Strain Characteristics of Underground Model Pipe During Incomplete Liquefaction

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Masaru KITAURA* and Masakatsu MIYAJIMA*

Abstract

This paper describes the dynamic behavior of buried pipeline during incomplete liquefaction. Buried pipeline systems have been damaged extensively by soil liquefaction. The procedures of mitigating the damage due to soil liquefaction were recently proposed. The extent that the buried pipe are not damaged should be made clear in advance in case of ground improvement as countermeasures to liquefaction. Therefore it is important to understand the behavior of the buried pipeline during incomplete liquefaction.

Experiments were performed employing the pipe whose two ends were free. The pipe was buried in loose, saturated sand stratum. The model sand stratum was vibrated by a tansient harmonic wave with a frequency of 5 Hz. The experimental results showed that the dynamic strains took the maximum value as the ratio of excess pore water pressure was between 0.5 and 0.75. This was explained in terms of resonance of the system consisting of the buried pipe and model ground. Furthermore the effects of the buoyancy on the buried pipe and structure during incomplete liquefaction were discussed.

1. Introduction

Buried pipeline systems were damaged extensively by soil liquefaction in the 1983 Middle Japan Sea earthquake ($M=7.7$). Though many studies of soil liquefaction have been performed very actively from the 1964 Niigata earthquake ($M=7.5$)[1], extensive damage to buried pipeline systems subjected to soil liquefaction was caused again. The authors already conducted vibrating tests using a rubber pipe model in the liquefaction process[2),3)]. The experimental results and the field investigation of the damage to buried pipeline systems in the 1983 Middle Japan Sea earthquake[4] suggested that the factors concerned with the failure of the buried pipeline due to soil liquefaction were as follows.

(1) Large behavior of the ground due to resonance during incomplete liquefaction.
(2) The forces due to the buoyancy and groundwater flow acting on the buried pipelines during complete liquefaction.
(3) Large ground deformations subjected to soil liquefaction.

Meanwhile the studies on the procedure of mitigating the damage due to soil liquefaction have been carried on[5]. As concerned with improvement of the ground, the countermeasure using gravel pile was given attention[6),7)]. It is an important problem to evaluate the criteria of the maximum excess pore water pressure if the gravel pile is designed. In order to solve this

* Department of Construction and Environmental Engineering

problem, dynamic behavior of the buried pipeline during incomplete liquefaction should be made clear.

This paper deals with vibrating tests using a rubber pipe model in liquefaction process. Dynamic behavior of buried pipe during incomplete liquefaction is discussed. Furthermore the effects of the buoyancy on the buried pipe and the structure during incomplete liquefaction are investigated. In the present paper, incomplete liquefaction means such a state as the ratio of excess pore water pressure is less than 1.0.

2. Dynamic Behavior of Pipe During Incomplete Liquefaction

2.1 Testing Procedure

General view of experimental apparatus is shown in Fig. 1. The size of the model sand stratum was 500 mm in width, 1,500 mm in length and about 250 mm in height. In these experiments, the loose saturated sand stratum was used. The procedure for making it was as same as that written in Ref. 2). The physical properties of sands and those of the model ground are shown in Table 1 and Table 2, respectively. The model of buried pipe was simulated by a rubber stick with 20 mm diameter and 1,000 mm length. Its elastic modulus was 810 kgf/cm^2 (79.4MPa) and its weight per unit volume was 1.14gf/cm^3 (11.2kN/m^3). Fifteen strain gauges were pasted up on the model pipe and were waterproofed (see Fig. 2). The two ends of the model pipe were free as this model simulated the pipeline buried in a free field. A pore pressure transducer was buried in the same depth as that of the buried pipe to measure the excess pore water pressure in the liquefaction process. This pore pressure transducer was able to measure the transient pressure with accuracy because this was miniature type pressure transducer using

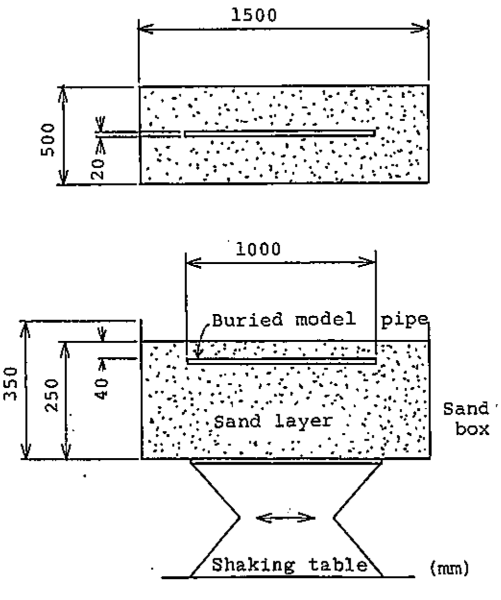

Fig. 1 General view of experimental apparatus.

Table 1 Physical properties of sands.

Coefficient of uniformity	2.96
Specific gravity	2.665
Maximum void ratio	0.982
Minimum void ratio	0.717
Coefficient of permeability	0.0176 (cm/sec) for e_{max}
	0.0157 (cm/sec) for e_{min}

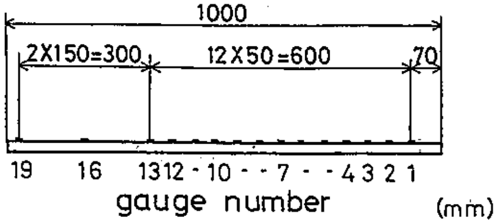

Fig. 2 Model of pipe.

Table 2 Physical properties of model ground.

Moist unit weight	(gf/cm³)	1.884
Dry unit weight	(gf/cm³)	1.560
Water content	(%)	32.78

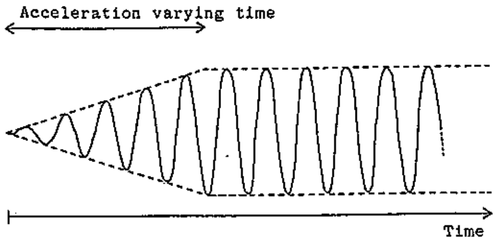

Fig. 3 Input transient harmonic wave.

semiconductor and its predominant frequency was very high (8kHz). The model sand stratum was vibrated by a transient harmonic wave with a frequency of 5 Hz (see Fig. 3) because the state of incomplete liquefaction lasted for an long time by such an input wave.

2. 2 Experimental Results and Discussions

Fig. 4 shows the records of excess pore water pressure, acceleration of the shaking table,

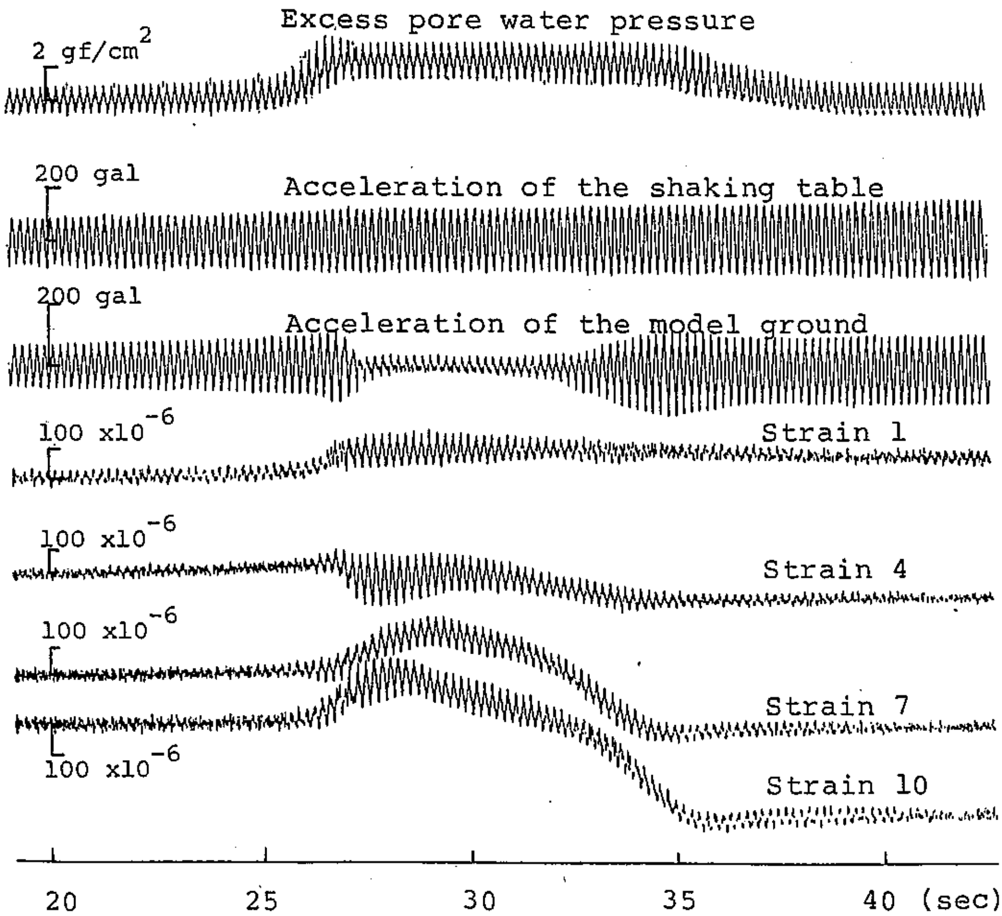

Fig. 4 Records of excess pore water pressure, acceleration of the shaking table, acceleration of the model ground and pipe strains.

acceleration of the model ground and pipe strains at gauge 1, 4, 7 and 10. Acceleration varying time was 50 sec in this case. The excess pore water pressure increased after 25 seconds shaking and kept a constant value for about 10 seconds and then decreased. The maximum value of the excess pore water pressure was about 2 gf/cm^2 (196Pa). This figure indicates that the state of incomplete liquefaction remained for about 10 seconds because the ratio of excess pore water pressure was about 0.6. The ratio means the value of excess pore water pressure devided by the initial vertical effective stress. The acceleration of the model ground was large when the excess pore water pressure was accumulating and dissipating. It was explained in terms of temporary resonance of the model ground which was softened in that process. Dynamic strains of the model pipe began to increase when the excess pore water pressure began to accumulate. And the dynamic strains remained large during incomplete liquefaction. Three generation factors of dynamic strains were considered[2] as follows:

(1) Degree of transmission of the input waves through the model ground.
(2) Degree of transmission of the ground strains to the pipe.
(3) Flexibility of the systems.

The change of the system flexibility during softening of the model ground and resonance of the system were considered in factor (3). Ref. 2) suggested that the buried pipe became most unfavorable state when the model ground resonated during the ground softening due to accumulating of the excess pore water pressure. Fig. 4, however, shows that the dynamic strain was large when the acceleration of the model ground was small. Therefore it is difficult to consider that the large dynamic strains was caused by resonance of the model ground. These reults were interpreted in terms of the effects as follows; decrease of transmission of the ground strains to the pipe due to accumulating of the excess pore water pressure and small exposure area of the model pipe to dynamic pressure of the liquefied sand because the shaking direction is the same as the axis of the model pipe in these experiments. Fig. 4 suggests that the system consisting of the model pipe and the ground resonated and the large dynamic strain was caused in higher excess pore water pressure than that when the model ground resonated. Fig. 4 also displays that the neutral axis in the strain records moved during incomplete liquefaction, especially at gauges 7 and 10. It is explained as follows; (1) The initial vertical effective stress decreased as the excess pore water pressure increased. (2) The state of the model ground changed to liquid. (3) The unit weight of the pipe was smaller than that of the model ground in this process. (4) Therefore, the pipe deformed due to the buoyancy.

Fig. 5 shows the relationship between the ratio of excess pore water pressure and the dynamic strains at gauge 10 which is pasted up in the center of the pipe. Each line in this figure indicates the experimental results in the acceleration varying time of 3 sec, 20 sec, 50 sec and 70 sec, respectively. The arrow means the process of time. The experimental results in the acceleration varying time of 3 sec shows that the model ground completely liquefied because the maximum value of the ratio of excess pore water pressure took 1. 0. While the others indicate the model ground did not liquefy completely because the maximum values of that were 0.71, 0.59 and 0.43, respectively. The dynamic strains became the maximum value when the ratio of excess pore water pressure was 0.5 to 0.75. The periods when the dynamic strains became more than

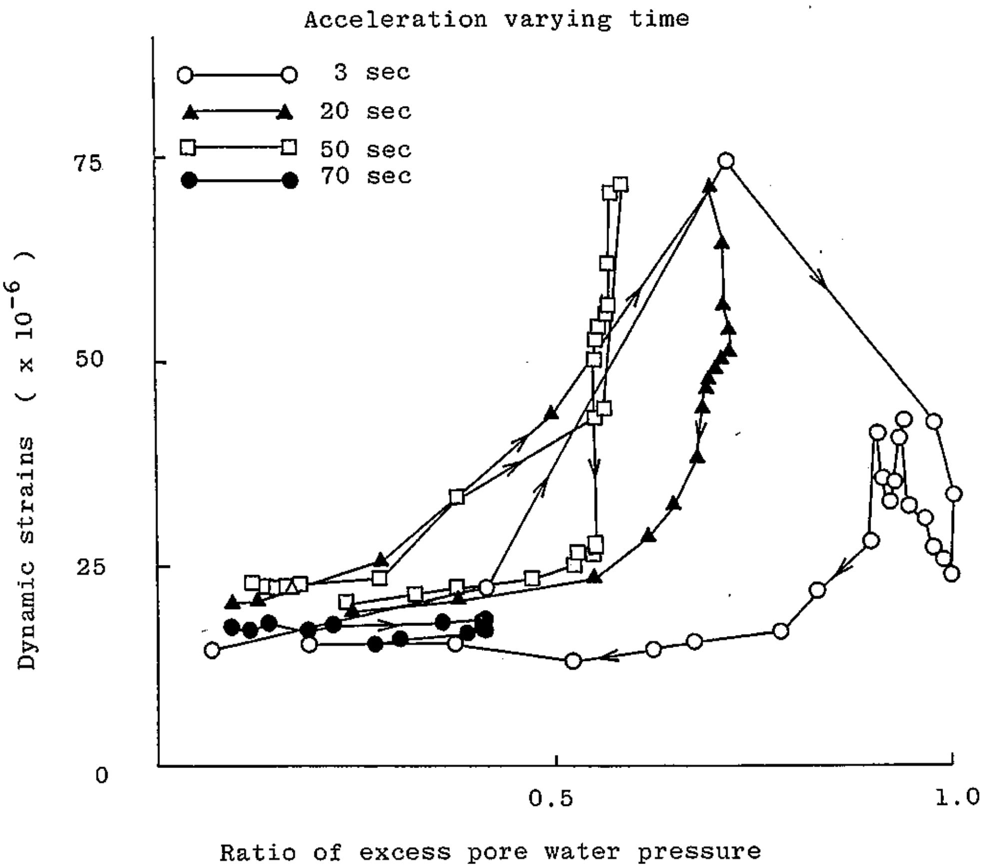

Fig. 5 Relationship between ratio of excess pore water pressure and dynamic strains.

50×10^{-6} were about 8 sec in the experiments in the acceleration varying time of 20 sec and 50 sec. While the line of the acceleration varying time of 3 sec indicates the period was about 1 sec. The value of 50×10^{-6} means relatively large dynamic strain in these experiments. Fig. 5 suggests that the period when the dynamic strains became large coincides with that when the ratio of excess pore water pressure is 0.5 to 0.9, that is, when the model ground incompletely liquefies. These results confirm that the longer the period of the incomplete liquefaction of the ground remains, the longer that of large dynamic strains is. Put otherwise, we can say that the probability of failure at the joints becomes high due to large dynamic behavior when the incomplete liquefaction lasts for a long time.

3. Effects of Buoyancy on Buried Pipe During Incomplete Liquefacion

Fig. 6 shows the schematic diagram of forces acting on buried pipe in liquefaction process. Let the buried pipe be regarded as a plate with D in thick and A in area of upper and lower sides, in simplicity. The total upward force F acting on the lower side of the plate can be expressed as:

$$F = W_l - (U + W_u + G_p) \quad (1)$$

where W_l is water pressure acting on lower side of the plate which contains the accumulating excess pore water pressure, W_u is that acting on the upper side, U is force due to the effective confining pressure acting on upper side and G_p is the weight. W_l and W_u increase as the excess pore water pressure accumulates, while U decreases. Eq. (1) can be rewritten as:

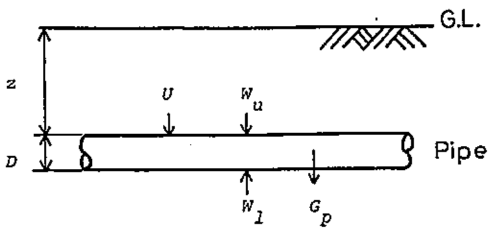

Fig. 6 Schematic diagram of force acting on buried pipe during liquefaction.

$$F = (z+D) \times P_w \times A - (\gamma_s \times z + \rho_p \times D) \times A \quad (2)$$

where $(z+D) \times P_w$ is the pore water pressure in the depth of $z+D$, P_w means the pore water pressure divided by the depth, that is, P_w is the assumed unit weight of water considering varying of the excess pore water pressure. γ_s is the moist unit weight of the ground and ρ_p is the unit weight of the plate. In Eq. (2), Z, D, A, γ_s and ρ_p are constants during liquefaction process and P_w varies from γ_w to γ_s. γ_w is the unit weight of water. Therefore Eq. (2) suggestes that the upward force does not begin to act on the buried pipe as soon as the excess pore water pressure increases but it begins to act when the excess pore water pressure accumulates to some extent. The ratio of excess pore water pressure, when the upward force begins to act on the buried structure, depends on the dimensions of the buried structure, its unit weight, moist unit weight of the ground and depth of the buried structure. For example, for the ductile iron pipe in nominal diameter 500 mm, the upward force begins to act when the ratio of excess pore water pressure reaches 0.82 and it becomes about 608 kgf (6.0 kN) per one pipe during complete liquefaction[8]. This means that the upward force does not always act on the buried pipe during incomplete liquefaction. Therefore, the probability of static failure markedly reduces if the criteria of the ratio of excess pore water pressure is less than 0.82 in the design of the ground improvement, in this case. On the other hand, in case of the hollow structure such as a manhole, the upward force acts when the ratio of excess pore water pressure is less than 0.5[8]. Then the relative displacement between the pipe and the manhole becomes large since the upward force does not act on the pipe but it acts on the manhole. Therefore the probability of failure is high at the joint of the buried pipe connected with the manhole during incomplete liquefaction, especially the ratio of excess pore water pressure is less than 0.5.

4. Conclusions

The present paper investigated experimentally on strain characteristics of the buried pipe during incomplete liquefaction. The effects of the buoyancy on the buried pipe and structure such as a manhole during incomplete liquefaction were discussed. The summary of the above results will be described below. The dynamic strains became maximum when the ratio of excess pore water pressure was between 0.5 and 0.75 in liquefaction process. This was explained in terms of the resonance of the system consisting of the buried pipe and ground. The longer the period of the incomplete liquefaction of the ground remained, the longer that of large dynamic strains was. Therefore the probability of failure at the joints becomes high due to large dynamic behavior in case that the incomplete liquefaction lasts for a long time. While in case of the buried pipe connected with the manhole, the probability of failure at the connector between them becomes high during incomplete liquefaction, especially when the ratio of excess pore water pressure is less than 0.5. Therefore, care should be taken of the behavior of a buried pipe during incomplete liquefaction.

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References

1) Yoshimi, Y. : Liquefaction of Sandy Soil, Gihodo shuppan, 1980.
2) Kitaura, M. and Miyajima, M. : Experimental Study on Strain Characteristics of Underground Pipe During Liquefaction, Memoirs of the Faculty of Technology, Kanazawa University, Vol. 15, No. 1, pp. 1-12, 1982.
3) Kitaura, M. and Miyajima, M. : Strain Characteristics of Model Pipe Fixed at One End During Liquefaction, Memoirs of the Faculty of Technology, Kanazawa University, Vol. 16, No. 1, pp. 17-24, 1983.
4) Kitaura, M., Miyajima, M. and Ikemoto, T. : Damage to Lifelines due to 1983 Middle Japan Sea Earthquake and Their Restoration, Memoirs of the Faculty of Technology, Kanazawa University, Vol. 17, No. 1, pp. 43-55, 1984.
5) Yoshimatsu, N., Yoshimi, Y. and Sasaki, Y. : Soil liquefaction 7. Countermeasure to Mitigate the Damage due to Soil Liquefaction, TSUCHI-TO-KISO, Vol. 30, No. 4, pp. 71-79, 1982.
6) Sasaki, Y., Taniguchi, E. and Ogasawara, H. : Shaking-Table Test on Gravel Drains for the Countermeasure to Liquefaction, Proc. of the 17th Japan National Conference on Soil Mechanics and Foundation Engineering, pp. 1685-1688, 1982.
7) Tanaka, Y., Kokusho, T., Esashi, Y. and Matsui, I. : On the Improvement of Potentially Liquefiable Sand Deposits using Gravel Pipes, Proc. of the 18th Japan National Conference on Soil Mechanics and Foundation Engineering, pp. 569-572, 1982.
8) Kakitani, T. : Earthquape Response Analysis of Buried Lifeline During Soil Liquefaction, Master's thesis in Kanazawa University, 1984.

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