SPECIAL ISSUE OF SOILS AND FOUNDATIONS 109-118, Jan. 1996
Japanese Geotechnical Society

SOIL CHARACTERISTICS AND GROUND DAMAGE

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ABSTRACT

Soil conditions in reclaimed islands which were devastated by extensive liquefaction during the Kobe earthquake are described together with the soil strength which was obtained from the laboratory tests on undisturbed samples. Interpretation of this soil strength is given in the light of back-calculated shear strength based on accelerations recorded on the ground surface.

Observed settlements of the ground surface resulting from liquefaction are described and evaluated in terms of the values estimated based on existing methodology. Finally, the outcome of an in-situ survey on permanent deformations of the land areas behind a quaywall or revetment is presented. As a result it was possible to identify the characteristic patterns of the displacements which vary with the distance from the waterfront inland.

Key words: coarse-grained soil, earthquake, liquefaction, settlement (**IGC:** D7/E8)

INTRODUCTION

The violent ground shaking during the Hyogoken-Nambu earthquake caused landfills in the port area to liquefy leading to widespread occurrence of ground deformation including settlement and lateral spreading. As a result of the ground distortion, various types of damage were incurred by infrastructure such as lateral shifting of quaywalls, breakage of gas and water mains. The landfills in the several affected areas were first reclaimed around 1953 by transporting soil from borrow areas at the foothill of the Rokko Mountains. Large amount of the soils was transported by a long-distance belt conveyer system to the sites of reclamation along the old shoreline. Several islands north of Rokko Island were constructed during this period. The second phase of land reclamation began in 1968 to construct two large man-made islands further offshore to the south. Port Island with an area of 436 hectare was constructed during the period from 1966 to 1980 by transporting soils from Suma which were carried in bottom dump-barges and placed under water at the site of reclamation. The 580 hectare-Rokko Island was constructed between 1972 and 1990 by excavating soil materials from Suma and also from other sites in the Rokko Mountains. The grain size distribution of the materials used for the reclamation of Port and Rokko Islands are shown in Fig. 1.

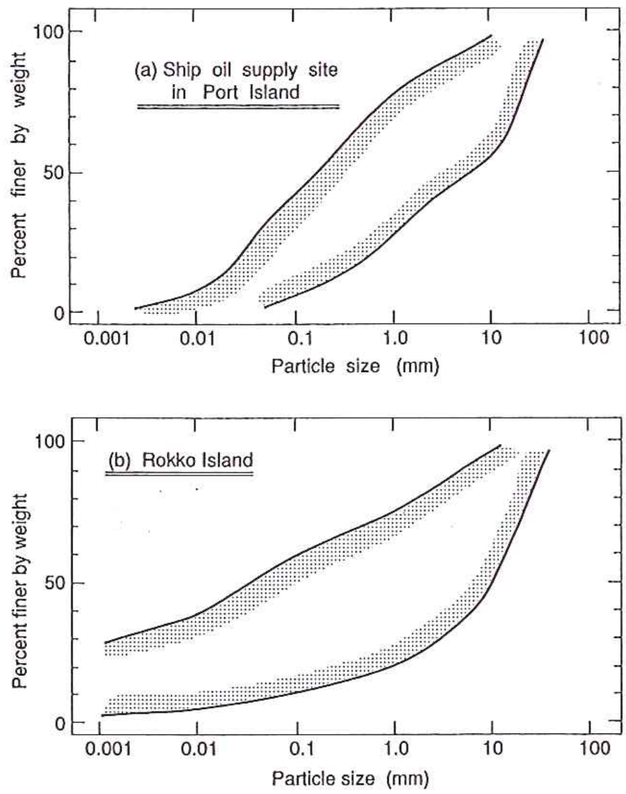

Fig. 1. Grain size distribution of the materials used for land reclamation: (a) Ship oil supply site in Port Island; (b) Rokko Island

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Manuscript was received for review on August 17, 1995.
Written discussions on this paper should be submitted before August 1, 1996 to the Japanese Geotechnical Society, Sugayama Bldg. 4F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101, Japan. Upon request the closing date may be extended one month.

LABORATORY TESTS

Prior to the earthquake a series of cyclic triaxial tests was performed by Nagase et al. (1995) on undisturbed samples of Masado soils recovered from the bottom of a cut. The location of the excavation is shown in Fig. 2. The reclaimed site in the northern section of Port Island was excavated to provide an approach to the gate of a freeway tunnel which was under construction. Undisturbed samples were recovered in block and frozen in the field before they were brought to the laboratory. After carefully trimming into specimens 7.5 cm in diameter and 15 cm in height, they were placed in the triaxial chamber and consolidated to varying effective ambient pressures. In some tests, the consolidation pressure was reduced to produce a state of overconsolidation with the OCR-value of 2 and 4.

The grain size distribution curves of the samples tested are shown in Fig. 3 where it can be seen that the soil consists of about 5% fines, 55% gravel and 40% sand. The gravel is defined here as a material having a grain size larger than 2 mm, and the fines as the soil having a particle size smaller than 0.074 mm. It should be noted that the Masado soil prevailing in the coastal area of Kobe is well-graded, containing a fairly large proportion of gravel. Such a soil has generally been considered as a material practically "immune to liquefaction" and not been specified for detailed studies in the laboratory as well as in the field.

The results of the cyclic loading tests are presented in Fig. 4 in terms of the cyclic stress ratio plotted versus the

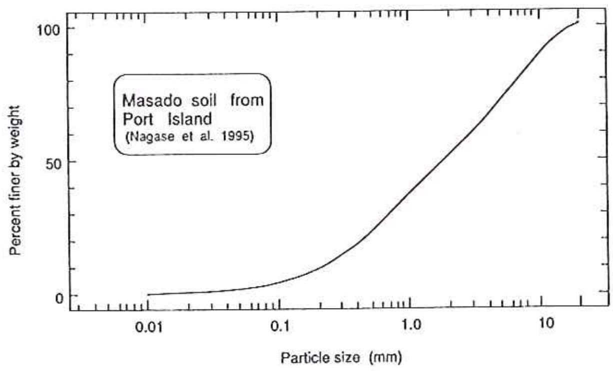

Fig. 3. Grain size distribution curve for the Masado soil tested

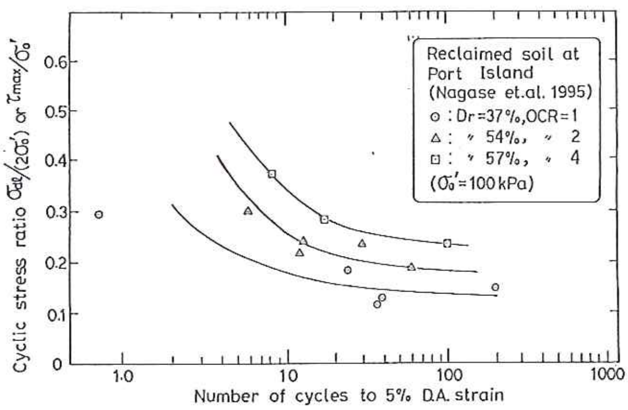

Fig. 4. Cyclic stress ratio versus number of cycles (Nagase et al. 1995)

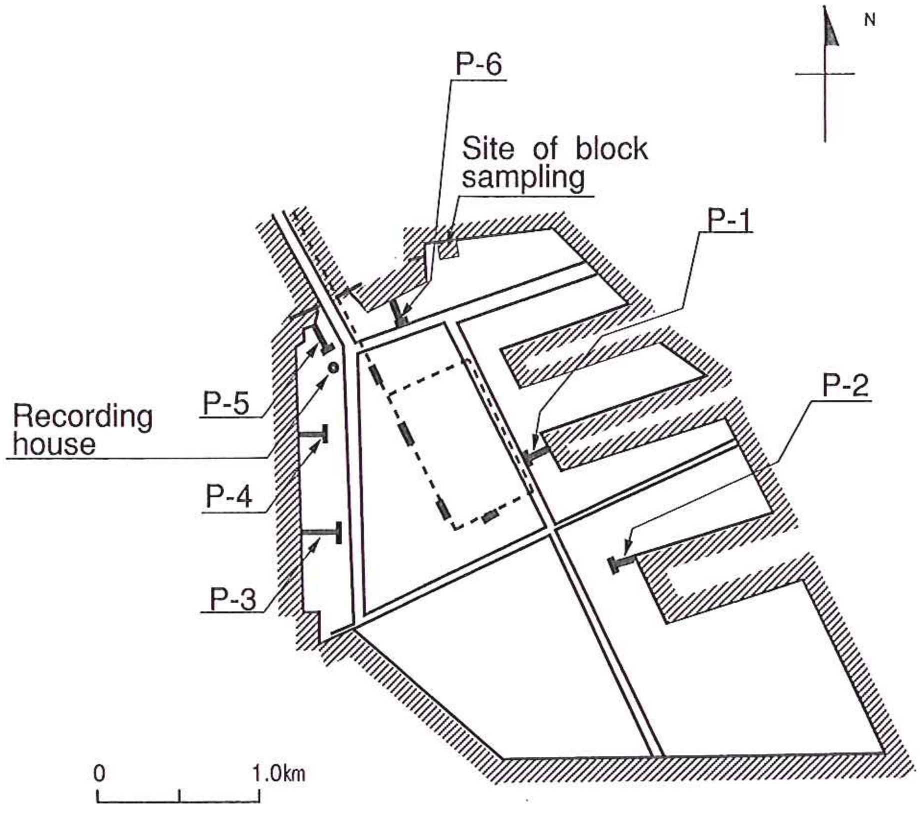

Fig. 2. Locations of the recording house and displacement measurements behind a quaywall in Port Island

number of cycles required to produce cyclic softening with development of 5% double-amplitude axial strain in the sample. Figure 4 indicates that the cyclic strength, defined as the cyclic stress ratio causing 5% D.A. axial strain in 20 cycles of load application, is about 0.18 for normally consolidated samples, which is a value commonly encountered in a loose to medium loose sand in a normally consolidated state. It can also be seen in Fig. 4 that the cyclic strength increases significantly with increasing ratio of overconsolidation, taking on a value of 0.3 and 0.4, respectively, for the OCR of 2 and 4. The influence of OCR as stated above may be put in an empirical formula,

$$\left(\frac{\sigma_{dl}}{2\sigma_o'}\right)_{\mathrm{OCR}}=(\mathrm{OCR})^n\cdot\left(\frac{\sigma_{dl}}{2\sigma_o'}\right)_N \tag{1}$$

where $(\sigma_{dl}/2\sigma_o')_N$ and $(\sigma_{dl}/2\sigma_o')_{\mathrm{OCR}}$ denote respectively the cyclic strength of normally and overconsolidated specimens. The exponent n was shown to have a value of 0.5 by Ishihara and Takatsu (1978) based on cyclic triaxial tests on clean sand. Nagase et al. (1995) showed a value of $n=0.40-0.45$ for the undisturbed samples obtained from the site in Port Island.

Another set of triaxial test results on undisturbed samples from Port Island was performed by Yasuda (1990). The samples were recovered by what is called triple-tube sampling and tested using the cyclic triaxial test apparatus. The outcome of these tests is presented in Fig. 7, together with data for the case of OCR=1 and 2 by Nagase et al. (1995) quoted from Fig. 4. It may be seen that the test data from Yasuda (1990) shows a cyclic strength of about 0.25 for 20 cycles of load application which is a value larger than the data obtained by Nagase et al. (1995) for the case of OCR=1.0.

RELATIVE DENSITY INTERPRETATION

According to the method of JSSMFE (Japanese Society of Soil Mechanics and Foundation Engineering), it is stipulated that the maximum and minimum void ratio $e_{\max}$ and $e_{\min}$ be determined for the predominantly sand portion having particle size smaller than 2 mm while keeping its fines passing #200 mesh less than 5%. For the Masado soil, $e_{\max}$ and $e_{\min}$ were determined by Nagase et al. (1995) in accordance with these requirements, yielding values of $e_{\max}=1.098$ and $e_{\min}=0.526$. These values may well be taken as the limiting void ratios for the sand portion contained in the Masado soil.

The void ratio of the sand portion in the undisturbed samples can be determined without difficulty as follows. Suppose the gross void ratio of an undisturbed sample is assumed to be $e=0.48$, for example. If the gravel fraction having particle size larger than 2 mm is 42% in weight, then the void ratio for the silt-sand portion e_s can be readily calculated as $e_s=0.48/(1-0.42)=0.828$. The void ratio thus determined for the silt-sand portion may be considered to represent the state of packing of the major constituent of soil which was responsible for the inducement of liquefaction during the Kobe earthquake. The limiting void ratios determined above by the method of JSSMFE are thus taken as the upper and lower bounds of the possible values of void ratio e_s for which the relative density of the silt-sand portion can be most logically defined. In the particular example stated above, the relative density is determined as 47% for the silt-sand portion. The relative density thus obtained is indicated in Fig. 4 for the specimens used in the cyclic triaxial tests on the undisturbed specimens from the site of Port Island. It may be seen that the cyclic strength as correlated with the relative density for the gravel containing silty sand is about the same as the correlation between these quantities that have been established for clean sand or silty sand. It should be noted therefore that if the relative density is determined for the silt-sand matrix by scalping the oversized gravel portion, it may be used as an index property to identify the liquefiability of a composite soil containing silt, sand and gravel. This concept is applicable only for the case of gravel content probably less then about 50% where gravel particles exist not in contact with each other in the matrix of silt and sand. For this condition, the presence of gravel may be assumed not to exert any influence on the overall resistance of the soil.

BACK CALCULATION OF CYCLIC STRENGTH OF SOIL

As described in the report by Toki (1995), a set of records was monitored by four accelerometers in a vertical array installed at a site in the northern part of Port Island. The soil profile and the time histories for two horizontal components obtained by Toki (1995) is reproduced in Figs. 5 and 6. The manmade fill to the depth of 18 m is composed basically of the same Masado soil as that at the tunnel site where undisturbed samples for the laboratory testing were recovered. In view of the low SPT N-value, about 5 to 10, the cyclic softening due to liquefaction appears to have taken place mainly in this near-surface deposit. In fact, the second thrust of acceleration in the south direction was shown to decline significantly at the surface (341 cm/sec^2) and at the depth of 16 m (340 cm/sec^2). A similar decrease in acceleration was also observed in the E-W component. It is, therefore, highly likely that the cyclic softening due to liquefaction occurred during one to two cycles of seismic load application. In addition, it may be reasonable to assume that the peak acceleration at the onset of liquefaction was $a=a_{\max}=340$ cm/sec^2. The cyclic stress ratio τ/σ_v' at any instant of seismic shaking is estimated by the equation

$$\frac{\tau}{\sigma_v'}=\frac{a}{g}(1-0.015Z)\frac{\sigma_v}{\sigma_v'} \tag{2}$$

where a denotes the acceleration and σ_v and σ_v' are the total and effective overburden pressures acting on a soil element located at a depth of Z (in meter). The depth of the ground water table at this location was estimated to be approximately 3 m. Introducing the value of $a=a_{\max}=340$ cm/sec^2 and $g=980$ cm/sec^2, the maximum

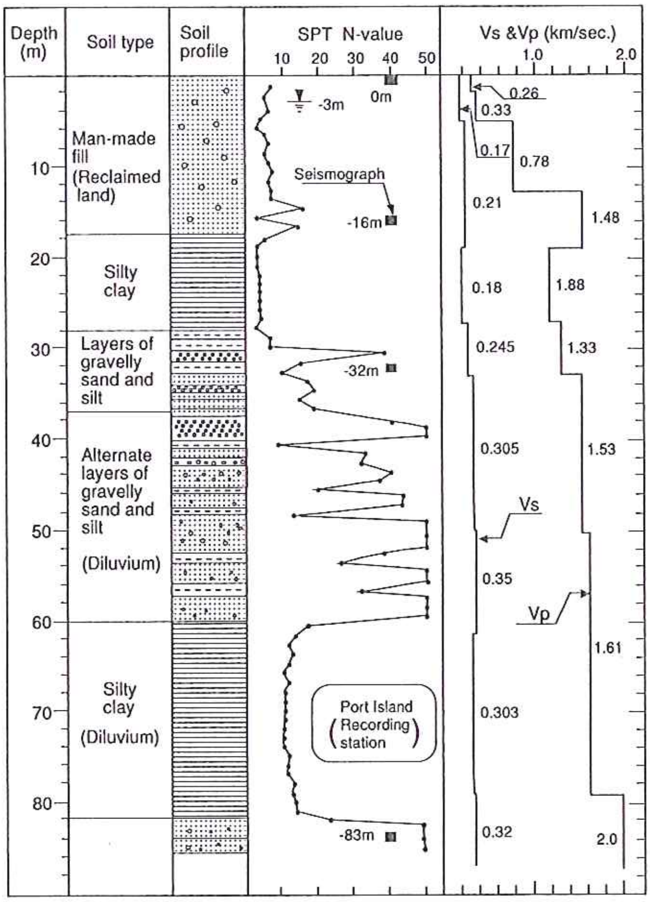

Fig. 5. Soil profile at the site of vertical array of seismograph (Toki, 1995)

stress ratio that must have been applied to soil elements at varying depths can be calculated. The maximum stress ratio thus computed from field observations was found to increase with increasing depth, to magnitudes of about 0.5 to 0.7 at shallow depths. This cyclic stress ratio is plotted in Fig. 7 versus the number of cycles which are assumed to be one to two as mentioned above. It should be noted that the cyclic stress ratio plotted is expressed in terms of the maximum stress divided by the mean confining stress σ_o' instead of the effective overburden stress σ_v'. The conversion from σ_v' to σ_o' was made simply through the use of the following relationship

$$\sigma_o'=\frac{1+2K_0}{3}\sigma_v' \tag{3}$$

where K_0 is the coefficient of earth presure at rest. This conversion represents the computed data in a way that permits direct comparison with the laboratory test data. While an estimate of an exact K_0-value is somewhat difficult, an assumed value between 0.5 and 0.75 is considered reasonable. Based on these values, the cyclic stress ratio τ_{max}/σ_o' was calculated and is shown in Fig. 7.

Comparing the two sets of data, one from the laboratory and the other from the computation based on field measurements, it can be mentioned that the laboratory cyclic strength is almost coincident with the value of cyclic strength computed from the observed acceleration at the time of strong shaking during the Kobe earthquake.

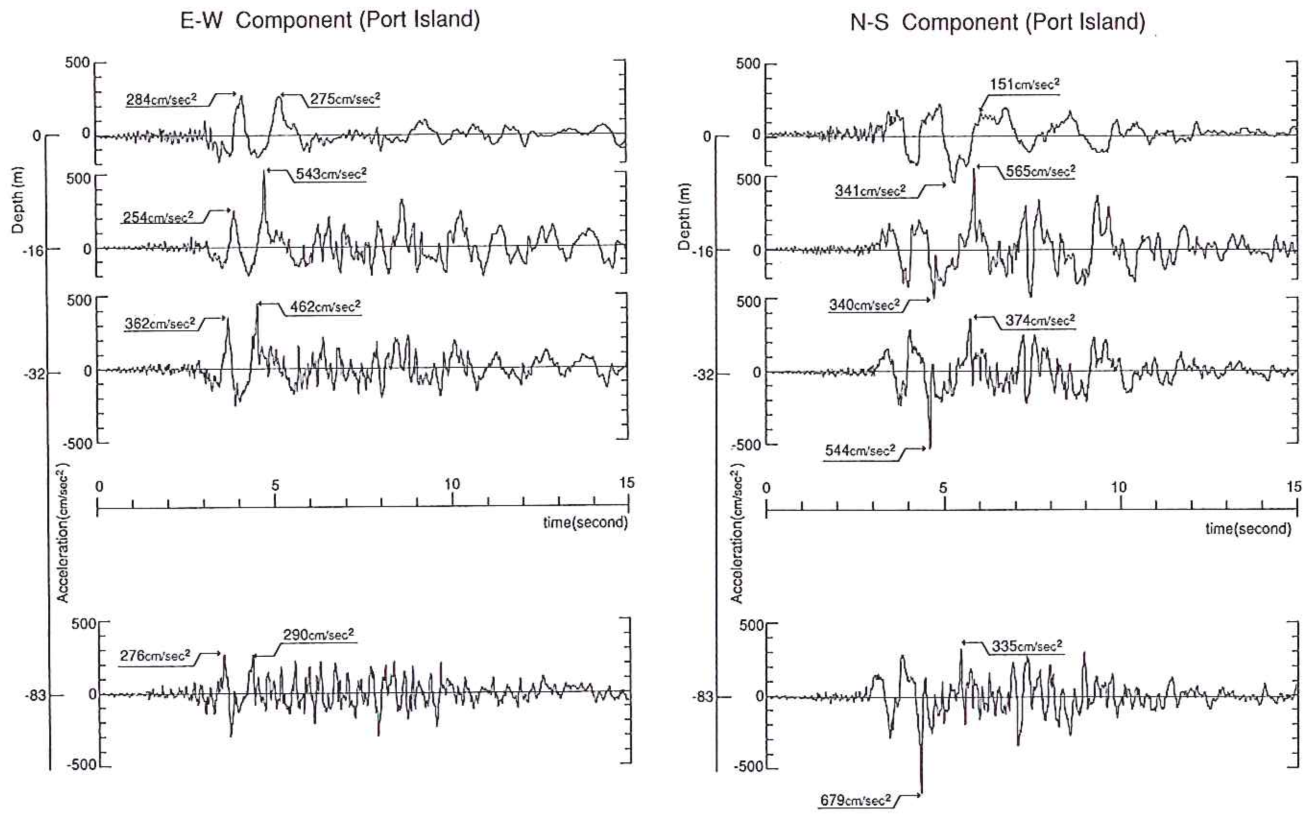

Fig. 6. Accelerations recorded at the site on Port Island (Toki, 1995)

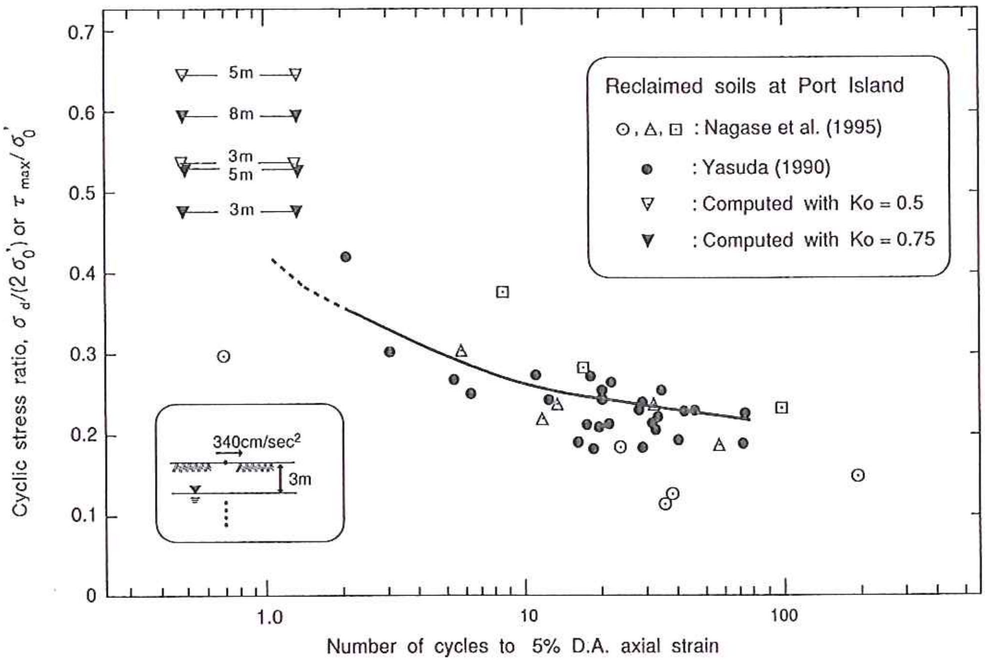

Fig. 7. Summary plot of cyclic strength from laboratory tests and from compution

GROUND SETTLEMENT FOLLOWING LIQUEFACTION

Following the earthquake, a large amount of silt- or sand-laden water spurted through pavement joints and in the shrubbery zone along the roads and spread over on the roads and paved areas. Figure 8 shows a mud puddle about 50 cm deep covering a wide area in Port Island. The shrubbery on the roadside is seen being splashed by the mud. Figure 9 shows the top of cast-in-place concrete piles that had protruded about 50 cm above the ground surface as a result of overall settlement of the surrounding ground due to extensive liquefaction. The settlement of the ground surface sufficiently far from the waterfront was estimated by surveying the difference in elevation between supposedly subsided flat ground areas and objects such as pile-supported buildings which were apparently free from any settlement. The results of such surveys made at many locations in the flat area of Port Island and Rokko Island are presented in Fig. 10. It may be seen that the settlements observed varied in a wide range with a maximum of 90 cm on Port Island. The average value of settlements was 50 cm on Port Island and 40 cm on Rokko Island.

The procedures for estimating ground settlements resulting from liquefaction have been developed by various researchers. Figure 11 shows a chart prepared by Ishihara and Yoshimine (1991) for estimating the post-liquefaction settlements of sand deposits. In this procedure, the volumetric strain is estimated as a function of the factor of safety F_l if the density of a deposit is made known in terms of SPT N-value or CPT q_c-value. If the

Fig. 8. Photo at a location on Port Island taken one day after the earthquake

Fig. 9. Top of cast-in-place piles protruding above the ground surface

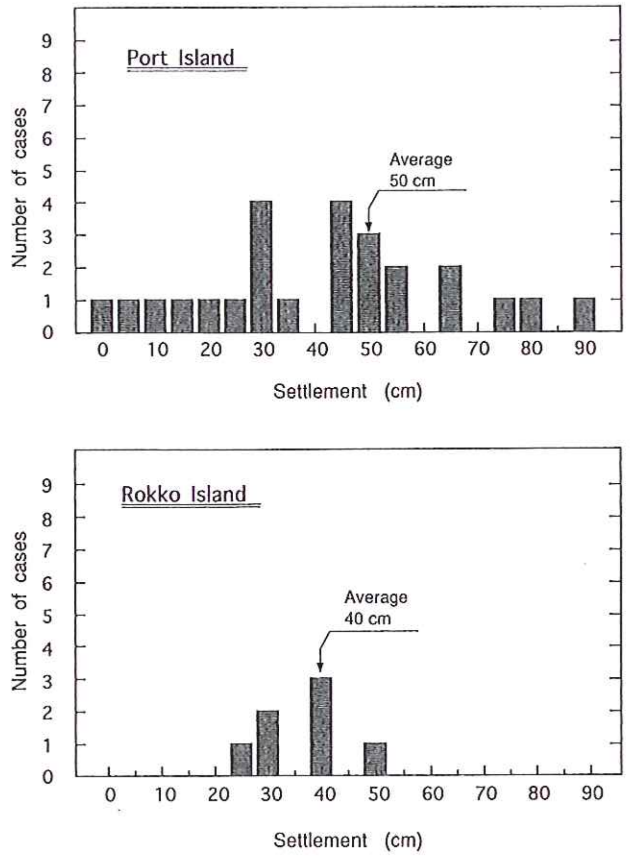

Fig. 10. Settlements observed on the ground surface at Port and Rokko Islands

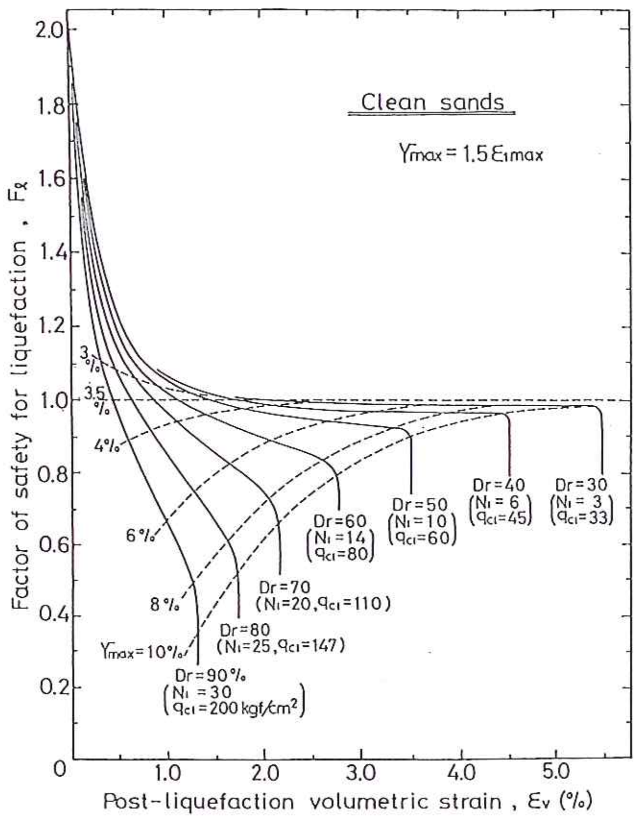

Fig. 11. Post-liquefaction volumetric strain as functions of factor of safety against liquefaction

near-surface soil deposit is assumed to have developed liquefaction with the development of single-amplitude shear strain of about $\tau_{max}=3.5\%$, the factor of safety is estimated from the chart as having been near unity at the onset of liquefaction. SPT N_1-value is estimated from the boring data in Fig. 5 as being $N_1=6\sim10$. Entering the chart with these values, the post-liquefaction volumetric strain is estimated to be approximately $\varepsilon_v=2\sim4\%$. Since the liquefaction appears to have penetrated to a depth of about 15 m below the ground water table, the settlement is calculated as $(0.02\sim0.04)\times15$ m$=30\sim60$ cm. This value is approximately in the range of observed settlements shown in Fig. 10. It should be noted, however, that the chart in Fig. 11 has been established based on the laboratory test data on clean sands and a more exact chart like the one in Fig. 11 should be established for the gravel-containing silty sand such as that encountered in the area affected by the earthquake in Kobe.

LATERAL DISPLACEMENT

It has been known that a soil deposit which softens as a result of liquefaction starts to move laterally if the ground is sloped or if there is a difference in elevation on the surface. The most conspicuous lateral displacement in the Kobe earthquake took place in the ground surface which is located behind the revetment line in the Kobe harbour area. Actually, about 25 km of quaywall line suffered damage involving outward displacement of about $1\sim3$ m as a result of liquefaction having taken place in the soil behind or possibly underneath the wall. The caisson blocks used for the quaywall construction sank and moved extensively towards the sea, accompanied by equally large movements of soil backfills behind the wall. This movement successively propagated rearwards, bringing about varying degrees of damage to industrial facilities and storage tanks located there. The lateral displacements were generally large near the waterfront and decreased with distance inland.

In order to identify the pattern of the retrogressive displacements, measurements of lateral movement were made on the ground by surveying openings of ground cracks starting from a point which is located sufficiently inland where there are no cracks observed. By summing up successively the width of the crack openings from the fixed inland point, the lateral displacements in the direction perpendicular to the revetment line were obtained as a function of the distance from the waterfront. Measurements were also made of vertical offsets of the crack openings and local slope between two successive cracks by means of an inclinometer. By summing up successively the local settlements thus obtained, it became possible to obtain a pattern of settlement distribution as it varies inland in the direction perpendicular to the revetment line. An example of such measurements is presented in Fig. 12 for a cross section of the northern wharf in Port Island. As indicated in the inset, the quaywall at this location was constructed by placing concrete caissons 6 m high and 4 m wide on top of a mound composed of dumped

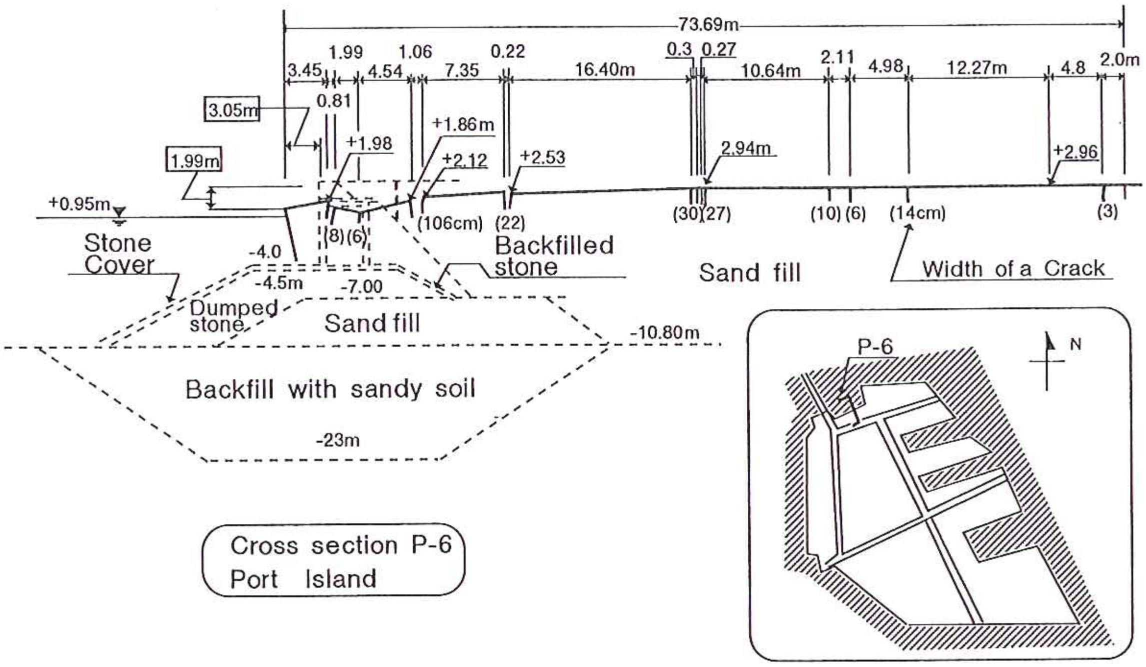

Fig. 12. Ground deformation behind a quaywall in Port Island

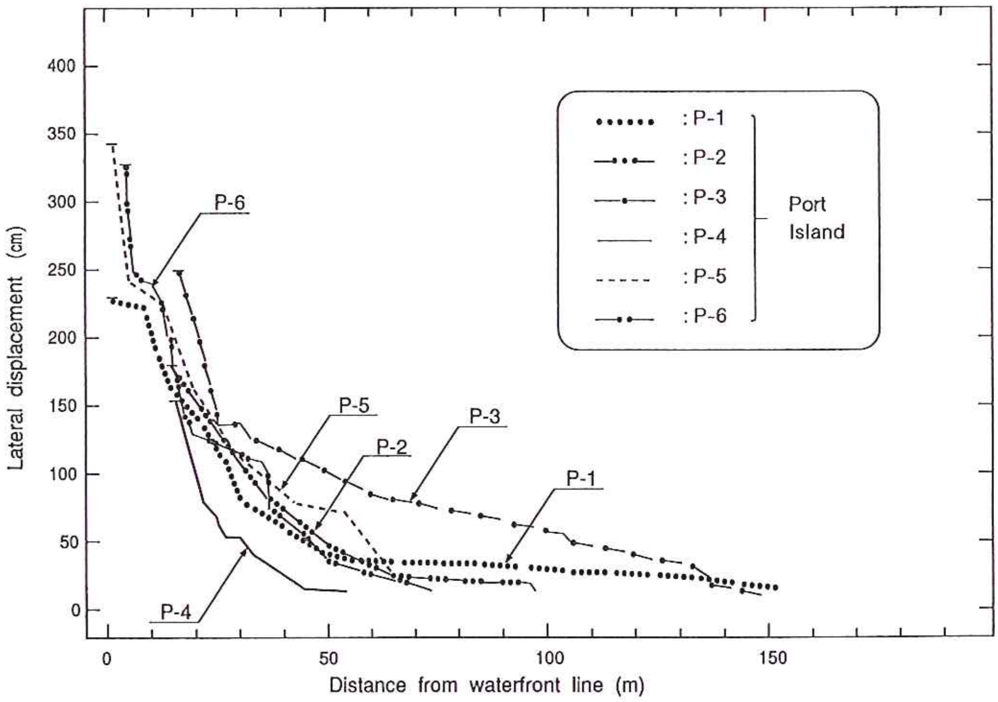

Fig. 13. Distribution of lateral displacement behind the quaywall

stone. The soft silty soil in the original seabed was initially removed down to a depth of 11 m from the mudline and replaced with Masado soil up to an elevation of 7 m below the sea water level. Stones were then hauled to the site and placed in open water to construct the mound. Behind the caissons, the Masado soil was placed directly over the soft seabed deposit. In the particular example shown in Fig. 12, the lateral displacement was found to have propagated backward for a distance of 74 m. The lateral displacement at the quaywall is shown to be 3.05 m and a settlement of 1.99 m occurred. Apparently, these settlements do not include the overall land subsidence due to liquefaction which must have occurred almost equally over the waterfront area surveyed in this investigation.

A similar survey of the ground distortion was made at several locations along the quaywall line in the area of Port Island. The exact locations are shown in Fig. 2. The results of such in-situ survey are summarized in Fig. 13 where the lateral displacements are plotted versus the inland distance from the waterfront. It may be seen that the lateral displacement decreases with increasing distance from the waterfront, but in some instances it propagated as far back as 150 m from the revetment line. A displacement of about 50 cm still existed at a location 50 m behind the waterfront, if the quaywall was displaced seawards by 2.0 to 3.0 m as a result of liquefaction in the surrounding soil deposit. The lateral displacements at any distance inland are divided by the displacement at the revetment line and shown in Fig. 14. The curves normalized in this way indicate a general pattern in which the lateral displacement is distributed.

The distribution of ground settlements in the direction perpendicular to the quaywall line is shown in Fig. 15 in a summary form where it is also seen that the ground subsidence tends to decrease with increasing distance from the waterfront. The measured displacements as stated above are apparently associated with the ground distortion

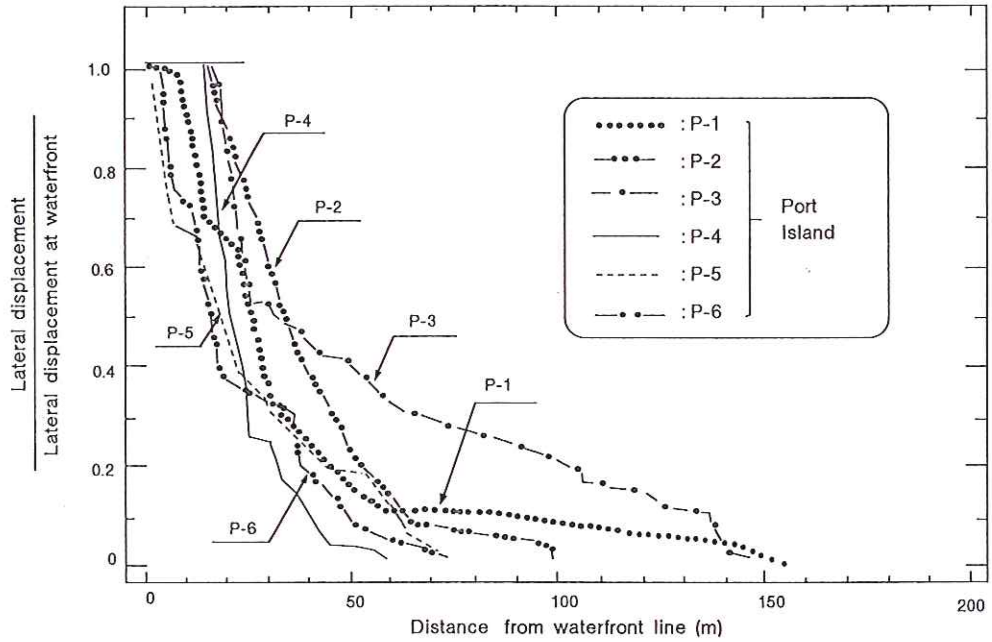

Fig. 14. Normalized distribution of lateral displacement behind the quaywall

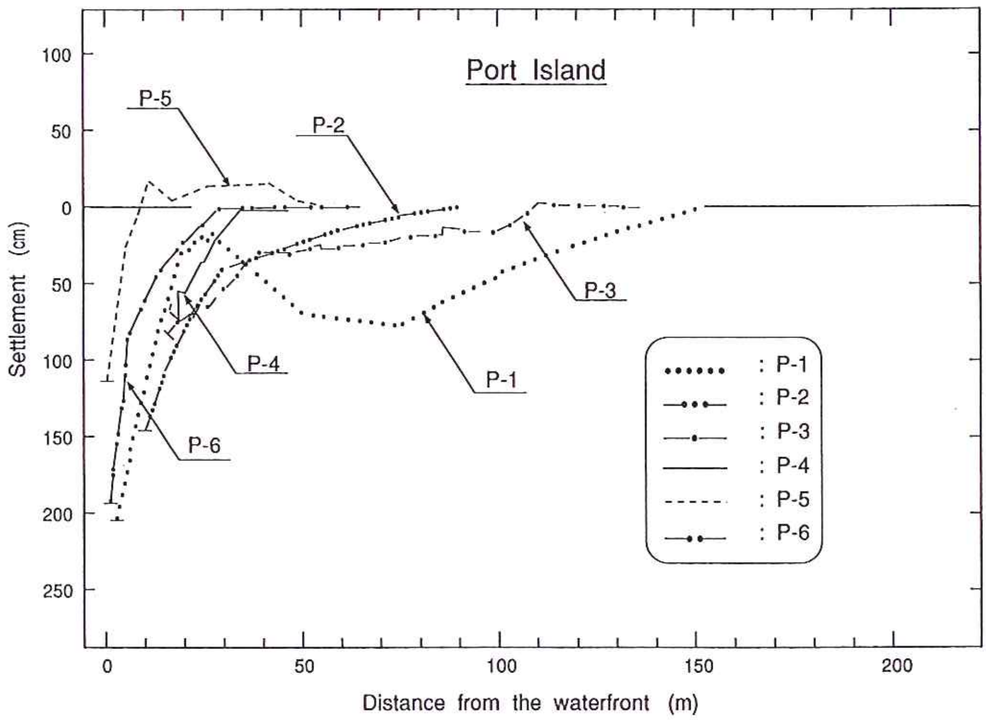

Fig. 15. Settlement of the ground surface behind the quaywall

which is manifested on the surface as differential settlement or uneven movements.

In order to examine the distorsional characteristics of the ground surface, the extensional strain was estimated by taking the difference of lateral displacements at two adjacent points divided by the horizontal distance between them. The distribution of the extensional strain on the ground surface thus calculated is shown in Fig. 16 as a function of the distance inward from the waterfront line. While it fluctuates because of the volatile nature of the crack openings near the quaywall, the extensional strain of about a few percent was found to be produced in the zone up to 50 m from the waterfront. The characteristics of differential settlement may be evaluated in terms of vertical distortion defined as the ratio of the difference in settlement between two adjacent points to the horizontal distance between them. The vertical distortion calculated in this way is presented in Fig. 17 as a function of the dis-

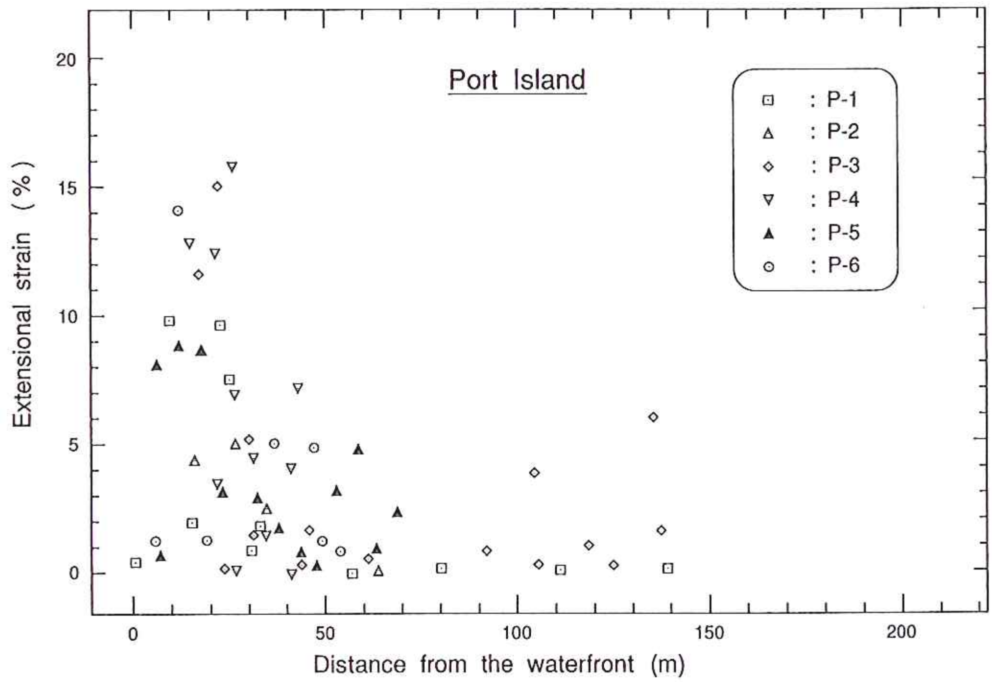

Fig. 16. Extensional strain in the horizontal direction plotted versus distance inland

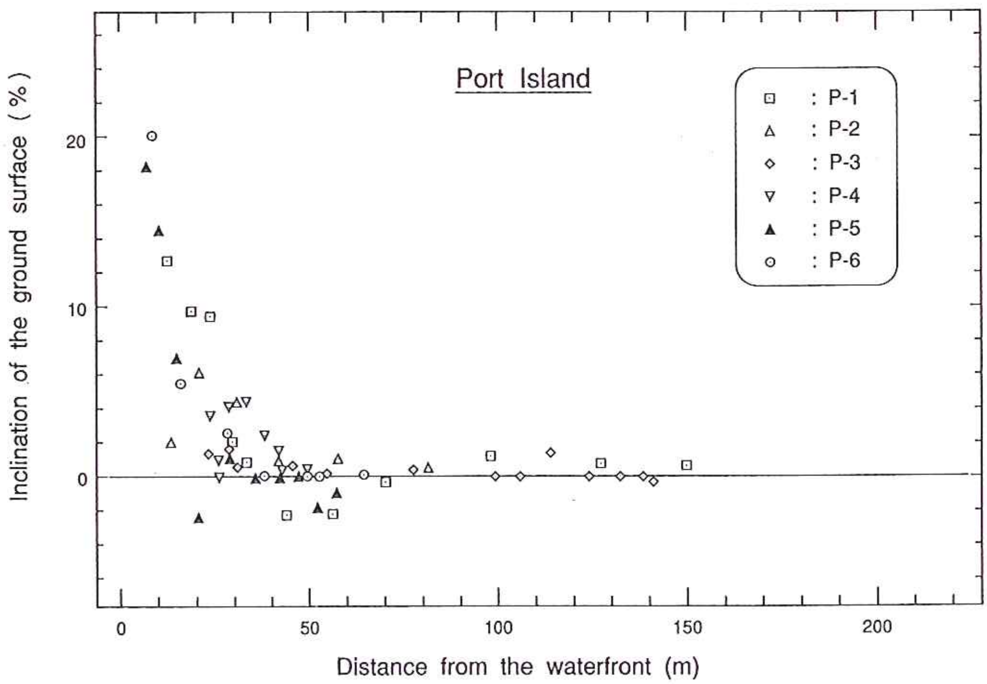

Fig. 17. Vertical deflection behind the quaywall

tance inward from the waterfront line. It should be noted that the vertical distortion defined above is equal to the local inclination of the ground surface. While it could fluctuate locally, the maximum value of aligned inclination was shown to be about 5% in the belt zone in close proximity to the quaywall line.

CONCLUSIONS

As a result of the cyclic triaxial tests made on undisturbed samples, the Masado soil used for landfilling in the Kobe area was shown to exhibit resistance to liquefaction which is mostly equivalent to that for loose to medium loose clean sand. This cyclic strength was also shown to coincide approximately with the value estimated from computations based on recorded accelerations at the ground surface.

The settlements surveyed at Port and Rokko Islands indicated values as high as 40 to 50 cm on the average. These values were also shown to be approximately coincident with those estimated from a current analytical method.

Measurements were made of both lateral and vertical displacement of the ground surface behind the revetment line which were devastated by the earthquake. As a result, the patterns of distribution of lateral and vertical displacement were made known to be functions of the distance inward from the waterfront. It was shown that if the quaywall is displaced seawards by an amount of the order of 2 m, the displacement can progress successively for a distance of 150 m from the revetment line.

ACKNOWLEDGMENTS

The measurements of displacements behind the quaywall were made by Dr. K. Ito of Konoike Co. with the assistance of the Kobe city office. Their cooperation and assistance are gratefully acknowledged. The acceleration records obtained at Port Island were provided through the courtesy of Dr. Y. Iwasaki of Geo- Research Institute Osaka.

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