State-of the-art of Stability Assessment of Underground Structures during Liquefaction

by

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ABSTRACT

Soil liquefaction often induces damage to underground structures. The most typical mode of the liquefaction-induced damage is uplift displacement of the structures. The paper presents the state-of-the-arts of the problem. First, case records on damage to underground structures induced by soil liquefaction are briefly reviewed. Then, the stability assessment techniques that are used in practice and that was developed before/after the 1995 Hyogoken-Nanbu earthquake are introduced, and the research issues are summarized. In the last part of the paper, a recent research works is described to study the mechanism of the damage and to improve the currently used stability assessment techniques.

KEY WORDS: Liquefaction
Underground Structure
Uplift

1. INTRODUCTION

During past earthquakes, underground structures have suffered damages caused by liquefaction of surrounding soils, which include sewerage treatment ponds and pipes, petroleum storage tanks, etc. The most typical mode of the damage is uplift movement. In Japan, a stability assessment procedure for the uplift behavior was developed in 1986 (Japan Road Association, 1986) and the method has been used widely with slight modification. The method, however, is based on limit equilibrium and thus cannot evaluate the extent of damage or magnitude of uplift displacement.

Since the occurrence of the Hyogoken-Nanbu earthquake of 1995, a consensus has been developed that for any civil engineering structures the Level 2 earthquake, which is conceptually defined as the possible maximum earthquake, should be considered in the design, and that the design should be based on seismic performance. From this viewpoint, the stability assessment procedure mentioned above has limitations and requires to be elaborated.

In this paper, case records on damage to underground structures induced by soil liquefaction during past earthquakes are briefly reviewed. Then, the stability assessment techniques that are used in practice and that was developed before/after the 1995 Hyogoken-Nanbu earthquake are introduced and the research issues related to the methods are summarized. In the last part of the paper, a recent research works is presented to study the mechanism of the damage and to improve the currently used stability assessment techniques.

2. CASE HISTORIES

In Japan, several types of underground structure having relatively light unit weight were damaged by liquefaction of subsoils during past earthquakes. Case histories on the damaged underground structures in Japan are briefly summarized in Table 1. The observed modes of damage were uplift, settlement, and extrusion

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and distortion at pipe joint, among which liquefaction-induced uplift is the most typical mode of damage to underground structures. Uplift distortion occurred to sewage treatment pipelines, ponds and petroleum tanks during the 1964 Niigata earthquake (PWRI 1965), the 1983 Nihonkai-Chubu earthquake (Taniguchi and Morishita 1985), the 1993 Kushiro-Oki earthquake (Koseki et al. 1997, Japanese Society of Soil Mechanics and Foundation Engineering 1994), the 1993 Hokkaido-Nansei--Oki earthquake (PWRI 1994), the 1994 Hokkaido-Toho-Oki earthquake (JGS 1998) and the 1994 Sanriku-Haruka-Oki earthquake (Koseki et al. 1998a). During the 1995 Hyogoken-Nanbu earthquake, the pile foundation of the Higashinada sewage treatment plant suffered serious damage caused by lateral flow of liquefied ground. In this section, case records on damage to underground structures induced by soil liquefaction are briefly reviewed.

(1) Damage to sewage treatment tanks during the 1964 Niigata earthquake

Fig.1 shows a typical damage to a sewage treatment tank during the 1964 Niigata earthquake. The tank located at Zoshobori, Niigata City, left of the Shinanogawa River, suffered severe uplift distortion induced by subsoil liquefaction and broke at the construction joint. The mechanism of the damage would have been as follows. The tank was subjected to a buoyant force of the liquefied soil. The left half segment had a maintenance house on it, thus the center of self weight was eccentric to the house side. Accordingly, the left half segment was pushed up unevenly, leading to opening of the construction joint. The similar damage was reported for the Koshinanogawa and Kawabatacho sewage pomp stations as well.

Sewer pipes were also suffered severe damage by this earthquake. A total length of the damaged pipes amounted to about 31 km, which were about 60 % in length of all the sewer pipes in Niigata City. Damage modes were uplift, settlement of underground structure cased by ground subsidence after liquefaction and extrusion and displacement of pipe at joints. The maximum uplift displacement in the damaged sites ranged from 10 cm to 2 m.

(2) Damage to underground fuel tank during the 1983 Nihonkai-Chubu earthquake

Eighty underground fuel tanks and two underground water tanks were suffered uplift damage during the 1983 Nihonkai-Chubu earthquake in Aomori prefecture and Akita prefecture. Most of the damaged sites were located in flood plain. Maximum uplift displacement of the damaged tanks was about 2 m, which is shown in Fig. 2 The ground surface of surrounding soil around the fuel tank subsided about 70 cm. Fig.3 shows grain size distributions of sand boils or backfill soils retrieved around the damaged sites. The range of potentially liquefiable distributions was also shown in Fig.3.

(3) Damage to sewer manholes during the 1993 Kushiro-Oki earthquake

During the 1993 Kushiro-Oki earthquake, damage to sewer pipes took place in Kushiro City, Kushiro Town, Shibecha Town. During the 1994 Hokkaido-Toho-Oki earthquake, that hit the same area, sewer pipes were damaged in Kushiro City, Kushiro Town, Shibetsu Town. In Kushiro City, sewer pipes of a total length of about 19 km were damaged by the two earthquakes, which corresponded to about 2 % of all the sewer pipes in length. In Shibetsu Town, those of 10 km were damaged by the Hokkaido-Toho-Oki earthquake, which corresponded to about 56 % of all the sewer pipes. Especially, during the 1993 Kushiro-Oki earthquake, several sewer manholes in Kushiro Town were ejected up to 1.5 m (see Fig.4).

In order to evaluate their failure mechanism, the level and boring surveys, cut-off investigation were carried out on the sewer manholes in Kushiro Town, which ejected up during the 1993 Kushiro-Oki earthquake. Liquefaction resistance ratio F_L of backfill and original subsoil was evaluated by using the SPT N-values and other soil parameters. The location of the area surveyed is shown in Fig.5. Typical soil profile and uplift displacements are shown in Figs.6 and 7, respectively, which were estimated based on the results of an observation of the soil profile during the restoration work and the SPT N-values measured. Based on the results of the surveys and analyses of liquefaction resistance ratio F_L, backfill soil of the manholes was estimated to have liquefied extensively and to have moved laterally toward the bottom of the ejected manholes as shown in Fig.8 (a). In the original subsoil surrounding the manholes which were ejected by more than 50 cm, relatively clean sands with a fines content less than 15 % existed at the same depth as the bottom of the manhole, and impermeable peat and clay layers thicker than 1 m also existed adjacent to the saturated backfill soil. Based on liquefaction resistance ratio F_L, the sand layer in the original subsoil surrounding the manholes might have liquefied moderately and squeezed the backfill soil as shown in Fig.8 (b). The low permeability of peat and clay layer was considered to have affected the extent of liquefaction of backfill.

Based on the above results, liquefaction of the backfill soil was considered to be a major cause of the damage, while existence of peat and clay layer and/or a liquefiable sand layer in the original soil was considered to have affected the extent of the damage. Additional research is required to reveal the possible effect of the original subsoil on the local amplification of earthquake motion. The similar results were concluded by Koseki et al. (1998b) and Sasaki et al. (1999), who conducted statistical causal analyses on damaged/undamaged sewer pipes in Kushiro City during the 1993 Kushiro-Oki earthquake and the 1994 Hokkaido-Toho-Oki earthquake and those in Shibetsu Town during the 1994 Hokkaido-Toho-Oki earthquake, respectively.

(4) Behavior of common utility ducts during the 1995 Hyogoken-Nanbu earthquake

The 1995 Hyogoken-Nanbu earthquake caused severe damage to buried structures (PWRI 1996). These damages were mainly caused by the earthquake-induced ground displacements including lateral flow and settlement of ground, and by excessive inertia force during earthquake. During this earthquake, however, despite very strong ground motion, uplift damage to buried structures, especially to common utility ducts, in liquefied area was insignificant. Fig.9 shows the typical cross section of common utility ducts buried in liquefied area in Kobe City during the earthquake and soil profile based on boring survey result, which was conducted after the earthquake. The liquefaction resistance ratio F_L which were evaluated by using SPT-N-value and other soil parameter and in accordance with "Specifications for highway bridges (1997)" are also shown in Fig.9. The results indicate that liquefaction had occurred in the subsoil surrounding the common utility ducts. Actually, sand boils were observed around those sites after the earthquake (Hamada et al. 1995). These common utility ducts, however, suffered no uplift damage despite of liquefaction of the surrounding subsoil. The possible reason for the performance will be discussed later in 5.(3).

3. CURRENT PRACTICES AND ISSUES ON STABILITY EVALUATION DURING LIQUEFACTION

(1) Current practices on stability evaluation during liquefaction

The previous section has suggested that the major damage mode of underground structures due to liquefaction is uplift damage. In this section, the stability assessment techniques against uplift that are used in practice and that was developed before/after the 1995 Hyogoken-Nanbu earthquake are introduced and the research issues related to the methods are summarized.

Table 2 summarizes the effects of liquefaction taken into account in the existing design manuals or guidelines which treat underground structure including common utility ducts, underground parking facilities, sewage facilities, water supply facilities or open-cut railway tunnels. The effects of liquefaction considered in the manuals are uplift, settlement, lateral spreading and increased earth pressure during earthquakes. The manuals for sewage facility, water supply facility and open-cut railway tunnels, which were revised after the 1995 Hyogoken-Nanbu earthquake, require consideration of high intensity (Level 2) ground motion according to importance of structures, and the effect of lateral spreading of surrounding subsoils. The effects of subsidence of surrounding subsoil are considered in the manuals for sewer pipes and open-cut railway tunnels. In the manual for sewer pipes, settlement of surrounding soil estimated according to thickness of liquefied layer is considered in designing flexible joints. For open-cut railway tunnels, settlement is required to be less than an allowable value. Lateral spreading displacements of the ground are considered in the manuals for sewer pipes and water supply pipes in designing flexible joints, and cross section of open-cut railway tunnels based on the response displacement method. For open-cut railway tunnels, increase of intermediate component of earth pressure during liquefaction is considered designing cross section of structures. If the tunnels are penetrated into non-liquefiable layer, then cyclic component of total earth pressure, which is formulated as the modified Westargaard's dynamic pressure, is considered in designing the cross section. All of the manuals, however, treat uplift.

Table 3 summarizes the methods of uplift stability techniques adopted in the manuals. Safety factor against uplift F_U specified in "Design manual for common utility ducts" (Japan Road Association 1986) is defined as shown in Fig.10, which is a ratio of the total weight to the total uplifting force acting on the underground structure, given by

$$F_U = \frac{W_S + W_B + Q_S + Q_B}{U_S + U_D} \qquad (1)$$

where, W_S: weight of overburden soil, W_B: weight of structure, Q_S: shear resistance acting on side wall of overburden soil block, Q_B: frictional resistance acting on side wall of structure, U_S: buoyancy force due to hydrostatic pore pressure, U_D: buoyancy force due to excess pore pressure. Buoyancy force due to excess pore pressure U_d is given from the liquefaction resistance ratio F_L by using the empirical relationship as shown in Fig.11. Shear resistance is assumed to be dependent on the excess pore water pressure ratio, given by

$$\tau_s = K_0 \sigma_{n0}' \tan \phi' \qquad (Lu > 1.0)$$
$$= 0 \qquad (Lu <= 1.0)$$

Frictional resistance acting on side wall of structure is given by

$$\tau_s = K_0 \sigma_{n0}' \tan 2/3 \phi' \qquad (Lu > 1.0)$$
$$= 0 \qquad (Lu <= 1.0)$$

The method has been used widely with slight modification for other manuals.

(2) Issues on the uplift stability evaluation method

During the 1995 Hyogoken-Nanbu earthquake, no common utility duct suffered uplift damage despite of liquefaction of the surrounding subsoil. The applicability of uplift stability method based on "Design manual for common utility ducts" is investigated on the case history of common utility ducts during the 1995 Hyogoken-Nanbu earthquake.

Common utility ducts, which were located in liquefied areas, were evaluated. According to the visual inspection conducted after the quake, the ducts were in alignment, suggesting that virtually no uplift displacement had taken place. Safety factors against liquefaction of surrounding soil were evaluated based on "Specifications of highway bridges (1996)", in which a PGA of 0.6 was employed.

Fig.12 shows the results of calculated values of F_U in relation to the observed residual uplift displacement. This figure also shows the result for aforementioned case histories of damaged manhole in Kushiro City during the 1993 Kushiro-Oki earthquake (Koseki et al. 1997). Although these common utility ducts in Kobe City were suffered no insignificant uplift damage, the values of F_U was less than unity in all sites. The method evaluates the limit equilibrium of vertical force acting on underground structures and does not evaluate the extent of damage or magnitude of uplift displacement.

This method is still widely used for several manuals including sewage facilities, water supply facilities and cut-off railway tunnels, which were revised after the 1995 Hyogoken-Nanbu earthquake and consider level 2 design earthquake. It is, however, presumed that if aforementioned evaluation method of uplift stability such as that provided by "Design manual for common utility ducts (1986)" is employed for a very strong design earthquake like the Hyogoken-Nanbu earthquake, it may yield too conservative design. In manual for railway, although the relationship between F_L and R_U has been revised to consider strong design earthquake (see Fig.11), basis for this relationship are not explained.

To enable a more rational design of the buried structures, a more reasonable uplift stability design method for strong earthquake is required. In the following section, recent studies are presented that has been conducted in order to improve the evaluation method of uplift stability.

4. EXPERIMENTAL STUDIES

A series of dynamic centrifugal model tests was conducted in order to investigate the effects of several factors on the stability of the structures against uplift. The test results were compared with a conventional stability analysis (Japan Road Association (1986)) in order to verify applicability of this method.

(1) Test procedure

A series of dynamic centrifuge tests at a scale of 1/50 was conducted by using dynamic geotechnical centrifuges in the Public Works Research Institute, Japan (Koga et al. (1986) and Matsuo et al. (1998)).

Cross sections of the models used in the centrifugal tests are shown in Fig.13 and the test conditions are summarized in Table 4. In the tests, density of sand layer, amplitude and waveform of input acceleration and apparent unit weight of the buried structure were varied. The models were prepared in a rigid steel container with inner dimensions of 80 cm long, 20 cm wide, and 30 cm high. The model consists of sand layer with a thickness of 20 cm and acrylic box assuming underground structure with a base area of 10 cm by 19 cm and a height of 7.5 cm in model scale. The apparent unit weight of the model box was controlled at designated value, shown in Table 4.1, by putting lead shot inside.

Toyoura sand and Edosaki sand were used for the tests. Edosaki sand has a fines content of approximately 11%. A sand layer was prepared by pouring air-dried Toyoura sand through air in a rigid soil container for most cases, while in case 98-6 and case 98-8, Edosaki sand with a water content of approximately 21% was compacted to a designated relative density by tamping. To fulfill the requirement in the similarity law, the sand layer was saturated by silicon oil having a viscosity of 50 centi-stokes (50 times as viscous as water).

In the tests, after applying a centrifugal acceleration of 50 G a horizontal shaking was conducted. A sinusoidal wave of 20 cycles and 60 Hz was applied to the most of models, while in several cases, the earthquake motion of N-S component recorded at the Hachinohe-Harbor during the 1968 Tokachioki-Oki earthquake or N-S component recorded at the Kobe Maritime Observatory during the 1995 Hyogoken-Nanbu earthquake was applied.

During the tests, horizontal accelerations of the shaking table, sand layer and the model box, excess pore pressure in the sand layer and at the bottom of the model box, and uplift displacement of model box and settlement of ground surface, were measured. Locations of the measurements are shown in Fig.13. After the tests, the ground deformation was sketched.

(2) Results and discussions

a) Permanent deformation
Typical ground deformation that was sketched after the test in case 98-1 is presented in Fig.14. Sand layer beside model box settled and distorted toward the bottom of the model box, accompanying uplift displacement of model box and overlying soil. This deformation mode is very similar that given in Fig.8.

b) Time histories
Fig.15 plots the time histories of selected measurements during shaking for cases 97-4, 98-2 and 98-3. The following are seen from these data.

Uplift movement of the model box initiated after the surrounding sand layer had attained liquefaction. Uplift displacement proceeded at nearly constant rate during shaking in cases with sinusoidal input motion. After the shaking, it proceeded at a slow rate. In case 98-3 with earthquake shaking, uplift displacement developed significantly during the main shocks.

The excess pore pressure was reached on initial effective overburden pressure at first 2 or 3 cycles in all cases. In the case 98-2 with high sand density, the pore pressure exhibited spiky drops due to positive dilatancy. Although pore pressure after shaking maintained the value equal to that during shaking, uplift displacement rate decreased after shaking. Therefore, it is suggested that uplift displacement be affected by shaking intensity in addition to pore pressure.

c) Input acceleration vs. uplift displacement
Relationships between input acceleration and uplift displacement of the model box are shown in Figs.16. Fig.16 (a) plots the data for the cases with the ground water level of 0 mm and the apparent unit weight of the model box ρ_m of 0.8 in order to investigate influences of density of sand layer and input acceleration. Fig.16 (b) shows the effect of the apparent unit weight of the model box, and Fig.16 (c) shows the effect of the ground water level. The following can be seen from the data.

From Fig.16 (a), if the amplitudes of input acceleration are same, the amount of uplift displacements increases with the decrease of sand density. The amount of uplift displacement increases as the amplitude of input acceleration increases. Regarding the effect of input wave, if maximum amplitudes of input acceleration are equal, the amounts of uplift displacement in the cases with sinusoidal wave are larger than in cases with earthquake wave.

From Fig.16 (b), if both sand densities and magnitude of input acceleration are equal, the amounts of uplift displacement are almost same despite of different apparent unit weight of model. The effect of the apparent unit weight of model box is insignificant.

From Fig.16 (c), the amount of uplift displacement in case with ground water level of -6.5 cm is almost same as in case with that of 0 cm. However, the amount of uplift displacement in case with ground water level of -11.5 cm is much smaller than in case with that of 0 cm.

5. STUDIES ON UPLIFT STABILITY EVALUATION DURING LIQUEFACTION

The results of aforementioned centrifugal model tests and the case records during the 1995 Hyogoken-Nanbu earthquake were analyzed with the conventional stability procedures (Japan Road Association (1986) and Koseki and Matsuo (1995)) in order to verify applicability of those method.

(1) Safety factor against uplift

Safety factor against uplift *Fu* based on "Design manual for common utility ducts" (Japan Road Association 1986) is defined as already shown in Fig. 7. For the centrifugal model tests, buoyancy force due to excess pore pressure *Ud* were calculated based on the recorded excess pore pressure during tests.

Relationship between minimum safety factor F_{Umin} against uplift during shaking and amount of residual uplift displacement of model box is shown in Fig.17. This figure also shows the aforementioned results of the case histories of the common utility ducts during the 1995 Hyogoken-Nanbu earthquake and the results for previous shaking table and centrifugal model tests by Koseki and Matsuo (1995). Fig.17 demonstrates that F_{Umin} was smaller than 1.0 when the model box suffered residual uplift. However, the amount of uplift displacement is not differentiated by F_{Umin} but depend on density of sand layer and amplitude of input acceleration.

(2) Safety factor against sliding

Koseki and Matsuo (1995) proposed the uplift stability evaluation method based on safety factor against sliding, in which uplift of underground structure was regarded as lateral movement of surrounding liquefiable soil toward the bottom of underground structure as shown in Fig.18. The safety factor against sliding is defined as the equilibrium of moments due to weight of soil block and structure and due to shear resistance acting on sliding surface, given by

$$F_S = \frac{M_{r1} + M_{r2} + M_{r3} + M_{r4}}{M_{d1} + M_{d2} - M_{d3}} \qquad (1)$$

where, Mr1: resistant moment due to weight of overburden soil and structure, resistant moment due to shear resistance acting on sliding surface, Mr1-3: active moments due to weight of sliding soil blocks. Shear resistance is assumed to be dependent on the excess pore water pressure ratio, given by

$$\tau_f = (1 - R_U)\sigma_n' \cdot \tan\phi' \qquad (2)$$

where, τ_f: resistant shear stress, R_U: excess pore water pressure ratio, σ_n': component of initial overburden effective pressure perpendicular to sliding surface.

Fig.19 shows the relationship between minimum safety factor against sliding F_{Smin} and residual uplift displacement for aforementioned centrifugal model test and case history during the 1995 Hyogoken-Nanbu earthquake. This figure also shows results for the previous shaking table and centrifugal model tests by Koseki and Matsuo (1995). From Fig.19 demonstrates that F_{Smin} was smaller than 1.5 when the model box suffered residual uplift. If values of F_{Smin} are relative large, uplift displacements are considered to relatively relate to the value of F_{Smin}. However, if F_{Smin} was smaller than 1.0, the amount of uplift was not clearly related to the value of F_{Smin} but depend on density of sand layer and amplitude of input acceleration. Especially, although the values of F_{Smin} were almost same, uplift displacement was increased when density of sand layer was higher and magnitude of input acceleration was larger. Consequently, the uplift stability evaluation methods based on safety factor against sliding can not considered the effect of density of sand layer and magnitude of input acceleration for uplift displacement.

As for the case histories, the values of F_{Smin} at all site were less than unity despite of no uplift damage. Consequently, the uplift stability evaluation method based on safety factor against sliding is considered to provide conservative result.

(3) Subject for the reasonable uplift stability evaluation method

These results demonstrates that the conventional uplift stability methods, which include safety factor against uplift Fu and safety factor against sliding Fs, can not estimate the residual uplift displacement

The major defect of the methods would be that for liquefied soil, shear resistance that calculated by Eq.(2) is regarded as completely zero. However, liquefied soils actually have some shear resistance due to so-called cyclic mobility. In addition, based on result of model tests, uplift displacement progressed during shaking and was related to the magnitude of input acceleration. These factors are not considered in these methods. This post-liquefaction shear resistance depends on the soil density and the degree of liquefaction. Therefore, it is required to reveal relationship between the shear resistance of liquefied soil and the effects of input acceleration and density of soil.

5. SUMMARY

Case histories of damage to underground structures caused by liquefaction were briefly reviewed. As a result, uplift damage was the most typical damage mode of underground structures.

The current practices of stability assessment for underground structures were briefly summarized, all of which are based on the safety factor approach. Back analyses of the model tests and the case histories of the common utility ducts during the 1995 Hyogoken-Nanbu earthquake demonstrated that the conventional uplift stability technique yield conservative results.

A series of dynamic centrifugal model tests was conducted in order to investigate the effects of several factors on the stability of the structures against uplift. The tests showed. Uplift displacement of the model box increased with duration of shaking and also after shaking. The rate of displacement decreased when shaking stopped. Residual uplift displacement of the model box increased as magnitude of input acceleration increased. The amounts of uplift displacements in the cases with sinusoidal wave are larger than in cases with earthquake wave. Residual uplift displacement of the model box decreased as the sand density increased.

Minimum safety factors against uplift F_{Umin} and against sliding F_{Smin} of model tests were smaller than 1.0 and 1.5 respectively when the model box suffered residual uplift. The amount of uplift displacement was not clearly related to the value of F_{Umin} and F_{Smin} but depended on density of sand layer and amplitude of input acceleration.

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Table1 Case histories of earthquake induced damage to underground structures in Japan

Earthquake	Damaged underground structure	References
Niigata (1964), M=7.5	Sewage treatment tank, sewer manhole and pipe (*Niigata City*)	PWRI(1965)
Nihonkai-Chubu (1983), M=7.7	Fuel tank, Water tank, sewer pipes (*Noshiro City*)	Morishita and Taniguchi (1985)
Kushiro-Oki (1993), M=7.8	Sewer manhole (*Kushiro Town*), sewer pipe (*Kushiro Town, Kushiro City and Shibetya Town*)	Koseki et al. (1997a), JSSMFE (1994)
Hokkaido-Nansei-Oki (1993), M=7.8	Fuel tank (*Esashi Town*), sewer manhole (*Oshamanbe Town and Kamiiso Town*)	PWRI(1994)
Hokkaido-Toho-Oki (1994), M=8.1	Sewer manhole (*Shibetsu Town*), sewer pipe (*Kushiro City*)	JGS (1998)
Sanriku-Haruka-Oki (1994), M=7.5	Sewer pipe (*Towada City*)	Koseki et al.(1998a)
Hyogoken-Nanbu (1995), M=7.2	Water tank (*Nishinomiya City*), Sewage treatment facility (*Kobe City*)	PWRI(1996)

Table2 Effects of liquefaction considered the existing design manuals

Titles	Year published and organization	Object structure	Design ground motion		Effects of liquefaction taken into account							
					Uplift		Post-earthquake subsidence		Lateral spread		Seismic earth pressure	
			L1	L2	Effects of liquefaction	Check items	Effects of liquefaction	Check items	Effects of liquefaction	Check items	Effects of liquefaction	Check items
Design manual for common utility ducts.	1986 Japan Road Association	Common utility ducts	○	–	Excess pore water pressures and reduction of	Uplift stability	–	–	–	–	–	–
Design and execution manual for underground parking lots	1992 Japan Road Association	Underground parking facilities	○	–	Excess pore water pressure	Uplift stability	–	–	–	–	–	–
Guidlines of aseismic measures for sewage facilities	1997 Japan Sewage Association	Sewage pipes	○	○	–*	–*	Subsidence of surrounding ground	Design of flexible joint	Lateral spread displacements	Design of flexible joint	–	–
		Sewage treatment facilities	○	○	Excess pore water pressures and reduction of	Uplift stability	–	–	Lateral spread forece	Design of pile	–	–
Guidlines of earthquake resistant methods for water supply facilities	1997 Japan Waterworks association	Water supply facilities	○	○	Excess pore water pressures and reduction of shear	Uplift stability	–	–	Lateral spread displacements	Design of flexible joint	–	–
Design standards for railway structures, Seismic design	1998 Railway Technical Research Institute	Open-cut railway tunnels	○	○	Pore water pressure and reduction of shear resistance	Uplift stability	Subsidence of surrounding ground	allowable settlement	Lateral spread displacements	Design of cross section of structure	Intermediate and cyclic component of seismic earth pressure	Design of cross section of structure

*using non liquefiable backfill soil

L1: Ground motion highly probable to occur during service period

L2: Ground motion with high intensity, though less probable to occur during the service period

Table3 Uplift stability methods in each design manual

Titles	Established year and organ	Object structures	Formula	Excess pre water pressure	Shear resistance	Safety factor	Design lateral forece coefficient	Notes
Design manual for common utility ducts.	1986 Japan Road Association	Common utility ducts	$F_S = \frac{W_S + W_B + Q_S + Q_B}{U_S + U_D}$	$L_U = \begin{cases} F_L^{-1} & (F_L \geq 1) \\ 1 & (F_L < 1) \end{cases}$	considered	1.1	0.15	
Design and execution manual for parking	1992 Japan Road Association	Underground parking facilities	$F_S = \frac{W_S + W_B}{U_S + U_D}$	$L_U = \begin{cases} F_L^{-1} & (F_L \geq 1) \\ 1 & (F_L < 1) \end{cases}$	–	1.0	0.15	provide soil inprovement for bottom of
Guidline of aseismic measure for sewage facilities	1997 Japan Sewage Association	Sewage facilities	· Sewer pipes –* · Sewage treatment facilities $F_S = \frac{W_S(1-K_V)+W_f+P_\gamma}{V_w \cdot \gamma_w + V_l \cdot \gamma_l}$	–	considered	1.2		*using non-liquefiable backfill soil
Guidline of earthquake resistant method for water supply	1997 Japan Waterworks association	Water supply facilities	$F_S = \frac{W_B + Q_l}{V_l \cdot \gamma_l}$	–	considered	1.0	0.16 ~ 0.8	
Design standerd for railway structure, seismic design	1998 Railway Technical Research Institute	Cut and cover tunnel	$F_S = \frac{W_S + W_B + Q_S + Q_B}{U_S + U_D}$	$\begin{cases} \ln(L_U) = 0.6 - 1.2F_L & (F_L \geq 0.5) \\ L_U = 1 & (F_L < 0.5) \end{cases}$	considered	according to importance	acceleration resposse spectrum for Level 1 and Level 2	unnecessary to examin if temporary sheet piples were leaved

F_S: Safety factor against uplift、 W_S: Weight of overburden soil、 W_B: Weight of structure、 Q_S: Shear resistance acting on side wall of overburden soil block、 Q_B: Frictional resistance acting on side wall of structure、 U_S: Buoyancy force due to hydrostatic pore pressure、 U_D: Buoyancy force due to excess pore pressure、 P_γ: Pulling resistance of a pile、 V_0: Volume of structure、 V_w: Volume of part of structure below ground water level except liquefied layer、 V_l: Volume of part of structure in liquefied layer、 γ_w: Unit weight of water、 γ_l: Unit weight of liquefied soil、 L_U: Excess pore water pressure ratio、 Δu: Excess pore water pressure、 F_L: Liquefction resistance ratio、 K_v: Design vertical force coefficient

Table 4 Test conditions

Case	centrifugal Acc.(G)	unit weight of model	ground material	relative density Dr(%) [Dc(%)]	preparation method	ground water level (cm)	shaking condition wave type	(G)
97-1	50	1.3	Toyoura sand	20	air pluviated	0	sinusoidal	5.6
97-2	50	1.3	Toyoura sand	71	air pluviated	0	sinusoidal	16.0
97-3	50	0.8	Toyoura sand	49	air pluviated	0	sinusoidal	8.9
97-4	50	0.8	Toyoura sand	71	air pluviated	0	sinusoidal	14.5
97-5	50	0.8	Toyoura sand	28	air pluviated	0	sinusoidal	6.8
97-6	50	0.8	Toyoura sand	28	air pluviated	0	sinusoidal	15.7
97-7	50	0.8	Toyoura sand	54	air pluviated	0	sinusoidal	14.4
97-8	50	0.8	Toyoura sand	47	air pluviated	0	Hachinohe wave	19.6
97-9	50	0.8	Toyoura sand	47	air pluviated	-6	sinusoidal	10.0
97-10	50	0.8	Toyoura sand	67	air pluviated	-6	sinusoidal	14.9
98-1	50	0.8	Toyoura sand	52	air pluviated	0	sinusoidal	16.0
98-2	50	0.8	Toyoura sand	84	air pluviated	0	sinusoidal	40.0
98-3	50	0.8	Toyoura sand	53	air pluviated	-11.5	sinusoidal	16.0
98-4	50	0.8	Toyoura sand	83	air pluviated	0	Kobe wave	40.0
98-5	50	0.8	Toyoura sand	54	air pluviated	0	Kobe wave	40.0
98-6	50	0.8	Edosaki sand	14 [75]	wet tamping	0	sinusoidal	15.0
98-7	50	1.6	Toyoura sand	52	air pluviated	0	sinusoidal	15.0
98-8	50	0.8	Edosaki sand	57 [85]	wet tamping	0	sinusoidal	15.0
98-9	50	0.8	Toyoura sand	93	air pluviated	0	Kobe wave	40.0
98-10	50	0.4	Toyoura sand	51	air pluviated	0	sinusoidal	15.0

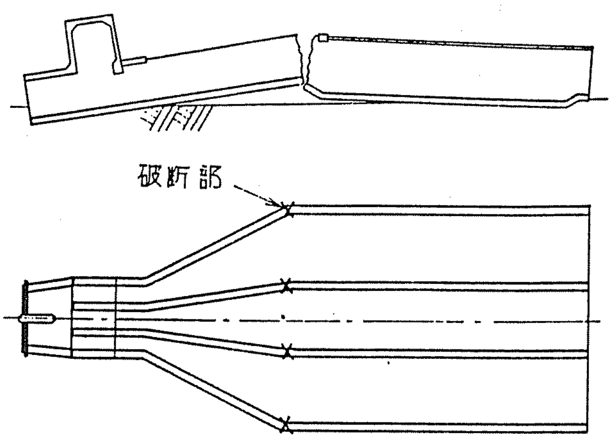


Fig.1 Schematic uplifted sewage treatment tank during the 1964 Niigata earthquake (the Zoshobori sewage pomp station)

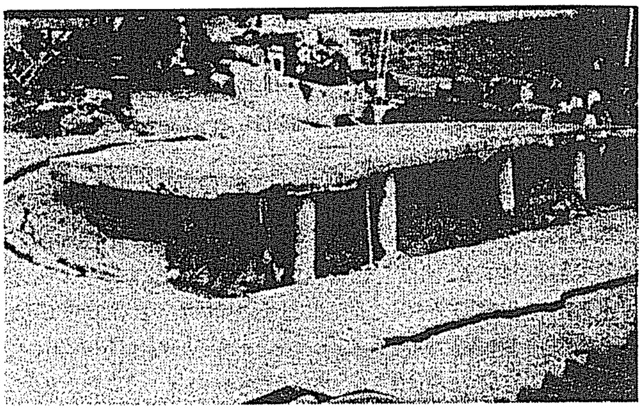


Fig.2 Uplifted fuel tank during the 1986 Nihonkai-Chubu earthquake

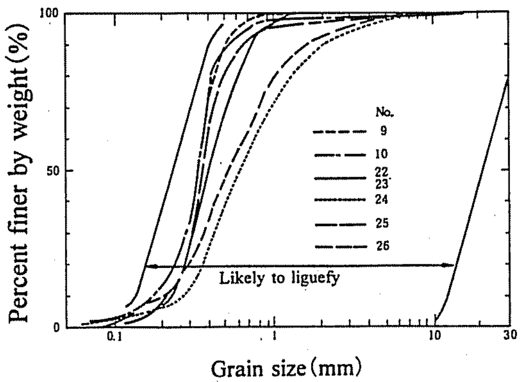


Fig.3 Grain size distribution of sand boil and backfill soil

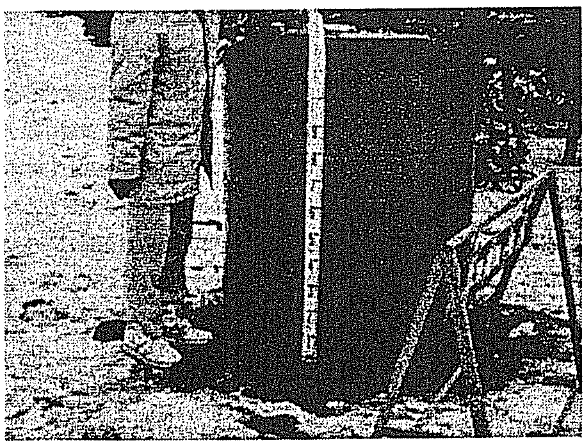


Fig.4 Uplifted sewage manhole during the 1993 Kushiro-Oki earthquake

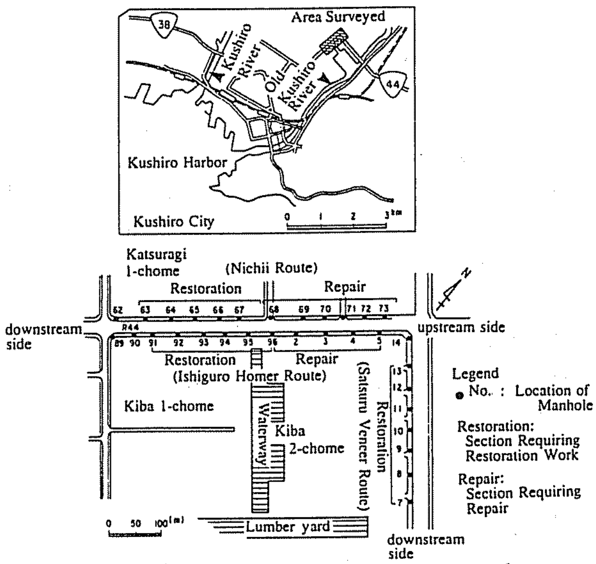


Fig.5 Location of area surveyed

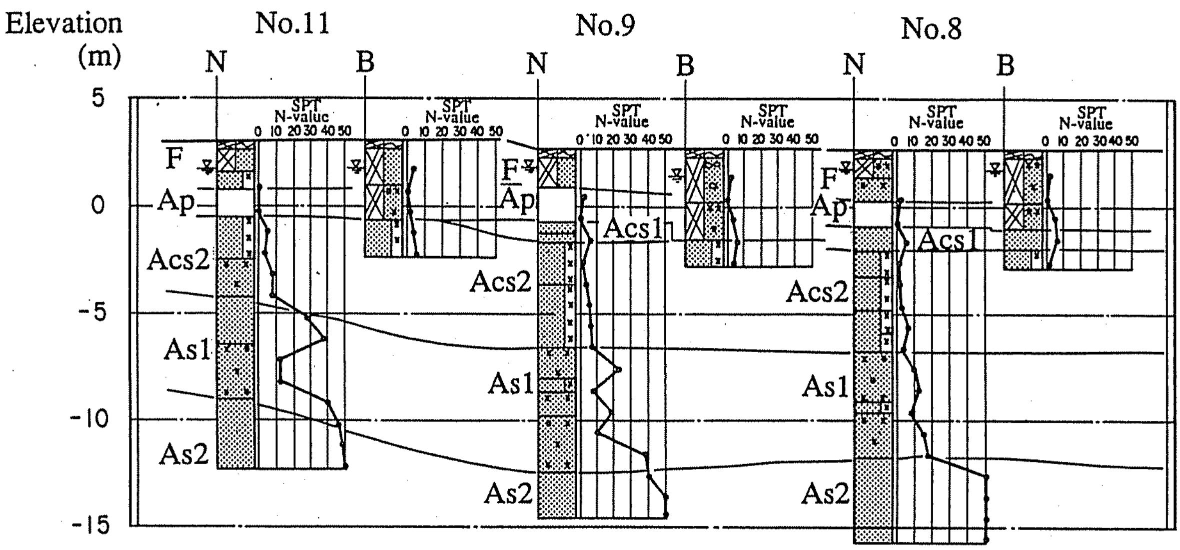


Fig.6 Typical estimated soil profile (Satsuru Veneer Route)

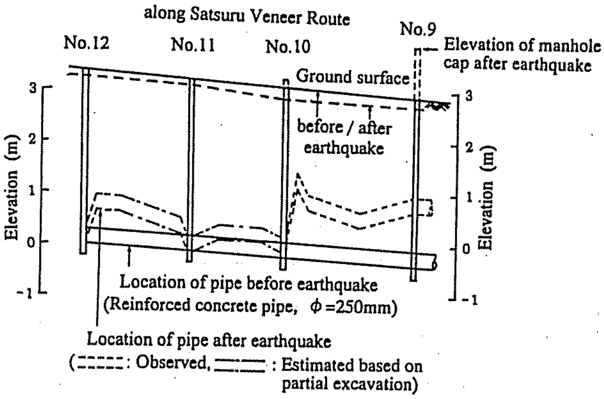


Fig.7 Vertical displacement of sewer pipes along Satsuru Veneer Route

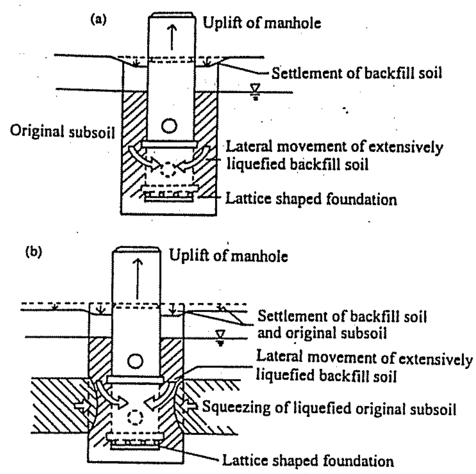


Fig.8 Schematic mechanism of manhole uplift

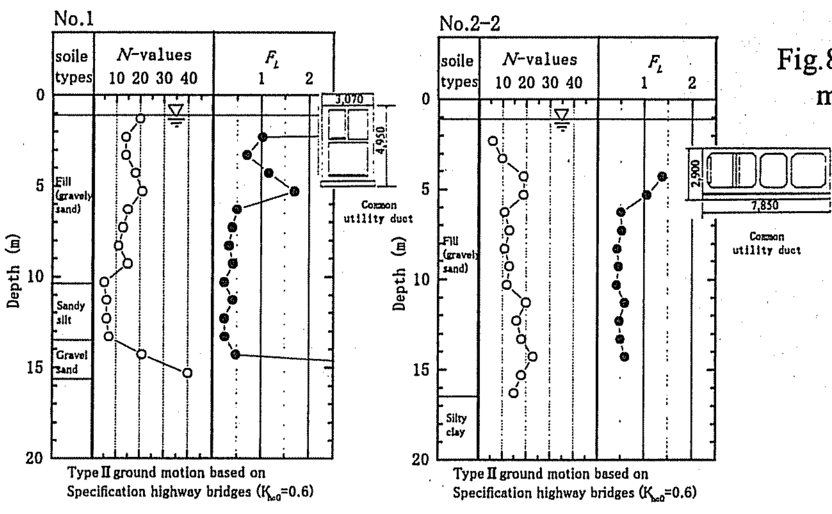


Fig.9 Typical soil profiles

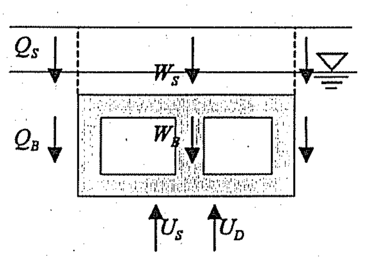


Fig 10 Definition of safety factor against uplift F_U

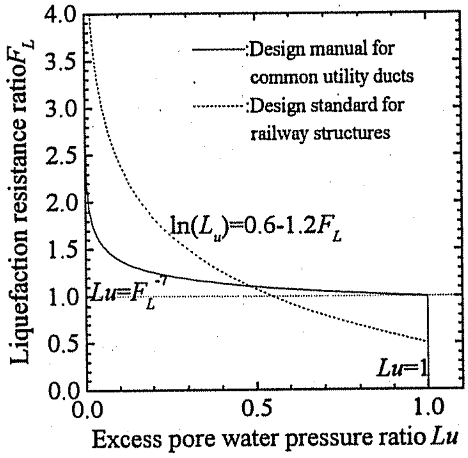


Fig.11 Relationship between Excess pore water pressure ratio R_U and liquefaction resistance ratio F_L

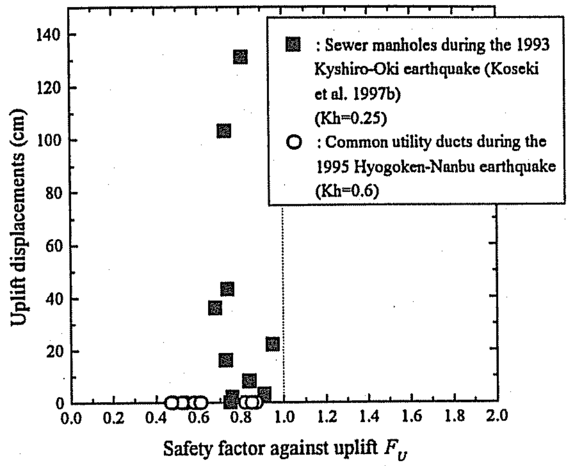


Fig.12 Relationship between uplift displacement and safety factor against uplift F_U

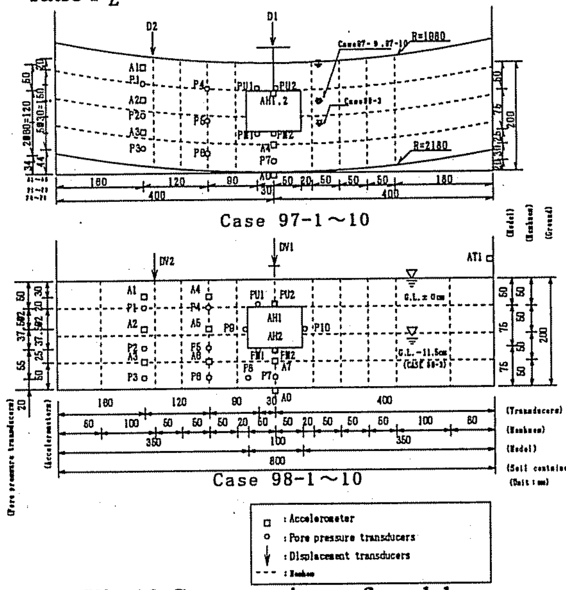


Fig.13 Cross sections of models

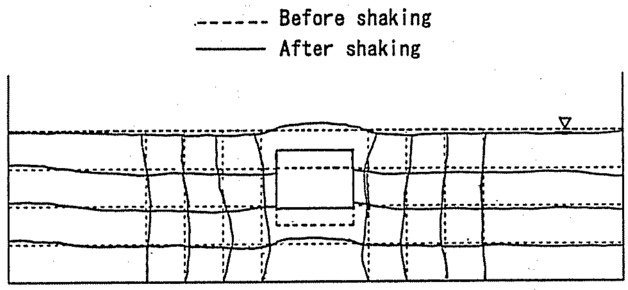


Fig.14 Observed deformation of sand layer after shaking (case 98-1)

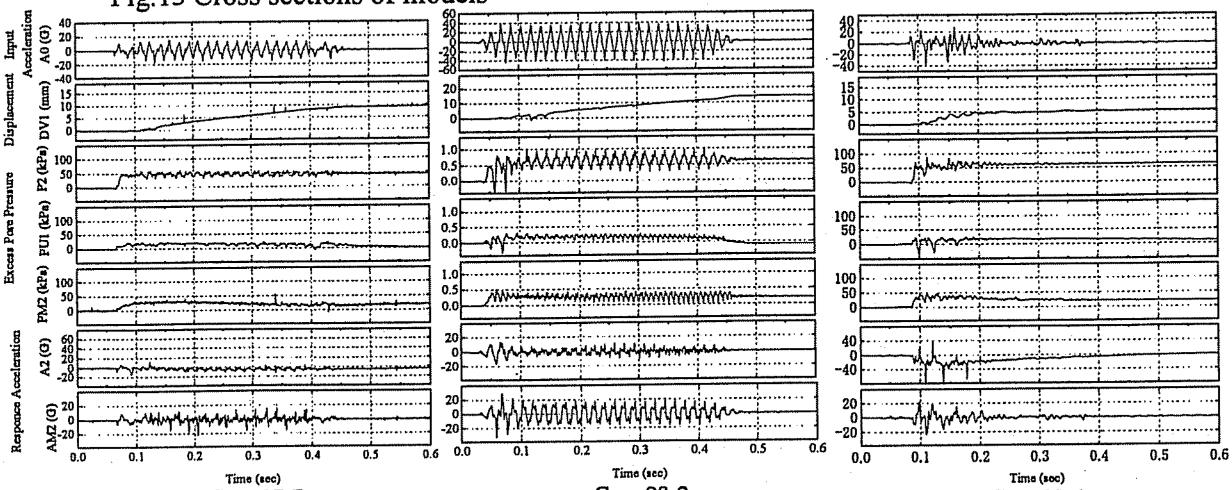
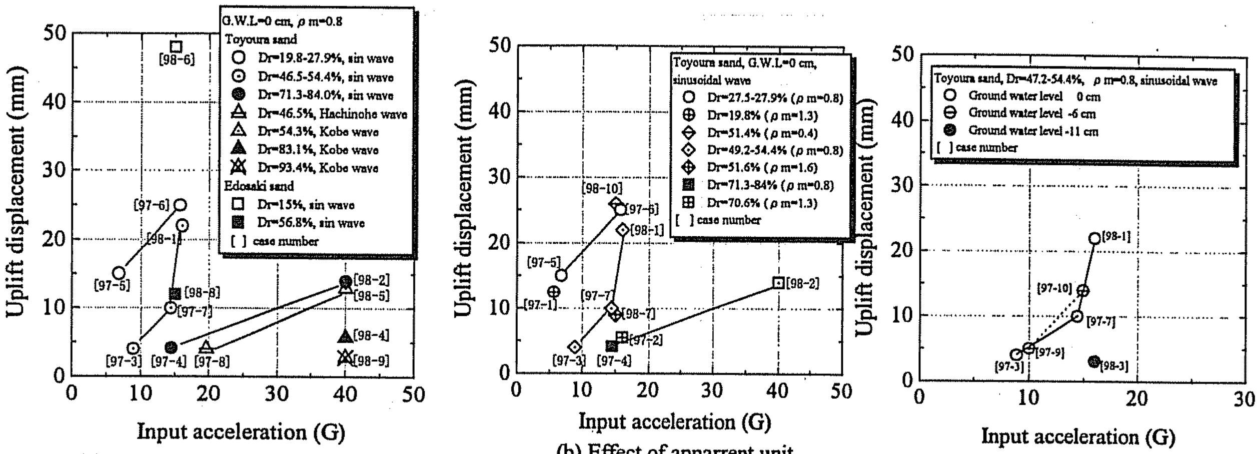


Fig.15 Time histories of selected measurements



(a) Effects of density of sand layer and input acceleration

(b) Effect of apparent unit weight of acrylic box

(c) Effect of ground water level

Figs.16 Relationship between input acceleration and uplift displacement

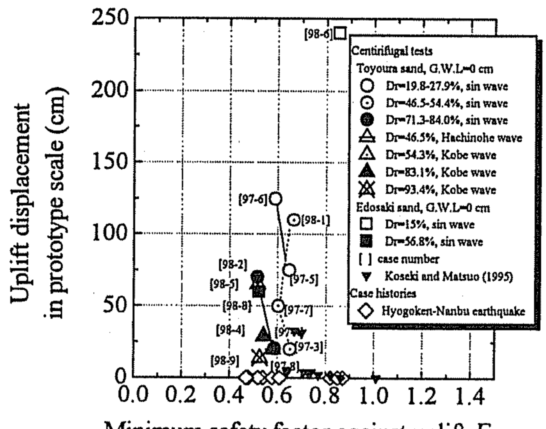


Fig.17 Relationship between minimum safety factor against uplift F_{Umin} and uplift displacement

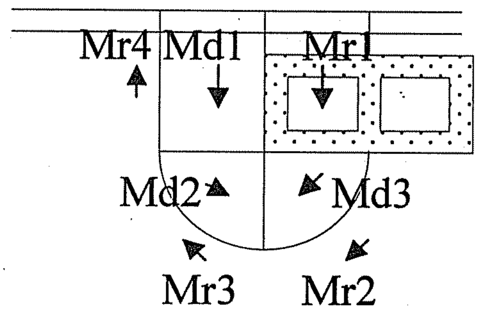


Fig.18 Definition of safety factor against sliding F_S

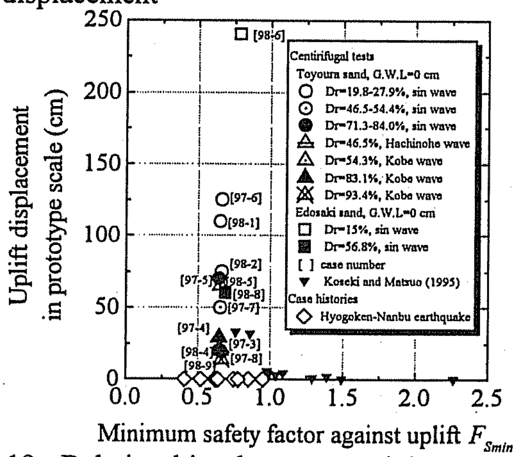


Fig.19 Relationship between minimum safety factor against sliding F_{Smin} and uplift displacement