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STUDY OF LIQUEFACTION DAMAGES OF QUAY-WALLS AND BREAKWATERS DURING KOBE EARTHQUAKE

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ABSTRACT

During Kobe Earthquake, very extensive damages of harbor facilities such as quay-wall and breakwater occurred in Kobe Port and also along the coastal areas between Kobe and Osaka cities. Major causes of the damages were the liquefaction of sands underlying and behind the concrete caisson and also strong earthquake shaking force on the caisson. The degree of damage varied considerably depending on location and also on the size of structure. In order to understand the mechanism of damage as well as the factors that controlled the degree of damage, it was necessary to examine and analyze the case records of damages of these structures.

This paper describes the result of such study on liquefaction damage of quay-walls and breakwaters. Through the study, it was found that the movement of sand at shallow depth below the caisson base is mainly responsible for a large settlement of caisson, but the mode of deformation is different between quay wall and breakwater. Also an effective stress liquefaction analysis was performed on the damaged quay-walls and breakwaters in order to check the applicability of effective stress liquefaction analysis on damage assessment. It was found that the effective stress analysis may be used to establish the overall trend of damage variation with the intensity of seismic motion, but problems exist in setting the dynamic parameters for the analysis, such as damping parameters, in order to obtain a reliable result.

INTRODUCTION

Due to the Great Hanshin (Kobe) Earthquake of 1995.1.17, severe earthquake damages occurred on almost all of infrastructures in Kobe City and surrounding cities between Kobe and Osaka. Among these, the harbor facilities along the north shorelines of the Osaka Bay have suffered very extensively due to liquefaction so that the function of Kobe Port and other port has ceased completely. In order to investigate the cause of the infrastructure damages in relation to the geotechnical condition in those areas, a research committee, which was headed by Prof. T. Matsui of Osaka University, was formed by the Kansai Chapter of Japanese Society of Civil Engineers. In total, nine study teams were formed in the aforementioned research committee, and this paper deals with the earthquake damages of harbor facilities as studied by one of the teams. The team member consists of 11 engineers and academics from university, local governments, construction companies, and geotechnical consultants.

This team has gathered the data of earthquake damages, such as settlements and lateral deformations of port facilities, extensively from nearly all of port facilities in the Osaka Bay in order to study the cause of the liquefaction damage at quay-walls and breakwaters. This paper describes the research outcome of this study team, and it presents the correlation study between the degree of damage and possible factors that controlled the degree of damage. It also discusses the use of numerical analysis to examine the variation of liquefaction damages at four selected locations that have different distances from the fault.

OUTLINES OF LIQUEFACTION DAMAGES

The earthquake damages of port facilities in the Osaka Bay were surveyed by dividing all port facilities into 7 zones, such as Suma to Han-Nan as shown in Fig.1. Damaged quay-walls and breakwaters in Fig.1 were identified by the amount of measured vertical settlement that reached in excess of 20-30cm. As shown in the figure, the significant damages are only found in the port facilities in the vicinity of Kobe Port, Fig.1 also shows four locations, points A to D, that are selected for numerical liquefaction analysis.

Almost 90% of quay-walls in Kobe Port were constructed by placing a massive concrete caisson on a granular fill with thickness varying from 10 to 15m that was placed by excavating the superficial soft marine clay of seabed. A typical cross section of quay wall is shown in F ig.2. Due to

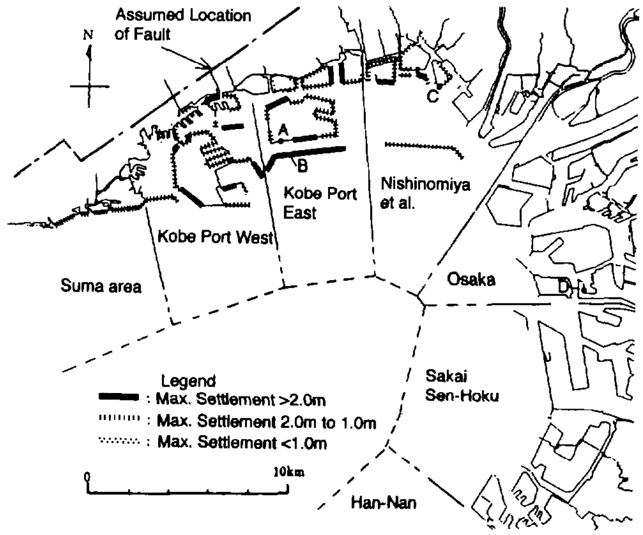

Fig. 1 Distribution of damages of harbor facilities

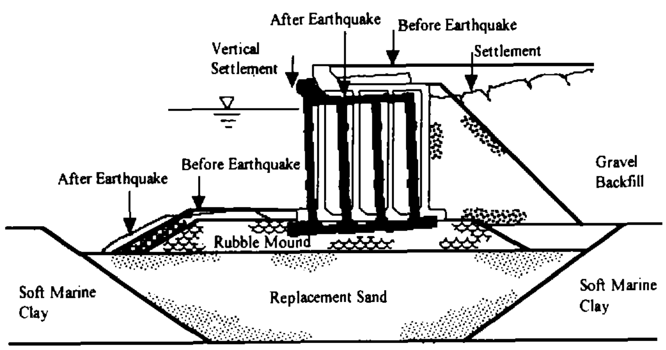

Fig. 2 Typical section of quay wall and damages

the earthquake, the granular fill materials behind the caisson and also below the caisson liquefied. Also a large horizontal thrust was exerted on the massive caisson due to the earthquake. Because of combined effect of these phenomena, the concrete caisson of quay-wall moved seawards extensively at many reclaimed lands, for example at Kobe Port Island and Rokko Island, as much as 5 to 6 meters and settled more than 1 to 2 meters along the perimeter.

To assess the extent of earthquake damage, various measurements of quay wall and breakwater movements were taken at these ports by using field survey or more sophisticated GPS (Global Positioning System) technique. In the following, an assessment of damage is mainly made based on the measured vertical settlements of these structures. As to the measurement of horizontal movement of quay walls, it was often very difficult to establish an absolute amount of movement by field survey. Examination of the gathered data showed that the horizontal movement is often reported as a relative movement or deviation of quay wall from a straight line that is drawn by connecting the two end corners of the quay wall under surveying. The measurement by GPS may not result in such error, but such measurement was not available in most cases.

STUDY ON FACTORS CONTROLLING THE DEGREE OF DAMAGES

In order to find what was the main factor that controlled the degree of damage of quay-wall and breakwater structures, information on the following five parameters such as geometry of caisson and ground acceleration are gathered and compared against the measured amounts of movement. It may be noted that the ground acceleration is estimated by using an empirical attenuation equation that relates the attenuation of ground vibration based on the distance from the fault as proposed by Fukushima and Tanaka (1990) as given below;.

$$\log A = 0.41M - \log(R + 0.032 \times 10^{0.41M}) - 0.0034R + 1.30$$

where A: the maximum ground acceleration (gal),
R: distance from the fault (km),
M: magnitude

The above equation was originally proposed for the maximum ground acceleration at a firm base ground, but it was used here to estimate the difference of earthquake intensity at various points.

Table 1. Parameters for correlation study on damages

Parameter	Definition
A	Horizontal acceleration of ground
H	Height of caisson
L	Depth of underlying granular fill
R	Ratio between L and the thickness of soft marine clay
q_u	Average unconfined compression strength of soft clay

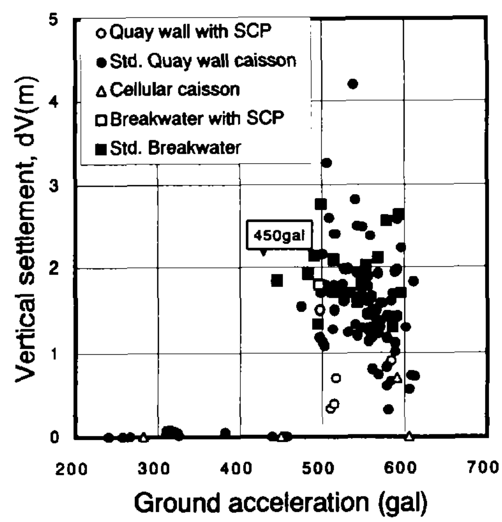

Fig. 3 Relationship between ground acceleration and damages

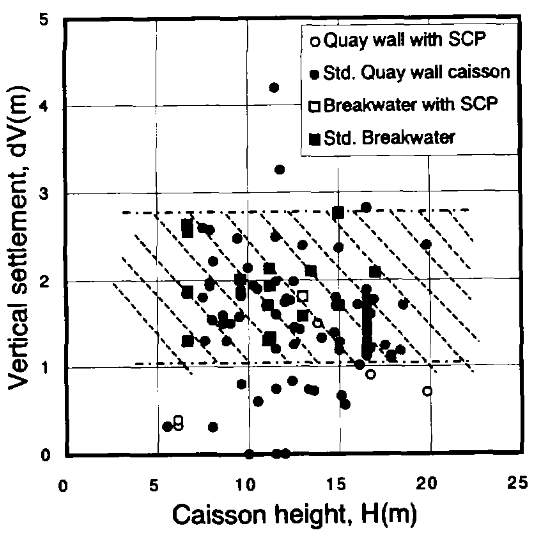

Fig.4 Relationship between caisson height, H, and settlement, dV.

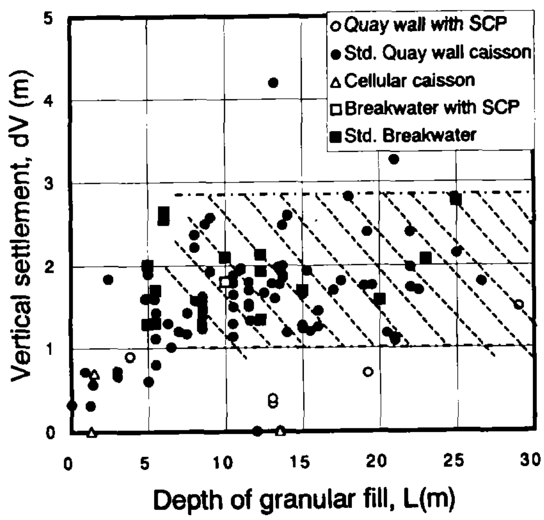

Fig.5 Relationship between depth of granular fill, L, and settlement, dV.

First the relationship between the ground acceleration and the degree of damage was examined as shown in Fig. 3. The vertical settlement, δV, of all structure, including those with sand compaction piles (SCP) and cellular caisson of light weight, was compared against the estimated ground acceleration. The figure shows that the damage occurs when the acceleration exceeds A=450 gal irrespective of structural differences. The amount of settlement is smaller for those structures with SCP and cellular caisson because of their less liquefaction potential and light weight respectively. Thus in the foregoing analysis, only the damage records that correspond A>450 gal were used for analysis.

The height of caisson, H, represents the total weight on the granular-fill, and therefore it might be possible to correlate the amount of settlement, dV, to the height of caisson, H. The relationship between dV and His shown in Fig.4, but there seems to be no relation between the two.

Next, the relationship between the depth of granular fill, L, below caisson and the damage was examined in Figs. 5 and 6. In Fig.5, the vertical settlement, δV, was depicted against the depth, L. The amount of settlement ranges about the same value between 1 to 3 m when L exceeds 5m. In Fig.6, the vertical settlement, δV, was normalized by the depth of granular fill or the length of SCP, L, and the ratio, $\delta V/L$, decreases steadily with the increase of L value. This indicates that the amount of settlement stays nearly constant, about 10 to 20% of L, with the increase of depth of granular fill. Again, those structures with SCP and cellular caisson show smaller settlement.

Most of quay wall and breakwater foundations have their entire original soft clay base being replaced by the granular fill or SCP. However, in some cases, the soft clay foundation was not entirely replaced or improved by the granular fill replacement or SCP respectively. In Fig. 7, the settlements of those structures with their foundation partially replaced or improved are examined. A parameter, R, which is the ratio between L and the thickness of soft clay, represents the

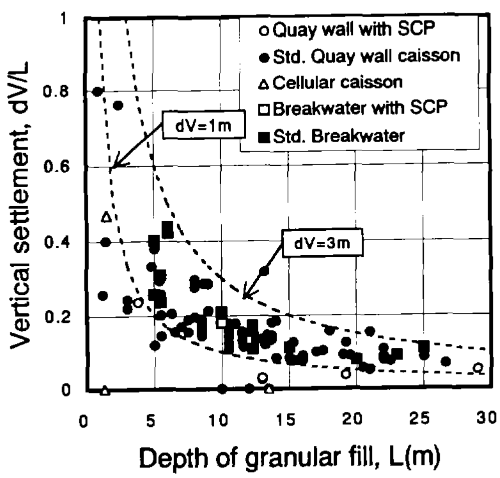

Fig.6 Relationship between depth of granular fill, L, and normalized settlement, dV/L.

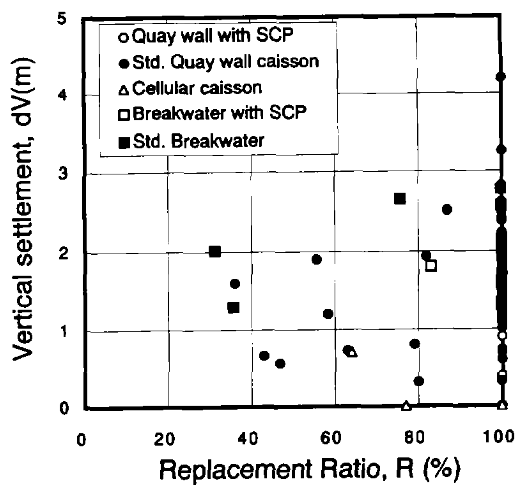

Fig.7 Relationship between replacement ratio, R, and settlement, dV.

improvement ratio of the foundation. It can be seen that the structures with R value less than 100% also show large settlement ranging from 0.5 to 2.5 m. This result suggest that the deformation or compression of granular fill to cause the structural settlement is mostly occurring in the upper zone of sand fill replacement.

It may be appropriate here to discuss the different modes of structural deformation between the quay wall and the breakwater. From the survey records, it was clear that the horizontal movement of quay wall was very large ranging from 2 to 5 meters, while that of breakwater is nearly zero. Thus there is a clear difference between the deformation mode of foundation soil between these two structures. Iai et al. (1998) has shown by using numerical analysis that the soil beneath the quay wall fails rotationally in the direction of a combined static force resulting from the caisson weight acting vertically and the horizontal thrust of back fill. They also pointed out that the granular soil beneath the caisson does not entirely liquefy. On the other hand, for the breakwater, the soil beneath the structure liquefies extensively and large deformations and compressions in simple shear mode take place near the surface of granular fill. They further noted for the breakwater that the compression of soil due to the re-consolidation after liquefaction is large, about 3% of fill thickness, L. The difference in the deformation modes of foundation soil as discussed by Iai et al. (1998) is illustrated in Fig. 8. The results as given in Figs. 4 to 6 have all indicated the amount of settlement is independent of the total thickness granular fill, thus this suggests that the deformation is contained only in upper part of granular fill. Such results are consistent with the mode of foundation deformation as illustrated in Fig.8.

Additional data to indicate the magnitude of recompression after liquefaction for breakwater were obtained by the settlement records of breakwaters at south of Kobe Port. Fig. 9 shows a variation of settlement along a long stretch of breakwater along Line A-A in Fig.10. It shows a continuous and consistent change in the amount of settlement from the west to the east. The settlement amount was compared against the thickness of soft clay that was replaced by the granular fill in Fig. 11. A comparison between the settlement and the thickness of granular fill shows an almost linear relationship, excluding the data for Nishinomiya area. The breakwater in Nishinomiya area stretches South-East direction along which

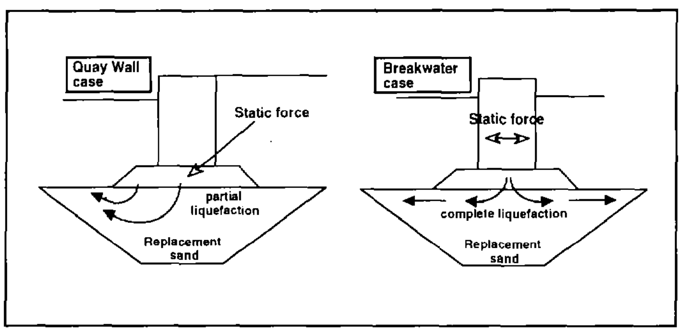

Fig.8 Schematics of different failure modes of granular fill under the caisson.

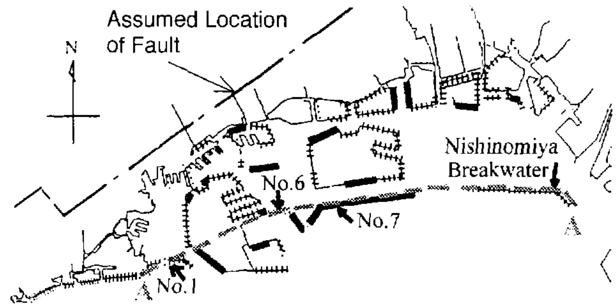

Fig.9 Location of breakwater surveyed

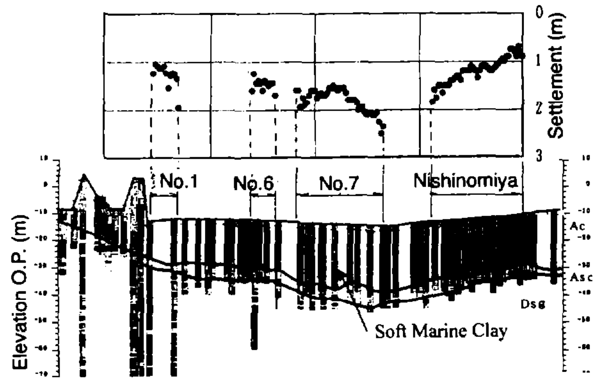

Fig.10 Variation of vertical settlement of breakwater along Section A-A.

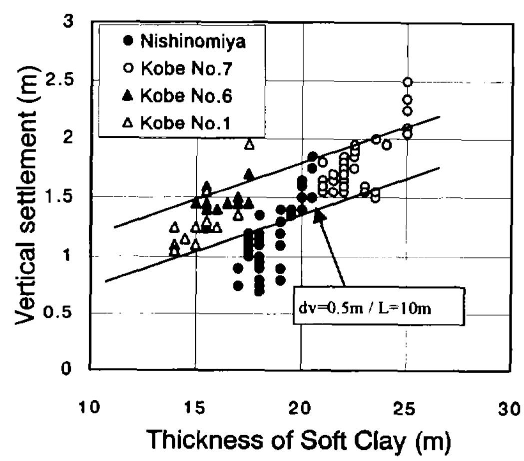

Fig.11 Relationship between the thickness of soft clay and the vertical settlement, dV

direction the ground acceleration reduces significantly. Thus the trend of these data was neglected from further analysis. For the settlement of breakwater, it has been noted that the re-consolidation after liquefaction takes a large part in the total amount of settlement. The results shown in Fig. 11 suggest the re-compression of granular fill increases about 0.5m over a 10m increase of fill thickness, and this may be suggested that

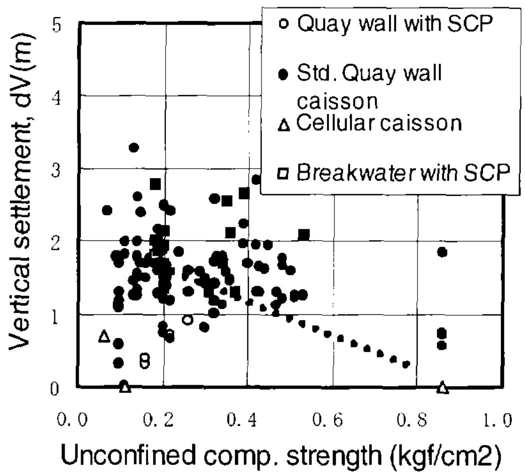

Fig.12 Relationship between the unconfined compression strength of soft clay and the vertical settlement, dV

the recompression was about 5% compression of the fill thickness.

Lastly, the strength properties of soft marine clay was compared with the amount of settlement in Fig.12. The strength in Fig.12 represents an average strength for a 5m thick surface layer of soft clay. The trend is not so clear, but the strength of clay near the surface may influence the amount of lateral movement of granular fill as suggested by the above arguments.

NUMERICAL ANALYSIS OF LIQUEFACTION DAMAGES

In order to study how the liquefaction damage of port facilities varies among various locations, it was decided to perform a numerical liquefaction analysis at those facilities where the degree of damage and other geotechnical condition vary significantly. In total, four locations were selected for the analysis, and they are namely, quay wall at Rokko Island (Point A), breakwater at south of Rokko Island (Point B), quay wall at Koshien-reclaimed-land (Point C) that suffered medium damage, and quay wall at Osaka Port (Point D) that had only a slight movement with a trace of liquefaction. The locations of these are shown in Fig.1.

It was also decided to perform the liquefaction analysis by carefully choosing the seismic base elevation for the dynamic analysis. The liquefaction analyses of damaged quay walls for Kobe Earthquake have been performed often by setting the seismic engineering base at the bottom of soft marine clay. However, in actual field situation, there are thick alternating layers of sand, gravel and clay further beneath. The seismic velocity and standard penetration value in the stratum just below the soft clay is not large enough so that it is not justified from the design point to choose the seismic engineering base at the bottom of the soft clay stratum. The stratigraphy of

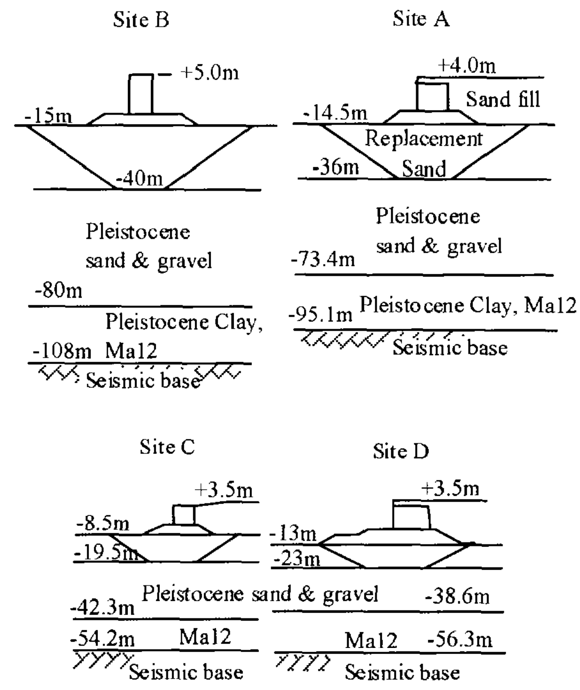

Fig.13 Modeling of geotechnical conditions at four sites of liquefaction analysis.

seabed at Kobe Port has been described previously by Tanaka (1996). More appropriate seismic engineering base would be at the bottom of upper-most Pleistocene clay, i.e., locally called Ma12 stratum that is located nearly 60 to 80m below the seabed.

Fig.13 shows the geotechnical conditions of seabed at the four locations. Based on these geotechnical profiles, the finite element mesh was constructed for the liquefaction analysis by using a computer program of FLIP as developed by Iai (1992). A typical FEM mesh for liquefaction analysis is shown in Fig.14. At the base of Ma12 clay, the acceleration record that was measured at Kobe Port Island at the bottom of same stratum was applied. Only adjustment was to change the maximum acceleration according to the value obtained from the attenuation equation (1) as discussed earlier. Table 2 lists the computed results and the actual measurements of both

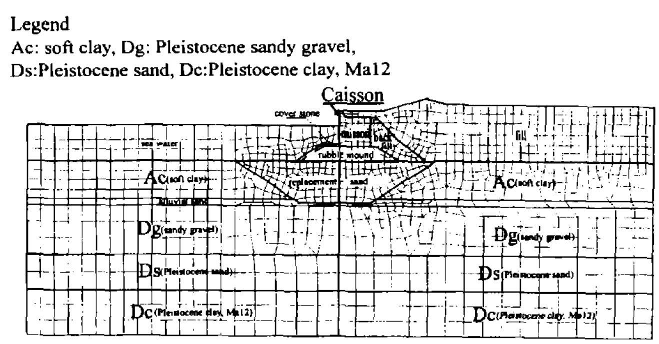

Fig.14 Example of FEM mesh of liquefaction analysis.

horizontal and vertical movements of caisson at these four locations.

Table 2. Comparison of analysis and measurement

	Site A	Site B	Site C	Site D
D(km)	5.0	5.7	7.8	19.5
A(gal)	539	523	484	324
dH(cm) (Analysis)	255 (180)	9.5 (0.7)	135.1 (76)	0 (37)
dV(cm) (Analysis)	108 (80)	193.7 (87.3)	135.3 (33)	4.9 (22)

Where D, A, dH, and dV represent distance from the fault, acceleration at the base, measured horizontal movement, and measured vertical movement respectively. The numbers in the bracket are the results of analysis.

The results in the table shows that computed movements are generally much smaller for all sites expect that of Site D. The disagreement at two sites, C & D, with smaller accelerations is more pronounced. Also it is interesting to note that there is a large difference in the measured vertical settlements at Site A & B while the analysis predicted nearly the same amount for the two sites. Breakwater at Site B had a very thick layer of granular fill, about 25m, and this large difference may be due to the recompression of the granular fill after liquefaction. The settlement data in Fig.11 suggests a recompression of 1.25m if the ratio is 5%, while the computation above indicates 1.05m of recompression.

A comparison of the computations and the measurements is depicted in Fig.15. As can be seen from the figure, there is a fair agreement in the changing trend of damage level that increases significantly with the increase of acceleration. However, the amounts of predicted movement are much smaller than the actual, especially under smaller acceleration.

In addition to the discrepancy in the numerical values, it was also difficult to obtain a stable computation result without paying a large effort on selection of damping parameter for the analysis. Further work on the selection of suitable input parameters for the analysis is warranted.

CONCLUSIONS

Attempts were made to delineate the factors that controlled the degree of liquefaction damages of quay wall and breakwater at various ports along the shore of Osaka Bay. Examinations of liquefaction damage records and the results of numerical liquefaction analysis result in the following conclusions.

1) It is clear from the study, the deformation modes of quay wall and breakwater are quite different. For the breakwater, the settlement due to recompression of granular fill after liquefaction seems to be very important. For the both structures, the amount of soil deformation at shallow zone of granular fill seems to be a controlling factor for the liquefaction damage.

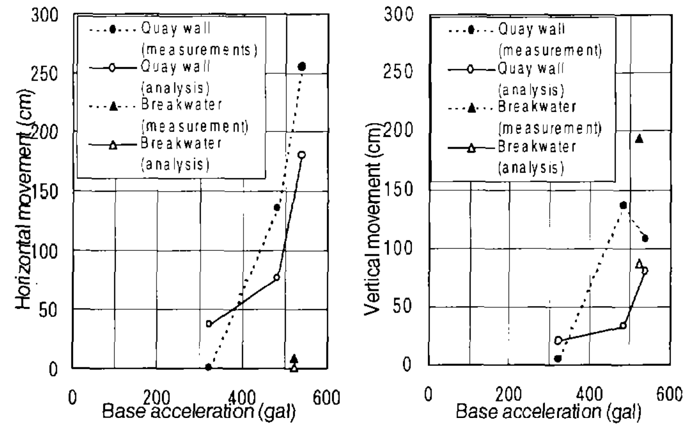

Fig.15 Comparison of horizontal and vertical movements with the results of liquefaction analysis.

2) The numerical liquefaction analysis may be used to examine the variation of liquefaction damage for these structures under different accelerations. However, the accuracy of predicted results is still in need of improvement especially for the locations where the magnitude of accelerations is not so large. Further work on the selection of suitable input parameters for stable analytical result is warranted

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