DEVELOPMENT OF SENSOR FOR MONITORING SEISMIC LIQUEFACTION

YOSHIHISA SHIMIZU¹, SUSUMU YASUDA¹, IWAO MORIMOTO¹ and ROLANDO ORENSE¹

ABSTRACT

Recent studies have revealed that damage to gas pipelines buried underground is mainly caused by liquefaction of the surrounding soil mass. Therefore, if the occurrence of liquefaction in underground soil where gas pipelines are buried can be detected immediately after an earthquake, a more efficient and quick measure can be implemented to prevent the expansion of damage. With this in mind, a “liquefaction sensor” which is based on the measurement of water level within a hollow pipe inserted underground is developed, instead of the conventional pore water pressure meter which is less durable. Several experiments, ranging from laboratory boiling tests and shaking table experiments to field flow and vibration tests, were carried out to examine the adaptability of the sensor. From these tests, it was observed that the water level inside the pipe increases as the excess pore water pressure in the adjacent ground is increased, and the two variables have a nearly proportional relation to each other. Therefore, the occurrence of liquefaction can be detected by measuring the water level within the pipe of the liquefaction sensor.

Key words: earthquake, engineering development, in-situ test, liquefaction, model test, pore pressure, sandy soil (IGC: C8/E8/E14)

INTRODUCTION

Underground structures, notably buried gas pipelines, are highly susceptible to damage during seismic liquefaction. This was clearly shown during the 1964 Niigata Earthquake and 1983 Nihonkai Chubu Earthquake, where underground gas pipelines suffered extensive damage as a result of liquefaction of the adjacent ground (Japan Gas Association, 1965, 1984).

Therefore, if the occurrence and extent of liquefaction can be detected soon after an earthquake in locations where gas pipelines are buried and where the adjacent soil is suspected to be highly susceptible to liquefaction, efficient measures can be immediately implemented to prevent the expansion of damage to such facilities.

One of the most commonly used method to directly detect the occurrence of liquefaction during or within a short time after the earthquake is to measure the rise in the excess pore water pressure at the target site. For this purpose, pore water pressure transducers are usually embedded into the soil deposit to measure such variation in excess pore water pressure. For example, pore pressure transducers clearly captured the development of liquefaction in the Wildlife site during the 1987 Superstition Hills earthquake (Holzer et al., 1989; Youd and Holzer, 1994). Ishihara et al. (1987) also reported pore pressure readings monitored during the 1985 Chiba- baragi (Japan) earthquake.

However, conventional pore water pressure transducers have a number of problems, especially in terms of durability. Experience shows that they are often out of order or fail to function properly after having been embedded for a long time underground in a saturated condition. Ishihara et al. (1987) described the malfunctioning of piezometers in Owi No. 1 Island in 1981, necessitating the re-installation of a new set of instruments at the site prior to the 1985 earthquake. It is also reported that a number of pore pressure transducers in the Treasure Island array site were damaged due to the penetration of water through the seals protecting the electronics (Scott and Hushmand, 1995a). There are also existing references (Hushmand et al., 1992a, 1992b; Scott and Hushmand, 1995b) which claim that some of the transducers installed by the United States Geological Survey (USGS) in the Wildlife site malfunctioned during the 1987 Superstition Hills earthquake and that the piezometers probably did not measure the actual pore water pressures.

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For long term continuous monitoring, it is necessary to check the proper functioning of the meter prior to its use, and this is cumbersome and time consuming. Furthermore, replacing an old pore pressure meter with a new one before the expiry date requires digging the ground to as deep as the site where the old meter has been embedded, and this increases the installation cost. Consequently, the maintenance required may be troublesome and costly. Therefore, a more durable sensor is necessary for continuous monitoring of pore pressure variation in the ground.

**PRINCIPLE OF LIQUEFACTION SENSOR**

To counteract the limitation of conventional pore pressure transducers, a "liquefaction sensor" has been developed to detect the occurrence and extent of liquefaction at a site. The liquefaction sensor is a device that can quickly detect the occurrence of soil liquefaction during an earthquake. It must meet the following requirements:

1. It can maintain stable operation for a long period of time (durability)
2. The working principle must be simple
3. It must be economical
4. It must provide some information regarding the magnitude of excess pore water pressure build-up in the ground

One possible alternative to the conventional pore pressure meter is the detection of soil liquefaction through the rise of water level inside a hollow pipe inserted into the ground. This method consists of inserting a hollow pipe with a strainer at its middle part into a sandy layer susceptible to liquefaction, and measuring the water elevation in the pipe as a substitute for excess pore water pressure.

The principle of the liquefaction sensor is depicted in Fig. 1. When an earthquake occurs, excess pore water pressure, $\Delta u$, is generated in the saturated ground and increases with time. On the other hand, the water pressure in the open-ended hollow pipe inserted in the same place temporarily remains the same, thus producing a pressure gradient between the hollow pipe and the surrounding soil. This pressure gradient causes the entry of underground water into the pipe through the small openings in the strainer section, resulting in change in the water level within the hollow pipe inserted underground. As the water level in the pipe increases, the pressure gradient between the hollow pipe and surrounding soil decreases. When the magnitude of excess pore water pressure generated is high and the supply of underground water is unlimited, water in the pipe will rise up to a level equal to the excess pore water pressure head.

It can be seen that the working principle for the liquefaction sensor is simple. It is also comparatively free from the problems associated with conventional transducers because the water level inside the pipe can be measured by appropriate gauges which can be positioned on top of the sensor above the ground, e.g., the capacitance-type level gauge which is discussed later. Therefore, no delicate component of the sensor is embedded underground. Hence, this type of sensor is more adapted for long and continuous monitoring.

Thus, it can be said that the main advantages of a liquefaction sensor over conventional pore pressure meters are its durability and easy maintenance. Nevertheless, there is a need to illustrate the reliability and usefulness of the readings obtained by the sensors. For this purpose, various tests were performed to check the applicability of liquefaction sensors, and the results are presented below.

![Fig. 1. Principle of liquefaction sensor](image1)

![Fig. 2. Experimental set-up for shaking table tests (Series 1)](image2)
EXPERIMENTS ON WATER LEVEL RISE INSIDE HOLLOW PIPES

As mentioned earlier, the liquefaction sensor is based on the measurement of water level rise within a hollow pipe buried underground. Although the working principle is quite simple, several experiments were performed in the initial phase of the study to confirm the relation between the water level in a hollow pipe and the excess pore water pressure generated in the adjacent ground. Shaking table tests and boiling tests involving a soil container and model pipes were carried out under several test conditions.

Shaking Table Tests (Series 1)

The experimental set-up employed in the study is shown in Fig. 2. The soil container measures 90 cm long, 50 cm deep and 60 cm wide. Three types of soils were employed: mountain sand with low fines content ($F_c=8\%$), mountain sand with high fines content ($F_c=16.4\%$), and the Japanese standard Toyoura sand ($F_c=0\%$). The model ground was prepared by water pluviation. The ground surface was flat and the water level was equal to the ground surface. The relative density of the soil was between $12~24\%$. Two hollow pipes with strainers, one with a diameter of 11.8 mm and the other 46.5 mm, were placed in the soil container. At the strainer section, small holes were drilled in the pipes and covered with nylon screen to prevent clogging. The numbers shown in Fig. 2 correspond to the pore pressure transducers employed to monitor the development of excess pore water pressure in the ground.

The effect of various parameters, such as the thickness of the saturated sandy deposit and the location of the

![Graphs showing water level rise and pressure ratio](image)

Fig. 3. Time histories of excess pore water pressure, water level and input acceleration for shaking table test
strainers were also investigated. The model ground was then subjected to 3 Hz harmonic excitation. For each test, the amplitude of input motion was controlled such that the excess pore water pressure ratio, \( r_e = \Delta u / \sigma' \) (\( \Delta u \): excess pore water pressure, \( \sigma' \): initial effective overburden pressure), in the ground would rise to a predetermined level (i.e., \( r_e = 0.2, 0.5, 0.9 \)). The water elevation in the pipes as well as the excess pore water pressures were measured.

To illustrate typical time histories obtained in the tests, Fig. 3 shows the variation with time of the water level in the hollow pipes, the excess pore water pressures, and the input acceleration in the case of ground consisting of loose Toyoura sand. As shown in the figure, the excess pore pressure in the ground increased up to the maximum values in two or three seconds after shaking. In the same fashion, the water level in the 11.8 mm diameter pipe increased simultaneously with the excess pore water pressure. On the other hand, the water level in the 46.5 mm-diameter pipe increased up to the maximum value in about 7 seconds.

Figure 4 shows the relation between the maximum water elevation measured in the pipes and the fines content for the case where the ground is shaken such that the pore water pressure, \( r_e = 0.9 \). It can be seen that for both pipe diameters, there is a tendency for the maximum water level to decrease with the increase in fines content. On the other hand, a clear tendency for the maximum water level to increase with the thickness of saturated deposit can be observed, as shown in Fig. 5. Also indicated in the figure are the maximum excess pore water pressure heads computed for the case of \( r_e = 0.90 \) in the adjacent ground. These values refer to the theoretical increase in water level inside the pipe, \( \Delta H \), computed based on the initial overburden pressure at the level of the strainer \( \sigma' \), for the specified excess pore pressure ratio, i.e., \( \Delta H = \Delta u / \gamma_w = r_e \sigma' / \gamma_w \) where \( \gamma_w \) is the unit weight of water. It can be seen that the water elevation rise within the pipes is much smaller than the computed value.

Figure 6 shows the relation between the maximum water level in the pipes and the maximum excess pore water pressure ratio, as monitored by Pore Pressure Transducer No. 5 located at the same level as the strainer (see Fig. 2). Note that in the figure, three sets of the data points correspond to an embedment depth (or strainer depth) of 25 cm, while one set is for a depth of 35 cm. It can be seen from the figure that as the pore pressure ratio in the adjacent ground increases, the water level within the pipes also increases. However, it can also be seen that the increase in water level is more pronounced for the deeper strainer level corresponding to \( h = 35 \) cm.

A comparison between the maximum water elevations observed for the large (46.5 cm) diameter pipe and small (11.8 cm) diameter pipe is given in Fig. 7. These two water elevations correspond to a single magnitude of excess pore water pressure ratio in the ground. It can be seen that the water level recorded in the large diameter pipe is much smaller that those observed for small-diameter pipe. This is expected since the volume of water that accumulates within the pipe for the given time period is of finite quantity, and therefore, for a given volume of water inflow, a larger diameter would imply lower water elevation.

This series of shaking table tests clearly showed that the
water level within a hollow pipe embedded into the ground increases with the excess pore water pressure developed on the adjacent ground. Various parameters, such as fines content, depth of liquefiable layer and the diameter of the hollow pipe affect the monitored water level. However, in all the tests performed, it was noted that the monitored maximum water levels in the pipes are much smaller than the calculated values considering the effective pressure head. One possible explanation for this was the size of model ground employed in the study. Due to the small size of the ground, the volume of water available is limited. In addition, the excess pore water pressure dissipated immediately after shaking and the pore water pressure can not be maintained for a long time. Hence, the water that entered the strainer and accumulated within the pipe was not sufficient to reflect the actual pore pressure condition in the adjacent ground, i.e., not enough time was provided for the water level inside the pipe to equilibrate with the surrounding ground. In actual ground, however, there is an unlimited supply of water and the excess pore water pressure can be maintained for a long period of time due to the presence of liquefiable surface layers above the liquefiable deposit.

**Boiling Tests**

To circumvent the limitation of small scale shaking table tests, and to simulate the condition wherein a surface liquefiable layer overlays the liquefiable deposit, laboratory boiling tests were performed.

The tests were performed in a small tank into which the model ground was placed, as shown in Fig. 8. The model pipe was set-up in the middle of the deposit, and various pore pressure transducers were installed. A gravel filter (grain diameter $D_{50}=2-10$ mm) was placed around the pipe in the location of the strainer to prevent clogging of the strainer holes. Various parameters, such as fines content, relative density, thickness of saturated sand deposit, embedment depth and length of strainer, pipe diameter, and surface drainage condition were considered. To investigate the case wherein the surface is in undrained condition, an impermeable layer consisting of 5 mm thick aluminum plate supported by steel channels attached to beams positioned on top of the box was placed on the surface of the model ground.

During the test, water was introduced into the model ground from the bottom of the soil container such that the pattern of pore pressure build-up was set constant, as shown in Fig. 9. In the first case, i.e., pattern (1), the water supply was immediately stopped once the target level of pore pressure ratio $r_u$ was achieved. In patterns (2) and (3), the water supply was maintained such that the level of $r_u$ was kept constant for 15 seconds and 100 seconds, respectively. Such patterns were controlled by a computer. Five levels of pre-determined excess pore water pressure ratios (as monitored by pore pressure No. 4 in Fig. 8) and three patterns of pore pressure time histories were used, resulting in a total of 15 water pressure supply conditions. Note that since the movement of water towards the strainer may affect the readings, the farthest pore pressure transducer from the strainer, i.e.,
No. 4, was selected to represent the excess pore water pressure of the ground at the level of the strainer.

The experimental results for the case where $F_e = 2.5\%$ are shown in Fig. 10, where the relation between the maximum water level and the maximum excess pore water is depicted. In this case, the surface condition is drained, i.e., no impermeable layer was placed on the surface. It can be seen that as the pore pressure in the “free field” increases, there is a proportional increase in the water level height, for each specific water supply duration time. However, as expected, the longer duration time of water supply causes a higher water level inside the pipe. Also shown in the figure are the results for the case where the fines content $F_e = 8.4\%$. It can be seen that for model ground with higher fines content, the rate of increase in water level height with pore water pressure is much slower than those with lower fines content. Moreover, the monitored maximum water level is much slower than the pore water pressure in the ground.

For the same cases (i.e., $F_e = 2.5\%$ and $8.4\%$) but with the surface condition set undrained by placing the impermeable layer, the results are shown in Fig. 11. It can be seen that the rate of increase for both model grounds with different fines content are practically the same. When the duration time of water supply is long ($t = 100$ sec.), the maximum water level is almost equal to the maximum excess pore water pressure, even for the case of $F_e = 8.4\%$. The most probable reasons for this are as follows: (1) the over-all permeability of the ground is low; (2) a less permeable layer was formed within the deposit during the preparation of the model ground; (3) either a thin film where water can not pass through easily developed near the outer portion of the filter section or the filter is clogged; and (4) the model ground was not fully saturated.

A comparison of the effect of pipe diameter is shown in Fig. 12, where the response of 46.5 mm diameter and 11.8 mm-diameter pipes are investigated. In both cases, the surface condition is drained. It can be observed that for the smaller diameter pipe (11.8 mm), the water level is almost equal to the excess pore water pressure, regardless of the duration time of water supply. Note that the same tendency was observed both for surface drained and surface undrained conditions. Thus, for the same rate of flow into the pipe, a smaller pipe diameter results in a higher water level.

The results of the shaking table tests and boiling tests confirmed that the amount of water level rise in the pipe
is nearly proportional to the increase in excess pore water pressure in the adjacent ground. The results of the above mentioned laboratory experiments led to the development of "liquefaction sensors".

OUTLINE OF LIQUEFACTION SENSOR

The liquefaction sensor is a device which can quickly detect the occurrence and extent of soil liquefaction during earthquakes. Presently, two types of sensors have been developed: the open-type sensor which is based on the measurement of the water level rise inside the hollow pipe buried underground; and the closed-type sensor, which detects the variation of water level inside a hollow pipe with a closed upper end by measuring the changes in the air pressure inside the hollow pipe.

Figures 13 and 14 show schematic drawings of the open-type and closed-type sensors, respectively. The sensor basically consist of a hollow pipe (SUS-type) buried underground with a strainer that takes in underground water and raises the water level within the pipe, and a water level detection sensor that measures the amount of water elevation within the pipe.

Initially, the open-type sensor was developed. In this sensor, a modified capacitance-type level gauge, widely used as a level gauge in water tanks, is used as the detection sensor. In effect, the liquefaction sensor is designed to act as a concentric condenser and measures the water level inside the pipe by detecting changes in the capacitance induced by the variation in water level inside the sensor. This method allows the accurate and continuous measurement of water level (less than 1% error) inside the pipe. The capacitance type level gauge consists of a capacitance probe that detects the water level and a probe head that converts the detected water level into electric signals. Because the capacitance probe is in wire form, a stabilizing weight is attached at the tip so that the probe will not swing and touch the wall of the pipe during an earthquake.

One problem with the open-type sensor is the limitation on the height of pipe. The depth of the strainer is generally set at a depth where liquefaction potential has been evaluated to be high. For example, when the effective stress at the level of the strainer is 98 kPa, calculations show that the water level in the pipe could rise to as much as 10 m when the soil liquefies. Considering a typical case wherein the ground water level is located 2 m below the ground surface, the height of the pipe required for the open-type sensor should be longer than 8 m; otherwise, water will spout out of the sensor. However, because of stability problems and other limitations, it is impractical to have a free-standing pipe of this height. Therefore, in order to effectively detect the occurrence of liquefaction without using hollow pipes of great lengths, closed-type sensors were developed.

![Fig. 13. Schematic diagram of open-type liquefaction sensor](image)

![Fig. 14. Schematic diagram of closed-type liquefaction sensor](image)
For the closed-type sensor, an air pressure transducer is employed instead of the capacitance-type level gauge. The pore water pressure at the strainer location is inferred from the air pressure at the top of the closed pipe using the principle based on the ideal gas law, which states that

$$\frac{P \cdot V}{T} = \text{constant}$$  \hspace{1cm} (1)

where $P$ is the air pressure, $V$ is the volume of air, and $T$ is the temperature.

Now, considering the constant temperature and cross-sectional area of the pipe, Eq. (1) can be written as

$$P_0 \cdot H_0 = P_1 \cdot H_1$$  \hspace{1cm} (2)

where $H$ is the height of air inside the pipe, and the subscript 0 and 1 refer to the initial (before quake) and final (after quake) conditions. By setting

$$\Delta H = H_0 - H_1 \quad \Delta P = P_1 - P_0$$  \hspace{1cm} (3)

and noting that the change in excess pore water pressure, $\Delta u$, is given by

$$\Delta u = \Delta H \cdot \gamma_w + \Delta P$$  \hspace{1cm} (4)

where $\gamma_w$ is the unit weight of water, the combination of Eqs. (3) and (4) results in

$$\Delta u = \frac{\Delta P \cdot H_0}{P_0 + \Delta P} \cdot \gamma_w + \Delta P$$  \hspace{1cm} (5)

Thus, if the initial air pressure is set to atmospheric pressure (98 kPa), and when $H_0$ and $\Delta P$ are known, the excess pore water pressure $\Delta u$, can be calculated.

The hollow pipe used for both open- and closed-type liquefaction sensors is stainless steel (SUS) pipe, whose inside diameter is 32.9 mm (outside diameter is 42.7 mm). The whole section of the pipe is divided into the cone-shaped tip that supports the head, the strainer that takes in the underground water, and the extension rod which sets the strainer section at the desired depth. The strainer section, which is 100 mm long and has openings of 2 mm in diameter, is positioned in the sandy layer which is evaluated to be highly susceptible to liquefaction. The outline of the strainer section is shown in Fig. 15(a).

A filter is provided in the vicinity of the strainer. For this sensor, a filter which utilizes the gravel screening method was adopted. To fix the filter consisting of gravels to the periphery of the pipe, a PVC tube with window-like holes was employed. This is a thin-walled water pipe with outside diameter of 76 mm and length of 1.0 m. The holes are squares with side length of 8 cm, and the ratio of the opening to the peripheral area is 42.9%. The filter used for the liquefaction sensor is shown in Fig. 15(b). In addition, a vinyl net with 2 mm sieve openings is mounted inside the PVC tube to hold the gravel filter together.

Because the monitoring system for the closed-type sensor is affected by variations in temperature and atmospheric pressure, transducers to monitor these variables are installed inside the manhole. Moreover, the manhole is wrapped with insulators to minimize the effect of temperature variation. In addition, measures have been undertaken to eliminate the effects of dissolved gases in ground water and of methane gas produced underground which may affect the performance of the closed-type sensor.

Note that since there is no sophisticated instrument employed underground, this sensor is more durable compared to the conventional one. Moreover, all of the detecting devices can be accessed above the ground and, therefore, maintenance work is easier.

**TESTS ON THE ADAPTABILITY OF LIQUEFACTION SENSORS**

The liquefaction sensors that have been developed and outlined above have been subjected to several field tests and shaking table tests in order to confirm their adaptability. The following sections discuss the results of these tests.

**Full-Scale Vibration Tests**

Full-scale vibration tests using prototype open-type sensors were conducted in a reclaimed land in Tokyo Bay. Figure 16 shows the boring data at the site where the liquefaction sensors were buried. From the figure, a loose layer of fine sand (SPT N-value <10) exists at depths ranging between GL-1.0 m and GL-3.25 m. Two open-type liquefaction sensors with different strainer lengths were inserted in the ground, with the center of the strainer at the same level as the fine sand layer. Sensor No. 1 (L1) has a 5 cm-long strainer with 1.4% open area surface ratio (i.e., ratio between the area of the strainer hole to the total area of the strainer), while Sensor No. 2 (L2) has a 100 cm-long strainer and 6% open area surface ratio. They have the same inner diameter of 32.9 mm, and both are fitted with a capacitance-type level gauge to detect the
water level. In addition, seven pore pressure transducers and one accelerometer were installed at the site.

The arrangement of the measuring devices are shown in Fig. 17. The centers of the strainers of the sensors were positioned at a depth of GL-2.8 m. Six of the pore pressure transducers were buried at the same depth as the strainers, while one transducer was buried to a depth of GL-1.5 m.

Ground vibration was induced by the insertion of a pair of vibration rods into the ground by vibro-hammers. As shown in Fig. 18, the vibro-hammer employed has a capacity of 1176 N·m with frequency of 8.5 Hz. The vibration rods, with 508 mm diameter and 22 mm thickness, were located symmetrically with respect to the sensors, as shown in Fig. 17. Figures 19 and 20 show photos of the liquefaction sensor and accelerometers used in the field vibration tests, respectively. Note that the tests were carried out by varying the distance between the sensors and the vibration rods. During the tests, the excess pore water pressures as well the water level inside the pipes were monitored.

The maximum value of excess pore water pressure, $\Delta u$, and the water elevation of the sensor, $\Delta H$, measured during the experiment are shown in Table 1. On the other hand, Fig. 21 shows the relationships between the maximum excess pore water pressure measured by the pore pressure meters, and the maximum water elevation in the sensors. Considering the location of the transducers, the recorded pore pressure in P7 was compared with Sensor L1, while that recorded in P2 was compared with L2. It was observed that there is almost linear relationship between the two values, confirming the effectiveness of the sensors as a device to detect liquefaction. Moreover, during the test, water was ob-
served to spew out from the nozzle of the liquefaction sensor. Note that under this in-situ condition, there is enough time and volume of water available for the water level inside the pipe to reach a level equal to the excess pore water pressure developed in the adjacent ground.

**Field Flow Tests**

Flow tests were conducted in the compound of the Pipeline Engineering and Development Center of the Tokyo Gas Co. Ltd., where both open-type and closed-type liquefaction sensors, as well as conventional pore pressure transducers and a probe-type pore pressure sensor were installed. The probe-type pressure sensor forms part of an underground water monitoring system installed at the site which can also be used to collect samples of underground water and gases. All the gauges mentioned above were arranged in the site such that they were, on average, 1.4 m apart from each other, as illustrated in Fig. 22. Water was supplied into the observation well through a pump under varying pore pressure (0.05–0.15 kPa), volume of flow (0.014–0.021 m³/min) and supply duration time (3–10 min.) A transducer was also installed at the bottom of the well to monitor the pore pressure.

The results of the tests are shown in Fig. 23, which plots the relation between the maximum elevation of water inside the closed-type and open-type sensors, and the maximum pore water pressure monitored by the transducers located at the same level as the strainers. It is observed that, under the test conditions mentioned above, the readings of the closed-type sensors are almost

<table>
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<th>Exp. no.</th>
<th>Liquefaction sensor</th>
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<th>Water elevation, ΔH (m)</th>
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equal to those registered by the pore pressure transducers, while the reading obtained by the open-type sensors are smaller. A possible explanation for this is that the small amount of excess pore water pressure generated in the surrounding soil (approximately $r_0 = 0.06$) was not enough to raise the water level in the open-type sensor sufficiently. However, it is predicted that a large-scale earthquake would generate high pore water pressure and the open-type sensor would increase to a level equivalent to the pore pressure in the adjacent ground.

**Shaking Table Tests (Series 2)**

To further test the adaptability of the sensors, small-scale shaking table tests were conducted using the prototype sensors. The soil employed in the experiment consisted of fine sand mixed with silt to obtain a silty sand having $F_s = 20\%$ and $D_s = 40\%$ to simulate the soil condition in the field. In addition, two tests with only clean sand ($F_s = 3\%$) were performed in conjunction with the open-type sensor. In order to approximate the actual overburden pressure in-situ, the test box shown in Fig. 24(a) was used, where additional overburden pressure $P_o$ can be applied on the top of the box. In the experiment, the $P_o$'s used correspond to 0.0, 9.8 and 19.6 kPa.

The saturated model ground was prepared by water pluviation technique. When the level of sand reached the top of the test box, the cover was placed after which the pre-determined overburden pressure was applied. When equilibrium was obtained, the sensor was attached and the box was shaken at a specified acceleration level and duration time until liquefaction was detected.

A total of 13 tests were conducted, 5 of which involved the closed-type sensor; the rest used open-type sensors. Various values of $P_o$'s, acceleration level, duration of...
shaking, fines content and sensor pipe diameters were employed. A prototype sensor with an inside diameter of 33 mm was used in all the tests, except for one test with an open-type sensor where a 10 mm pipe was employed. The location of the pore pressure transducers is also shown in Fig. 24(b).

Closed-type sensors: The relation between the maximum pore pressure reading monitored by the transducers at the level of the strainer and the water level monitored by the sensor is shown in Fig. 25. In the figure, $\Delta H_{\text{free}}$ refers to the water level in the sensor when the pore pressure transducer registered its maximum reading, $\Delta H_{\text{max}}$ (i.e., the time when the "free field" liquefies), while $\Delta H_{\text{max}}$ denotes the maximum water level. It can be observed from the figure that $\Delta H_{\text{free}}$ and $\Delta H_{\text{max}}$ are almost equal to $\Delta H_{\text{max}}$, indicating that the closed-type sensor can detect liquefaction quite accurately. The time lag between the occurrence of $\Delta H_{\text{free}}$ and $\Delta H_{\text{max}}$ is about 0 ~ 8 sec. This finding is supplemented in Fig. 26 which plots the development of water level in the sensor, $\Delta H$, with the increase in pore water pressure in the ground, $\Delta u$, for various levels of confining pressure. It can be seen that the relation is more or less a straight line, indicating a simultaneous increase in $\Delta u$ and $\Delta H$.

Open-type sensor: Figure 27 shows the variation of $\Delta H_{\text{max}}$ and $\Delta H$ for the open-type sensor. It can be seen that in cases involving silty sand ($F_i = 20\%$), the values of $\Delta H_{\text{max}}$ are generally smaller than those of $\Delta H_{\text{free}}$, while in cases where clean sand ($F_i = 3\%$) is used, the two values are almost equal. This implies that the permeability of the ground affects the flow of water into the pipe. This finding is substantiated by the plots of the development of $\Delta u$ and $\Delta H$ for various $P_o$'s shown in Fig. 28. It is seen that the increase in $\Delta H$ proceeds at a slower rate than the increase in $\Delta u$, and when $\Delta u$ is maintained at a high level as a result of liquefaction, $\Delta H$ continues to increase, indicating a time lag between $\Delta H_{\text{free}}$ and $\Delta H_{\text{max}}$. In general, this time lag is between 16 ~ 91 min for model ground with $F_i = 20\%$.

The effect of $F_i$ is illustrated in Fig. 29 where it can be seen that in the case where clean sand ($F_i = 3\%$) is used, $\Delta H$ increases almost simultaneously with $\Delta u$, while for silty sand ($F_i = 20\%$), the rate of increase of $\Delta H$ is much slower. Finally, the effect of pipe diameter is shown in Fig. 30, where the results for a 33 mm pipe diameter is

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*Fig. 25. Relation between $\Delta u_{\text{max}}$ and $\Delta H$ (closed-type)*

*Fig. 26. Correlation between change in water elevation and excess pore water pressure for various overburden pressures, $P_o$ (closed-type sensor)*

*Fig. 27. Relation between $\Delta u_{\text{max}}$ and $\Delta H$ (open-type sensor)*
Fig. 28. Correlation between change in water elevation and excess pore water pressure for various overburden pressures, \( P_0 \) (open-type sensor)

Fig. 29. Correlation between change in water elevation and excess pore water pressure in ground with various fines contents, \( F_c \) (open-type sensor)

DISTRIBUTION OF LIQUEFACTION SENSORS IN TOKYO

With the promising results of the laboratory and in-situ tests performed to investigate the adaptability of liquefaction sensors, Tokyo Gas Co. Ltd. has installed liquefaction sensors in areas near medium pressure gas pipelines which are judged to be highly susceptible to soil liquefaction.

The procedure in installing the open-type sensor is shown schematically in Fig. 31. After an 86 mm diameter hole is drilled and cleaned, the sensor body is placed inside the hole. The strainer section, wrapped around by the gravel filter drain, is back-filled with sand and bentonite pellets. After the backfill materials are placed, a simple check is made on whether clogging occurs in the strainer section by supplying water into the hollow pipe. After confirming that the strainer works well, the support pipe is placed. Finally, the probe is attached to the open end of the sensor. Initially, most of the sensors installed were open-type; however, they were replaced over the years by closed-type sensors.

At present, liquefaction sensors have been installed at 20 sites all over the Tokyo Metropolitan area. Each of these sites is located in medium pressure gas pipeline blocks as arranged by Tokyo Gas Co. Ltd. Prior to sensor installation, standard penetration tests were conducted, and liquefaction potential evaluation analyses based on the methodology incorporated in the Japanese Code for Highway Bridges (JRA, 1980) were performed at suitable sites to determine the most appropriate location of the sensor.

The strainer section is set generally in the fine sandy layer of the deposit and its location varies between 2.50 m–8.80 m below the ground surface. For each site, SPT
$N$-values, and grain size distributions are available. In addition, results of physical tests, liquefaction tests, dynamic tests and PS logging tests are available for most sites. To supplement the sensors, a 3-component accelerometer, a conventional pore pressure transducer, and the probe-type pore pressure sensor mentioned earlier are also installed in the vicinity of each of the sensors.

Note that based on the results of liquefaction potential evaluation conducted at each site, the degree of pore pressure build-up at various elevations within each site can be estimated from the magnitude of pore pressure that develops at the level of the strainer. Therefore, once the water level inside the sensor during an actual earthquake is monitored, the extent of pore pressure build-up in the other layers can be interpolated, and the extent of liquefaction in terms of depth can be estimated.

These liquefaction sensors, together with 332 SI (Spectral Intensity) sensors and 5 bedrock seismometers, form part of Tokyo Gas Co.'s seismic information monitoring system called SIGNAL (Seismic Information Gathering Network Alert System), which has been in operation since June, 1994 (Yoshikawa et al., 1995). The distribution of these sensors is shown in Fig. 32, while a schematic diagram of SIGNAL is illustrated in Fig. 33. It can be seen that the data recorded by the sensors are
transmitted quickly to the head office through a wireless network. If certain areas are estimated to have some damage to gas pipelines based on the data, the supply of gas is closed to prevent the expansion of damage to other structures.

RESPONSE OF SENSORS TO ACTUAL EARTHQUAKES

In recent years, the Tokyo Metropolitan area has been shaken by several relatively small-scale earthquakes. Notable examples are those which occurred on February 3, 1992 (M5.9; maximum intensity of V on the JMA
scale), and on May 21, 1993 (M5.4; maximum intensity of IV). Although these earthquakes were of small magnitude and the pore pressures were generally small, they nevertheless triggered an increase in water level inside some of the liquefaction sensors installed in the area. Note than when these earthquakes occurred, most of the sensors were still the open-type ones.

Figure 34(a) shows the soil profile at the Ohmori Higashi site. The strainer section is located at a depth of 5.5 m, where the soil consists predominantly of fine sand with SPT N-value of 6. A pore pressure meter is also buried at the same depth as the strainer. Figure 34(b) shows the recorded surface acceleration, and excess pore water pressure registered by the transducer and water level inside the sensor during the February 2, 1992 earthquake. With the peak ground acceleration of 143.5 cm/sec², the water level rose to about 9 cm, which was only about 10% of the maximum excess pore water pressure in the adjacent ground. Note, however, that the water level was still increasing although the monitored excess pore water pressure was already decreasing. This tendency is the same as those observed in shaking table tests. Because of the time lag between the maximum excess pore water pressure and maximum water level, the water level inside the sensor can continue to increase even when the excess pore water pressure is already decreasing.

On the other hand, the soil profile at the Sasamegawa site is presented in Fig. 35(a). The strainer is also set at a depth of 5.5 m where the soil consists of fine sand. To supplement the readings, a pore pressure transducer is also installed at the same depth as the strainer. The recorded values during the May 21, 1993 earthquake are shown in Fig. 35(b). The peak ground acceleration registered is 113.7 cm/sec² and the water level in the sensor rose to about 6 cm, which was almost equal to the maximum recorded pore water pressure in the adjacent ground.

Since 1993, no major increase in the water levels inside the sensors have been observed. Judging solely on the two observations discussed above, it is noted that the water levels in the sensors increased when sufficient excess pore water pressure was generated in the ground. It is believed that during large earthquakes when the pore water pressure is normally high, the rise in the water level inside the sensor would be indicative of the state of soil as far as liquefaction is concerned.

Note that both shaking table tests and field tests show that there is a time lag between the occurrence of maximum excess pore water pressure and maximum water level within the liquefaction sensor. Figure 3 indicates a time lag of about 3-4 seconds between the occurrence of the maximum values of excess pore water pressure in the ground and maximum water level in the 46.5 mm-diameter pipe. Similar observations can be made on the results of shaking table tests on an open-type sensor, as shown in Figs. 27-30. In the actual recording in
the field shown in Figs. 34 and 35, it can be seen that about 30 to 60 seconds was necessary for the water level to reach the maximum value, although the excess pore water pressure increased rapidly just after the peak acceleration. As shown in these tests and observations, several tens of seconds were necessary to reach the maximum water level. However, it should be borne in mind that the main purpose of the liquefaction sensor is not to estimate excess pore water pressure, but to detect liquefaction sites. Once the occurrence of liquefaction at target sites is confirmed and some pipe damages are reported, the valves of main gas pipelines may be closed so as not to cause secondary fires. In the present system, the detection of liquefied sites can be done in 10 to 20 minutes after the occurrence of the earthquake. Therefore, if about 1 minute is necessary to reach the maximum water level within the sensor, it is more than adequate for this system.

CONCLUSIONS

For the purpose of mitigating the damage to underground pipelines induced by the liquefaction of the ground, liquefaction sensors have been developed based on the measurement of the rise in water elevation inside a hollow pipe inserted underground. The adaptability of the sensors has been extensively investigated through laboratory shaking table tests and boiling tests, as well as in-situ vibration and flow tests. Through the tests, it was observed that the water elevation is proportionately related to the excess pore water pressure.

Based on the experience of the shaking table tests using a small soil container, it was demonstrated that the water level is affected by several factors, such as diameter of the hollow pipe, fines content of sand and thickness of liquefied layer, among others. Note that if the liquefiable layer is thin and small in area as in the case of ground in the soil container, these effects are important because accumulation of water is limited. However, if the liquefiable layer is thick, such as more than three meters, and widely spread in a horizontal direction, these effects are negligible because the accumulation of water is unlimited and easy. In reality, the field tests demonstrated that water level can increase up to the theoretical height within the liquefaction sensor. Therefore, it can be judged that the sensor developed is satisfactory to detect the water level in the field.

Several small-scale earthquakes have triggered a rise in water level inside the sensors. Since the magnitudes of these earthquakes were relatively small, the water level rise was not sufficient to raise the level comparable to those of the pore pressures in the adjacent ground.

The importance of disaster prevention measures in urban areas has been highlighted, and it is necessary to further improve the measures to be implemented to protect lifelines when an earthquake occurs. Among
these, measures to protect lifelines against the hazards of soil liquefaction are important. Under such circumstances, further refinements on the operating conditions of the liquefaction sensors have to be carried out to improve their performance and to ensure that they are effectively used as one of the methods to mitigate the damaging effects of soil liquefaction during earthquakes.

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