UPLIFT OF SEWER PIPES CAUSED BY EARTHQUAKE-INDUCED LIQUEFACTION OF SURROUNDING SOIL

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ABSTRACT

A series of shaking table tests are made to investigate the uplift behavior of sewer pipes caused by liquefaction of surrounding soil. It is shown that uplift displacement of pipes buried in unliquefiable original soil, which does not allow a dissipation of excess pore water pressure from liquefied backfill soil, increases drastically when they are subjected to extensive shaking. Uplift displacement of pipes buried in liquefiable original soil, on the other hand, accumulates rather gradually even at moderate shaking. A different behavior on residual deformation of the backfill and the original soils is also observed. In relation to the effects of soil condition, damage to sewer pipes in Towada City caused by the December 28, 1994 Sanriku-Haruka-Oki earthquake is reported and discussed. It is estimated that upper backfill soil consisting of sandy gravel may have liquefied because it was under loose and saturated condition and was subjected to earthquake motion under nearly undrained condition due partly to its own low permeability and also to the existence of surrounding original soil with low permeability.

Key words: case history, liquefaction, model test, (Sanriku-Haruka-Oki earthquake), (sewer pipe), (uplift) (IGC: E8/H8)

INTRODUCTION

Sewer pipes and manholes are usually constructed by the open-cut method, in which excavation is backfilled with sandy soil. This backfill soil is potentially liquefiable if it is loose and saturated below the ground water level. Similarly to other underground facilities having relatively light weight, sewer pipes are susceptible to uplift damage by earthquake-induced liquefaction of surrounding soil (e.g. Koseki et al., 1997a). Compared with other large or middle-scale underground facilities such as semi-buried roads and common utility ducts having a cross-sectional width of several meters or more, sewer pipes and manholes having a typical diameter of about one meter or less will be, therefore, relatively largely affected by the seismic behavior of the backfill soil. Effects of the seismic behavior of the original soil surrounding the backfill soil could also be significant when, for example, a sub-layer having some thickness liquefies as the backfill soil, or an impermeable layer prevents horizontal dissipation of excess pore water pressure generated in the liquefied backfill soil.

In order to evaluate the mechanism of uplift of buried pipes and manholes, shaking table tests have been conducted by several researchers (Ohtomo et al., 1987; Yasuda et al., 1995; Nagase et al., 1996). Effects of the backfill and the original soil condition have, however, not yet been well understood. A series of similar shaking table tests were also carried out by the authors (Koseki et al., 1997b) on several types of underground structures including sewer pipes and manholes. In these tests, model pipes did not uplift largely, due possibly to the fact that the input motion was increased step by step and the backfill soil was densified by previous shaking histories. Another series of shaking table tests were, therefore, conducted by reducing the number of shaking steps from five to three and also by changing the condition of the backfill and the original soils. A brief comparison between the results from the first and the second test series has been made elsewhere (Koseki et al., 1997c).

This paper describes in detail the results from the second test series, particularly on the effects of the seismic behavior of both original and backfill soils including their deformation modes. It reports that the test results

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can be interpreted satisfactorily based on safety factor against uplift, which was defined by Koseki et al. (1997b) based on the equilibrium of vertical force acting on the model pipe. It also reports typical damage to sewer pipes in Towada City, Aomori Prefecture, caused by the Sanriku-Haruka-Oki earthquake on December 28, 1994 and discusses factors affecting the damage, in particular the effects of seismic behavior of the surrounding soil layers.

PROCEDURES OF MODEL TESTS

A cross-section of the model used in the shaking table test is shown in Fig. 1; it consists of six models of underground pipes. The test condition for each model is summarized in Table 1. The geometrical scale of the model is about 1/6, and the considered prototype is a PVC (polyvinyl chloride) sewer pipe with a diameter of 30 cm,

Fig. 1. Cross-section of test model

Table 1. Model test condition

Case	Original soil	Backfill soil (D_r in %)	Foundation	Shaking step, 5 Hz sin. wave
M-9	Toyouura sand ($D_r=60\%$)	F sand (16)	—	First:
M-10		F sand (20)	Ladder	200 cm/sec ² , 20 cycles
M-11		Upper: G sand (15) Lower: F sand (15)	—	Second: 306 cm/sec ² , 20 cycles
M-12*	Toyouura sand ($D_r=98\%$)	F sand (47)	—	Final: 410 cm/sec ² , 100 cycles
M-13*		F sand (44)	Ladder	
M-14*		F sand (52) with large thickness below pipe	—	

* No transmission of pore fluid between original and backfill soils
F sand: Fine silica sand, G sand: Gravelly sand

buried at a depth of 180 cm from the ground surface. It should be noted, however, that any rigorous similitude with respect to the dynamic response and the migration speed of pore water of saturated soil was not employed. For simplification, the original and the backfill soils were saturated with normal water. The water level was adjusted to the ground surface.

In a rigid soil container 250 cm long, 60 cm high and 68 cm deep, a model deposit was prepared by using Toyoura sand (the maximum diameter $D_{\max}=0.42$ mm, the mean diameter $D_{50}=0.20$ mm, and the fines content $FC=0\%$). For cases M-9 to M-11, air-dried sand particles were pluviated through air to prepare relatively loose original soil layers at a D_r of about 60%. For cases M-12 to M-14, sand particles were pluviated through water and then densified to a D_r of about 100% by shaking.

A PVC tube with a diameter of 5.1 cm was buried at a depth of 30 cm; this was underlain by a wooden ladder foundation supported by a gravel mat ($D_{\max}$ =about 20 mm) for cases M-10 and M-13. The apparent unit weight of the pipe was controlled at 6.0 kN/m³ by putting lead shot inside. The space surrounding the pipe was back-filled by pluviating a fine silica sand ($D_{\max}=0.42$ mm, $D_{50}=0.11$ mm, and $FC=6\%$) through air (for cases M-9 to M-11 at a D_r of about 20%) or water (for cases M-12 to M-14 at a D_r of about 50%). In case M-11, a gravelly sand ($D_{\max}=26.5$ mm, $D_{50}=1.1$ mm, and $FC=6\%$) was placed loosely ($D_r=15\%$) as the upper half of the backfill; this sand had been retrieved from the upper backfill at a construction site for restoration work on damaged sewer pipes in Towada City, as will be described later, and had been sieved to remove grains larger than 26.5 mm. In case M-14, the backfilled space was made deeper without changing the depth of the pipe. In order to observe the deformation of the backfill soil, a colored sand was used as part of the backfill to form alternating horizontal layers (see Fig. 12).

For cases M-9 to M-11, rubber sheets were placed vertically in the middle of the original soil between different cases to prevent lateral migration of excess pore water pressure. For cases M-12 to M-14, the backfill soil was wrapped-around with rubber sheets keeping the top open; these were glued in a box shape to prevent the dissipation of excess pore water pressure from the loose backfill soil to the dense original soil. In relation to the effects of dissipation of excess pore water pressure, Yasuda et al. (1995) demonstrated that a model pipe with backfill soil wrapped-around with vinyl sheets uplifted largely, while another pipe with backfill soil without the vinyl sheets did not uplift.

For all the cases, horizontal shakings were applied by using a sinusoidal wave at a frequency of 5 Hz as input acceleration, where the amplitude of acceleration was increased from 200 cm/sec² to 410 cm/sec² in three shaking steps. The number of cycles was 20 in the first and second shaking steps, while it was increased to 100 in the final shaking step. Based on the authors' previous experiences, this shaking frequency was considered to be much lower than the natural frequencies of the tested

models at small strain levels which exclude the effects of liquefaction, although the former value was determined rather arbitrarily without measuring the latter values.

Effects of the seismic behavior of the original soil were investigated by comparing behaviors of cases M-9 to M-11 surrounded by relatively loose original soil, which allowed transmission of excess pore water pressure at their interfaces between the original and the backfill soils, with those of cases M-12 to M-14 surrounded by dense original soil, which did not allow the transmission of excess pore water pressure at the interfaces. Effects of the use of a wooden ladder foundation were studied by comparing the results between cases M-9 and M-10 (surrounded by relatively loose original soil), and between cases M-12 and M-13 (surrounded by dense original soil). The effects of backfilling by sandy gravel and of the vertical extent of the backfilled portion were examined, respectively, by comparing the behaviors of cases M-11 and M-14 with those of otherwise similar cases.

RESULTS OF MODEL TESTS

Excess Pore Water Pressure and Response Acceleration

Change of excess pore water pressure and response acceleration at several locations of the test models were monitored during and after each shaking step. The loca-

tions of transducers referred to in this paper are shown in Fig. 2.

Relationships between the input acceleration and the maximum excess pore water pressures observed at the bottom of the pipe in the backfill soil and at the same depth as the bottom of the pipe in the original soil are shown in Figs. 3 and 4. The transducers were located in the central section of the model except for case M-10, where the maximum excess pore water pressures measured in the backfill soil at front and rear ends of the bottom of the pipe were averaged and plotted in Fig. 3. The ranges of initial effective overburden pressures σ'_{v0} , which are estimated one-dimensionally by using the unit weight of the original soil for Fig. 4 and those of the backfill soil and the model pipe for Fig. 3, respectively, are also indicated in the figures.

Based on Fig. 3, it was estimated that the backfill soil liquefied completely, generating maximum excess pore water pressures larger than σ'_{v0} at the bottom of the pipe from the first shaking step for cases M-9 to M-11 (surrounded by relatively loose original soil), and from the second shaking step for cases M-12 to M-14 (surrounded by dense original soil). It should be noted that a sudden decrease in the maximum excess pore water pressure in the final shaking step was observed in the central section for case M-10 as shown in Fig. 5. This peculiar behavior was attributed to the existence of air near the diaphragm

Fig. 2. Location of transducers referred to in this paper

Fig. 3. Relationship between input acceleration and maximum excess pore water pressure in backfill soil

Fig. 4. Relationship between input acceleration and maximum excess pore water pressure in original soil

Fig. 5. Relationship between input acceleration and maximum excess pore water pressure in backfill soil for case M-10

of the pore pressure gauge, because the maximum excess pore pressures observed at front and rear ends of the pipe were larger than those observed at the central section even in the first and second shaking steps, as demonstrated in the same figure. The sudden decrease of the maximum excess pore pressure in the central section in the final shaking step may be due to additional intrusion of air into the pore pressure gauge from the surrounding soil, which liquefied during the previous shaking steps. The average of the maximum excess pore water pressures measured at the front and rear ends for case M-10 was, therefore, plotted in Fig. 3. It should be noted that such peculiar behavior was not seen in the maximum excess pore water pressures measured at other locations shown in Figs. 3 and 4.

As shown in Fig. 4, in all cases the maximum excess pore water pressures in the original soil at the same depth as the bottom of the pipe did not reach the range of σ'_{v0} . This was particularly true of the dense original soil for cases M-12 to M-14, in which the maximum excess pore water pressure did not reach half of σ'_{v0} , except at the location between cases M-13 and M-14 in the final shaking step. On the other hand, the maximum excess pore water pressures in the relatively loose original soil for cases M-9 to M-11 were about 70% to 90% of σ'_{v0} in all the shaking steps.

Ratios of the maximum response accelerations recorded in the original soil and at the top of the pipe, respectively, to the input acceleration are shown in Figs. 6 and 7. Time histories of these accelerations during the final shaking step are shown in Figs. 8 and 9. As shown in Fig. 6 and typically demonstrated in Fig. 8, remarkable amplification of the response accelerations of the relatively loose original soil for cases M-9 to M-11 occurred from the first shaking step, which was followed by a gradual reduction in their amplitude. On the other hand, no amplification or reduction was observed for cases M-12 to M-14 with the dense original soil. These responses suggest that the former soil liquefied partly, while the latter soil did not; this is qualitatively consistent with the behavior of excess pore water pressure as mentioned above,

Fig. 6. Relationship between input acceleration and response acceleration ratio of original soil

Fig. 7. Relationship between input acceleration and response acceleration ratio of pipe

Fig. 8. Time histories of response acceleration in original soil during the final shaking step

with the exception of the location between cases M-13 and M-14 in the final shaking step. Since the accelerometers in the original soils were buried 10 cm deeper than the pore pressure gauges as shown in Fig. 2, possible par-

Fig. 9. Time histories of response acceleration of pipe during the final shaking step

tial liquefaction of the upper part of the dense original soil in the final shaking step may not have been reflected in the accelerometer record.

In terms of the maximum response acceleration of pipes, no significant difference among the different cases was observed, as shown in Fig. 7. Overall response of pipes, however, was very different, as shown in Fig. 9; the responses of pipes for cases M-9 to M-11 (with the relatively loose original soil) decreased suddenly at an early stage of the final shaking, while those for cases M-12 to M-14 (with the dense original soil) did not show any reduction. Considering that in all cases, complete liquefaction in the backfill soil was estimated to have occurred during the final shaking step, it is likely that the overall response of pipes was rather dependent on the density of the original soil.

Uplift Displacement of Pipes

Vertical displacement of pipes was recorded by using displacement gauges fixed to the soil container and connected to the pipes at their central sections by steel wires. Relationships between the input acceleration and the cumulative uplift displacements of the pipes at the end of each shaking step are shown in Fig. 10. Residual uplift displacements of the pipes induced at each shaking step are plotted in Fig. 11. In the first shaking step, the pipes of cases M-9 to M-11 with the relatively loose original soil uplifted by about 5 to 15 mm, while those of cases M-12 to M-14 with the dense original soil did not uplift at all. As shown in Fig. 11, in the second shaking step, the pipes of the former cases uplifted approximately by the same amount as in the first shaking step, and those of the latter cases also uplifted slightly by 3 to 8 mm. In the final shaking step, all pipes suffered much larger uplift displacements than in the previous steps, due partly to the increased number of cycles (100 cycles instead of 20 cycles). Interestingly, the amount of uplift displacement was remarkably larger for cases with the dense original soil than for cases with the relatively loose one. This

Fig. 10. Relationship between input acceleration and cumulative uplift displacement of pipes

Fig. 11. Relationship between input acceleration and residual uplift displacement of pipes induced in each shaking step

peculiar response resulted in the largest cumulative uplift displacements by about 100 mm at the end of the final shaking step for cases M-13 and M-14, as shown in Fig. 10. Note that 100 mm is about 1/3 of the initial depth of the pipes. It should be noted, however, that the pipe for case M-14 suffered a residual rotation as will be shown in Fig. 12, which wound the steel wire and resulted in overestimation of the uplift displacement.

Deformation of Backfill Soil

Deformation of backfill soil at the central section is shown in Fig. 12, which was sketched by carefully removing the dewatered soil after the final shaking step. The following different trends of behavior were observed:

1) Case M-9:

The backfill soil was squeezed inwards by the loose original soil at a lower part, while it was stretched outwards near the surface. Vertical displacement of the backfill soil took place continuously, with distortion of soil below the pipe, in harmony with the uplift of the pipe.

2) Case M-10:

The overall behavior was almost similar to that of case M-9, except that a portion of the backfill soil which was located just above the pipe penetrated into the overlying

Fig. 12. Observed deformation of backfill soil at the central section after the final shaking step

soil at an angle of about 30 degrees measured from the vertical direction. The wooden ladder foundation uplifted slightly.

3) Case M-11:

Compared with case M-9, the extent of stretching of the upper backfill soil consisting of gravelly sand and the vertical displacement of the lower backfill soil were both relatively small.

4) Case M-12:

The outer shape of the backfill soil was almost unchanged, while it was deformed internally in accordance with the uplift of the pipe; the soil above the pipe was largely distorted. On the contrary, the soil below the pipe was not disturbed.

5) Case M-13:

The backfill soil above the pipe was deformed in such a manner that the pipe punched out the overlying soil. In contrast to case M-10, the wooden ladder foundation was found at its initial location.

6) Case M-14:

The tendency of the overlying backfill soil to be punched out by the pipe was the most remarkable, and was accompanied by the downward movement of lateral soil. In addition, the soil at a depth just below the pipe was slightly disturbed.

In Fig. 12, relative settlements of the surface of the backfill soil and the surrounding original soil are also

shown. It should be noted that the surface of the backfill soil exhibited a relative upheaval for cases M-9 to M-11 with the relatively loose original soil, while it exhibited a relative subsidence for cases M-12 to M-14 with the dense original soil.

DISCUSSIONS ON MODEL TESTS

Effect of Seismic Behavior of Original Soil

A different tendency was observed in Figs. 10 to 12 with respect to the effect of original soil on the uplift behavior of the pipes. For cases M-9 to M-11 where the original soil was liquefiable, the residual uplift displacement of the pipes accumulated gradually, and the surface of the backfill soil exhibited a relative upheaval. For cases M-12 to M-14 where the original soil was unliquefiable and the dissipation of excess pore water pressure from the backfill soil to the original soil was prevented by the rubber sheets, the pipes suffered sudden large uplift displacement during the final shaking step, and the surface of the backfill soil exhibited a relative subsidence. The sudden increase in uplift displacement for the latter cases may have been related to the difference in the response acceleration of pipes as shown in Fig. 9, where the pipes, possibly together with the completely liquefied backfill soil around them, for cases M-12 to M-14 were forced to vibrate horizontally by the surrounding unliquefiable original soil.

Based on these observations, the following behavior is expected for actual sewer pipes.

- 1) Pipes buried in liquefiable original soil will uplift relatively easily when an earthquake motion is strong enough to initiate the liquefaction of the original soil. This may cause overall uplift failure of pipes in an affected area with similar ground conditions.
- 2) Pipes buried in unliquefiable original soil may not uplift largely even if the backfill soil liquefies due to a moderate earthquake motion. However, if they are subjected to an extensive earthquake motion when, for example, the permeability of the original soil is very low, the uplift displacement of the pipes will increase drastically. This may cause a wide variation in the uplift displacement in the longitudinal direction of the pipes if the level of the earthquake motion is marginally large enough to trigger the uplift.
- 3) Occurrence of liquefaction in the original soil may be detected by comparing the surface level of the backfill soil with that of the surrounding original soil; if the backfill soil is relatively upheaved, then the original soil may have liquefied; otherwise, only the backfill soil may have liquefied.

Effect of Backfilling Condition

With respect to the backfilling condition, attentions should be paid to the following based on Figs. 10 to 12.

- 1) When compared to case M-9, it is seen that the existence of gravelly sand in the upper part of the backfill soil for case M-11 reduced the uplift displacement of the pipe by about 50% in the final shaking step.

Its effect was also prominent in the extent of deformation of the backfill soil. In order to reduce uplift damage to pipes, it is desirable to replace loose backfill sand, which is potentially liquefiable if it is saturated, to as large an extent as possible with a material having a high resistance to liquefaction and deformation, such as gravelly soil and cement-treated soil.

- 2) When compared to case M-12, it is seen that backfilling of the deeper portion with loose sand for case M-14 resulted in larger uplift displacement of the pipe, which may have been caused by the slight deformation of backfill soil associated with its liquefaction at a depth just below the pipe. When excavation for pipes is to be backfilled with liquefiable soil, it is recommended that the depth of excavation be minimized in order to reduce the uplifting damage to pipes.
- 3) When compared to cases M-9 and M-12, respectively, it is seen that the existence of the wooden ladder foundation in cases M-10 and M-13 did not reduce the uplift displacement, and it did not affect the deformation mode of the backfill soil either. It should be noted, however, that the pipes supported with a wooden ladder foundation were located in the middle part of the soil container where the effect of rigid end walls to constrain the cyclic shear deformation of the soil was relatively small. In the present model tests, this effect may have counterbalanced the potential effect of the wooden ladder foundation to reduce the uplift displacement of the pipes. In order to evaluate the seismic behavior of actual pipes, the latter effect should be reexamined independently in future studies.

Safety Factor against Uplift

The equilibrium of vertical forces acting on the pipe and the overburden soil block (Koseki et al., 1997b) is schematically shown in Fig. 13. The safety factor F_s of pipes against uplift was defined with an assumption of vertical slip surfaces in the overlying soil, as indicated in

Fig. 13. Definition of safety factor against uplift and assumed vertical slip surfaces in overburden soil block

the same figure. The assumption was shown to be reasonable when the pipe punched out the overlying soil upwards typically for cases M-10, M-13 and M-14 as shown in Fig. 12. Changes of F_s during and after shaking were evaluated by using the excess pore water pressure measured at the bottom of the pipe and in the underlying soil layer. The seepage force F was found to be zero or negligibly small, since the degree of liquefaction was more extensive in the backfill soil than in the underlying original soil. For simplicity, the frictional resistance Q was set to zero because the surrounding backfill soil was estimated to have liquefied nearly totally.

Relationships between F_s and the observed uplift displacements of the pipes in the central section are shown in Figs. 14, 15 and 16 during the first, the second and the final shaking step, respectively. Relations for case M-10 are not included, because measurement of the excess pore water pressure at the bottom of the pipe may have been incorrect, as mentioned before. In each step, uplift was triggered when F_s became between 0.8 and 1.0. This behavior was consistent with previous model test results reported by Koseki et al. (1997b). It was reconfirmed that this safety factor can be used to evaluate whether the uplift of buried pipes will be triggered or not.

It should be noted that when the bottom of the uplifting pipe is in contact with the underlying soil layer having positive effective vertical stress, F_s should be larger than 1.0 in principle, because upward force due to the effective vertical stress in the soil skeleton beneath the pipe is not considered in its definition. The effective vertical stress in the liquefied backfill soil may, however, be negligibly small. In addition, the actual value of Q in the overlying soil may not be zero, due possibly to mobilization of shear resistance along the assumed vertical slip surfaces by the uplifting movement, all of which results in un-

Fig. 14. Relationship between safety factor against uplift and uplift displacement of pipes in the first shaking step

Fig. 15. Relationship between safety factor against uplift and uplift displacement of pipes in the second shaking step

Fig. 16. Relationship between safety factor against uplift and uplift displacement of pipes in the final shaking step

Fig. 17. Relationship between minimum safety factor against uplift and residual uplift displacement of pipes

derestimation of F_s during uplift of the pipes.

In order to examine possible correlation, the minimum safety factor $F_{s_{min}}$ against uplift during each shaking step,

and the amount of residual uplift displacement were plotted in Fig. 17. It was demonstrated that $F_{s_{min}}$ was smaller than 1.0 when the pipe suffered residual uplift. On the other hand, the amount of uplift was not clearly related to the value of $F_{s_{min}}$ but was rather dependent on the number of cycles of the input acceleration.

TYPICAL DAMAGE TO SEWER PIPES IN TOWADA CITY CAUSED BY THE SANRIKU-HARUKA-OKI EARTHQUAKE

On December 28, 1994, the Sanriku-Haruka-Oki earthquake with a magnitude of 7.5 on the JMA (Japan Meteorological Agency) scale hit Aomori and Iwate Prefectures, which are located in the north-eastern part of Japan (Fig. 18). It was accompanied by the largest aftershock with a JMA magnitude of 6.9 on January 7, 1995.

Damage to sewer pipes caused by these earthquakes is reported to have occurred in Hachinohe City and Towada City and along Towada Lake (HIT, 1995 and JGS, 1996). Among these, the damage suffered in Towada City in Aomori Prefecture is of particular importance, because the upper portion of the backfill soil consisted of sandy gravel, which may be considered to be less liquefiable according to the aseismic design specification (JSWA, 1981) that was current at the time of construction.

In Towada City, sewer pipes were damaged at 18 sites as listed in Table 2; these were located in the eastern area

Fig. 18. Location of Towada City and epicenters of the main shock and the largest aftershock of the 1994 Sanriku-Haruka-Oki earthquake

Table 2. Sewer pipes in Towada City damaged by the 1994 Sanriku-Haruka-Oki earthquake

No.	Location	Pipe type, diameter, damaged length (m)	Foundation type, width of excavation (m)	Thickness of overburden soil (m)
1	V	PVC, 0.2, 90	A, 0.89	2.6 to 3.5
2	VI	PVC, 0.2, 45	A, 0.89	3.8 to 4.1
3	VI	Clay, 0.2, 84	B, 0.74	1.7 to 2.9
4	III	Clay, 0.2, 232	B, 0.74	1.1 to 3.2
5	III	Clay, 0.2, 146	B, 0.74	2.5 to 3.6
6	II	Clay, 0.2, 65	B, 0.89	3.4 to 4.1
7	II	PVC, 0.2, 37	A, 0.74	2.6 to 2.7
8	II	PVC, 0.2, 41	A, 0.74	1.9 to 2.3
9	I	PVC, 0.25, 116	A, 0.91	1.5 to 1.7
10	I	RC, 0.4, 138	C, 0.89	2.7 to 3.1
11	IV	RC, 0.3, 97	B, 0.91	1.7 to 2.2
12	I	Clay, 0.2, 47	B, 0.91	1.6
13	II	Clay, 0.15, 76	B, 0.91	1.5 to 2.2
14	III	PVC/Clay, 0.2, 108	A/B, 0.74	2.1 to 3.7
15	III	Clay, 0.2, 49	B, 0.74	1.9 to 2.4
16	III	PVC, 0.2, 34	A, 0.91	1.9 to 2.0
17	V	RC, 0.4, 44	B, 0.89	2.8 to 3.2
18	VI	Clay, 0.2, 89	B, 0.91	1.7 to 2.2

PVC: Polyvinyl chloride pipe

Clay: Vitrified-clay pipe

RC: Reinforced precast-concrete pipe

* Refer to Figs. 19 and 21 for location and foundation type

of the city as shown in Fig. 19. In this area, Kuroboku soil or andosol, which is an organic-volcanic ash soil known locally as "Shikibo," was predominant as the original soil near the surface. A typical soil profile estimated along line A in Fig. 19 is shown in Fig. 20, where the Kuroboku soil (denoted as Kb) overlies Holocene

Fig. 19. Network of sewer pipes in Towada City and location of affected area **Fig. 20. Typical soil profile in the affected area (along line A in Fig. 19)**

sandy soils (As2 and As3) partially containing pumice or volcanic ash, Pleistocene volcanic sandy soil (Dvs) and Pleistocene volcanic clayey soil (Dvc). The Kuroboku soil is usually underlain by artificial fill (F) and Holocene sandy soil (As1), which are mostly above the ground water level. At borehole No. H3-2, a thin layer of peat (Ap) is sandwiched between the Holocene sandy soil layer (As2).

Standard cross-sections of sewer pipes in Towada City are shown in Fig. 21. They were laid on a bed of crushed stones with a thickness of 150 mm or on a wooden ladder foundation. To backfill the upper part, sandy gravel ($D_{\max}=53$ mm, D_{50} =about 5 mm, FC =about 4%) was chosen from an economical viewpoint. On the other hand, to backfill the lower part up to a depth of 300 mm above the top of the pipe, coarse sand ($D_{\max}=19$ mm, D_{50} =about 1 to 2 mm, FC =about 5%) was used instead of the sandy gravel in order not to damage the pipes during compaction of the backfill soil. Typical grain size distribution curves of these upper and lower backfill soils together with those of soils used for the aforementioned model tests are shown in Fig. 22.

In the earthquake, the pipes suffered residual uneven displacement (Photo 1), which was mainly caused by up-lifting of pipes at free sections and by constraint of uplift-

Fig. 21. Standard cross-sections of sewer pipes

Fig. 22. Grain size distribution curves of backfill soils for sewer pipes in Towada City and soils used for model tests

ing at sections connected to manholes and branch pipes (Photo 2). Failure or opening of joints also occurred (Photo 3). They were accompanied by residual settlement at the surface of the backfill soil which amounted to about 300 mm at largest (Photo 4), while there was no residual settlement at the surface of the original soil surrounding the backfill soil (Photo 5). During reconstruc-

Photo 1. Residual uneven displacement of reinforced precast-concrete pipes (part of site No. 11 in Table 2, by courtesy of Towada City)

Photo 2. Uplift of polyvinyl chloride pipes constrained partly by branch pipe (site No. 7 in Table 2, by courtesy of Towada City)

tion work after the earthquake, it was observed that some of the pipes uplifted into the region of the upper backfill soil consisting of sandy gravel as typically shown in Figs. 23 to 25. On the other hand, the horizontal residual displacements of these pipes, which had been buried

Photo 3. Opening of joint for vitrified-clay pipes (site No. 6 in Table 2, by courtesy of Towada City)

Photo 4. Residual settlement of backfill soil (site No. 6 in Table 2, by courtesy of Towada City)

Photo 5. Ground surface condition and ejected manhole by settlement of backfill soil (site No. 6 in Table 2, by courtesy of Towada City)

Fig. 23. Elevation and horizontal alignment of reinforced precast-concrete pipes (part of site No. 11 in Table 2)

straight with respect to their horizontal alignments before the earthquake, were smaller than the vertical ones as shown in these figures.

DISCUSSIONS ON THE DAMAGE TO SEWER PIPES

The original soil surrounding the damaged pipes at a depth about 2 to 3 m from the ground surface consisted mainly of Kuroboku soil (Kb, organic-volcanic ash soil) and volcanic sandy soils with an SPT N -value of about 10 or larger (As3 and Dvs) as typically shown in the downstream side of Fig. 20. They can be regarded as non-liquefiable and less-liquefiable, respectively. This inference is consistent with the observation that residual settlement did not occur at the surface of the original soil as typically shown in Photo 5. In addition, residual settlement of up to 300 mm at the surface of the backfill soil may be explained by the fact that the upper backfill sandy gravel liquefied as well as the lower backfill sand. It is not reasonable to assume that only the lower layer with a total thickness of 450 to 700 mm at the side of the pipe liquefied and reduced its volume by 43 to 66% due to post-liquefaction drainage, causing such a large settlement at the surface.

“Masa” soil (granite-origin sandy soil), which was used for reclamation work to construct Port Island in

Fig. 24. Elevation and horizontal alignment of polyvinyl chloride pipes (site No. 7 in Table 2)

Fig. 25. Elevation and horizontal alignment of vitrified-clay pipes (site No. 6 in Table 2)

Kobe City, Hyogo Prefecture, is considered to have liquefied during the 1995 Hyogoken-Nambu earthquake. The range of the grain size distribution of Masa soil is shown in Fig. 26 (after Yasuda et al., 1996) together with that of reclaimed soil at Rokko Island. The grain size distribution curves of the sandy gravel used as upper backfill and the sand used as lower backfill in Towada City, as

Fig. 26. Grain size distribution curves of reclaimed soils in Port Island and Rokko Island, Kobe City (after Yasuda et al., 1996)

shown in Fig. 22, were roughly similar to those of the Masa soil. It can be inferred, therefore, that relatively loose and saturated deposits of these backfill soils may have liquefied when subjected to the earthquake motion under nearly undrained condition, as is the case for usual loose clean sands.

In order to evaluate the drainage condition for the upper backfill sandy gravel, permeability tests were conducted using the rest of the gravelly sand. The sample was retrieved from the upper backfill and was sieved to eliminate larger grains as mentioned earlier. A specimen of 100 mm in diameter and 126 mm in height was prepared by compaction using a vibrator and saturation using carbon dioxide and deaired water. Because the in-situ density of the upper backfill soil was not known, the dry density of the specimen was set arbitrarily to about 1.5 g/cm^3 , which corresponded to a D_r of about 60%. The results showed that the coefficient of permeability k corrected for water temperature at 15 centigrade was as low as $3.7 \times 10^{-3} \text{ cm/sec}$, which may have been due to the existence of fines about 6% in weight as shown in Fig. 22. This result suggests that the backfill soils were subjected to the earthquake motion under nearly undrained condition. In addition, in-situ permeability test results for the original soils surrounding the damaged sewer pipes are shown in Fig. 20. These permeability values were almost as low as or even lower than the aforementioned value estimated for the upper backfill soil. This result excludes the possibility of the excess pore water pressure generated in the backfill soils to dissipate into the surrounding original soils. The impermeable sheets used to prevent flowout of the backfill soil, which were used for pipes with foundation type A as shown in Fig. 21, may also have affected the performance of the pipes in terms of maintaining the undrained condition during the earthquake for the lower backfill sand.

It should be noted, however, that coarse and well-graded soils such as this sandy gravel with uniformity coefficient U_c of about 10 are reasonably expected to be much more compactible than sands that are conventionally used as backfill materials. In using such coarse and well-graded soils for backfilling of pipes, great care

should be taken to compact them thoroughly, while preventing any damage to the pipes during the compaction work.

CONCLUSIONS

Results of the model tests performed in this study can be summarized as follows;

- 1) When the original soil liquefied, the residual uplift displacement of the pipes accumulated gradually, and the surface of the backfill soil exhibited a relative upheaval.
- 2) When the original soil did not liquefy and the dissipation of excess pore water pressure from the backfill soil to the original soil was prevented, the pipes suffered sudden large uplift displacement at relatively large shaking amplitude, and the surface of the backfill soil exhibited a relative subsidence.
- 3) Uplift displacement of the pipes was reduced by backfilling the upper part with gravelly sand, while it was increased by backfilling to a deeper portion with loose fine sand. The effect of the wooden ladder foundation beneath the pipes could not be clearly demonstrated.
- 4) Uplift was triggered when the safety factor F_s against uplift, which was defined based on the equilibrium of vertical forces acting on the buried pipe and the overburden soil block, was reduced to about 0.8 to 1.0.

Based on these results, a different behavior is expected for actual sewer pipes with respect to the effect of original soil on the uplift characteristics of the pipes; pipes buried in liquefiable original soil may suffer overall uplift failure, while pipes buried in unliquefiable original soil may suffer uplift failure with a wide variation in the uplift displacement if the level of the earthquake motion is marginally large enough to trigger the uplift.

On the use of coarse soils as backfill material, investigation into the damage to sewer pipes in Towada City by the 1994 Sanriku-Haruka-Oki earthquake revealed another feature:

- 5) Based on the large amount of residual settlement at the surface of the backfill soil, the upper backfill sandy gravel, which was possibly loose and subjected to earthquake motion under nearly undrained condition, was judged to have liquefied as well as the lower backfill sand. This inference was supported by the grain size distributions which were similar to "Masa" soil at Port Island, Kobe City, by the low permeability, and by the existence of low permeable layers as surrounding original soil.

This case history warns us that, in using coarse and well-graded soils as preferable backfill material in terms

of the reduction of liquefaction induced damage to pipes, great care should be taken to compact them as thoroughly as possible.

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