SOILS AND FOUNDATIONS Vol. 51, No. 2, 215–226, Apr. 2011
Japanese Geotechnical Society

MICROZONATION FOR SEISMIC GEOTECHNICAL HAZARDS AND ACTUAL DAMAGE DURING THE 2005 FUKUOKA-KEN SEIHO-OKI EARTHQUAKE

SUSUMU YASUDA[i], HIDEO NAGASE[ii] and YUTAKA TANOUE[iii]

ABSTRACT

The 2005 Fukuoka-ken Seiho-oki earthquake caused damage to structures due to liquefaction, slope failure and strong shaking in Fukuoka City. One of the authors had conducted microzonation for liquefaction, slope failure and strong shaking about 17 years before the earthquake. After the earthquake, the authors compared the zoning map with the actual damage. The results showed that liquefied sites coincided fairly well with predicted zones in the reclaimed lands. However, they were slightly different in a big sand spit named Uminonakamichi. Failure occurred at several slopes inside the predicted areas, while damage to buildings occurred due to strong shaking along the Kego Fault. The damaged area was slightly different from the predicted area. Furthermore, a liquefaction analysis was conducted and the results of this analysis were compared with the results of microzonation in 1988. In this paper, the authors discuss the validity of microzonation based on this comparison.

Key words: earthquake damage, geotechnical hazard, liquefaction, microzonation, slope failure, strong shaking (**IGC**: E8/D7)

INTRODUCTION

The seismicity around Fukuoka City is of the lowest class in Japan. However, microzonation for probable earthquake effects was needed because Fukuoka City is the biggest and most important city in southern Japan. Microzonation for liquefaction, strong shaking and slope failure was conducted based on several analyses in 1988 (Fukuoka City Waterworks Bureau, 1988 and 1989, Yasuda and Matsumura, 1991). The base acceleration for these analyses was assumed to be 150 cm/s^2, although the maximum acceleration at the ground surface was assumed to be 200 cm/s^2 in the microzonation for liquefaction and slope failure, considering the results of the analysis for strong motion. According to the results of the analyses, liquefaction was predicted in almost all of the artificially reclaimed lands. Slopes considered susceptible to failure were located along several mountains and hills. Areas considered susceptible to strong shaking were distributed along the Kego Fault and central lowlands of the city.

Seventeen years later, in 2005, a big earthquake with a magnitude of 7.0 occurred near Fukuoka City. The maximum surface acceleration in the downtown of Fukuoka was 276 cm/s^2 which was almost the same as the maximum acceleration assumed for the microzoning map. The authors conducted site investigations just after the earthquake and compared the actual geotechnical hazards with those projected in the zoning map. Furthermore, a liquefaction analysis was performed and the results were compared with the results of microzonation in 1988. Here, the authors discuss the accuracy and problems of microzoning by comparing the actual damage and that projected in the microzoning map.

MICROZONATION CONDUCTED IN 1988

Geotechnical, Geomorphological and Seismological Conditions of Fukuoka City

The seismicity in and around Fukuoka City is low, compared with that in other cities Japan, because Fukuoka City is almost 300 km from the subduction zone of the Philippine Sea Plate, where huge earthquakes occur periodically. Only small earthquakes, rarely close to the city, have caused damage. An earthquake occurred in 1898 at the southwest boundary of Fukuoka City. A survey of literature about this earthquake revealed that liquefaction was induced at several sites in the western part of the city. However, the damage to structures due to the liquefaction was light. Though the seismicity in Fukuoka City is low, microzonation for probable earthquake effects was needed because Fukuoka City is the biggest

[i] Professor, Department of Civil and Environmental Engineering, Tokyo Denki University, Japan (yasuda@g.dendai.ac.jp).
[ii] Professor, Department of Civil and Architectural Engineering, Kyushu Institute of Technology, Japan.
[iii] Senior Engineer, Kiso-jiban Consultants Co., Ltd., Japan.
The manuscript for this paper was received for review on May 24, 2010; approved on December 7, 2010.
Written discussions on this paper should be submitted before November 1, 2011 to the Japanese Geotechnical Society, 4-38-2, Sengoku, Bunkyo-ku, Tokyo 112-0011, Japan. Upon request the closing date may be extended one month.

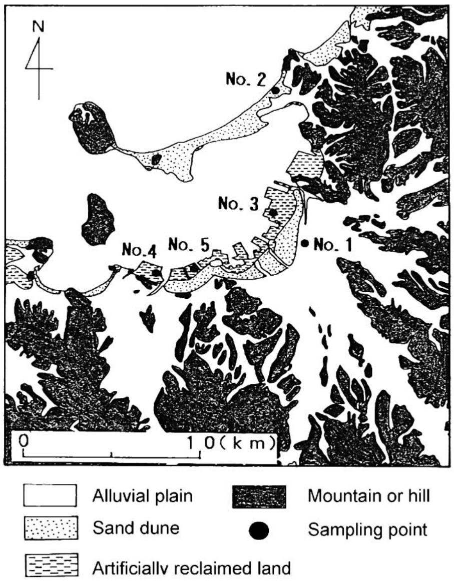


Fig. 1. Topography in Fukuoka City

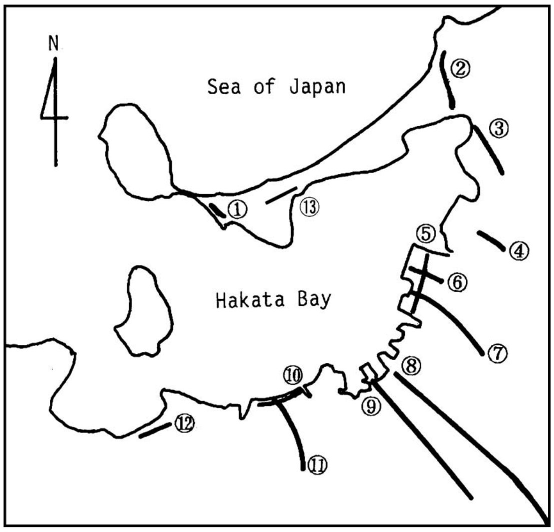


Fig. 2. Selected sites for 1988 soil cross sections (After Fukuoka City Waterworks Bureau, 1988)

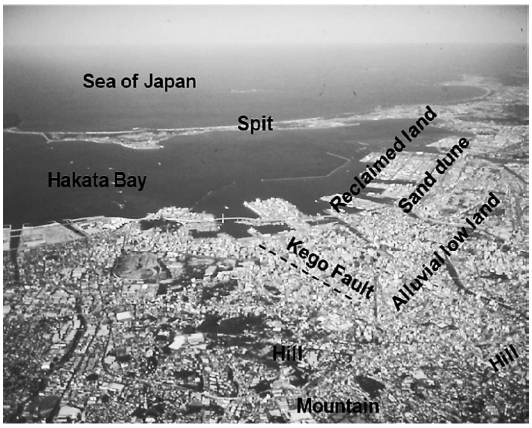


Photo 1 Bird's-eye view of Fukuoka City

and most important city in southern Japan, with a population of about 1.4 million.

The north side of the city faces Hakata Bay, and other sides are surrounded by mountains of 300 to 1000 m in height, as shown in Fig. 1 and Photo 1. From the mountains, several small rivers carry soil to Hakata Bay, forming an alluvial plain. Along the foot of the mountains are hills. A fairly large spit, named Uminonakamichi, lies between Hakata Bay and the Sea of Japan. Along Hakata Bay, a small sand dune runs continuously. In front of this small sand dune are several areas of artificially reclaimed lands. A big fault named Kego Fault crosses the central part of the city from northwest to southeast. Therefore, structures in the city may be damaged during an earthquake which produces: (1) the liquefaction of reclaimed lands and the sand dunes along the coast, (2) the failure of mountain or hill slopes, and (3) strong shaking near the fault and on the soft ground.

Estimation of Soil Layers

As the first stage of the 1988 microzoning, almost 1,200 sets of borehole data were collected. Based on this data, soil cross sections at the 13 areas shown in Fig. 2 were estimated. These sites were selected to cover the whole alluvial plain. Figure 3 shows a typical soil cross section, that in area (7)–(7′). A layer of artificially filled sand or silt, with a thickness of 1 to 13 m, is formed near the coast of Hakata Bay. Behind this area is a sand dune layer with a thickness of 3 to 6 mm which changes to the deltaic alluvial sand inland. Therefore, the sand layers in Fukuoka City are classified into the following three layers:

(1) an alluvial sand layer which covers almost all of the alluvial plain
(2) a dune sand layer which runs along the natural coast
(3) an artificially reclaimed sand layer

Microzonation for Liquefaction

As the first step, undisturbed samples of the three sand layers were taken at the five sites shown in Fig. 1, and undrained cyclic triaxial tests were conducted to measure the liquefaction strength. Sampling methods at the five sites were as follows:

(1) No. 1, No. 2 and No. 3: Undisturbed samples were taken by the block sampling method. Thin wall tubes of 75 mm in inner diameter, 79 mm in outer diameter and 15 cm in length were pushed into the ground from the bottom of the excavated ditches, as shown in Photo 2.
(2) No. 4 and No. 5: Undisturbed samples were taken from bore holes by stationary piston samplers. Thin walled tubes of 75 mm in inner diameter, 79 mm in outer diameter and 100 cm in length were used.

The reclaimed lands had been formed several years prior to the tests. Tests on reconstituted samples of the reclaimed sands with the same density were also carried out. Figure 4 shows the typical grain-size distribution

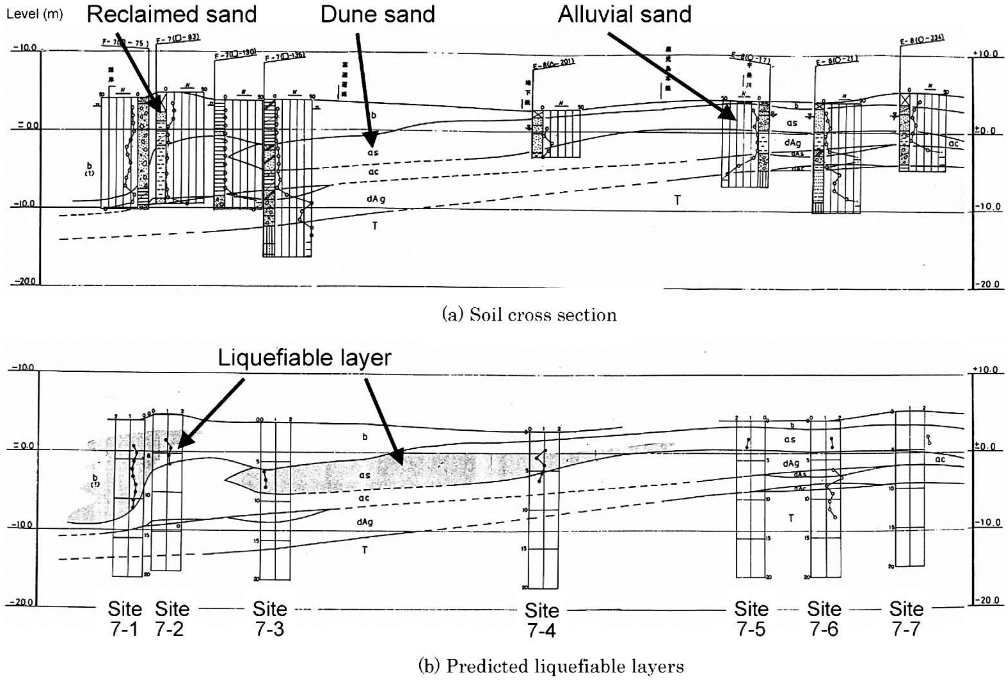


Fig. 3. Soil cross section in the area 7–7′ and predicted liquefiable layers for 1988 microzonation (After Fukuoka City Waterworks Bureau, 1988)

Photo 2 Block sampling at site No. 1

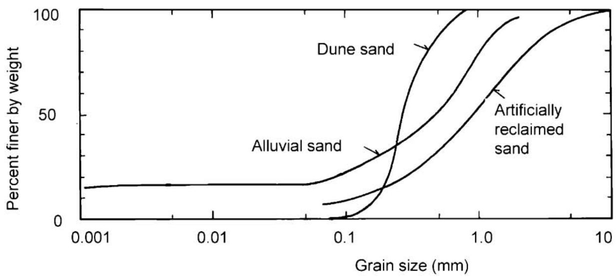


Fig. 4. Grain-size distribution curves of tested sands (After Fukuoka City Waterworks Bureau, 1988)

curves of the alluvial, dune and reclaimed sands. Dune sand and artificially reclaimed sand have few fine particles, but alluvial sand has a considerable amount of fine particles. The fines content, F_c, of the alluvial sand was 15%. The relationships between F_c and the liquefaction strength (Undrained cyclic shear stress ratio), R_L ($N=20$,

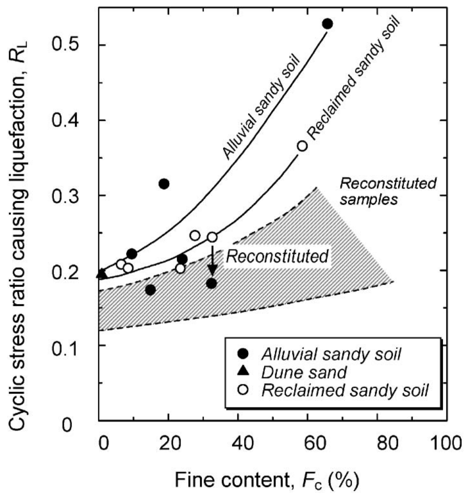


Fig. 5. Relationships between liquefaction strength and fine content of soils in Fukuoka City

$DA=5\%$) are plotted in Fig. 5. As shown in the figure, the liquefaction strength of alluvial sand was higher than those of the reclaimed sand, the dune sand and the reconstituted alluvial sand.

In Japan, the method of estimating liquefaction used in the specification for highway bridges (Japan Road Association, 1980), which was derived from Iwasaki et al.'s formula (1978) with slight modification, and which uses SPT N-values, was the one commonly used at the time. Therefore, after comparing the cyclic triaxial test results, shown in Fig. 5, with the formula, the following modified formula was derived and applied to soil in Fukuoka City:

$$R=R_1+R_2+R_3 \tag{1}$$

where, R: liquefaction strength (undrained cyclic shear strength ratio)

$R_1 = 0.0882\sqrt{N/((\sigma'_v/98)+0.7)}$
$R_2 = 0.19$ for $0.02\text{ mm} \leqq D_{50} < 0.05\text{ mm}$
$0.225 \log(0.35/D_{50})$ for $0.05\text{ mm} \leqq D_{50} < 0.6\text{ mm}$
-0.05 for $0.6\text{ mm} \leqq D_{50} < 2.0\text{ mm}$
$R_3 = 0$ for dune sand and reclaimed sand
0.187 for alluvial sand

A value of 0.187 was added to the liquefaction strength ratio R of alluvial sand, on the basis of the results shown in Fig. 5. The alluvial sand has much fines though its mean diameter is fairly large, as shown in Fig. 4. This is the reason why the value of 0.187 must be added to the formula used in the specification for highway bridges.

Assuming a maximum surface acceleration of 200 cm/s^2, liquefiable layers were predicted at all soil cross sections. The hatched zones in Fig. 3(b) show the predicted liquefiable layers along section (7)–(7′). The reclaimed sand layer and dune sand layer were judged to be liquefiable along this section.

It was necessary not only to predict the liquefiable layer but also to predict the possibility of damage to structures due to liquefaction. The amount of possible damage was judged based on the relationship between the thickness of the liquefiable layer, H_2, and the thickness of the upper non-liquefiable layer, H_1, proposed by Ishihara (1985). Figure 6 shows the 1988 microzoning map for liquefaction thus determined. In general, structures on artificially reclaimed land or partly on the sand dune were considered susceptible to damage due to liquefaction. Liquefaction was predicted to occur throughout most of the sand dune, but damage due to liquefaction was not expected. Various soils have been used for the fill soils in the artificially reclaimed lands. In general, sandy soils and clayey soils have been filled in old and new reclaimed lands, respectively. Therefore, it was judged that liquefaction would not occur in newly reclaimed lands. These reclaimed lands were located in Meino-hama, Odo, Atago-hama and Kashii-hama.

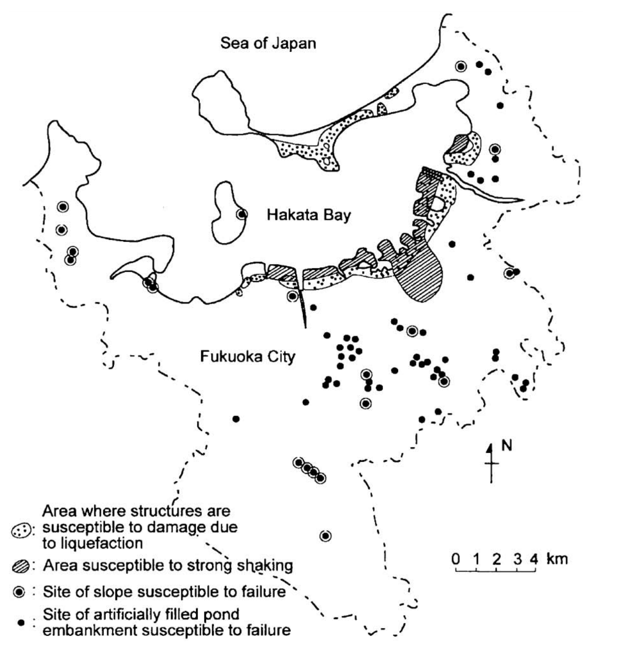


Fig. 6. Microzoning map of Fukuoka City (After Fukuoka City Waterworks Bureau, 1989)

Microzonation for Slope Failure

Microzonation for slope failure was conducted by the following two methods:

(1) Slopes susceptible to failure during heavy rain had been investigated in Fukuoka City. Assuming these slopes had the potential to slide during earthquakes also, detailed site investigations of the slopes were conducted. Determining which slopes were susceptible to failure during an earthquake was based on the slope angle, hardness of the slope and several other factors, as shown in Fig. 6.
(2) A simple method to predict the susceptibility to slope failure had been developed by Kanagawa Prefecture (Kanagawa Prefecture, 1986) and introduced in the seismic zoning manual prepared by TC4 (TC4, 1994). This method was derived from the main factors which affected slope failures during four earthquakes in Japan. The weightings of these factors in one grid (about 500 m times 500 m) were also clarified in this method. The factors identified were: (a) maximum surface acceleration, (b) the length of slope in one grid, (c) the variation of level in one grid, (d) the length of the fault in one grid, (e) the length of the cut slope, and (f) the angularity of slope. Based on this method, grids of slopes susceptible to failure were constructed. Figure 7 shows the predicted zoning map.

As shown in Fig. 7, slopes susceptible to failure are prevalent in several mountains and hills. Most failure susceptible slopes predicted by the first method shown in Fig. 6 are in the grids of slopes susceptible to failure predicted by the second method.

Relating to slope failure, the locations of artificially filled ponds were found by comparing a recent map with a

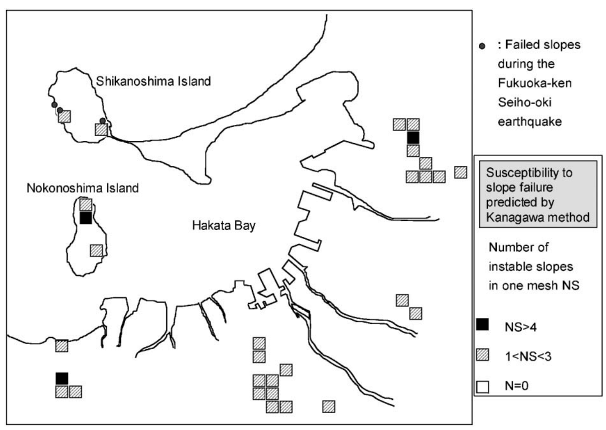


Fig. 7. Microzoning map for slope instability predicted by Kanagawa method (After Fukuoka City Waterworks Bureau, 1988)

map of about 100 years ago, as shown in Fig. 6. The stability of these ponds could not be evaluated quantitatively. However, these sites were pointed out as being possibly susceptible to failure during an earthquake.

Microzonation for Strong Shaking

Microzonation for strong shaking was conducted based on seismic response analyses. In addition to the collection of boring data mentioned before, PS logging data were collected. By comparing these data, the following relationships between SPT N-values and shear velocity, V_s (m/s), were derived:

$$\left.\begin{array}{ll} V_s = 80\,N^{0.333} & \text{for dune sand and reclaimed sand} \\ V_s = 144\,N^{0.159} & \text{for other sands} \\ V_s = 100\,N^{0.333} & \text{for reclaimed clay} \\ V_s = 157\,N^{0.180} & \text{for other clays} \end{array}\right\} \quad (2)$$

Data on the relationships between the shear strain, γ, the dynamic shear modulus ratio, G/G_0, and the damping ratio, h, could not be collected for soil in Fukuoka City. Therefore, the following relationships, derived from cyclic torsional shear tests of almost 100 undisturbed samples in Japan (Yasuda and Yamaguchi, 1985), were used for the analyses.

$$\left.\begin{array}{l} G/G_0 = (A_1 + A_2 \log D_{50}) \times (p'/98)^{(B_1 + B_2 \log D_{50})} \\ h = (C_1 + C_2 \log D_{50}) \times (p'/98)^{(D_1 + D_2 \log D_{50})} \end{array}\right\} \quad (3)$$

where,

D_{50}: mean diameter of soil (mm)

A_1 to D_2: coefficients shown in Fig. 8

p': effective mean confining pressure (kN/m^2)

Two kinds of seismic response analysis were conducted: (1) one dimensional analysis, and (2) two dimensional analysis. One dimensional seismic response analysis was applied to 37 typical borehole sites. The dynamic shear modulus at low strain level, G_0, was determined from the SPT N-value using Eq. (2). The relationships between γ and G/G_0, and γ and h, were estimated using Eq. (3). Assuming that a big earthquake would not hit Fukuoka City in the near future, as mentioned before, a seismic wave which had been recorded during a past medium earthquake was used for an input base seismic motion. The wave used was recorded at the Institute of Industrial Science of the University of Tokyo, Chiba Campus, during the 1987 Chibakentoho-oki earthquake. Moreover, as a representative seismic wave obtained during a big earthquake, the wave recorded in Hachinohe Harbour during the 1968 Tokachi-oki earthquake was also applied. The peak acceleration of these waves was adjusted to 150 cm/s^2 at outcrop motion. The computer program used was "SHAKE" (Schnabel et al., 1972).

Figure 9 shows the relationships between the analyzed amplification ratio of acceleration, $A_{\max,s}/A_{\max,b}$, the predominant period of the ground, T_G, and the maximum surface displacement, $D_{\max}$. T_G (sec.) was calculated using the following equation:

$$T_G = \sum_{i=1}^{n} (4H_i/V_{si}) \quad (4)$$

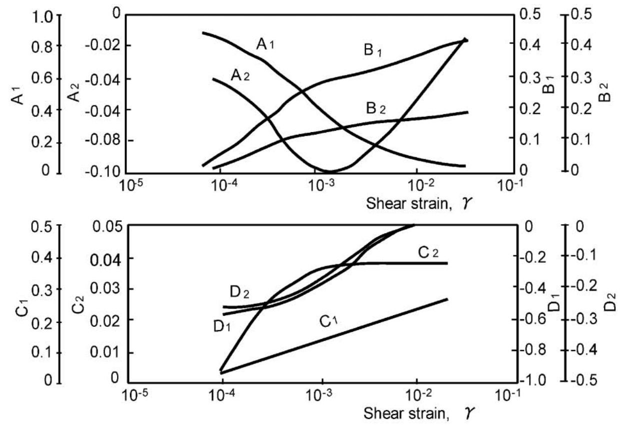


Fig. 8. Coefficients of Equation (3) (Yasuda and Yamaguchi, 1985)

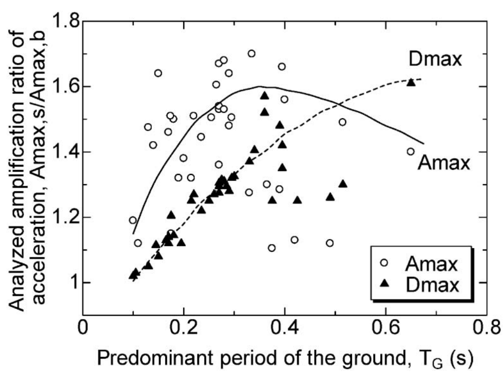


Fig. 9 Relationship between amplification ratio of acceleration, maximum displacement and predominant period of the ground for 1988 microzonation (After Fukuoka City Waterworks Bureau, 1989)

where,

i: number of the soil layers

H_i: thickness of the (i)th soil layer (m)

V_{si}: shear wave velocity of the (i)th soil layer (m/s)

The dotted points in Fig. 9 are scattered to some extent. However, assuming that there are comparatively unique relationships between T_G and $A_{\max,s}/A_{\max,b}$, and T_G and $D_{\max}$, $A_{\max,s}$ and $D_{\max}$ in all boring sites along the 13 soil cross sections were predicted.

As mentioned before, a fault, named the Kego Fault, crosses the central part of the city. Moreover, there are many quay walls along the coast. Therefore, two dimensional seismic response analyses across the Kego Fault and a quay wall at Hakozaki reclaimed land were conducted to ascertain the effect of the fault and the quay wall on the seismic response. The computer program used was "FLUSH" (Lysmer et al., 1975). Figures 10 and 11 show the cross sections across the Kego Fault and the quay wall at Hakozaki, respectively, and the relationships between γ and G/G_0 or h were estimated using Eqs. (2) and (3), respectively. The input wave was the recorded wave at the Institute of Industrial Science of the University of Tokyo, and the peak acceleration of this wave was adjusted to 150 cm/s^2. The quay wall in Fig. 11 is a sheet pile type wall.

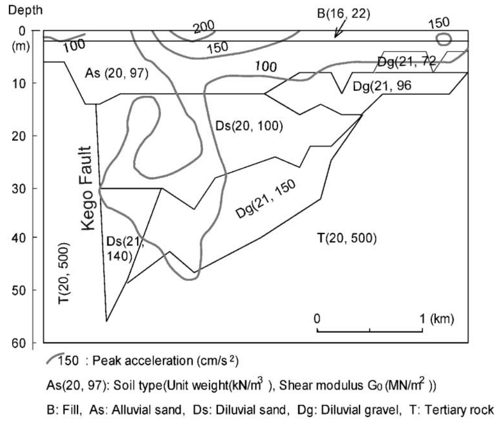


Fig. 10. Soil cross section and analyzed peak acceleration across the Kego Fault (After Fukuoka City Waterworks Bureau, 1989)

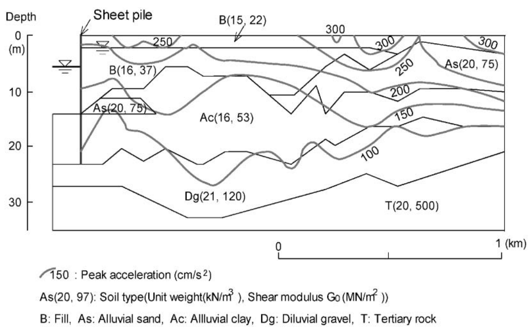


Fig. 11. Soil cross section and analyzed peak acceleration across a quay wall at Hakozaki (After Fukuoka City Waterworks Bureau, 1989)

The contour lines in Figs. 10 and 11 show the distributions of analyzed peak acceleration in the grounds. Fairly strong shaking was likely to occur at the ground surfaces about 1 km east of the Kego Fault and the ground behind the quay wall. Then the hatched zones shown in Fig. 6 were judged as the areas susceptible to strong shaking.

DAMAGE DURING THE 2005 FUKUOKA-KEN SEIHO-OKI EARTHQUAKE AND COMPARISON WITH PREDICTED HAZARDS

Outline of Damage

The Fukuoka-ken Seiho-oki earthquake, with a magnitude of 7.0 (Mj), occurred in Japan on March 20, 2005. The epicenter of the earthquake was about 30 km northwest of downtown Fukuoka City. The direction of the fault is estimated as NW-SE. The maximum horizontal surface acceleration recorded in the center of Fukuoka City was 276 cm/s^2. The seismic intensity in Fukuoka City, according to the JMA scale, was 5 Lower to 6 Lower, which was almost the same as the assumed seismic intensities for the microzoning. Therefore, it was considered appropriate to compare the actual geotechnical hazards that occurred during the earthquake with the predicted ones.

The earthquake caused severe damage to houses due to

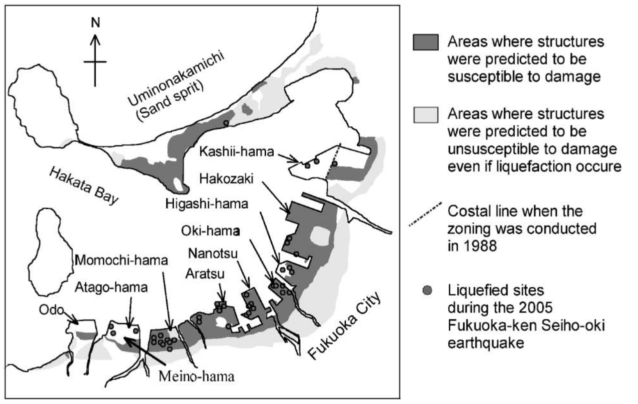


Fig. 12. Hazard map for liquefaction and liquefied sites during the Fukuoka-ken Seiho-oki earthquake

the failure of retaining walls on a tiny island named Genkai-jima which is only about 10 km from the epicenter. A lot of tiled roofs of timber houses were damaged due to shaking over a wide area. Several buildings suffered damage in the central area of Fukuoka City. Liquefaction occurred in many reclaimed lands and caused damage to quay walls, tanks and sheds. Slope failures occurred at several sites on Shikano-shima Island, which is located about 20 km from the epicenter.

Damage to Structures due to Liquefaction

The closed circles plotted in Fig. 12 indicate the liquefied sites. Almost all the sites are located in the reclaimed lands along Hakata Bay. No liquefaction occurred on the alluvial low land. One site in the Uminonakamichi sand spit liquefied. However, the liquefied site was newly filled land on a pond. Therefore, it can be said that liquefaction occurred in only artificially reclaimed lands.

Among the reclaimed lands, Oki-hama, Nanotsu, Higashi-hama, Aratsu, Hakozaki and Momochi-hama were constructed before 1990. These reclaimed lands were filled mainly with sandy soil. Other reclaimed lands, Meino-hama, Odo, Atago-hama, Kashii-hama and Island City were newly filled with clayey soil. Therefore, few boiled sands were observed in the latter newly reclaimed lands. Liquefaction did not occur in Meino-hama and Odo. A few places with boiled sands were observed on pavements in Kashii-hama and Island City, but no structures were damaged. In Atago-hama, liquefaction occurred at two school grounds, a fishing port and along roads. A shed settled, quay walls tilted and a road subsided due to the liquefaction.

In contrast, in the former reclaimed land, liquefaction occurred at many sites. In Oki-hama, a quay wall tilted, as shown in Photo 3. The horizontal displacement of the quay wall was about 1 m. The ground behind the quay wall flowed toward the bay and subsided. The flow zone extended about 10 m from the quay wall. Several quay walls tilted a little in Nanotsu, Higashi-hama and Aratsu. Two small tanks settled about 10 to 20 cm due to liquefaction in Aratsu. In Momochi-hama, sand boils were observed at many sites, as shown in Photo 4. However,

Photo 3 Displacement of a quay wall and subsidence of the ground in Oki-hama

Photo 4 Boiled sand in Momochi-hama

Photo 5 Liquefaction at the man-made beach in Atago-hama

no damage to buildings and timber houses occurred and obvious ground subsidence did not occur at the liquefied sites. Therefore, it is estimated that the liquefied layer was thin.

There are two man-made beaches at the front of Momochi-hama and Atago-hama. Many large sand volcanoes were observed on the beaches, as shown in Photo 5. The ground was very loose and the water table was

Photo 6 Liquefaction-induced flow at Uminonakamichi Seaside Park

shallow at both beaches.

Photo 6 shows liquefaction in Uminonakamichi. This site was newly developed for a park, named Uminonakamichi Seaside Park, by filling part of a pond with sand. Liquefaction occurred at this site and induced ground flow toward the remaining pond. The maximum horizontal displacement due to the flow was about 10 m. Simple piers and promenades were damaged due to the flow.

In Itoshima Peninsula, except for the playground of Shima Junior High School, little liquefaction occurred during the 2005 Fukuoka-ken Seiho-oki earthquake, though liquefaction occurred during the earthquake in 1898.

Comparison of Liquefied Sites with Predicted Liquefaction Areas

The shadowed zones in Fig. 12 show the areas where structures had been predicted to be susceptible to damage due to liquefaction. These areas compare with the liquefied sites as follows:

(1) Almost all the liquefied sites are located in the areas where structures had been deemed susceptible to damage. Therefore, roughly speaking, the microzonation for liquefaction was fairly valid.

(2) Liquefaction occurred at three sites in Atago-hama, though it was predicted that liquefaction would not occur there because this land was filled with clayey soil. The liquefied sites are located along old shore protections. In general, sandy soil is filled just behind shore protections during reclamation work. As such, it could be that there was sandy soil at the sites which liquefied during the earthquake.

(3) In the Uminonakamichi sand spit, liquefaction did not occur in natural ground, though some areas at the toes of sand dunes had been predicted to liquefy. There was little borehole data from this sand spit because there were no heavy structures there. Therefore, in the microzoning, the grounds at the toes of sand dunes were judged to be liquefiable, based on experience with past earthquakes. For example, liquefaction occurred in the grounds at the toes of sand dunes during the 1964 Niigata and 1983 Nihonkai-

Photo 7 Slope failure in Shikano-shima island

Photo 8 A damaged apartment above the Kego Fault

chubu earthquakes. The difference in the ground conditions between the liquefied sites and Uminonakamichi is not clear. Further study is necessary.

Damage to Slopes

Comparatively large slope failures occurred at three sites in Shikano-shima Island, as shown in Fig. 7. Photo 7 shows the biggest slide, which occurred on the east side of the island. Two other slides occurred on the west side of the island. Shadowed grids in Fig. 7 show the zones where several of the slopes were predicted to be unstable. The three sites in Shikano-shima are located mainly inside the landslide susceptible grids. Therefore, it can be said that the microzonation by the Kanagawa method was fairly valid for Shikano-shima island. However, in the east and south of Fukuoka City, no slope failure occured in the grids where unstable slopes had been predicted. As the epicenter was located in the northwest, seismic intensity in the eastern and southern areas might have been too low to cause slope failure.

In Fig. 6, the sites of slopes that had been predicted to be susceptible to failure based on site investigations and the sites where damage of artificially filled ponds had been predicted are plotted. These sites were not damaged during the 2005 Fukuoka-ken Seiho-oki earthquake. Seismic intensity at these sites was low because these sites are located mainly in eastern and southern areas also.

Damage to Buildings

As shown in Photo 8, several buildings suffered damage near the Kego Fault in the downtown area of Fukuoka City. The closed circles in Fig. 13 show the locations of the damaged buildings, investigated by the Seibu Branch of the Japan Society of Civil Engineers (2005). The area predicted to be susceptible to strong shaking is also drawn in the figure. Four damaged buildings are located inside the predicted area. However, other damaged buildings are located slightly outside the predicted area. The reason why the damaged area was slightly different from the predicted area is not clear. However, one of the authors found a small differential settlement on a road where buildings were damaged, as shown in Photo 9. The

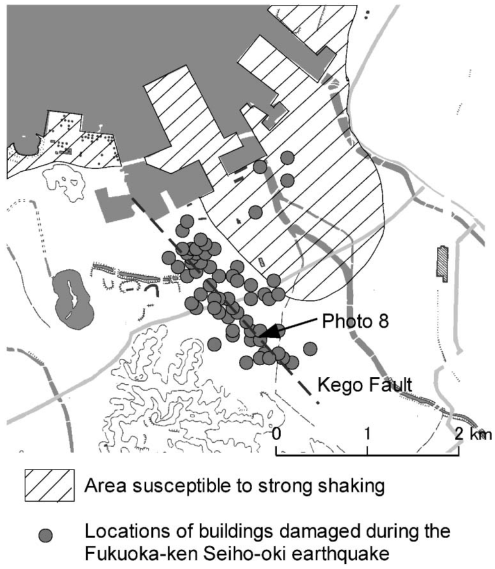


Fig. 13. Hazard map for strong shaking and locations of buildings damaged during the Fukuoka-ken Seiho-oki earthquake

Photo 9 Differential settlement observed above the Kego Fault

site is just above the Kego Fault. One possibility is, therefore, that some vertical differential displacement occurred along the fault and caused the damage to the buildings. More study on the mechanism of the damage to buildings is necessary in this area.

LIQUEFACTION ANALYSES CONDUCTED AFTER THE EARTHQUAKE

Analytical Procedure

The sites liquefied during the 2005 Fukuoka-ken Seiho-oki earthquake were mostly restricted to the reclaimed lands mentioned before. In order to clarify the reason for this peculiarity in the liquefaction phenomenon liquefaction analyses were performed based on the actual acceleration during the earthquake. In the analyses, seven boring logs along cross section 7–7′, shown in Fig. 3, were selected. These boring data are the same as the boring data used for the 1988 microzoning. Two borings near Hakata Bay at Sites 7–1 and 7–2 are located in reclaimed land. Other boring logs are located in alluvial plain. Liquefaction-induced sand boil and damage to structures occurred near Site 7–1 and Site 7–2, but no trace of liquefaction was observed near other sites during the 2005 Fukuoka-ken Seiho-oki earthquake. The analytical procedure is similar to the one used by the Fukuoka City Bureau (1988), with some slight differences, as follows:

1) A program for a one-dimensional seismic response analysis, "SHAKE," was used to obtain the maximum accelerations on the ground surface at the selected seven sites during the 2005 Fukuoka-ken Seihou-oki Earthquake.

2) The input seismic waveform from the basement layer at the seven sites for the analysis was estimated from the seismic record of the EW component measured on the ground surface at Fukuoka public hall, by reconverting into the basement layer. Figure 14 shows the input seismic waveform for the analysis, which was utilized by Nagase et al. (2006). The maximum basement acceleration is 174.9 cm/s^2.

3) The shear wave velocities of the soil layers at the seven sites were obtained from the empirical formulae shown in Eq. (2).

4) The strain dependencies of the dynamic shear modulus ratio, G/G_0, and damping ratio, h, were expressed using the empirical formulae shown in Eq. (3).

5) The soil boring logs used in the analysis are the seven boring logs shown in Fig. 3. Two borings near Hakata Bay are located in reclaimed land. Other boring logs are located in alluvial plain.

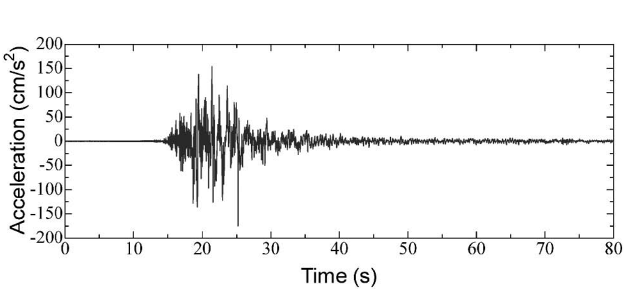


Fig. 14. Waveform used for one-dimensional seismic response analysis

6) The shear stress ratio during the earthquake, L, can be estimated directly from the analysis using the "SHAKE" program. However, for comparison with the previous estimate by the Fukuoka Waterworks (1988), the equation used by the Fukuoka Waterworks was used in this study. That equation is:

$$L = (1 - 0.015\,Z) \cdot k_{hc} \cdot \sigma_v / \sigma'_v \tag{5}$$

Z and k_{hc} denote the depth from the ground surface and the ratio of the maximum acceleration on the ground surface to the gravity acceleration, respectively. σ_v and σ'_v denote the vertical total and effective stress, respectively. In the estimate by the Fukuoka Waterworks, k_{hc} was assumed to be 200 cm/s^2, as mentioned before. On the contrary, k_{hc} during the 2005 Fukoka-ken Seiho-oki earthquake was evaluated from the seismic response analysis using the "SHAKE" program. The evaluated k_{hc} for the seven boring sites ranged from 0.181 to 0.220.

7) The liquefaction strength ratio R, was estimated by two simple methods. The first method is the above mentioned Eq. (1), which was derived in 1988 based on the test data taken in Fukuoka. The other method was introduced in the Specification for Highway Bridges after the 1995 Hyogoken-nambu earthquake based on the test data from many sites in Japan. These methods were called the Fukuoka method and the JRA method, respectively. The equation for the JRA method is as follows (JRA, 1996):

$$\left.\begin{aligned} &R = C_w R_L \\ &R_L = 0.0882\sqrt{N_a/1.7} && (N_a < 14) \\ &R_L = 0.0882\sqrt{N_a/1.7} + 1.6 \times 10^{-6}(N_a - 14)^{4.5} \\ & && (N_a \geq 14) \end{aligned}\right\} \tag{6}$$

where,

$$C_w = \begin{cases} 1.0 & (R_L \leq 0.1) \\ 3.3R_L + 0.67 & (0.1 \leq R_L \leq 0.4) \\ 2.0 & (0.4 \leq R_L) \end{cases}$$

for a near-field earthquake

$C_w = 1.0$ for a plate boundary earthquake

$N_a = c_1 N_1 + c_2$ for sandy soil

$$N_1 = 1.7N/((\sigma'_v/98) + 0.7)$$

$$c_1 = \begin{cases} 1 & (0\% \leq F_c \leq 10\%) \\ (F_c + 40)/50 & (10\% \leq F_c \leq 60\%) \\ F_c/20 - 1 & (60\% \leq F_c) \end{cases}$$

$$c_2 = \begin{cases} 0 & (0\% \leq F_c \leq 10\%) \\ (F_c - 10)/18 & (10\% \leq F_c) \end{cases}$$

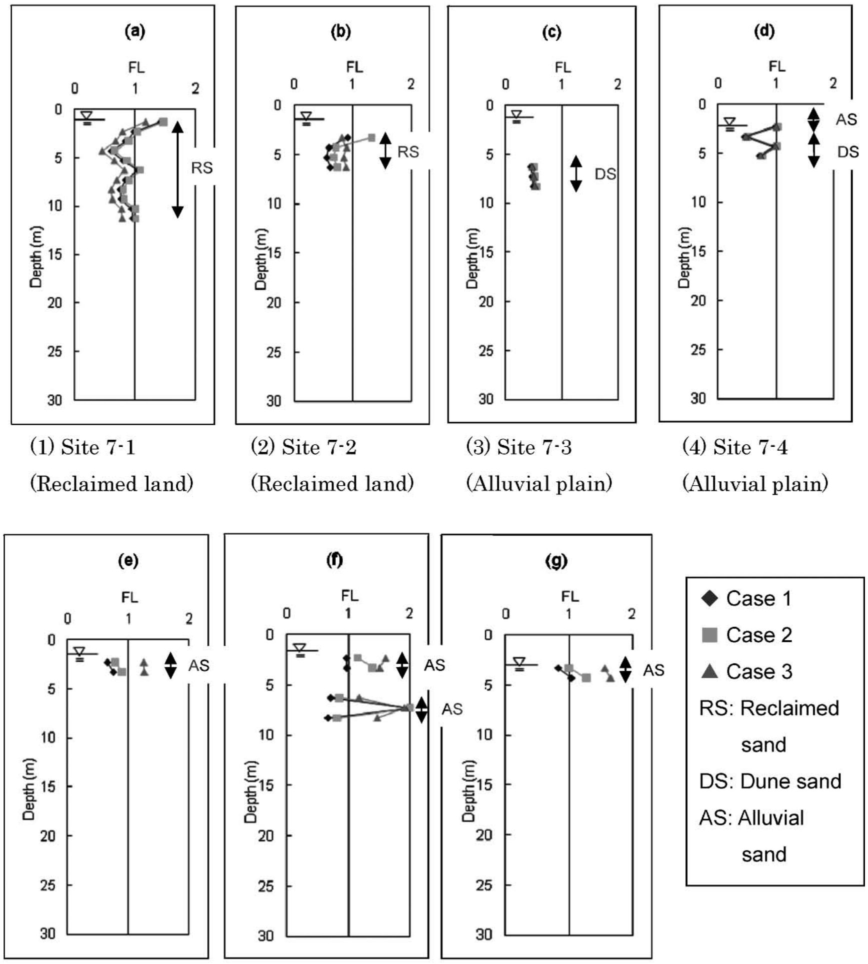
$N_a = [1 - 0.36 \log_{10}(D_{50}/2)] \times N_1$ for gravelly soil

In the JRA method, C_W and F_C must be determined. C_W was decided as 1.0 because the epicentral distance was about 25 km. As no soil tests at the seven sites were conducted, two sets of F_C for the soil layers at these sites were assumed based on Fig. 4. Then, the three cases of analysis shown in Table 1 were analyzed as follows:

Case 1: The JRA method was used. The fine content F_c

Table 1. Estimation method for L and R and assumed grain size of soil layers in liquefaction analyses

Case	Estimation method for L and R		Assumed grain size			
	L	R	Alluvial and artificially reclaimed soils		Artificially reclaimed soil	Dune sand
			Sand	Silty or clayey sand	Gravelly sand	
Case 1	Eq. (5)	JRA method	$F_C=20\%$	$F_C=60\%$	$D_{50}=2.0$mm	$F_C=0\%$
Case 2		JRA method	$F_C=40\%$	$F_C=80\%$	$D_{50}=2.0$ mm	$F_C=0\%$
Case 3		Fukuoka method	$D_{50}=0.35$ mm	$D_{50}=0.025$ mm	$D_{50}=2.0$ mm	$D_{50}=0.35$ mm



(1) Site 7-1 (Reclaimed land)
(2) Site 7-2 (Reclaimed land)
(3) Site 7-3 (Alluvial plain)
(4) Site 7-4 (Alluvial plain)
(5) Site 7-5 (Alluvial plain)
(6) Site 7-6 (Alluvial plain)
(7) Site 7-7 (Alluvial plain)

Fig. 15. Results of liquefaction analysis

was assumed to be 20% and 60% for the sand and silty or clayey sand, respectively, in the artificially reclaimed and alluvial soil layers. The value of 20% for the alluvial soil layer was decided from the grain size distribution curve shown in Fig. 4. The value of 60% for the alluvial silty or clayey sand layer was determined on the basis of the grain size distribution curves estimated by the Fukuoka City Waterworks Bureau (1988). The F_c values of the artificially reclaimed soil layer were assumed to be the same as those of the alluvial soil layer. The F_c value of the dune sand was assumed to be 0%, according to the data shown in Fig. 4. D_{50} was assumed to be 2.0 mm for the gravelly sand in the artificially reclaimed soil layer.

Case 2: The JRA method was used. The F_c values were assumed to be 40% and 80%. According to the Fukuoka City Waterworks Bureau (1988), the grain size distribution curves of the alluvial soil layer varied exceedingly, as the F_c values ranged form 20% to 80%. Therefore, in Case 2, the F_c values were assumed to be 40% and 80% for the sand layer and the silty or clayey sand layer, respectively. The F_c value of the dune sand was assumed to be 0%, as in Case 1. The D_{50} of the gravelly sand in the

Table 2. Estimated thicknesses of liquefied and surface non-liquefied layers

Case	Liquefied or non-liquefied layer	Site 7–1	Site 7–2	Site 7–3	Site 7–4	Site 7–5	Site 7–6	Site 7–7
Case 1	Liquefied layer	9.3 m	4.3 m	3.3 m	3.2 m	2.2 m	2.0 m	1.0 m
	Non-liquefied layer	2.3 m	2.5 m	5.8 m	2.3 m	1.6 m	1.8 m	3.0 m
Case 2	Liquefied layer	9.3 m	3.0 m	3.3 m	3.2 m	2.2 m	0 m	0 m
	Non-liquefied layer	2.3 m	3.8 m	5.8 m	2.3 m	1.6 m	3.8 m	4.0 m
Case 3	Liquefied layer	9.9 m	4.3 m	3.3 m	3.2 m	0 m	0 m	0 m
	Non-liquefied layer	1.8 m	2.5 m	5.8 m	2.3 m	3.8 m	3.8 m	4.0 m

artificially reclaimed soil layer was 2.0 mm.

Case 3: The Fukuoka method was used. The D_{50} value of the sand, silty or clayey sand and gravelly sand in the artificially reclaimed and alluvial soil layer were assumed to be 0.35 mm, 0.025 mm and 2.0 mm, respectively. The values of 0.35 mm and 0.025 mm were almost equal to the average values decided from the grain size distribution curves with F_c values from 20% to 40% and from 60% to 80%, respectively, on the basis of the data gathered by the Fukuoka City Waterworks Bureau (1988). The D_{50} value was assumed to be 0.35 mm for the dune sand layer based on Fig. 4.

8) The ratio of F_L, a safety factor against liquefaction, was obtained from the following equation:

$$F_L = R/L \tag{7}$$

Results of Analysis

Figure 15 shows the results of the liquefaction analysis at the seven sites for Cases 1–3. The thicknesses of the liquefied layers and the non-liquefied layers in the three cases are estimated from this figure and summarized in Table 2. Moreover, the relationships between the thicknesses of the liquefied layers and the non-liquefied layers are plotted in Fig. 16 together with the boundary to cause sand boil and/or liquefaction-induced damage proposed by Ishihara (1985). The F_L-values for the reclaimed sands at Site 7–1 and Site 7–2 are mostly less than 1.0 in Cases 1, 2 and 3. The thicknesses of the liquefied and surface non-liquefied layers at Site 7–1 are about 9.3 to 9.9 m and 1.8 to 2.3 m, respectively. The liquefaction that occurred and caused sand boils and damage to structures at Site 7–1 can be explained in every case. The thicknesses of the liquefied and non-liquefied layers are about 3.0 to 4.3 m and 2.5 to 3.8 m, respectively at Site 7–2. Liquefaction-induced damage can be explained in Cases 1 and 3, where the estimated thickness of the liquefied layer is the maximum (4.3 m) and the estimated thickness of the non-liquefied surface layer is the minimum (2.5 m).

At Site 7–3 and Site 7–4, it is estimated that liquefaction occurred in the dune sand layers in Cases 1, 2 and 3, because the dune sand does not have fines. However, it can be estimated that sand boils could not be spouted out onto the ground surface at Site 7–3 because the surface non-liquefied layer is thick, at about 5.8 m. At

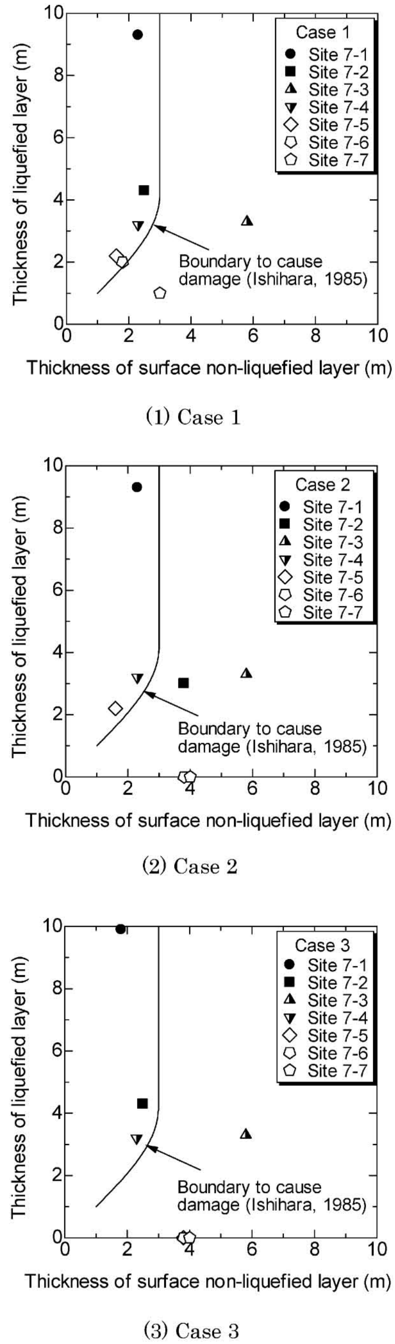


Fig. 16. Relationships between the thicknesses of liquefied layers and the non-liquefied layers estimated in three cases

Site 7–4, sand boil could be explained because the thicknesses of the liquefied and non-liquefied layers are near the boundary to cause sand boil, as shown in Fig. 16.

At Site 7–5 and Site 7–6, the estimated F_L of the alluvial soil layer are greater than 1.0 in Case 3. However, the F_L in Case 1 of the alluvial soil layer are less than 1.0. And the thicknesses of liquefied and surface non-liquefied layers are about 2.0 to 2.2 m and 1.6 to 1.8 m, respectively. Therefore, sand boil should have occurred at these sites under Case 1. There was, however, no trace of sand boil or liquefaction-induced damage at these sites. The apparent lack of liquefaction at these sites cannot be explained in Case 1 but can be explained in Case 3. In Case 2, liquefaction cannot be explained at Site 7–6 but can be explained at Site 7–5. Therefore, Case 3 was shown to be the most suitable for estimating liquefaction. This implies that the hazard map shown in Fig. 12 is appropriate under 177 to 216 cm/s^2 of maximum surface acceleration.

CONCLUSIONS

Microzonation for liquefaction, slope failure, and strong shaking during earthquakes in Fukuoka City, Japan was conducted in 1988 based on several analyses. Liquefaction was predicted in almost all of the artificially reclaimed land. Slopes susceptible to failure were located along several mountains and hills. Areas susceptible to strong shaking were distributed in the artificially reclaimed land and the central part of the city.

Seventeen years after the microzonation, the 2005 Fukuoka-ken Seiho-oki earthquake hit Fukuoka City. As the seismic intensity during the earthquake was similar to the intensity assumed for microzoning, actual damage was compared with the predicted damage. Liquefied areas coincided fairly well with the predicted areas. Slope failure occurred near the predicted zones, while damage to buildings occurred in slightly different zones.

Liquefaction analysis was performed on the same soil layer in which liquefaction had been predicted in the microzoning, using the waveform observed during the earthquake, and the results of liquefaction analysis were compared with the results of microzonation. The occurrence of liquefaction observed during the earthquake was suitably predicted by the microzonation.

ACKNOWLEDGMENTS

The analyzed results were discussed by a technical committee of Fukuoka City. Analyses for microzoning were conducted by Mr. Y. Yamamoto and Mr. S. Nakamura et al., former students at the Kyushu Institute of Technology. Site investigation on the damage due to the earthquake was conducted with Mr. K. Yamagata and Mr. S. Itoh, of Kiso-jiban Consultants Co., Ltd., and by Prof. S. Goto of Yamanashi University. The authors are grateful for this assistance.

REFERENCES

1) Fukuoka City Waterworks Bureau (1988): Evaluation on seismic behavior of water supply facilities and grounds, Part 1 (in Japanese).
2) Fukuoka City Waterworks Bureau (1989): Evaluation on seismic behavior of water supply facilities and grounds, Part 2 (in Japanese).
3) Ishihara, K. (1985): Stability of natural deposits during earthquakes, *Proc. 11th ICSMFE*, **1**, 321–376.
4) Iwasaki, T., Tokida, K., Tatsuoka, F., Watanabe, S., Yasuda, S. and Sato, H. (1982): Microzonation for soil liquefaction potential using simplified methods, *Proc. 3rd Int. Earthquake Microzonation Conf.*, **3**, 1319–1330.
5) Japan Road Association (1980): Specification for Highway Bridges (in Japanese).
6) Japan Road Association (1996): Specification for Highway Bridges (in Japanese).
7) Kanagawa Prefecture (1986): Prediction of Seismic Damage in Kanagawa Prefecture (in Japanese).
8) Lysmer, L., Udaka, T., Tsai, C. F. and Seed, H. B. (1975): FLUSH-A computer program for approximate 3-D analysis of soil-structure interaction problems, *Report No. EERC*75–30, University of California, Berkeley.
9) Nagase, H., Zen, K., Hirooka, A., Yasufuku, N., Kasama, K., Kobayashi, T., Maeda, Y., Uno, K., Hashimura, K. and Chen, G. (2006): Zoning for liquefaction and damage to port and harbor facilities and others during the 2005 Fukuoka-ken Seiho-oki Earthquake, *Soils and Foundations*, **46**(6), 805–816.
10) Schnabel, P. B., Lysmer, J. and Seed, H. B. (1972): SHAKE- A computer program for earthquake response analysis of horizontal layered sites, *Report No. EERC*72–12, University of California, Berkeley.
11) Seibu Branch of JSCE (2005): *Reconnaissance Report on the 2005 Fukuoka-ken Seiho-oki Earthquake* (in Japanese).
12) TC4, ISSMFE (1994): *Manual for Zonation on Seismic Geotechnical Hazards*, The Japanese Geotechnical Society.
13) Yasuda, S. and Yamaguchi, I. (1985): Dynamic soil properties of undisturbed samples, *Proc. 20th Japan National Conf. on SMFE*, 539–542 (in Japanese).
14) Yasuda, S. and Matsumura, S. (1991). Microzonation for liquefaction, slope failure and ground response during earthquakes in Fukuoka City, *Proc. 4th International Conference on Seismic Zonation*, **3**, 3–10.