

Seismic Inspection of Existing Structures Based on the Amount of Their Deformation Due to Liquefaction

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Abstract. If structures such as houses, or structures with very long lengths, such as river dikes, railway embankments, and sewers, have not been constructed with liquefaction in mind, it is necessary to extract repair their parts that could be easily damaged in the event of a future earthquake. In inspecting these structures, it is necessary to quantitatively estimate the points where countermeasures are required based on the amount of settlement or the amount of uplift of the structures. For that purpose, it is necessary to properly consider the process of settlement or uplift of the structures due to liquefaction based on past cases of damage and the results of model tests, and develop a method to easily estimate the amount of settlement or uplift. In Japan, there are many cases of structural damage due to liquefaction, and many model tests using shaking tables have been conducted. Simple estimation methods have been developed based on these and are being used for inspections. Furthermore, measures are being taken based on the inspection results.

Keywords: Liquefaction, Seismic inspection, River dike, Low-rise house, Sewer manhole

1 Introduction

During the last half century, methods for predicting the occurrence of liquefaction and countermeasures against damage due to liquefaction have been vigorously developed. These methods and countermeasures are now being considered when constructing large, new structures. However, they have not been applied to many previously constructed structures, and they should be to prevent damage stemming from future earthquakes. In these cases, the damage likely to be caused by liquefaction must be estimated. For important structures, such as bridges, mid-rise buildings, and tanks, detailed examinations, such as seismic response analysis, can be conducted to estimate the degree of damage. However, it is impossible to examine in detail all structures, such as houses in big cities, or very long structure, such as river dikes, railway embankments, and sewers. Therefore, simple methods are needed to estimate the damage to these structures and to extract repair parts of long structures that are likely to suffer damage.

As will be described later, the degree of actual damage to a house can be quantitatively evaluated by its penetration settlement and inclination angle. The damage to river dikes and railway embankments can be evaluated by the amount of their subsidence.

Damage to sewers can be evaluated by the amount of manhole uplift. A simple method for estimating these values can be derived by setting parameters related to damage and creating empirical formulae based on actual damage and model experiment results. This method can be easily created as long as there is data on past damage. However, the processes of the settlement and uplift of structures due to liquefaction are complicated, as described later, so it is important to derive simple methods that properly consider these processes.

These processes are clarified by examining the damage caused by past earthquakes in detail and by conducting model tests. In Japan, earthquakes that cause liquefaction have occurred once every one to two years, and many model tests using shaking tables have been conducted. Based on the damage caused by these earthquakes and on model test results, simple methods for estimating the amount of settlement or uplift have been developed and are being used for inspections. Furthermore, measures to prevent damage are being taken based on the inspection results. The current status of such efforts in Japan is introduced below.

2 Embankments

2.1 Earthquake damage cases where the amount of settlement due to liquefaction was measured

There are various types of embankments, such as earth dams, tailings dams, river dikes, road embankments, railway embankments, and filled housing lots, in Japan. These are damaged by sliding and/or subsidence due to liquefaction of their foundation ground and of the embankments. If it is estimated that a large slip collapse is likely to occur due to liquefaction, countermeasures must be taken, even for existing structures. On the other hand, if the amount of estimated subsidence is small enough that the structure could maintain its function, countermeasure are not necessary. For example, in the case of a river dike, it is a serious problem that the river water overflows into adjacent areas when the dike subsides, as shown in Fig. 1(1). However, even if subsidence occurs, countermeasures are not necessary if the top of a river dike is higher than any probable river water level. In the case of railway and road embankments, as long as subsidence does not prevent trains or cars from crossing the embankment, countermeasures are not needed. Fig. 1 (2) diagrams a railway embankment that is dysfunctional because it has subsided so much that a train cannot cross a bridge girder. Therefore, for long embankments, it is necessary to easily estimate the distribution of subsidence due to liquefaction during future earthquakes and to determine the priority of the locations where countermeasures will be taken.

About 70% of Japan's land area is mountainous, and the country is surrounded by the sea. Furthermore, the annual rainfall is about 1,700 mm, which is about twice as high as the world average, so there are more than 30,000 rivers. Furthermore, due to substantial seismic activity, some river dikes have been damaged every time an earthquake occurs. These damaged river dikes were usually restored just by filling the dikes with soil again, and earthquake countermeasures were hardly taken until the 1995 Hyogoken-nambu (Kobe) Earthquake. The 1995 Kobe Earthquake caused the Yodo River

dike was 50 cm high. Sand with a mean particle size of $D_{50} = 0.28$ mm and a uniformity coefficient of $U_C = 2.91$ was used for the sand of the ground and dike. The ground and dike were constructed with relative densities of $Dr = 59\%$ and 53% , respectively. Seventeen accelerometers and 40 pore water pressure transducers were installed in the ground. Settlement of the dike was also measured. The model ground was shaken at a sine wave with a frequency of 2 Hz and at an acceleration of 231 cm / s^2 . Figure 5 shows the time histories of excess pore water pressure measured by pore water pressure transducers in the ground. The vertical broken line in each time history indicates the time when the shaking is over. If the inside of the ground is roughly divided into four zones: the deepest A zone, the intermediate depth B zone under the dike, the shallowest C zone under the dike, and the D zone outside the dike, the time history of excess pore water pressure was different in each zone. In Zone B, the pore water pressure increased with the start of shaking and decreased with the end of shaking. In Zone A, the pore water pressure gradually increased with the start of shaking and rapidly decreased with the end of the shaking. In the C zone, the pore water pressure increased at the start of shaking as in the A zone. However, after a large drop in the middle of shaking, it soon returned to its original state, then, it decreased with the end of shaking. In the D zone, the pore water pressure increased considerably during the shaking, and after the shaking was over, the pore water pressure increased for a few more seconds and then began to decrease.

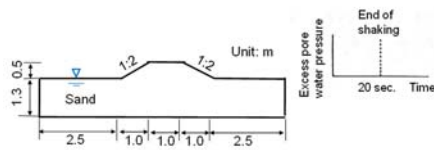


Fig. 4. Dike model [1].

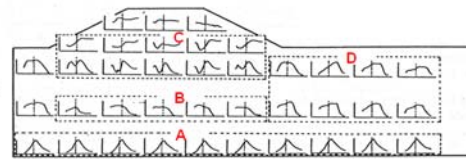


Fig. 5. Time histories of excess pore water pressure.

Figure 6 shows the distribution of excess pore water pressure 6 seconds after the start of shaking. High excess pore water pressure was generated in the B zone and the C zone. Figure 7 shows the time history of subsidence at the crest of the dike. It gradually subsided from the start of shaking, and subsided to about 15 cm. Judging from this, together with Figs. 5 and 6, it is considered that the pore water pressure increased and decreased and the dike subsided according to the following process from the start of shaking.

- (1) Pore water pressure increased in the B, C, and D zones with the start of shaking.
- (2) As the pore water pressure increased, the dike began to slip and subsidence began to occur. In the C zone positive dilatancy of the soil occurred due to the large deformation of the soil and the pore water pressure decreased once. After that, the pore water pressure increased due to the propagation of excess pore water pressure from Zone B.
- (3) Since the effective confining pressure in the D zone was smaller than that in the B and C zones, the excess pore water pressure due to shaking rose less in the D zone than in the B and C zones. However, since the excess pore water pressure from the B

and C zones propagated, the pore water pressure in the D zone increased even after the shaking was ended.

(4) Zone A was the deepest part of the ground, so it was slightly difficult to liquefy, and pore water pressure was generated later than in the other zones. When the shaking was ended, the excess pore water pressure dissipated toward the upper part, so that the pressure decreased rapidly.

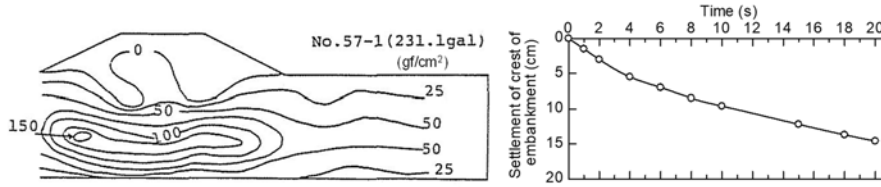


Fig. 6. Distribution of Excess pore water pressure. Fig.7. Time history of subsidence the dike.

2.3 Development of a simple method for estimating the amount of subsidence of river dikes

The amount of subsidence of river dikes due to liquefaction is thought to be affected by several factors, such as the height of the dike, the type and density of the dike soil and the surrounding ground, the groundwater level, and the seismic motion. However, if the height of the dike and the soil type of the ground are relatively constant in a certain area, it is considered that the thickness of the liquefied layer and the intensity of liquefaction greatly affect the amount of subsidence. Then the relationship between the liquefaction potential index P_L and the amount of subsidence was investigated for the dikes of three large rivers flowing near Nagoya City, Japan that were damaged during the 1944 Tohankai Earthquake, as shown in Fig.8 (Nakamura et al. [2]). Although the plotted points are a little scattered, the amount of subsidence increased as P_L increased, so it could also be used to estimate subsidence in future earthquakes. The liquefaction potential index P_L is calculated by the following formula based on the safety factor for liquefaction F_L , and is generally used to roughly estimate the magnitude of damage to a structure due to liquefaction (Iwasaki et al. [3])

$$P_L = \int_0^{20} (1 - F_L)(10 - 0.5z) dz \quad (1)$$

$$(1 - F_L) = 0 \quad (\text{for } F_L > 1)$$

On the other hand, in analyzing the stability of the dike slope, the safety factor against sliding FS can be expressed by the following equation, considering the horizontal seismic force k_h and the excess pore water pressure Δu .

$$F_{sd} = \frac{\sum \{c \cdot l + \{(W - u_0 \cdot b - \Delta u \cdot b) \times \cos \alpha - k_h \cdot W \cdot \sin \alpha\} \cdot \tan \phi\}}{\sum (W \cdot \sin \alpha + k_h \cdot W \cdot y/r)} \quad (2)$$

If F_S is small, the amount of subsidence at the crest of the dike may be large. As mentioned above, the time history of excess pore water pressure in the ground is

complicated, and it is necessary to obtain the distribution of excess pore water pressure in the cross section at a certain point in time and consider it in the calculation. Since this is time-consuming, it is common practice to simplify and estimate the distribution of excess pore water pressure from the distribution of F_L at the peak of cyclic shear stress, ignoring the time history of excess pore water pressure. After the 1995 Kobe Earthquake, using this simplified method, the safety factor F_S for slips was calculated for 27 river dikes damaged during six past earthquakes and compared with the settlement rate of the dikes (settlement/height of dike), as shown in Fig. 9 (Ministry of Construction [4], Matsuo [5]). However, in Eq. (2), both the safety factor $F_{sd}(\Delta u)$ when only the excess pore water pressure is considered and the safety factor $F_{sd}(k_h)$ when only the horizontal seismic intensity is considered are calculated, and the smaller one is shown in this figure. Although the plotted points are scattered, the settlement rate increased as the safety factor decreased. Therefore, based on the upper limit shown in the figure, damages to river dikes due to level 1 seismic motion (medium-scale seismic motion) have been inspected, and countermeasures have been applied.

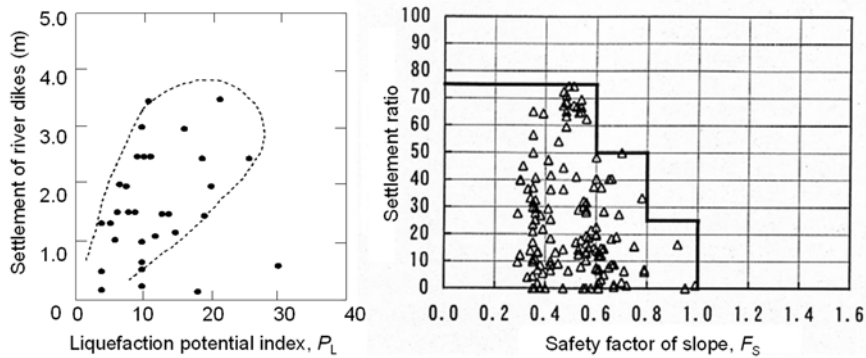


Fig. 8. Relationship between P_L and S [2]. **Fig. 9.** Relationship between F_S and S/H [4].

After that, it became necessary to investigate the seismic resistance of the river dikes against Level 2 seismic motion (large-scale seismic motion), design countermeasures, if necessary, and implement the countermeasures (Ishihara and Sasaki [6]). In order to estimate the amount of subsidence of the dike here, instead of the indirect method of estimating from F_S , methods of directly analyzing the deformation of the dike have been adopted. The optimal analysis method is two-dimensional seismic response analysis that can consider liquefaction. However, since this requires detailed soil tests and costly analysis, the applicability of a simpler static analysis method was examined. Then, ALID was taken up as a static analysis method, and it was used to design countermeasures that were included in a guidance in 2016 (Public Works Research Institute [7]). ALID is a residual deformation analysis method using the static finite element method. For details, see the references (Yasuda et al. [8]). Figure 10 shows the results of the reproduction analysis of the Yodo River dike in Photo 1. Figure 11 shows the results of the 27 river dikes damaged by the past six earthquakes, analyzed by ALID

and compared with the actual subsidence (Wakinaka et al. [9]). The amount of subsidence can be reproduced well even though ALID is a static analysis method. As shown in the schematic diagram in Fig. 12, the guidance covers the compaction method, the solidification method, the method using steel sheet piles, and the drain method that lowers the groundwater level inside the dike. These methods were designed by analysis using ALID. Currently, measures are being taken according to this guide. When using the compaction or solidification methods, improving the ground under the vicinity of the toe of an dike, as shown in Fig. 12, will increase F_s , though F_s does not increase if only the area directly under the crest of the dike is improved. However, according to the analysis using ALID, the amount of subsidence will be dramatically reduced if only the area directly under the dike is improved. Therefore, in addition to improving the ground under the toes of dikes, construction work is being carried out to improve the ground under the crests of dikes.

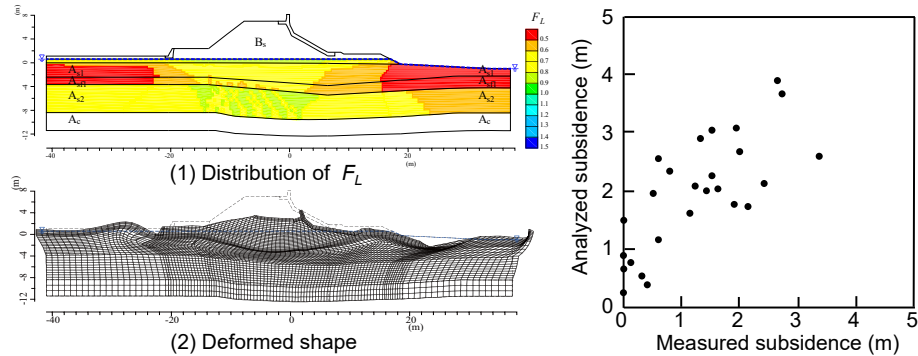


Fig. 10. Analyzed results of Yodo River dike. **Fig. 11.** Measured and analyzed subsidence [9].

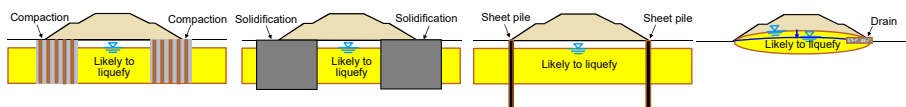


Fig. 12. Countermeasures against the subsidence of river dikes introduced in the guidance

2.4 Development of a simple method for estimating the amount of subsidence of railway embankments

Railway embankments are compacted well while being constructed, so excessive pore water pressure is unlikely to occur in these embankments during an earthquake. Therefore, it is considered that the shear rigidity of these embankments does not decrease during an earthquake, and that a decrease in the shear rigidity of the ground and the weight of the embankment cause subsidence. Then, a method for estimating the amount of subsidence was developed based on the results of several previous shaking table tests (Sawada et al. [10]). Factors that affect the subsidence of a railway embankment include the characteristics of the embankment, the characteristics of the ground beneath and around the embankment, and the seismic motion, but experimental results suggest that

the characteristics of the embankment have little effect. On the other hand, since it was clarified that there is a relationship between the thickness of the liquefaction layer and the layer's excess pore water pressure and amount of subsidence, the liquefaction potential index P_L was used as a representative index. If P_L is used, the influence of seismic motion can also be taken into consideration. Figure 13 (1) shows the relationship between P_L and the settlement ratio when the relative density is less than 60% and when it is 60% or more. The smaller the relative density, the larger the settlement. In Fig. 13 (2), the data on soil with a relative density of less than 60% are further divided by the number of shakings. The amount of subsidence increased as the number of shakings increased. These figures have been introduced into railway construction standards and are used to estimate the amount of embankment subsidence due to liquefaction (Railway Technical Research Institute [11]). Naganawa et al. [12] studied railway, roads and river embankments that were damaged due to liquefaction during eight past earthquakes and investigated the relationship between the P_L and the settlement ratio of these embankments. Their findings supported the relationships shown in Fig. 13.

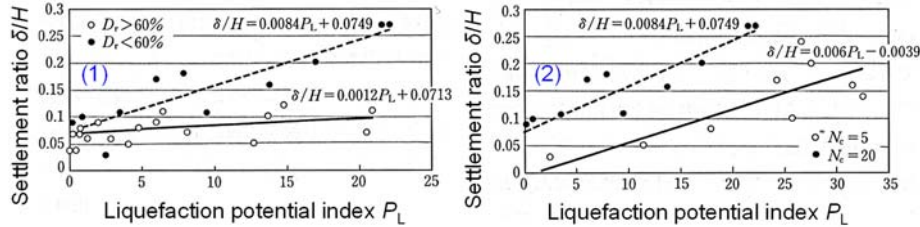


Fig. 13. Relationship between P_L and δ/H introduced into railway design standards [10] [11].

3 Low-rise housing

3.1 Earthquake damage cases in which the amount of settlement and inclination angle of buildings due to liquefaction were measured

When the ground liquefies, it softens, so low-rise houses and mid-rise buildings settle into the ground under their own weight, and in some cases tilt, as shown in Fig.14. The ground also subsides due to volume shrinkage as the excess pore water pressure disappears after liquefaction occurs. Therefore, it is necessary to distinguish between i) the absolute settlement of a building, ii) the penetration settlement of the building into the ground, and iii) the subsidence of the ground. If the thickness of the layer liquefied in a certain area is about the same in plane, the ground subsides uniformly in that area, so the building does not incline. Therefore, the amount of land subsidence does not damage the building much, but the amount of penetration settlement greatly affects damage.

Liquefaction has caused much damage to mid-rise buildings all over the world. The settlement and tilt angles of many buildings were measured in Niigata, Japan in 1964, in Dagupan, the Philippines in 1990, and in Adapazarı, Turkey in 1999 (Yasuda, et al. [13]). Since the settlement was measured locally as the difference from the surrounding ground surface, the amount of settlement corresponds to the amount of penetration settlement. In the 1964 Niigata earthquake, 340 reinforced concrete buildings were

damaged by liquefaction. Figure 15 shows the frequency distribution of the average settlement of buildings by taking out only the buildings that are spread foundations and whose settlement was measured. The maximum settlement was about 250 cm, and many buildings settled more than 100 cm. Figure 16 shows the relationship between the settlement and the inclination. Though the data are scattered, settlement increased as the inclination angle increased. Figure 16 plots a similar relationship for Dagupan and Adapazari, but in the case of Adapazari, settlement was smaller than in the other two cities. The cause of this is not clear, but the liquefied soil in Adapazari contained many fine particles, and it may have been difficult to settle.

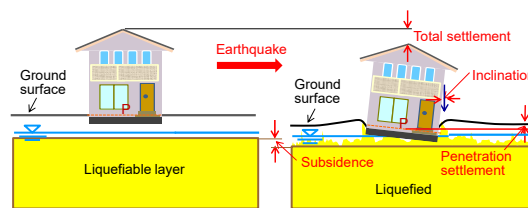


Fig. 14. Diagram of liquefaction-induced settlement of a house and the ground.

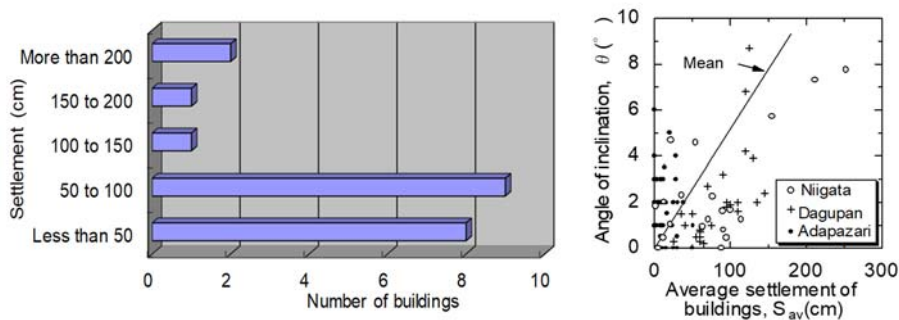


Fig. 15. Settlement of buildings in Niigata City [13]. **Fig. 16.** Inclination of buildings [13].

The liquefaction of ground beneath low-rise buildings with one or two floors has caused substantial damage all over the world. For example, in February 2011, such houses in Christchurch, New Zealand suffered a lot of damage, and in March 2011, the Tohoku Earthquake damaged about 27,000 houses. Low-rise houses suffered less subsidence and inclination due to liquefaction than mid-rise buildings and did not appear to be greatly damaged, so damage to low-rise houses has not received much attention in Japan. However, during the 2000 Tottori-ken Seibu Earthquake, it became clear that even a slight tilt would cause health problems, such as dizziness and nausea. Therefore, two months after the Tohoku Earthquake, the Cabinet Office provided new criteria, shown in Table 1, for judging the degree of damage from the amount of settlement and the angle of inclination. Consequently, the settlement and tilt angles of many houses in cities where liquefaction occurred were measured. Figures 17 (1) and 17 (2) show the

relationship between the amount of penetration settlement and the inclination angle measured in four cities (Yasuda [14]).

Table 1. New criteria for judging the degree of damage to a house.

Grade of damage		Totally collapsed	Large-scale half collapsed	Half collapsed	Partially damaged
Evaluation method	Inclination	> 50/1000	16.7/1000 to 50/1000	10/1000 to 16.7/1000	<10/1000
	Settlement	1m above the floor	Up to floor	25cm to the top of footing	

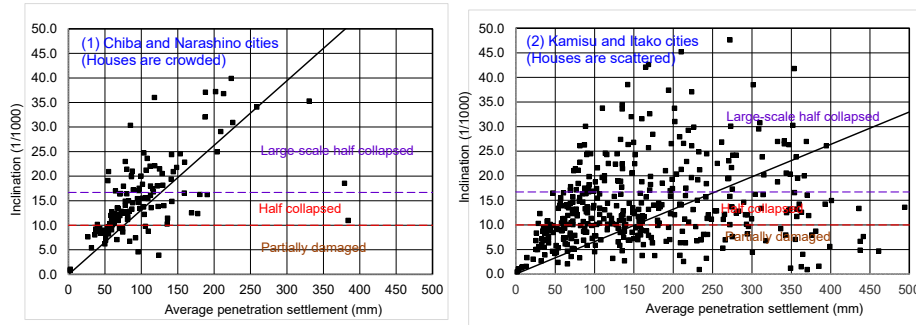


Fig. 17. Relationship between the penetration settlement and the inclination angle [14].

3.2 Shaking table tests to investigate the processes of settlement and tilting due to liquefaction

In order to investigate the process of settlement and inclination of low-rise houses due to liquefaction, many shaking table tests and analyses of disaster cases have been conducted. The timing of the start of settlement of buildings due to liquefaction during an earthquake was studied based on the testimony of eyewitnesses and on the results of a large shaking table experiment (Yasuda et al. [15]). According to Mr. Yuminamohi, who was at the airport terminal at the time of the 1964 Niigata Earthquake, about one and a half minutes after he felt the shaking, he heard a voice saying "The building is sinking!" He hurriedly started to take motion picture, and it was said that water began to spout from the side of the terminal building at Niigata Airport. In the shaking table test using a large soil container with a depth of 5 m, the ground liquefied after shaking for 30 seconds, and a concrete block of 1.86 tons in weight began to sink about 1 minute after the shaking ended. Therefore, it appears that the penetration settlement of a light, low-rise house does not start during the main shaking motion, but occurs slowly after liquefaction.

Regarding the process of subsidence due to liquefaction, an experiment was conducted by placing a large laminar soil container with the cross section shown in Fig.18 (1) on a large shaking table after the Tohoku Earthquake (Kaneko and Yasuda [16]). Figure 18 (2) and 18 (3) shows the time histories of excess pore water pressure for Case 2 of the experiment. Pore water pressure between the two house models and at GL-1.5m under the house model reached the maximum value in about 5 seconds, indicating liquefaction at these sites. On the other hand, at GL-0.3 m the house model did not generate excess pore water pressure until 5 seconds, and this pressure slowly increased

to the maximum value after 32 seconds. It is considered that the increase after 5 seconds was due to the propagation of excess pore water pressure from the lower layer. Figure 19 shows the time history of penetration settlement with photos at two points of time for Case 2. Based on case histories from past earthquakes and on model tests, the authors concluded that a structure such as a building or a house does not sink into a hole that was produced by spewing water, but penetrates the ground due to a decrease in the shear modulus of the surface layer following the outside lateral flow of the ground under the house and the heaving of ground surrounding the house, as schematically shown in Fig. 20. The ground surface settled slowly because the liquefied layer under and around the house densified gradually due the spewing of the pore water.

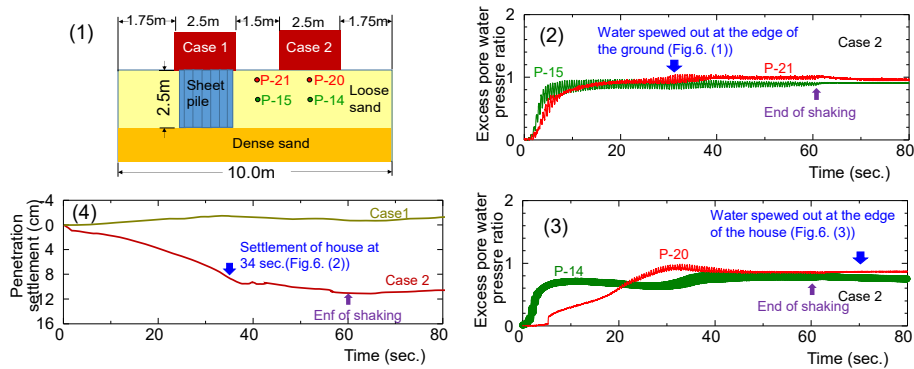


Fig. 18. Time histories of pore water pressure ratio and penetration settlement [16].

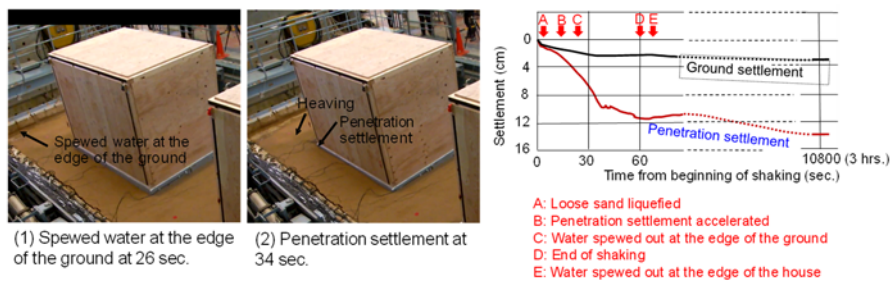


Fig. 19. Process of liquefaction, settlement and water spouting.

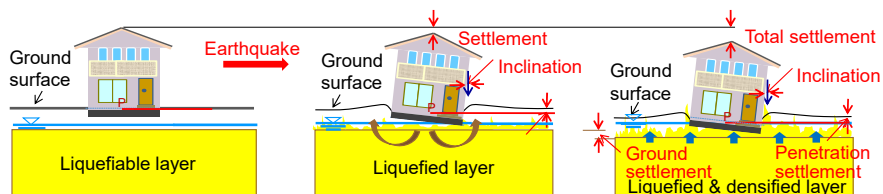


Fig. 20. Diagram of the mechanism of liquefaction-induced settlement of a house and the ground.

Regarding the process of tilting of houses in a densely populated urban area, there was a tendency to tilt inward when two houses were close to each other, as shown in Photo 2, taken after the 2000 Tottoriken-seibu Earthquake (Yasuda and Ishikawa [17]). However, when two houses were farther apart, they tended to tilt away from each other, as shown in Fig. 21. Figure 22 schematically illustrates these tendencies. These tendencies were also obtained by analysis. Figure 23 shows an example of analysis by ALID. The direction of inclination and the angle of inclination differed depending on the distance between the two houses. In addition, the analysis show that the groundwater level has a large effect on the inclination angle.



Photo 2. Adjacent houses tilted towards inside.

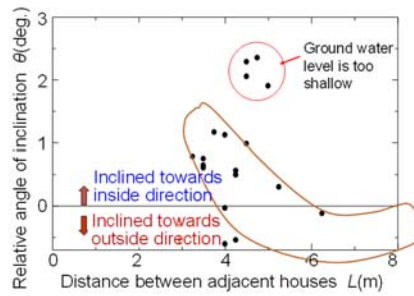


Fig.21. Relationship between L and θ [17].

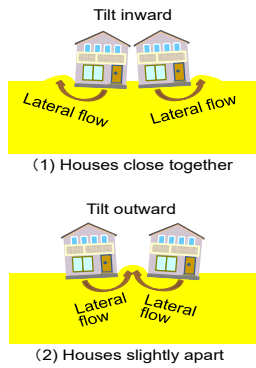
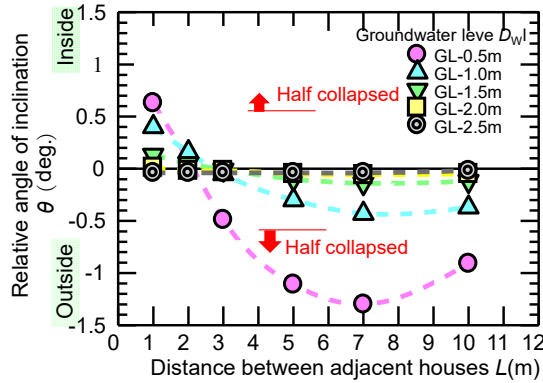


Fig. 22. Mechanism of tilting. Fig.23 Relationship among L , θ , and D_w analyzed by ALID [17].



3.3 Simple method to estimate the penetration settlement and inclination angle of low-rise houses

In Japan, although many low-rise houses were severely damaged by the Tohoku Earthquake, measures to prevent liquefaction damage are not being applied for new houses, for three main reasons: i) such measures are not required by law for low-rise houses, ii) the possibility of housing damage due to liquefaction is not fully recognized by residents and house makers, and iii) the probable damage to houses cannot be estimated

concretely from the liquefaction hazard maps currently being created. Therefore, as stated in Yasuda [18], the Ministry of Land, Infrastructure, Transport and Tourism issued a “Liquefaction Hazard Map Guide for Risk Communication” in order to promote the adoption of countermeasures. This guide suggests the following procedure to easily estimate the probable amount of penetration settlement and the angle of inclination of a house:

(1) The local government creates a liquefaction hazard map specialized for low-rise houses.

(2) Residents and house makers look at the hazard map, and if they are in a gray zone, conduct a simple soil investigation, such as screw weight sounding, to estimate the depth distribution of F_L .

(3) The house maker calculates the amount of penetration settlement of the house based on Stainbrenner’s method (Fig. 24), which calculates the settlement due to the weight of the house based on the elasticity theory. The shear rigidity of the liquefied layer is set according to F_L using Fig. 25 (1) used in ALID. However, while ALID assumes that the stress-strain relationship after liquefaction is bilinear, as shown in Fig. 25 (2), the elasticity theory can only consider a straight-line relationship with a slope of G_1 . Therefore, it was set that the decrease in shear rigidity due to liquefaction should be up to 1/300, so the amount of settlement does not become excessive.

(4) Estimate the inclination angle from the amount of settlement based on Fig. 17, depending on the density of the houses.

(5) Residents and house makers will judge the degree of likely damage based on Table 1 and take measures, if necessary.

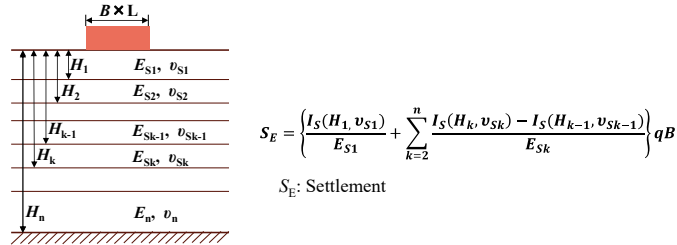


Fig. 24. Stainbrenner’s method to calculate the amount of penetration settlement.

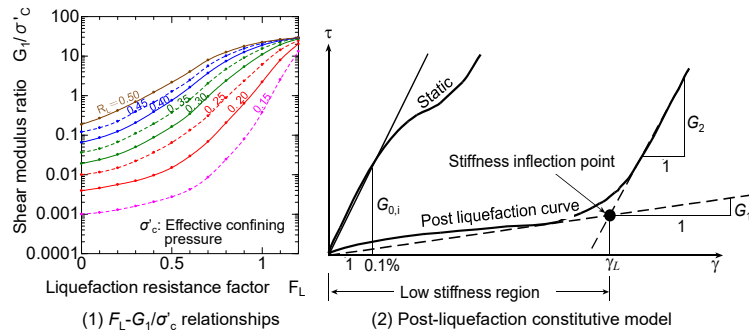


Fig. 25. Relationship among F_L , R_L and G_1/σ'_c [8].

4 Sewer manholes and pipes

4.1 Earthquake damage cases in which the amount of uplift of sewer manholes and pipes buried pipelines due to liquefaction was measured

Buried pipelines include water, sewer, gas, and telecommunications pipelines. Many sewer manholes and pipes are buried deeper than the groundwater level, and because the apparent unit volume weight of these pipes is lighter than the apparent unit volume weight of other buried pipes, they tend to float due to liquefaction. Liquefaction can also cause buried pipes to bend and to disconnect at their joints, but the following will focus on damage caused by the uplift of sewer manholes and pipes.

In Japan, the 1964 Niigata Earthquake and the 1983 Nihonkai-Chubu Earthquake caused uplift damage to buried water and gasoline tanks on liquefied ground. On the other hand, in peat ground that was not liquefied by the 1993 Kushiro-oki Earthquake, sewer manholes and sewer pipes were uplifted, as shown in Photo 3, which surprised Japanese researchers. When the damaged site was excavated and investigated, as shown in Photo 4, it became clear that the sand backfilled after excavating and installing manholes and pipes, as shown in Fig. 26, had liquefied. In the 2003 Tokachi-oki Earthquake, many manholes uplifted on peat ground. Of these, in Onbetsu Town, as shown in Fig. 27, the extent of manhole raising varied greatly, depending on the location, and while manholes along the B-B' survey line surfaced more than 60 cm, those along the A-A' survey line did not surface at all. When the cause was investigated, the former was clayey ground and the latter was sandy ground. The 2014 Niigata-ken-chuetsu Earthquake caused the uplift of about 1,400 manholes. Based on these cases, the authors consider the influence of the surrounding ground on the amount of rising, as shown in Fig. 28 (Yasuda and Kiku [19]). However, while the 2011 Tohoku Earthquake liquefied soil over a wide area in cities along Tokyo Bay, the manholes in this area did not rise much. Instead, as shown in Fig. 29 (1), prefabricated manholes were displaced horizontally, pipe joints were disconnected, and liquefied sand entered the pipes, causing severe damage. The author thinks that this is because the surrounding ground was also widely liquefied and the duration of the earthquake motion was long, so the liquefied ground continued to swing for 1 to 2 minutes (Yasuda et al. [20]). However, in places where the surrounding ground with cohesive soil was not liquefied, as shown in Fig. 29 (2), liquefaction of the backfill soil caused uplifting damage.



Photo 3. Uplifted sewer manhole in Kushiro Town. **Photo 4.** Investigation work.

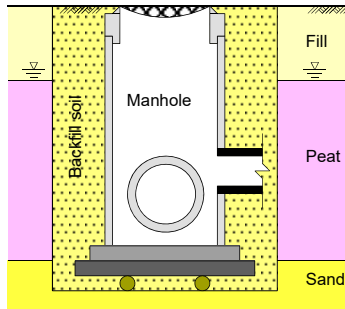


Fig. 26. Diagram of the cross section.

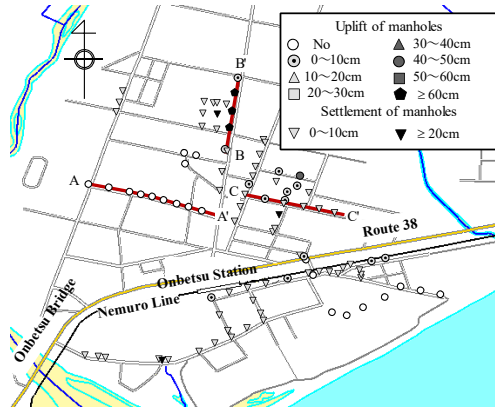


Fig.27. Uplift of manholes in Onbetsu Town [19].

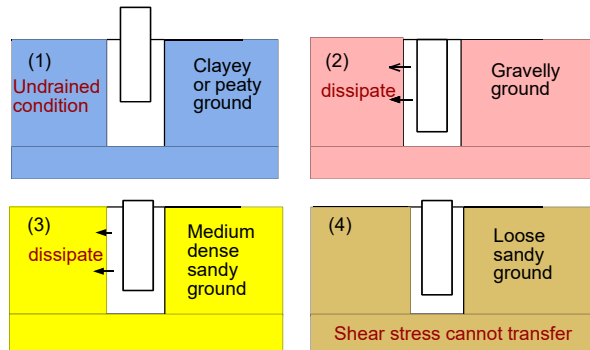
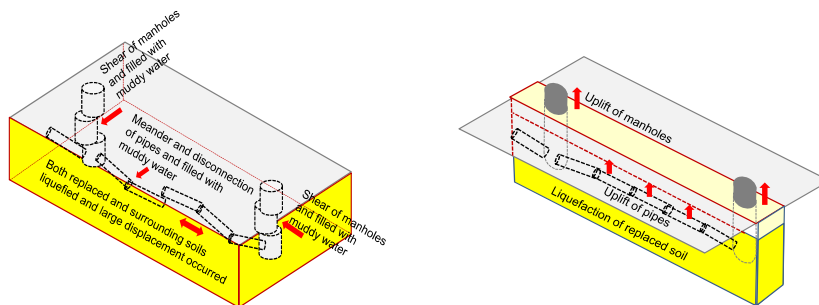


Fig. 28. Proposed relationship between uplift of a manhole and soil condition of ground [19].



(1) Damage due to the swing of liquefied ground (2) Damage due to the liquefaction of backfill soil

Fig. 29. Patterns of damage to manholes and pipes caused by the 2011 Tohoku Earthquake [20].

As shown in Photo 5, the Niigata-ken-chuetsu Earthquake caused damage, such as a car hitting a floating manhole. Therefore, the impact of a rise of sewer manholes on the lives of residents is not only that sewage water cannot flow after the earthquake, but also that the manholes become an obstacle to traffic. They can be a particularly serious obstacle when residents have to evacuate in an emergency due to a fire or tsunami

caused by an earthquake, or when emergency vehicles have to pass through. For example, when a hearing was held with the staff of the fire department, they answered that they can't pass if it rises from the road surface by about 10 cm to 20 cm as shown in Fig. 30.

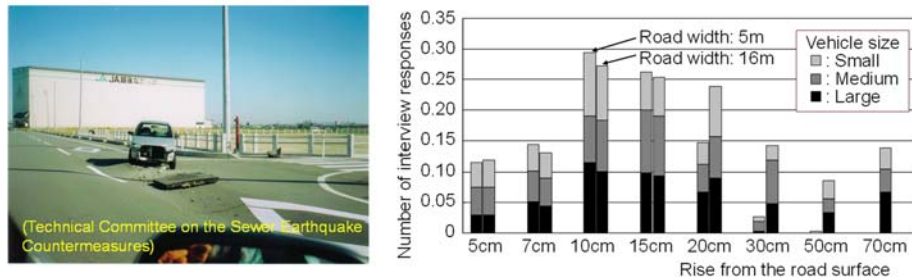


Photo 5. Car that collided with a manhole. Fig. 30. Levitation that a fire engine can pass through.

4.2 Shaking table tests to investigate the process of uplift of manholes and buried pipes due to liquefaction

After the Kushiro-oki Earthquake, the author and his colleagues conducted tests in which a small soil container, described schematically in Fig. 31, was placed on a shaking table and vibrated, in order to investigate the process of uplift of manholes and buried pipes [Yasuda and Kiku [19]]. Toyoura sand was used as the soil for the test, and pipes and manhole models were installed in the ground. There were two types of model ground: i) homogeneous ground with a relative density of about 30%, and ii) ground with only the backfill soil having a relative density of about 30% and the surrounding ground having a relative density of 90%, as shown in Fig. 31. In the latter case, the backfill soil was wrapped in a vinyl sheet so that the excess pore water pressure generated in the backfill soil would not propagate to the surrounding non-liquefied ground. Then, tests were conducted under many conditions by changing the width of the backfill soil, the groundwater level, the pipe diameter, and the thickness from the bottom of the manhole to the bottom of the backfill soil.

Since the front of this soil container was made of glass, “udon (Japanese soft noodle) chips” were installed at constant intervals between the glass and the ground. Then, the movement of soil particles was estimated from the displacement of the “udon chips”. The soil container was shaken with a large force of 3 Hz and 500 cm / s² so that liquefaction would occur in a few seconds. Figure 32 shows the time histories of the amount of rise when a pipe with a diameter of 11.4 cm was used and the width of the backfill soil was 25 cm, 30 cm, and 70 cm. Looking at the test results when the width was 70 cm, the pipe first rises a little with shaking (Stage 1), then the speed of uplifting slows down (Stage 2), and then it rises to the ground surface at once (Stage 3). Figure 33 shows the displacement amount of the udon chips up to the time points of Stage 2 and Stage 3. From these, it was considered that the uplift occurred according to the following process:

(1) First, the ground around the pipe was liquefied, and buoyancy was generated in the pipe, which floated about 5 cm at a rapid speed of about 4 cm / s. The uplift force of the pipe pushed the ground above the pipe upward, raising the ground surface a little. (Stage 1)

(2) When the soil on the pipe was pushed upward, it was sheared and deformed to generate a negative water pressure, and the shear rigidity of the liquefied soil was slightly restored. As a result, the uplift was temporarily stopped and the speed slowed down. (Stage 2)

(3) After that, excess pore water pressure from the surrounding soil propagated to the upper part of the pipe, and the soil above the pipe also returned to a liquefied state. For this reason, the soil on the upper part and sides of the pipe wrapped around the lower part of the pipe and floated it up at a speed of about 2 cm / s. (Stage 3)

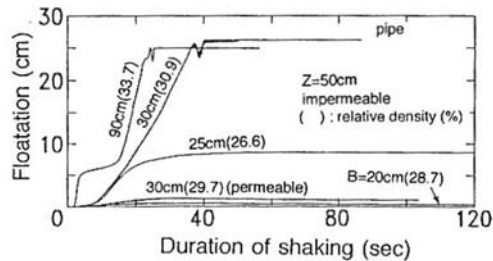
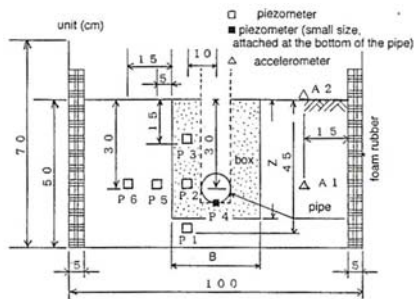


Fig. 31. Soil container used for tests [21]. Fig. 32. Time histories of the amount of rise of a pipe.

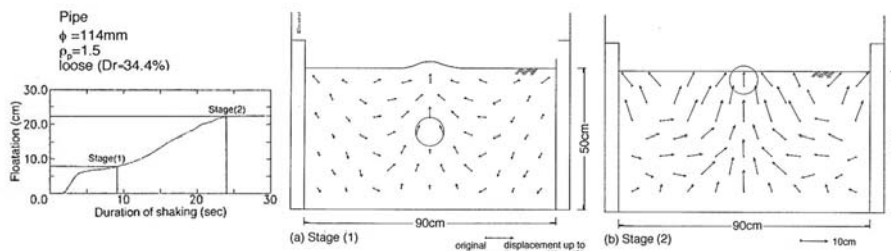


Fig. 33. Movement of “udon chips” up to the time points of Stage 2 and Stage 3.

When the width of the backfill soil was 30 cm, the speed of rising from the beginning was as slow as about 1 cm/s, and the pipe floated monotonously to the ground surface only in Stage 3 without Stages 1 and 2. Also, when the width became 25 cm, the pipe only floated about 8 cm and did not rise to the ground surface. Furthermore, in the test in which the vinyl sheet between the backfill and the surrounding dense ground was removed (“permeable data” in the figure), even if the width was 30 cm, the pipe hardly floated. It is considered that this was because the excess pore water pressure generated in the backfill soil was dissipated to the surrounding ground, as shown in Fig. 28 (3).

Figure 34 shows the time histories of the rise when the width of the backfill soil was 20 cm, 25 cm, and 30 cm using a model manhole. In each case, the manhole gradually

floated at an almost constant speed of 0.5 cm/s to 2 cm / s. The narrower the backfill soil, the slower the rise speed. The reason why there was no Stage 1 and 2 and only Stage 3 was because there was no soil on the manhole. The movement of the “udon chips” is shown in Fig. 35. The manhole floated as the surrounding soil wrapped under it. In addition, the size of pipes and manholes, soil density, backfill width, density of surrounding ground, degree of liquefaction, duration, etc. had an effect on the amount and speed of uplift of pipes and manholes.

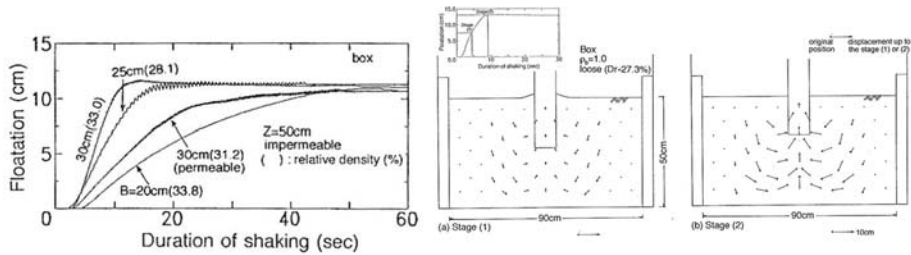


Fig. 34. Time histories of the rise for a manhole. Fig.35. Movement of “udon chips.”

4.3 Simple methods for estimating the amount of uplift for sewer manholes

The safety factor against uplift due to liquefaction F_S can be calculated by the balance between the weight of the structure, the frictional force with the ground, and the floating force. Figure 36 shows how to calculate F_S for sewer manholes and utility tunnels in Japan. This safety factor determines whether or not these structures float. However, the smaller this value, the smaller the amount of ascent may be. Koseki et al. [21] calculated the safety factor for manholes that uplifted due to the Kushiro-oki Earthquake and compared it with the observed uplift displacement. Figure 37 shows the result when the calculation is performed assuming that the backfill soil was liquefied. As can be seen in the figure, no clear relationship was found between the two.

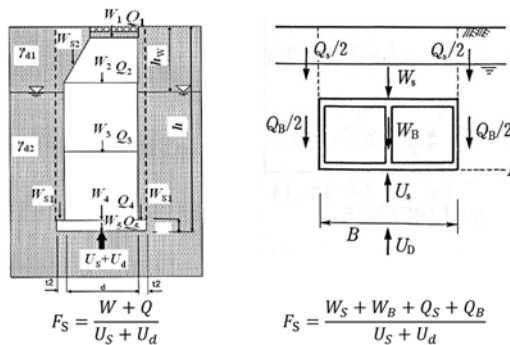


Fig. 36. Safety factor against uplift F_S in specifications for sewer manholes and utility tunnels.

In the case of sewer manholes, as described above, various factors affect the amount of uplift, and the connection of pipes to the manhole also reduces the amount of levitation. However, the maximum possible amount of uplift without these constraints can

be calculated from the balance between buoyancy and gravity. For manholes that were raised in Toyokoro Town during the 2003 Tokachi-oki Earthquake, the maximum amount of uplift was calculated from the manhole installation depth, groundwater level, unit volume weight of the ground, and the manhole's own weight and compared with their actual uplift displacements, as shown in Fig. 38. As can be seen in the figure, the amount of uplift that occurred was less than 1/4 of the calculated maximum amount of uplift.

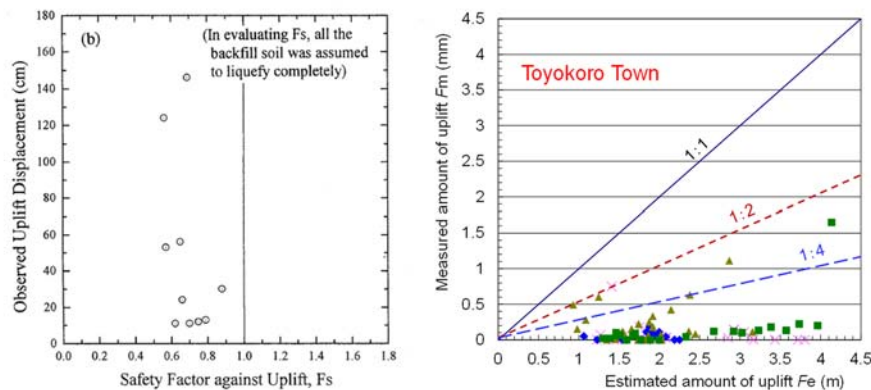


Fig. 37. Relationships between F_s s and uplift (Koseki et al. [21]). **Fig. 38.** F_e vs. F_m in Toyokoro.

Using this relationship of 1/4, the amount of uplift of a manhole in the Kitasenju district of Tokyo during a future earthquake was estimated. This area is one of the most densely populated residential areas in Tokyo, and there is a risk of fire in the event of an earthquake, and if a manhole rises, there is a risk of hindering the evacuation of residents and the passage of fire engines. Figure 39 shows the estimation results. The estimated amount of uplift differs depending on the area due to the influence of the groundwater level and the burial depth, but there are some areas where large uplift of about 60 cm is likely. In this district, the author and his colleagues created 250 m mesh soil profile models and created a liquefaction hazard map using the liquefaction potential index P_L (Yasuda and Ishikawa [22]). On the map, the areas where the manholes shown in Fig. 39 were calculated to rise by 60 to 80 cm are shown again by blue circles in Fig. 40. Comparing the two, the places where manholes were projected to rise by 60 to 80 cm have $P_L < 5$, and conversely, little rise was projected for areas of $P_L > 15$. In other words, the results were not related to the possibility of liquefaction of the ground. This is because Fig. 39 assumes floating due to liquefaction of the backfill soil. Therefore, it should be noted that the liquefaction of backfill soil may cause manhole floating damage even in zones that are judged not to be liquefied in the hazard map. In Japan, including the Kitasenju area, measures are being taken to prevent manhole floating, such as placing weights on the manholes and dissipating excess pore water pressure near them, for manholes buried in major roads.

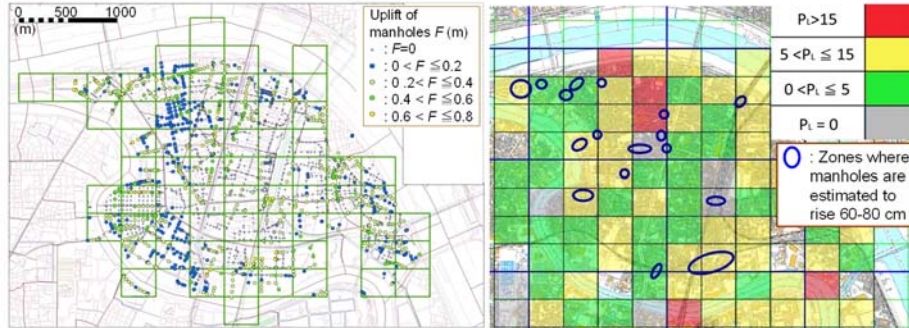


Fig. 39. Estimated uplift height of manholes.

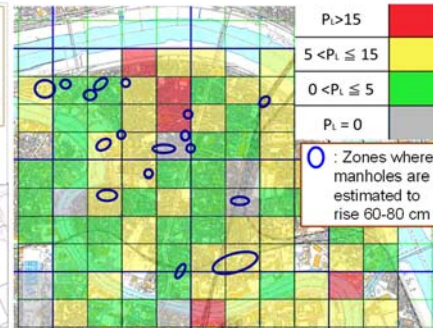


Fig. 40. Comparison with hazard map.

5 Concluding remarks

The current situation in which inspections are being conducted in Japan for areas that require countermeasures against liquefaction, targeting river dikes, railway embankments, low-rise houses, and sewer manholes is introduced. The Ministry of Land, Infrastructure, Transport and Tourism is inspecting river dikes and taking measures. As for low-rise housing, an inspection guide has just been created, and since residents and house makers are not aware of the damage caused by liquefaction, it is necessary to promote the adoption of countermeasures. Measures against uplift of manholes are being taken for existing sewer manholes, but inspection methods still need to be developed.

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