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EFFECT OF LIQUEFACTION SUSCEPTIBILITY ON BUILDING DAMAGE DURING THE 1995 KOBE EARTHQUAKE

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SUMMARY

Soil data at 165 damaged and non-damaged buildings due to the 1995 Kobe earthquake were used to evaluate liquefaction effects on ground surface and building. Liquefaction resistance factor, F_L, which is presented in the Specifications for Highway Bridges [1] and the liquefaction potential index, P_L, which is defined by Iwasaki et al. [2] were computed. The computed data from the Kobe and its neighboring cities indicate that P_L value correlate well with surface effects of liquefaction: P_L value for Level-1 ground motion, specified as design seismic force in the former Specifications, exceeds 5 where severe effects occurred, whereas P_L value for Level-2 ground motion, specified in the present Specifications, equals to or less than 15 where no surface effects were observed. To evaluate effects of liquefaction occurrence on damage to building, liquefaction susceptibility was classified by P_L values: the site with a $P_L > 5$ for Level-1 ground motion is defined as high susceptibility, sites with $P_L \leq 15$ for Level-2 ground motion is defined as low susceptibility, and all other sites are defined as moderate susceptibility to liquefaction. Damage level to superstructure and foundation of the buildings correlate with liquefaction susceptibility at each site, as well as ground shaking intensity.

INTRODUCTION

In response to an experience in the 1995 Kobe (Hyogoken Nambu) earthquake, performance-based design approach has been introduced in seismic design codes and guidelines of various structures in Japan, namely the structures should be designed to satisfy several performance levels or limit states for a specified earthquake ground motion. The design has to be based on two levels of earthquake input, Level-1 and Level-2 motions [3]: Level-1 motion is equivalent to the conventional level of earthquake motion, which will be experienced once or twice during the service life of a structure. Level-2 motion with extremely high level of seismic force although the probability that structures will be experienced Level-2 motion is very low. Two types of earthquakes can most likely cause it: interplate earthquake occurring under the ocean and intraplate earthquake associated with inland active faults. Level-1 motion is comparable to the seismic loadings, which have been used in Japanese seismic codes before the 1995 Kobe earthquake, whereas Level-2 motion is a new type of input motion to be considered in design for the most of structures.

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For example, in the Specifications for Highway Bridges [1], design standard horizontal seismic coefficients are specified at approximately 0.3–0.8, depending on types of seismic motion and ground condition. Here, the horizontal seismic coefficient, which has been generally used as seismic force in Japan, is defined as coefficient used to multiply the weight of structure for calculating the lateral inertial force in seismic resistant design.

In evaluating liquefaction potential subjected to the Level-2 ground motion, even medium density sand and gravely sand would result in being liquefied, which implies that the greater part of low-lying land and reclaimed land in Japan would be expose to be risk of liquefaction hazard. However, in engineering purposes, it is not usually important to judge whether or not liquefaction will occur at a site, but to evaluate its consequence for damage to structures.

The purposes of the present study are: to find an effective indexes for assessing occurrence and severity of liquefaction generated by the Level-2 ground motion; and to evaluate damage to buildings from liquefaction due to the Level-2 motion. First, data for the Kobe earthquake was collected on (1) the state of damage/non-damage to foundation and superstructures of building, (2) soil conditions at each surveyed building, and (3) occurrence of liquefaction effects such as sand boils and water spouting around the buildings. Quantitative analyses were then performed on correlation between the occurrence and severity of liquefaction during the Kobe earthquake and various parameters that are considered to be controlled to liquefaction susceptibility, such as SPT N-value, liquefaction resistance factor, F_L, and the liquefaction potential index, P_L to find appropriate indexes in liquefaction assessment. Finally, correlations between the index representing liquefaction susceptibility at the site and the damage to building foundations and superstructures due to the Kobe earthquake were investigated to evaluate the effect of liquefaction occurrences and severity on building damage.

DATA BASE

After the 1995 Kobe earthquake, a number of field investigation and analysis on damage to building foundations were conducted and some of which were published in papers and reports, e.g. [4]. However, the names and locations of the buildings, which are required to collect the information on the site condition, have not been disclosed. This has prevented to perform comprehensive statistical investigations and analyses on effects of liquefaction and other factors on damage to building foundations.

In the present study, data for 165 cases of foundation damage/non-damage were collected and compiled for the database by identifying the locations of these buildings. The locations of the buildings are plotted in Fig.1 together with the area of liquefaction during the Kobe earthquake. Types of superstructure and foundation of the buildings and damage due to the earthquake is respectively shown in Figs. 2 to 5.

Noted that the building data are not based on the complete count survey but on the case histories of the field investigations on building foundations through excavations of foundation, integrity tests for piles and inspections using a borehole camera and clinometers. This indicates that rates of damage are higher than that of completely count survey data.

SEVIRITY OF LIQUEFACTION AT STUDY SITES

In present study, surface evidences of liquefaction were used as an index of extent and severity of the subsurface liquefaction. They were classified into 5 levels, based on the severity of sand boils determined based on information from the maps of ground failures and ground displacement vector diagrams for the

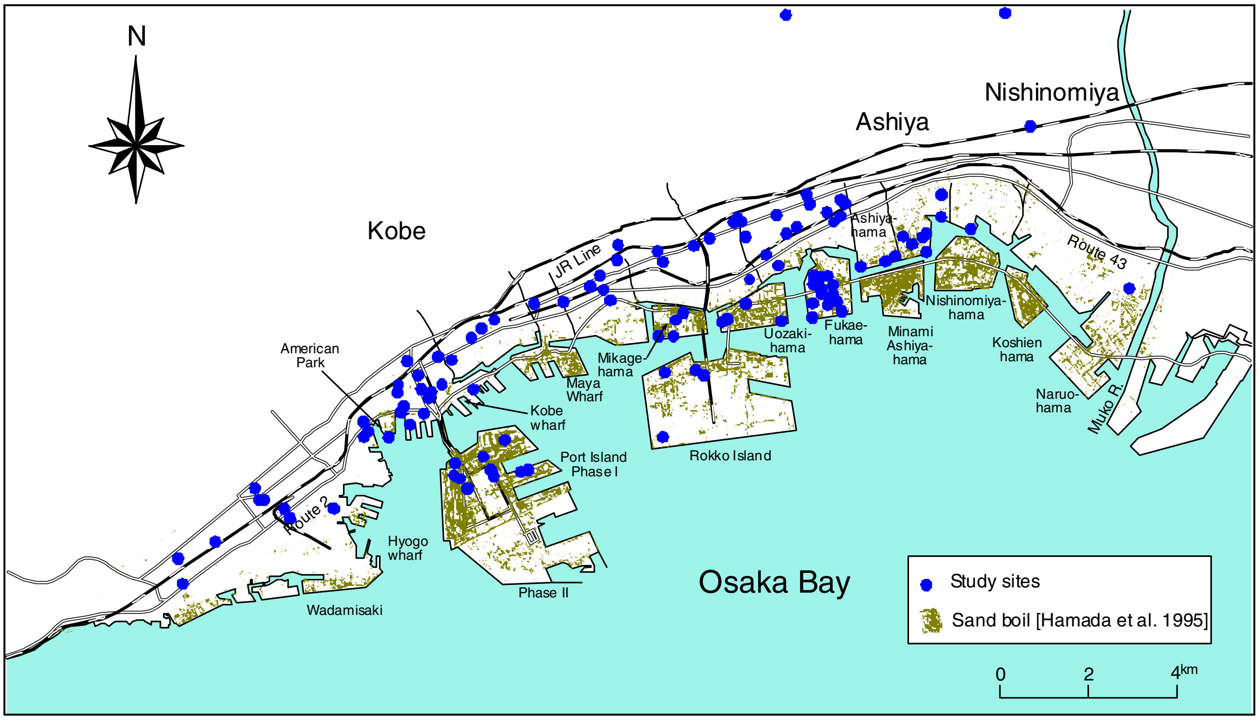


Fig.1. Locations of study sites and areas of liquefaction during the 1995 Kobe earthquake

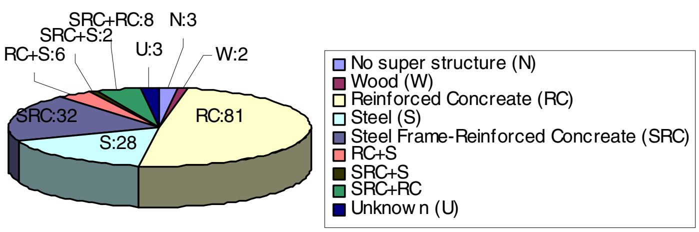


Fig.2. Structural types of surveyed buildings

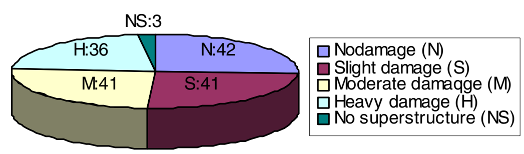


Fig.3. Level of damage to superstructures

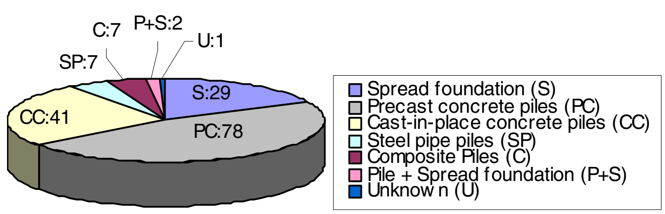


Fig. 4. Foundations type of the surveyed buildings

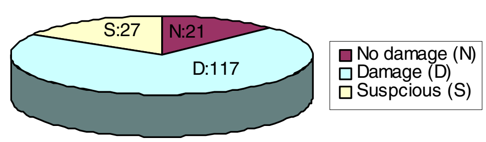


Fig.5. Presence or lack of damage to foundations

Kobe earthquake compiled by Hamada et al. [5], and from other reports and papers described the ground deformation at the building sites.

First, the study area was divided into coastal regions that experienced liquefaction over a wide area (zones a–d) and inland regions relative free from liquefaction (zone z). The coastal regions correspond to the area surveyed for ground failures in Hamada et al. [5]. In the inland regions falling outside Hamada's survey map, sand boils occurred only sporadically and on a significantly smaller scale than in coastal zones. The inland regions were thus classified as a single zone "z". Next, the coastal regions were classified into four zones, as summarized in Table 1, based on the severity of sand boils.

Table 1 Severity of liquefaction classified by surface evidences of liquefaction

Region	Severity of liquefaction	State of sand boils on the ground surface
Coastal region	a	Observed extreme amounts of sand boils in over 50% of the area within a 100-m radius of the subject building
	b	Observed large amounts of sand boils in 20–50% of the area within a 100-m radius of the subject building
	c	Observed sand boils in less than 20% of the area within a 100-m radius of the subject building
	d	No observed sand boils within a 100-m radius of the building but sand boils observed in the surrounding areas.
Inland region	z	No observed sand boils but no obvious sand boils observed in the surrounding areas.

ASSESSMENT OF LIQUEFACTION POTENTIAL

Liquefaction resistance factor, F_L, was computed with the simplified procedure introduced in the Specifications for Highway Bridges [1] for each building site. The method is similar to the Seed and Idriss [6] approach in that a soil liquefaction capacity factor, R, is calculated along with a dynamic load, L, induced in a soil element by the seismic motion. The ratio R/L is defined as the liquefaction resistance factor, F_L.

The liquefaction potential index, P_L defined by Iwasaki et al. [2], was also computed as an index to represent the severity of liquefaction effects expected on the surface. The index is defined as follows:

$$P_L = \int_0^{20} F(z)\,w(z)\,dz \qquad (1)$$

where z is the depth below the ground surface, measured in meters; $F(z)$ is a function of the liquefaction resistance factor, F_L, where $F(z)=1-F_L$ but, if $F_L>1.0$, $F(z)=0$; and $w(z)=10-0.5z$. Equation (1) gives values of P_L ranging from 0 to 100.

Soil layers requiring liquefaction assessment were selected based on the definitions in the Specifications for Highway Bridges [1]. When data on plasticity index, I_P, which is required to determine liquefiable soil layer were not obtained, layers with fines content, FC of < 35% were considered to be “liquefiable” for liquefaction assessment. The soil data collected in the present study were limited to those obtained from borehole logs and the SPT N-values. The soil constants such as fines content, FC and mean diameter of soil particle D_{50}, necessary for estimating liquefaction resistance and shear strength during earthquakes were assumed to be equivalent to the standard soil constants determined for the Kobe region for the respective soil types reported in the borehole logs [7].

Because the liquefaction factor, F_L, was evaluated as an index of susceptibility of liquefiable soils, input seismic force for calculating shear stress ratio, L, was adopted not observed peak ground acceleration at each site but design horizontal seismic coefficients defined in the specifications [1]. The seismic coefficients, k_{hc}, were assumed as follows in calculations of F_L values for Level-1 and Level-2 ground motions. For calculations of design horizontal seismic coefficients, the formula for Type II earthquake (near-field earthquake) was used for both Level-1 and Level-2 ground motions, since the present study deals case histories due to the Kobe earthquake. In the present Specifications for Highway Bridges [1], only the design seismic coefficient for Level-2 ground motion is defined. Thus, the coefficient for Level-1 ground motion was assumed to be 0.18, which was specified in the former Specifications [8], in the calculation of dynamic load induced in a soil element.

Based on the method specified in the 2002 Specifications for Highway Bridges, the design horizontal seismic coefficient, k_{hc}, which is the level-2 ground motion was calculated, assuming a modification factor, c_z, for zone A including the study area of 1.0 and design horizontal seismic coefficient at ground surface, k_{hG}, of 0.6 for ground type III. Dynamic shear strength ratio, R, was calculated using the modification factor based on earthquake motion properties, c_w, obtained with the formula for Type II ground motion.

INDEXES FOR SUSCEPTIBILITY TO LIQUEFACTION

One of important steps in evaluating liquefaction effects on structure is to identify the indexes representing liquefaction susceptibility at the site controlling liquefaction severity during an earthquake. In the following, correlation between various indexes and the severity of liquefaction during the Kobe earthquake will be examined.

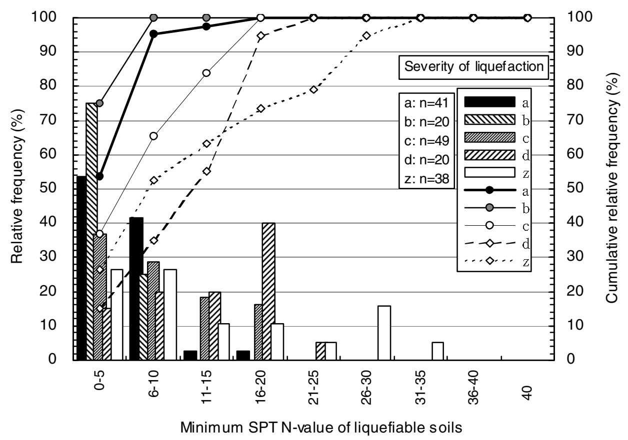


Fig. 6. Histogram and cumulative relative frequency of the minimum SPT-N value of liquefiable soils classified by the severity of liquefaction

Figure 6 is a histogram and cumulative relative frequency of the minimum SPT N-value of liquefiable soils at depths ranging from the groundwater table to 20 m, classified by the severity of liquefaction at each site. In sites a and b, which had suffered severe liquefaction, 95% or more of the sites had displayed minimum N-values of 10 or less. In contrast, sites with minimum N-values greater than 20, though small in number, experienced liquefaction severity of either d or z. The histogram indicates that in cases in which a layer with N-values of 20 or less is not present at depth to 20 m, surface ground disturbances are not expected to occur even during Level-2 ground motion.

Figure 7 shows histograms and cumulative relative frequencies of minimum liquefaction resistance factor, F_L, for Level-1 and Level-2 ground motions classified by the liquefaction severity. Zones a and b, which experienced a severe liquefaction, consist of sites with $F_L \leq 1$ for Level-1 ground motion in approximately 80-85% of the cases, and $F_L \leq 1.2$ in 90% or more of the cases. On the other hand, zones d and z consist of sites with $F_L > 1$ in approximately 75–85% of the cases.

For Level-2 ground motion, zones a and b consist of sites with $F_L \leq 0.4$ in approximately 95% or more of the cases,and zone c consists of grounds with $F_L \leq 0.6$ in approximately 90% of the cases. However, it must be noted that the minimum F_L value equals to or less than 1 in most cases for zone d, without any apparent liquefaction effects, and for zone z, which is unlikely to experience liquefaction. Although the liquefaction may partially occurred within the ground in zones d and z even when its effect is not apparent on the surface, the histograms may suggest that the F_L values calculated for Level-2 ground motions excessively under-evaluate the actual liquefaction resistance.

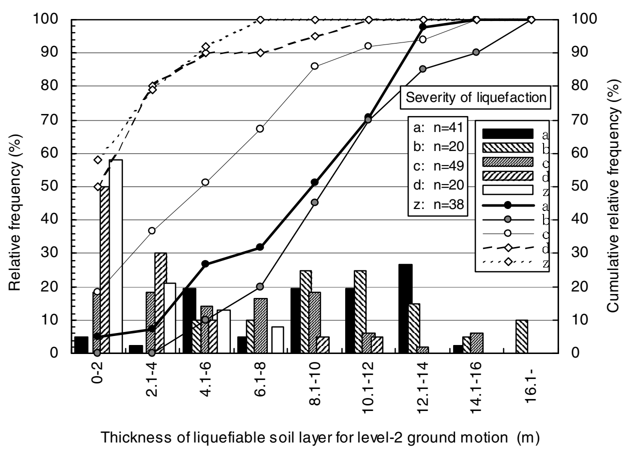
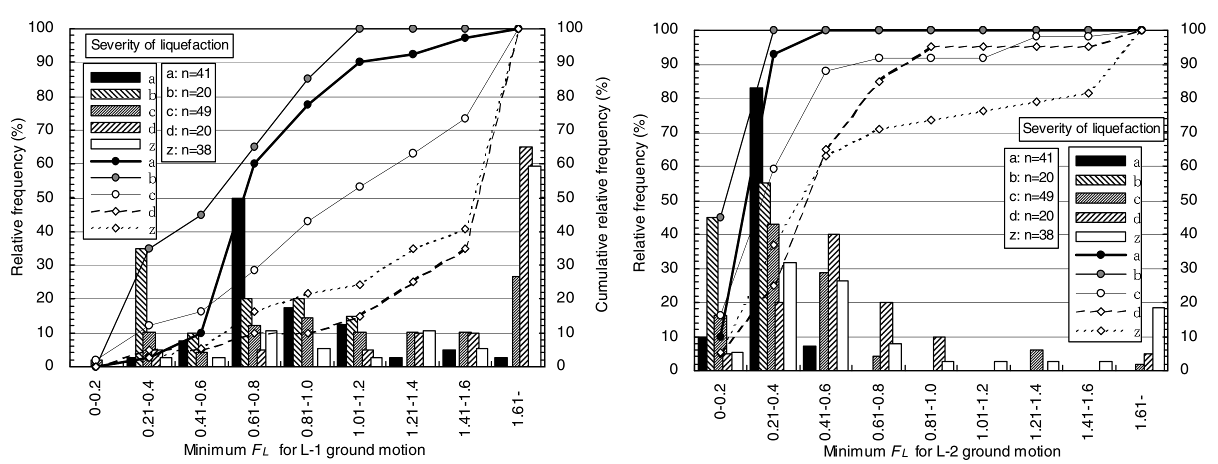


Fig. 8. Histogram and cumulative relative frequency of the thickness of the estimated liquefiable soil layer for Level-2 ground motion classified by the severity of liquefaction

Figure 8 is a histogram and cumulative relative frequency of the thickness of the estimated liquefiable layer for Level-2 ground motion classified by the liquefaction severity. Only the results for Level-2 ground motion are shown in the figure, as in most of the cases of Level-1 ground motion, none of the soil layers to a depth of GL-20 m displayed an F_L value of 1.0 or less, and the liquefiable soil layer could not be determined. The figure indicates that approximate 75–90% of the sites in zones a and b have liquefiable layers with $F_L \leq 1$ are 4 m or greater in thickness. In contrast, 50% or more of the sites in zones d and z are 2 m or less in thickness, and 90% of the sites are 6 m or less in thickness.



(a) Minimum F_L values for Level-1 ground motion (b) Minimum F_L values for Level-2 ground motion

Fig.7. Histograms and cumulative relative frequencies of minimum liquefaction resistance factor, F_L for Level-1 and Level-2 ground motions classified by the severity of liquefaction

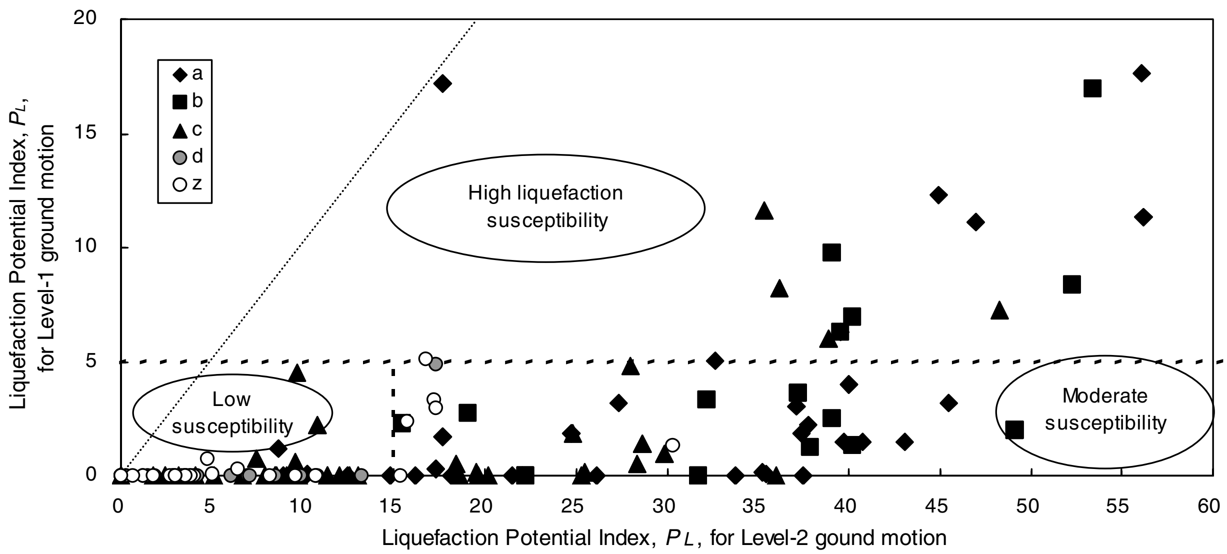
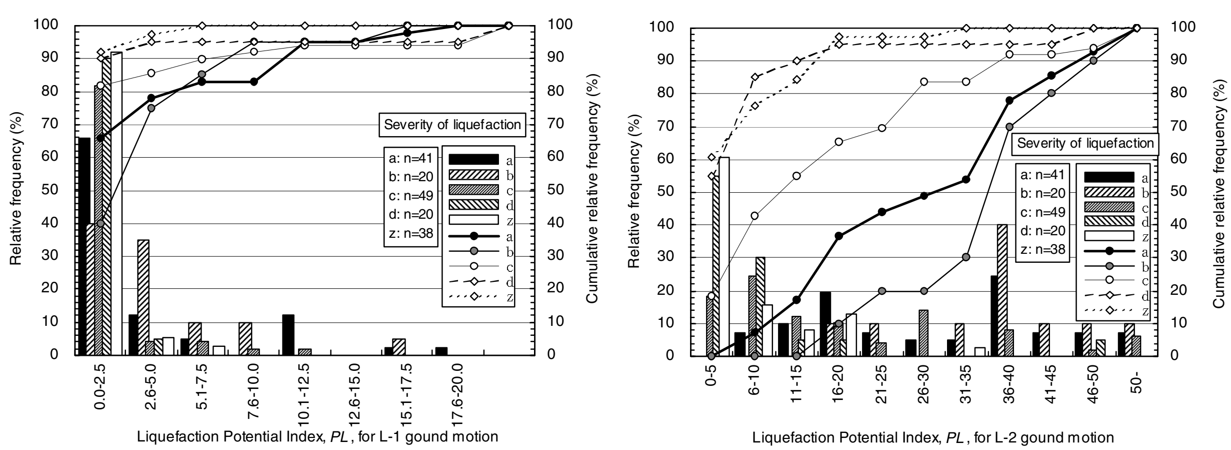


Fig. 10. Relationship between P_L values determined for Level-1 and Level-2 ground motions

Besides Figs. 6-8, a histogram of the depth to the upper surface of the estimated liquefiable layer was plotted. Of the sites assessed to occur liquefaction during Level-2 ground motion, 90% or over had depths to an upper surface of estimated liquefiable soil layer above GL-10 m, with 10% above GL-2 m. However, no clear trend was observed, respectively for zones a–z.

Figure 9 is a histograms and cumulative relative frequencies of the liquefaction potential index, P_L of the ground classified by the liquefaction severity for (a) Level-1 ground motion and (b) Level-2 ground motion. Figure 9 (a) indicates that the P_L value is equal to or less than 2.5 at 90% or more of the sites in zones d and z, and that there are only 2 sites with $P_L > 5$: one in zone z and another in zone d. For zones a and b, the P_L values are greater than 5 at approximately 25% of the sites. Sites with $P_L < 5$ are present in all zones, but sites with $P_L > 5$, which indicates that the sites underlain by a thick susceptible sand, are virtually restricted to zones a–c.



(a) Level-1 ground motion

(b) Level-2 ground motion.

Fig. 9. Histograms and cumulative relative frequencies of the liquefaction potential index P_L for Level-1 and Level-2 ground motions classified by the severity of liquefaction

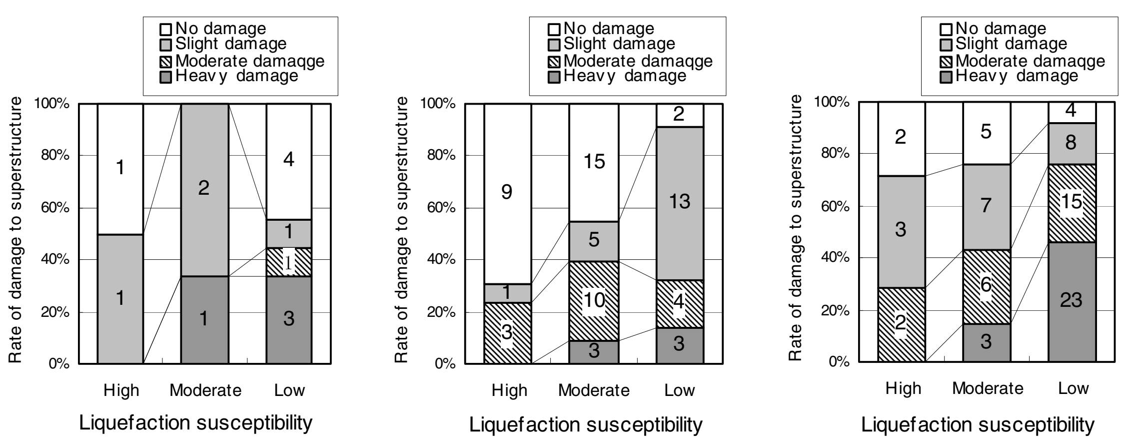
Figure 9 (b) for Level-2 ground motion indicates that in zones a and b, which have severe disturbance of liquefaction, 85% or more of the sites have $P_L \geq 15$. In contrast, 85% or more of the sites in zones d and z have $P_L \leq 15$. Accordingly, for Level-1 ground motion, P_L value of 5 may define the lower limit for loose deposit that is extremely susceptible to liquefaction, while a P_L value of 15 for Level-2 ground motion may define the upper limit for dense deposit, which is assessed to be free from the effects of liquefaction.

Relationships between P_L values determined for Level-1 and Level-2 ground motions are shown in Fig. 10. The dotted lines in the figure are the boundaries for degree of susceptibility to surface effects of liquefaction ("liquefaction susceptibility" hereafter), according the definition described above, i.e., locations with P_L values > 5 for Level-1 ground motion are defined as having high susceptibility to liquefaction, and locations with P_L values ≤ 15 for Level-2 ground motion are defined as having low susceptibility. All other locations correspond to having moderate susceptibility to liquefaction. Only the sites in zones a–c fall within the high liquefaction susceptibility, while most of the sites in zones d and z fall within the low susceptibility. Although approximately 15% of the sites in zones a and b is defined as low susceptibility, and 10–15% in zones d and z as high or moderate susceptibility, suggesting that the liquefaction susceptibility classified by the P_L values in Fig. 10 are sufficiently consistent with the actually observed liquefaction severity.

EFEECTS OF LIQUEFACTION SUSCEPTIBILITY ON BULDING DAMAGE

It is generally known that occurrences of liquefaction reduce the structural damage induced to ground shaking because of the attenuated ground motion due to nonlinear behavior of the liquefied deposits. However, the so-called "quake-absorbing effect" of liquefied deposits has not been confirmed quantitatively based on the case histories of past earthquakes.
To evaluate the effects of liquefaction occurrence on damage to superstructure and foundation of building, correlation between the building damage during the Kobe earthquake and liquefaction susceptibility defined by P_L value was examined.

Figure 11 shows rate of damage level of superstructures of the buildings classified by the liquefaction susceptibility of each building site for three levels of peak acceleration. Because observed peak accelerations at the ground surface during the Kobe earthquake were considered to be affected by



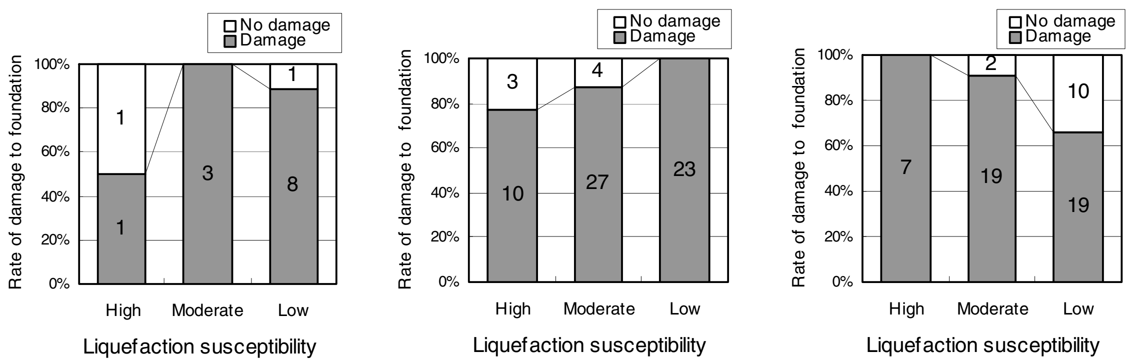
(a) 400–500 cm/s^2 (b) 500–600 cm/ s^2 (c) >600 cm/ s^2

Fig.11 Damage level of superstructures of the buildings classified by the liquefaction susceptibility of the site for simulated peak acceleration at the engineering seismic base layer

occurrence of liquefaction, values of the peak acceleration at the engineering seismic base layer with shear wave velocity ranging 500-600 m/sec, which was simulated by Sugito [9], were adopted in the present study.

No heavy damage to superstructures was observed in Fig. 11 for the sites with high liquefaction susceptibility regardless of the levels of peak acceleration at the engineering seismic base layer. In contrast, heavy damage to superstructures was observed for the sites with low susceptibility for all level of the peak accelerations studied here. Although heavy damage to superstructures was also observed for the sites with moderate liquefaction susceptibility, rate of the heavy damage is less than that for the sites with low susceptibility.

Figure 12 shows observed seismic intensity on JMA (Japan Meteorological Agency) Scale of each survey site classified by the liquefaction susceptibility for three levels of peak acceleration at the engineering seismic base layer. The sites with high susceptibility primarily experienced ground motion of the JMA intensity 6 (IX on the MM and MSK scales), regardless of the levels of peak acceleration at the base layer. In contrast, the sites with low susceptibility, present more areas of the JMA intensity 7 (X-XII on the MM



(a) 400–500 cm/s^2 (b) 500–600 cm/ s^2 (c) >600 cm/ s^2

Fig.13. Presence or lack of damage to foundation classified by the liquefaction susceptibility of the sites for simulated peak acceleration at the engineering seismic base layer

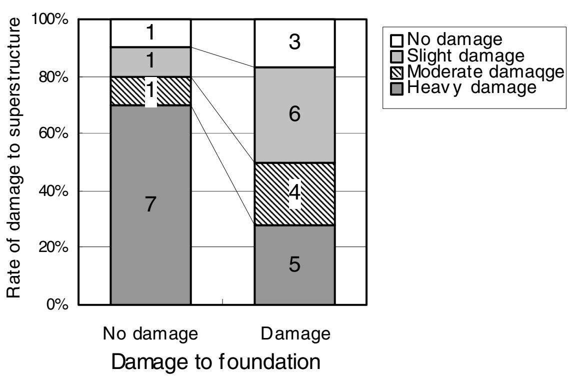
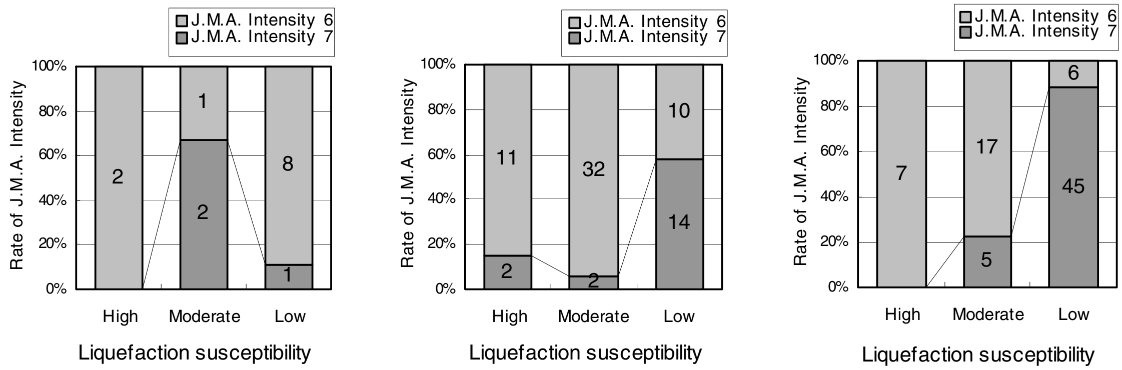


Fig.14 Level of the damage to superstructures classified by the presence or lack of damage to foundation, for the sites with low liquefaction susceptibility within the areas for peak acceleration exceeds 600 cm/s^2 at the engineering seismic base layer

and MSK scales), with higher levels of acceleration at the base layer. Although the sites moderate susceptibility, present some areas of the JMA intensity 7, rate of intensity 7 is less than that for the sites with low susceptibility except the sites with only three data points for peak acceleration of 400-500cm/s^2 in Fig.12 (a).

Figures 11 and 12 suggest that the surface ground at the sites with high liquefaction susceptibility behaved in a strongly nonlinear manner due to liquefaction, which controlled the amplification of ground motion, resulting in suppression of the heavy damage to superstructures of the buildings. In contrast, at the sites with low susceptibility, the intensity of the ground motion on the surface amplifies in proportion to the levels of the ground motion at



(a) 400–500 cm/s^2 (b) 500–600 cm/ s^2 (c) >600 cm/ s^2

Fig.12. JMA intensity of each survey site classified by the liquefaction susceptibility for simulated peak acceleration at the engineering seismic base layer

the engineering seismic base layer, resulting in inducing the heavy damage to superstructures. This implies the effect of so-called "quake-absorbing effect" of liquefied ground.

Figure 13 shows rate of presence or lack of damage to the building foundation classified by the liquefaction susceptibility at the sites for three levels of ground motion at the engineering seismic base layer. For the sites with high susceptibility, the rate of damage to foundations increases from Figs. (a) to (c), with increasing the levels of ground motion. In contrast, for the sites with low susceptibility, the rate of damage to foundations decreases from Figs. (a) to (c), with increasing the levels of ground motion.

With increases in the degree of liquefaction susceptibility, the rate of damage to foundation increases in the areas subjected a peak acceleration of excess of 600 cm/s^2 at the seismic base layer in Fig.13 (c). Whereas, the rate of damage to foundations decreases with increases in liquefaction susceptibility, in the areas experienced a peak acceleration equals to or less than 600 cm/s^2 in Figs. (a) and (b).

To identify the cause of above tendencies, the relationship between the presence or lack of damage to foundations and the level of the damage to superstructures was examined solely for the sites with low susceptibility within the areas where peak acceleration exceeds 600 cm/s^2 (Fig. 14). Seventy percent of the buildings with no damage to foundation which located in the areas with low liquefaction susceptibility suffered heavy damaged to superstructures and 10 % of the buildings with no damage to foundation suffered moderate damaged to superstructure. In contrast, 50 % of the buildings with damage to foundation corresponds to that with no or slight damage to superstructures. This implies that the inertial force due to the superstructure was reduced resulting from the collapse of superstructures of the buildings, located in the areas with low liquefaction susceptibility and subjected higher level of the ground motion at the engineering seismic base layer, which might resulted in reduction of seismic load on foundations.

CONCLUDING REMARKS

Liquefaction potential index, P_L correlate well with severity of surface effects during the 1995 Kobe earthquake, which indicates that it is useful index for assessing the susceptibility to surface effects of liquefaction (liquefaction susceptibility) of the sites subjected to the Level-2 ground motion, as well as level-1 motion for which Iwasaki et al. [2] applied and evaluated at 85 sites. Damage to superstructures of buildings and foundations correlate with liquefaction susceptibility defined by P_L value, as well as ground shaking intensity. With increases in the degree of liquefaction susceptibility, the rate of heavy damage to superstructures decreases regardless ground shaking intensity studied here, which suggests the quake-absorbing effect of liquefaction due to the nonlinear behavior of ground. The rate of damage to foundation also decreases with increasing the degree of liquefaction susceptibility, in the areas subjected peak acceleration of 600 cm/s^2 or less. However, rate of damage to foundation increase with increase in the degree of susceptibility in the areas experienced a peak acceleration of excess of 600 cm/s^2. The reason for this trend was found to be reduction of the inertial force from the superstructure due to collapse of superstructures in the sites with low liquefaction susceptibility within the areas of the highest acceleration.

To evaluate occurrence and severity of liquefaction, and its effects on building damage in future earthquakes, more investigations should be performed based on data sets from different areas and different earthquakes. Nevertheless, the present study found the unconfirmed trends of damage to superstructure and foundation of buildings in the severely liquefied and non-liquefied areas due to an extremely strong earthquake shaking.

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REFERENCES

1. Japan Road Association. “Specifications for highway bridges, Part V Seismic design.” Tokyo: Maruzen Company Ltd, 1996 and 2002 (in Japanese).
2. Iwasaki, T., Tokida, K., Tatsuoka, F., Watanabe, S., Yasuda, S. and Sato, H. “Microzonation for soil liquefaction potential using simplified methods.” Proceedings of 3rd International Conference on Microzonation, Seattle, 1982; Vol.3: 1319-1330.
3. The Central Disaster management Council, Cabinet Office, Government of Japan. “The basic disaster management plan.” 1995 (in Japanese).
4. Editorial Committee for the Report on the Hanshin-Awaji earthquake Disaster. “Damage to Building Foundation.” The report on the Hanshin-Awaji earthquake disaster, building series Vol. 4. Tokyo: Maruzen Company Ltd., 1998: 259-547.
5. Hamada, M., Isoyama, R. and Wakamatsu, K. “The 1995 Hyogoken-nanbu (Kobe) earthquake, liquefaction ground displacement and soil condition in Hanshin area.” Tokyo: Association of for Development of Earthquake Prediction, 1995.
6. Seed, H. B. and Idriss, I.M. “Simplified procedure for evaluating soil liquefaction potential.” J. Soil Mech. Foundation Division, American Society of Civil Engineers 1971; 97(9): 1249-1273.
7. Wakamatsu, K. and Numata, A. “Soil constants for liquefaction assessment in Hanshin area,” Proc. 56th Annual Meeting of Japan Society of Civil Engineers 2001; 3: 216-217 (in Japanese).
8. Japan Road Association. “Specifications for highway bridges, Part V Seismic design.” Tokyo: Maruzen Company Ltd, 1990 (in Japanese).
9. Sugito S. [1996]. “Simulation of near field ground motion by evolutionary spectrum model”. Proceedings 1st Symposium on the Mitigation of Urban Disasters Caused by Near-Field Earthquakes, Tokyo, 1996: 99-102 (in Japanese).