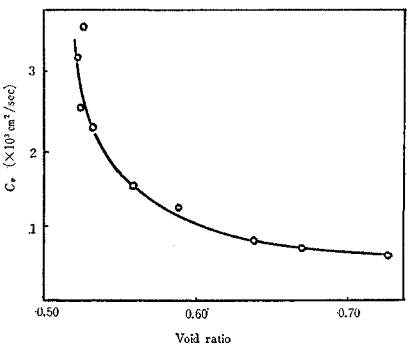

Fig. 8 Relationship between coefficient of consolidation and void ratio.

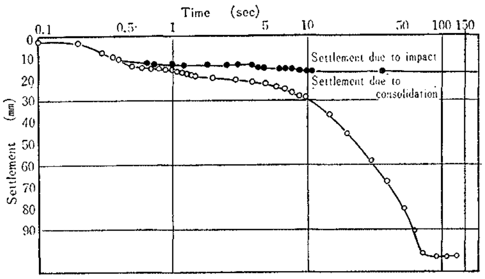

Fig. 9 Relationship between settlement and elapsed time (repeated impact test).

The values of c_v computed for other settlement data are summarized in Fig. 7 and 8, which indicate that the coefficient of consolidation increases with decrease in the initial void ratio determined by Eq. (1).

By using the same way as in the single impact test, the settlement curve of the repeated test plotted in Fig. 5 can also be divided into two parts as shown in Fig. 9. This figure presents that the rate of the settlement due to impact itself decreases gradually with increase in the elapsed tim, (or increase in the number of impact applications), and its accumulative settlement almost approaches a constant value at about 10 seconds after the first impact application. This may demonstrate that the settlement process after the time is mainly under the domination of the part due to consolidation.

5. Comparison Between Computed and Measured Excess Water Pressures in Single Impact Test

On the basis of the idealized process of surface settlements considered in the previous section, it may be assumed that the dissipating process of excess water pressures also follows the general consolidation process.

The basic equation of consolidation theory is expressed by

$$\frac{\partial u}{\partial t'}=c_v\frac{\partial^2 u}{\partial z^2} \cdots\cdots\cdots\cdots (2)$$

in which

u=excess water pressure,

t'=time elapsed after the time when the maximum excess water pressure occurred ($=t-t_0$), and

z=depth from the surface of specimen.

The record of a triangular distribution of the maximum excess water pressure with depth yields the initial condition

$$u(z,\ 0)=az \cdots\cdots\cdots\cdots (3)$$

in which a is a gradient depending on the initial void ratio of the sand mass.

Additionally the assumption of one-dimensional upward flow of water gives the mathematical expressions for the boundary conditions,

$$u(0,\ t')=0 \cdots\cdots\cdots\cdots (4)$$

$$\frac{\partial}{\partial z}u(H,\ t')=0 \cdots\cdots\cdots\cdots (5)$$

in which H is the thickness of the specimen at the initial stage of each impact application.

The solution of Eq. (2) which satisfies Eqs. (3) to (5) yields the expression for the excess water pressures at various depths of the specimen at any time,

$$u=\frac{8aH}{\pi^2}\sum_{n=0}^{\infty}(-1)^n\frac{1}{(2n+1)^2}\sin\frac{2n+1}{2H}\pi z\cdot \times\exp\left[-\frac{(2n+1)^2}{4}\pi^2 T\right] \cdots\cdots\cdots\cdots (6)$$

in which T is a time factor expressed by

$$T=\frac{c_v t'}{H^2} \cdots\cdots\cdots\cdots (7)$$

Fig. 10 presents several isochrones of both the measured and computed excess water pressures for the first impact application. In computations, the coefficient of consolidation of 6.4×10^2 cm²/sec and the gradient of the initial condition of $\gamma'=0.90$ g/cm³ (γ' is the initial submerged unit weight) were substituted in the first three terms of Eq. (6). The observed fact that the gradient of the initial conditions is equal to the initial submerged unit weight physically means the development of the complete liquefied state. Since the computed excess water pressures agree relatively well with the measured values, it may be suggested that the general consolidation theory is an adequate means for analy-

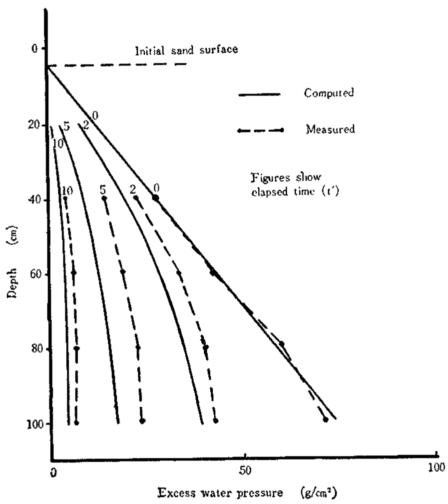

Fig. 10 Isochrones of measured and computed excess water pressures for lst impact in single impact test.

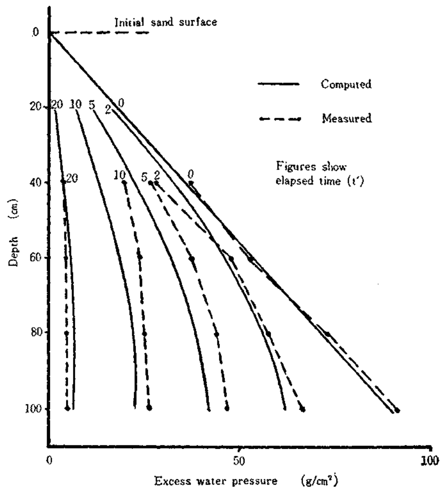

Fig. 11 Isochrones of measured and computed excess water pressures for 5th impact in single impact test.

zing the dissipation process of the excess water pressures recorded in this investigation.

Fig. 11, as another example, plotted by the same way for the fifth impact application may give the almost same conclusion as mentioned above. Even in this case in which the complete liquefaction state could not occur, the distribution of the initial excess water pressure was a triangular shape.

6. Relationship Between Observed Settlement Due to Consolidation and Calculated Accumulative Excess Water Pressure with Depth

According to the general consolidation theory, the settlement, ΔS, of the layer of thickness, Δz, at a depth, z, in the specimen can be given by

$$\Delta S = m_v \sigma' \Delta z \qquad (8)$$

in which m_v is a coefficient of volume compressibility, and as the pressure transducers are set to show zero when water pressure is hydrostatic, the effective stress, σ', is expressed by

$$\sigma' = \gamma' z - u \qquad (9)$$

As idealized in Fig. 12, let us assume that the whole sand mass can be divided into several layers, in each of which the excess water pressure at a time, $t_n (n=0, 1, 2, \cdots\cdots)$, is distributed uniformly with the recorded pressure at the correspondig time. Hence, as an example, the settlement, ΔS_i, of the i-th layer, from $t=t_{n-1}$ to $t_n (t_n > t_{n-1})$ can be given, by Eqs. (8) and (9), in the form:

$$\Delta S_i = m_v [(\gamma' z - u_{i,t_n}) - (\gamma' z - u_{i,t_{n-1}})] H_i \qquad (10)$$

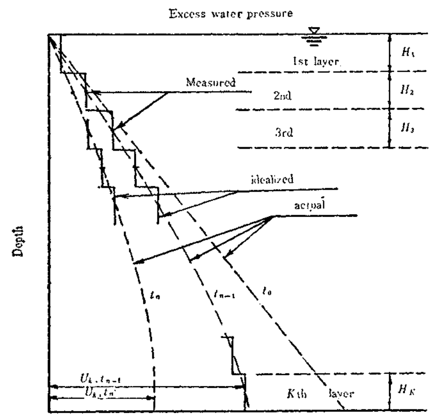

Fig. 12 Idealized distribution of excess water pressure with depth.

in which $u_{i,t_{n-1}}$ and u_{i,t_n} are the excess water pressures recorded at $t=t_{n-1}$ and t_n, respectively.

Accordingly, the accumulative settlement, S_i, of the i-th layer to a certain time, t_m, is

$$S_i = m_v \sum_{n=1}^{m} (u_{i,t_{n-1}} - u_{i,t_n}) H_i \qquad (11)$$

From Eq. (11), the accumulative settlement of the whole layer may be calculated by

$$S = \sum_{i=1}^{k} S_i = m_v \sum_{i=1}^{k} \sum_{n=1}^{m} (u_{i,t_{n-1}} - u_{i,t_n}) H_i \qquad (12)$$

where k is the number of layers.

On an adequate semi-logarithmic scale, Fig. 13 shows the observed settlement due to consolidation in Fig. 6 and the accumulative excess water pressure with depth calculated from the right side of Eq. (12). Fig. 14 is a figure of the same type derived from Fig. 9 and Eq. (12). The former is for the single impact test, and the latter for the repeated impact one. According to these figures, it may be recognized that the curve of the observed settlement due to consolidation has a similar characteristic to that of the accumulative excess water pressure, which means the settlement can be approximately

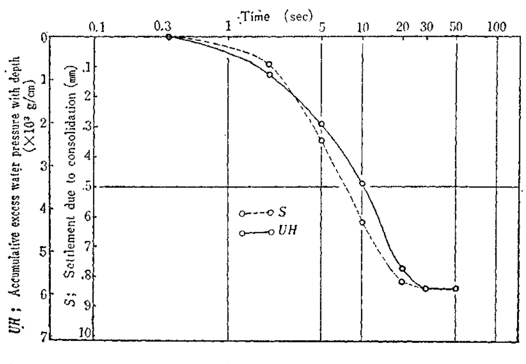

Fig. 13 Comparison of observed consolidation settlement and accumulative excess water pressure with depth (single impact test).

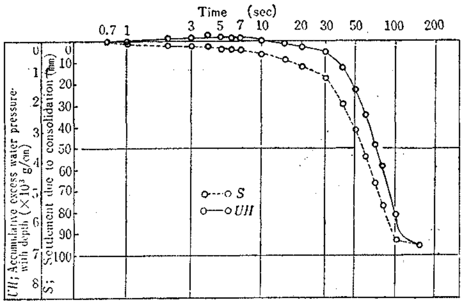

Fig. 14 Comparison of observed consolidation settlement and excess water pressure with depth (repeated impact test).

evaluated by an equation of settlement based on the general theory of consolidation.

Conclusions

This investigation on the model test of a saturated sand mass may conclude that:

(1) The maximum excess water pressures at various depths recorded during impact-vibrating motion are distributed in an approximately triangular shape with depth, even for an incomplete liquefied state, and these pressures become smaller as the initial void ratio decreases.

(2) All the surface settlements recorded in this investigation can be divided into two processes: one due to impact itself and the other due to consolidation. The former occurs rapidly in the early portion of the settlement process, and the latter gradually in the late portion.

(3) The coefficient of consolidation for the consolidation part in the single impact test increases with decrease in the initial void ratio.

(4) The settlement process due to impact itself in the repeated test can be almost neglected after a certain time (about 10 seconds).

(5) The settlement process due to consolidation is accompanied by the dissipation of excess water pressure. The dissipating process approximately follows the general consolidation theory of clays.

(6) The relationship between the observed settlement due to consolidation and the calculated accumulative excess water pressure with depth may be approximately analyzed by the general theory of consolidation, not only in the single impact test but also in the repeated one.

References

1) Florin, V.A. and Ivanov, P.L., Liquefaction of Saturated Sandy Soils, Proc. 5th Intenatl. Conf. on Soil Mech. and Found. Engg., Vol. 1, 1961, pp. 107-111.
2) Ohara, S., Experimental Study of Dynamic Pressure of Saturated Sand (in Japanese), Trans. J.S.C.E., No. 99, 1963, pp. 1-6.
3) Yamahara, H., Consideration on Failure of Sand Deposits under Simulated Earthquake Loading Condition (in Japanese), Soil Mech. and Found. Engg., Vol. 13, No. 2, 1965, pp. 3-9.
4) Kawakami, F., Ogawa, S. and Torase, M., Pore Water Pressure of Saturated Sand Developed by Impact Loads (in Japanese), Soil Mech. and Found. Engg., Vol. 13, No. 5, 1965, pp. 3-7.
5) Sunami, S., Character of Unstable Saturated Sand in Vibration, Parts 1, 2 and 3 (in Japanese), Trans. Architectural Inst. of Japan, Vol. 114, 1965, pp. 1-10; Vol. 1967, pp. 39-44 and Vol. 138, 1967, pp. 47-54.
6) Tsuchida, H., Experimental Studies on Liquefaction of Saturated Sandy Soils (in Japanese), Pre-print, 8th Lect. Meeting on Earthquake Engg., J.S.C.E., 1965, pp. 7-8.
7) Seed, H.B. and Lee, K.L., Liquefaction of Saturated Sands during Cyclic Loading, Jour. Soil Mech. and Found. Div., A.S.C.E., Vol. 92, 1966, pp. 105-134.
8) Lee, K.L. and Seed, H.B., Cyclic Stress Conditions Causing Liquefaction of Sand, Jour. Soil Mech. and Found. Div., A.S.C.E., Vol. 93, 1967, pp. 47-70.
9) Seed, H.B. and Idriss, A.M., Analysis of Soil Liquefaction : Niigata Earthquake, Jour. Soil Mech. and Found. Div., A.S.C.E., Vol. 93, 1967, pp. 83-108.
10) Yoshimi, Y., An Experimental Study of Liquefaction of Saturated Sands, Soil and Foundation, Vol. 7, No. 2, 1967, pp. 20-32.
11) Tanimoto, K., On Rise of Ground-Water Due to Dynamic Loads, Proc. Japan Earthquake Engineering Symposium 1966, pp.105-110.
12) Tanimoto, K., Liquefaction of Sand Layer Subjected to Shock and Vibratory Loads, Proc. 3rd Asian Regional Conf. on Soil Mech. and Found. Engg., 1967, pp. 362-365.

(Recvid Oct. 6, 1967)