

## LIQUEFACTION-INDUCED GROUND FAILURES IN THE 1995 HYGOKEN-NAMBU (KOBE) EARTHQUAKE

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### ABSTRACT

Extensive liquefaction occurred in artificially filled islands along Osaka Bay during the 1995 Hyogoken-nambu (Kobe) earthquake. Soil conditions in the islands are described together with the soil strength obtained in the laboratory tests on undisturbed samples. Interpretation of this soil strength is given in the light of back-calculated shear strength based on recovered accelerations on the ground surface. Observed settlements of the ground resulting from liquefaction are described and couched in terms of the values estimated by an existing methodology. Finally, outcome of in-situ survey on permanent deformations of the terrain behind the quay wall or revetment is presented.

### KEYWORDS

Ground settlement; Lateral spreading; Liquefaction; Sandy soil; Soil investigation; Soil test

### INTRODUCTION

The violent ground shaking during the Hyogoken-nambu (Kobe) earthquake caused the landfills in the port area to liquefy leading to widespread occurrence of ground deformation such as settlements and lateral spreading. Fig.1 shows liquefied sites in Kobe, Ashiya and Nishinomiya Cities. As a result of the ground distortion, various kinds of damage were incurred to infrastructures such as lateral shifting of quaywalls, breakage of gas and water lines.

The landfills in the several affected area were performed first around 1953 by transporting soils from borrow area at the foothill of the Rokko Mountains. A huge amount of the soils was carried by means of long-distance belt convey system to the sites of reclamation along the old shoreline. The several islands north of Rokko Islands were constructed in this period. The second round of artificial landfilling began in 1968 to construct two large man-made island further offshore to the south. Port Island with an area of 436 hectare was constructed in the period of 1966 to 1980 by transporting soils were carried in push-barges and dumped under water at the site of landfilling. The 580 hectare-Rokko Island was constructed in the period of 1972 to 1990 by taking soil materials from the site of Suma and also from other sites in the Rokko Mountains.



Fig.1 Liquefied sites during the 1995 Hyogoken-nambu Earthquake

### LABORATORY TESTS

Prior to the earthquake a series of cyclic triaxial tests had been performed by Nagase et al. (1995) on undisturbed samples of Masa soils recovered from the bottom of a cut. The location of the excavation is shown in Fig.2. The undisturbed samples were recovered in block and frozen in the field before they were brought to the laboratory. The results of the cyclic loading tests are displayed in Fig.3 in terms of the cyclic stress ratio plotted versus the number of cycles required to produce cyclic softening with development of 5% double-amplitude axial strain in the sample.

Another sets of triaxial results on undisturbed samples from Port Island had been performed by Yasuda. The samples were recovered by what is called triple-tube samples and tested using the cyclic triaxial test apparatus. These data are also shown in Fig.3.

### BACK CALCULATION

A set of records was monitored by four accelerometers in a vertical array installed by Kobe city office at a site in the northern part of Port Island. The soil profile and the time histories in two horizontal components is reproduced in Fig.4 and 5. The manmade fill to the depth of 18m is composed basically of the Masa soil. In

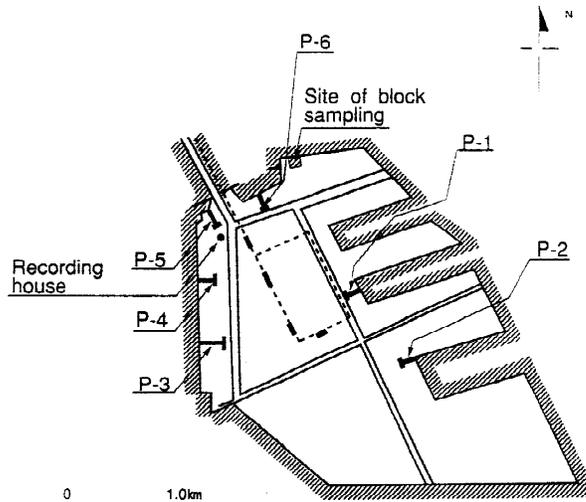


Fig.2 Locations of the recording houses and displacement measurements behind the quay wall in Port Island

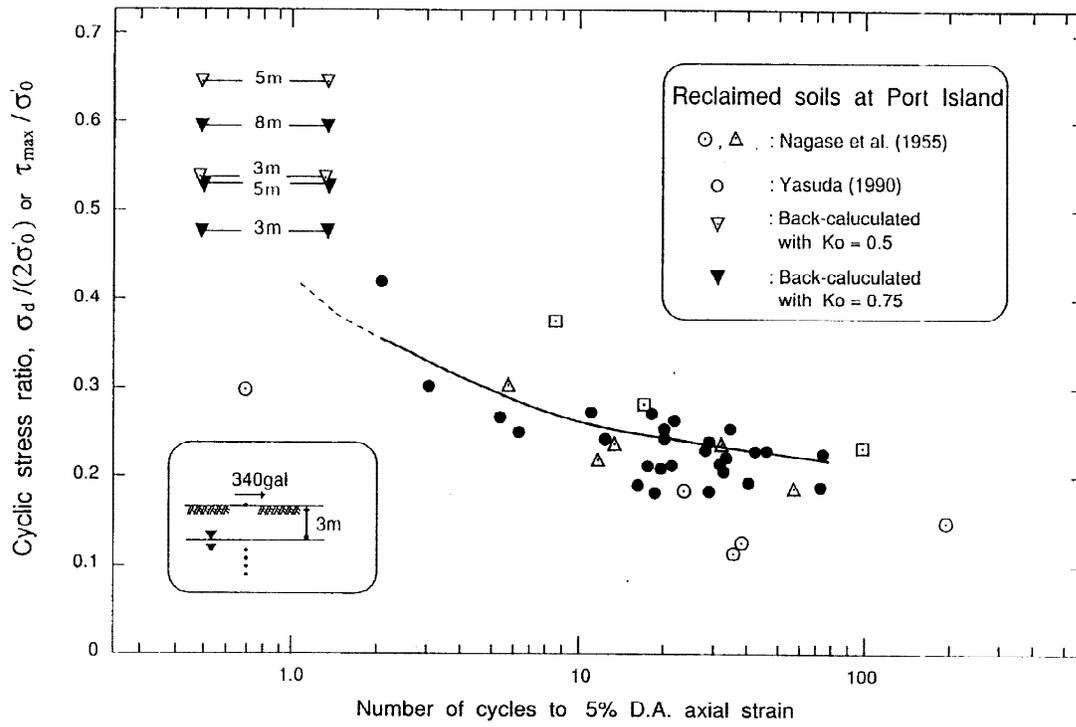


Fig.3 Summary plot of cyclic strength from laboratory tests and from back-calculation

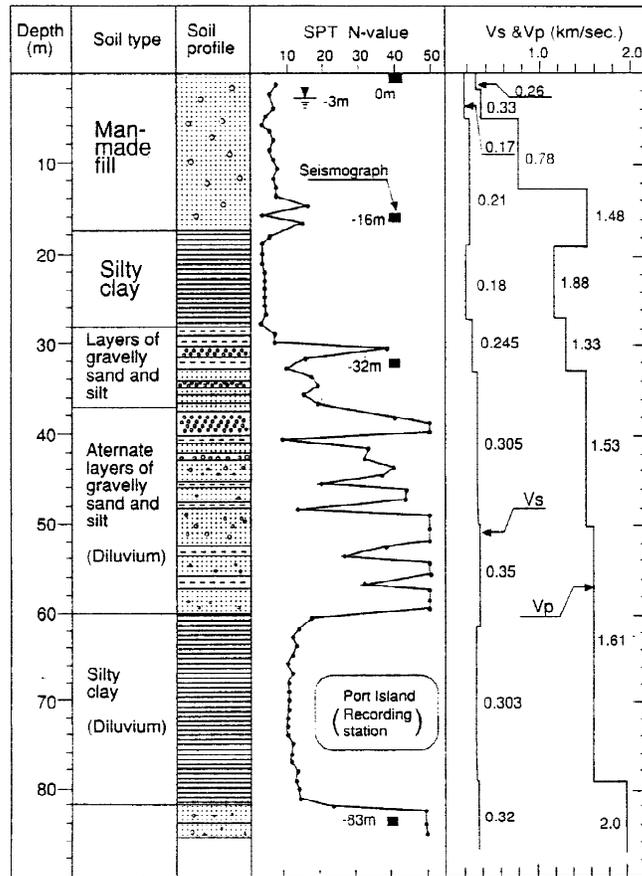


Fig.4 Soil profile at the site of vertical array of seismograph

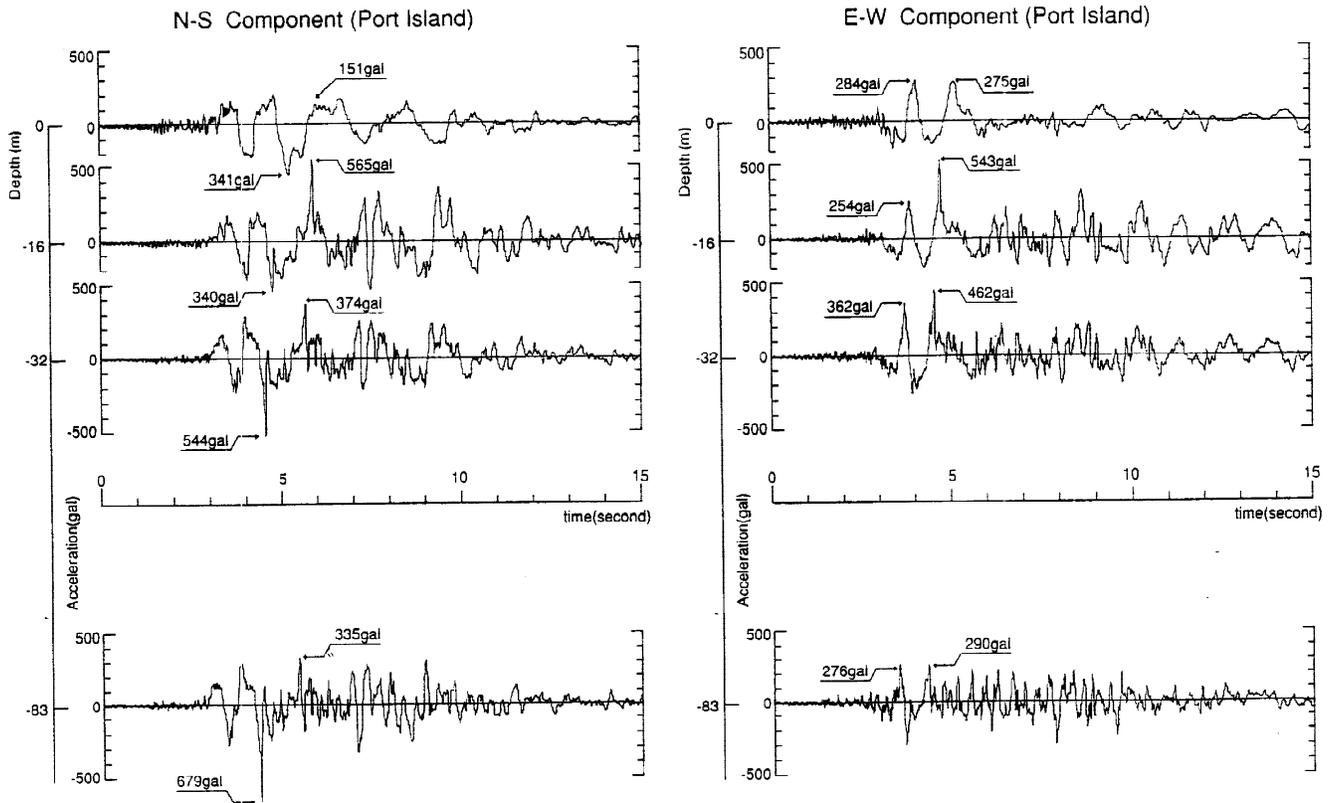


Fig.5 Recorded accelerations at the site in Port Island

view of the low N-value of SPT, being of the order of 5 to 10, the cyclic softening due to liquefaction appears to have taken place mainly in this near-surface deposit. In fact, the second spike of acceleration in the south direction are shown to decline significantly at the surface (341gal) and at the depth of 16m (340gal). Similar decrease in acceleration is also observed in E-W component. It is, therefore, highly likely that the cyclic softening due to liquefaction occurred in the course of one to two cycles of seismic load application. Furthermore, it may be reasonable to assume that the peak acceleration at the onset of liquefaction was  $A_{max}=340$  gal. The cyclic stress ratio  $\tau/\sigma_v'$  at any instant of seismic shaking is estimated by the equation

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

where A denotes the acceleration and  $\sigma_v$  and  $\sigma_v'$  are the total and effective overburden pressure acting on a soil element located at a depth of Z (in meter). The depth of the ground water table at this location is estimated to be approximately 3m. Introducing the value of  $A=A_{max}=340$  gal and  $g=980$  gal, one can calculate the maximum stress ratio that must have been applied to soil elements at varying depths. The maximum stress ratio thus back-calculated is found to increase with increasing depth, taking values of the order of 0.5 to 0.7 at shallow depths. The cyclic stress ratio is plotted in Fig.3 versus the number of cyclic which are assumed to be one to two as mentioned above. Note that the cyclic stress ratio plotted is expressed in terms of the maximum stress divided by the mean confining stress  $\sigma_o'$  instead of the effective overburden stress  $\sigma_v'$ . While estimate of exact Ko-value is somewhat difficult, it may well be assumed that it takes a value between 0.5 to 0.75. With these values, the cyclic stress ratio  $\tau_{max}/\sigma_o'$  is calculated and shown in Fig.3.

Comparing the two sets of data, one from the laboratory and the other from the back-determined cyclic strength is, by and large, coincident with the value of cyclic strength back-calculated from the observed acceleration at the time of strong shaking during the Hyogoken-nambu earthquake.

## GROUND SETTLEMENTS FOLLOWING LIQUEFACTION

Following the earthquake, a large amount of silt-or sand-laden water spurted through pavement joints and shrubbery zones along the roadside and spread over the road and paved areas. Overall settlement of the surrounding ground due to extensive occurrence of liquefaction. The settlement of the ground in the level ground sufficiently far from the waterfront were estimated by surveying difference in the elevation between supposedly subsided flat ground 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 displayed in Fig.6. It may be seen that the observed settlements vary in a wide range with its maximum of 90 cm in Port Island. The average value of settlements is shown to be 50 cm in Port Island and 40 cm in Rokko Island.

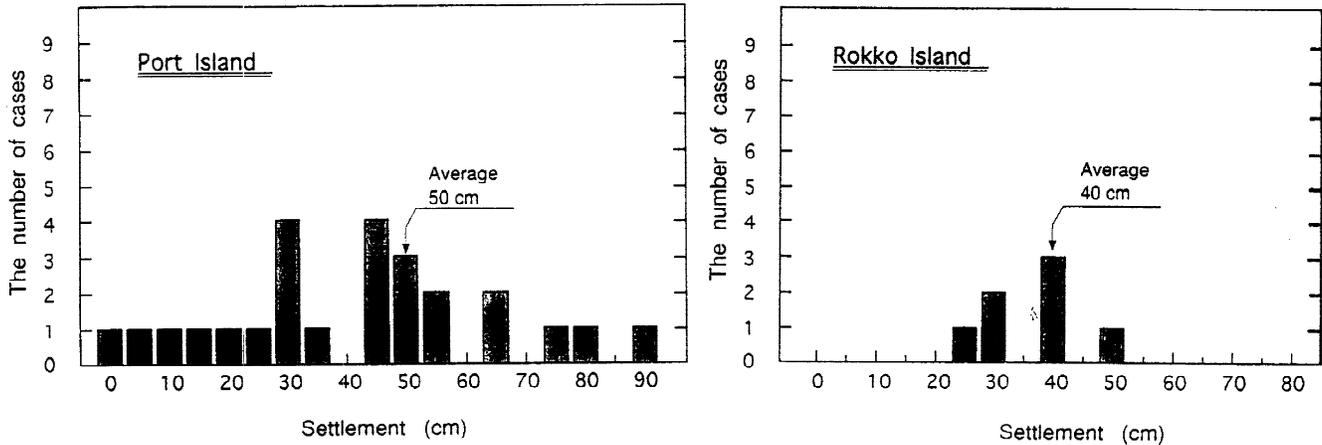


Fig.6 Observed settlements on the flat ground in Port Island and Rokko Island

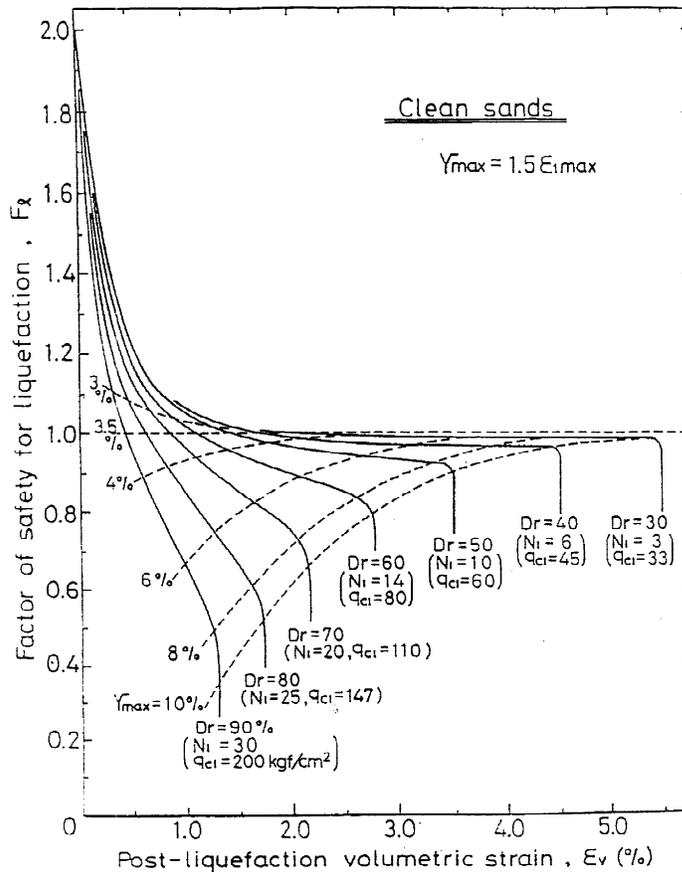


Fig.7 Post-liquefaction volumetric strain as functions of factor of safety against liquefaction

The procedures for estimating ground settlements resulting from liquefaction have been developed by various workers. Fig.7 shows a chart prepared by Ishihara and Yoshimine (1979) 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  $F_1$  if the density of a deposit is made known in terms of SPT  $N_1$ -value or CPT  $q_c$ -value. If the near-surface soil deposits 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. On the other hands, SPT  $N_1$ -values is estimated from the boring data in Fig.2 as being  $N_1=6 \sim 10$ . Entering in the chart with these values, the post-liquefaction volumetric strain is estimated as being approximately  $\Sigma v=2 \sim 4\%$ . Since the liquefaction appears to have penetrated to a depth of about 15m below the ground water tables, the settlement is calculated as  $(0.02 \sim 0.04) \times 15 \text{ m} = 30 \sim 60 \text{ cm}$ . This value is by and large in the range of observed settlements shown in Fig.6. It is to be noted, however, that the chart in Fig.7 has been established based on the laboratory tests on clean sands and more exact renewed chart like the one in Fig.7 needs to be established for the gravel-containing silty sand such as that prevalent in the Kobe area affected by the earthquake.

## LATERAL SPREADING

It has been known that the soil deposit softened as a result of liquefaction starts to move laterally if the ground is sloped or if there is different in elevation in the terrain. The most conspicuous lateral spreading in the Hyogoken-nambu earthquake took place in the terrain which is located behind the revetment line in the harbor area of Kobe. In fact, a total length of quay wall line as long as 25 km suffered the damage involving outward displacement of the order of 1 ~ 3 m as a result of liquefaction having taken place in the soil behind or possibly underneath the wall. The caisson blocks used for the quay wall construction sunk and moved largely towards the sea, accompanied by equally large movement of backfill soils behind the wall. This movement successively propagated backward, bringing about varying degrees of damage to industrial facilities and storage tanks sitting 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 opening of ground cracks starting from a point which is located sufficiently inland where there is no crack. 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 at 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 displayed in Fig.8 for a cross section of the northern wharf in Port Island. As indicated in the inset, the quay wall at this place was constructed by placing concrete caisson 6 m high and 4 m wide atop mound composed of dumped stones. The soft soil in the original seabed was removed first down to a depth of 11 m from the mudline and replaced by the Masa soil up to the elevation of 7 m below the sea water level. Then the stones were hauled into the sea to construct the mound. Behind the caissons, the Masa soil was placed directly over the soft seabed deposit. In the perpendicular example shown in Fig8, the lateral displacement is found to have propagated backward through a distanced of 74 m. The lateral displacement at the quay wall is shown to be 3.05 m and the settlement there was 1.99 m. Apparently, this 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.

Similar surveys of the ground distortion were made at several locations along the quay wall 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.9 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 instance it could propagate as far backward as 150 m from the revetment line. It is also seen that the

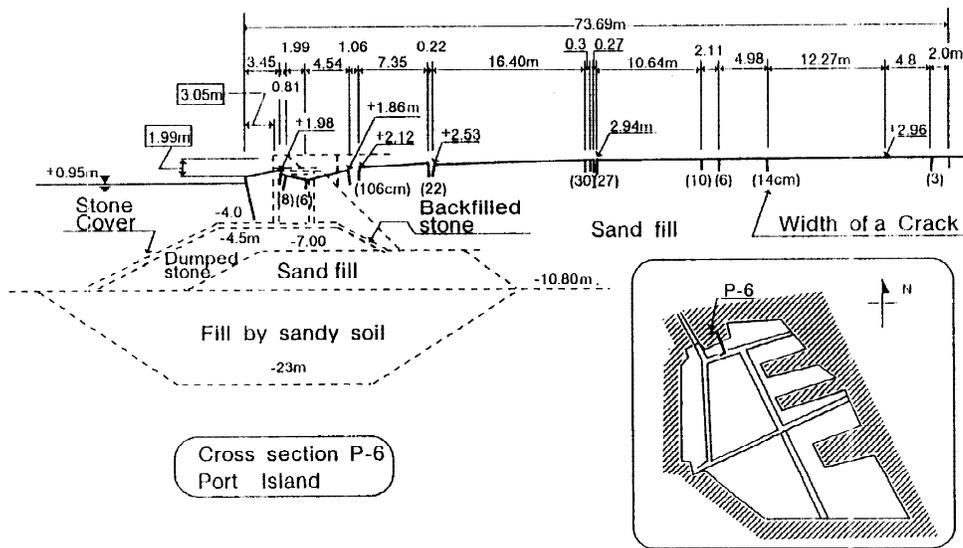


Fig.8 Ground deformation behind a quay wall in Port Island

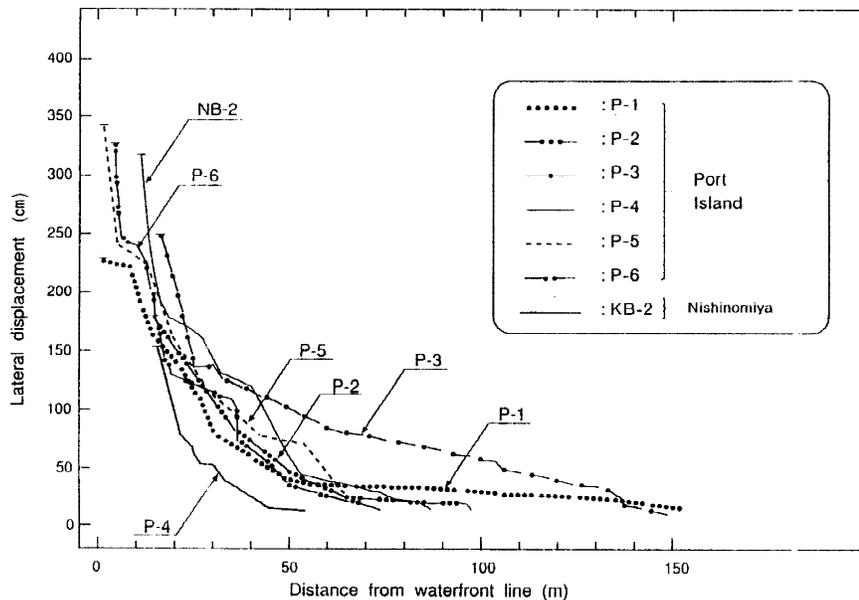


Fig.9 Distribution of lateral displacement behind the quay wall

displacement of the order of 50 cm is still existent even at a place 50 cm behind the waterfront, if the quay wall is dislodged seawards by 2.0 to 3.0 m as a result of liquefaction in the surrounding soil deposit.

The distribution of ground settlements in the direction perpendicular to the quay wall line is shown in Fig.10 in summary forms where it is also seen that the ground subsidence tends to decrease with increasing distance from the water front. The displacements as measured above are apparently associated with ground distortion which is manifested on the surface as differential settlements or uneven movements.

## CONCLUSIONS

Based on the survey at liquefied sites during the 1995 Hyogoken-nambu (Kobe) earthquake, the following conclusions were derived.

(1) Cyclic strength obtained by cyclic triaxial tests was shown to coincide approximately with the values estimated from back-calculation based on recovered acceleration on the ground surface.

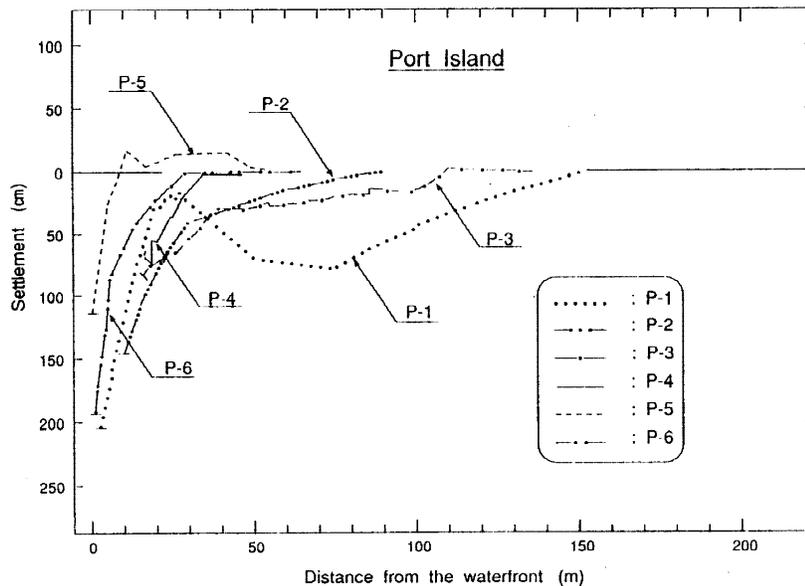


Fig.10 Settlements of the ground behind the quay wall

(2) The settlement surveyed in Port Island and Rokko Island indicated values as large as 40 to 50 cm on the average. These values are also shown to be approximately coincident with those estimated by a currently used procedure.

(3) Measurements were made of lateral and vertical displacements of the ground surface behind revetment line which were devastated by the earthquake. It was shown that if the quay wall is displaced seawards by an amount of the order of 2 m, the displacement can progress successively backwards as long as 150 m from the revetment.

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