PORE WATER PRESSURES AND GROUND MOTIONS MONITORED DURING THE 1985 CHIBA- IBARAGI EARTHQUAKE

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ABSTRACT

Following the successful measurements of accelerations and pore water pressure rise during the 1980 September 25 earthquake, a set of piezometers embedded in the sand deposit on the Owi No.1 Island in Tokyo Bay became out of order. A new set of piezometers was again installed at the same site in a bid to monitor pore water pressures awaiting a reasonably large earthquake to come. On October 4, 1985, an earthquake of magnitude of 6.2 shook the area of Tokyo Bay and pore water pressure rise was again monitored, together with the accelerations on the ground surface and in the deposit at a depth of 10 m. The maximum ground surface acceleration recorded was 71 gal and the pore water pressure increase at depths of 6 m and 14 m was 4.5% and 5.3% of the mean effective confining pressure, respectively. On the basis of the laboratory test results performed previously on undisturbed soil specimens, an estimate was made of the pore water pressures that might be produced in the deposit by the seismic shaking with a maximum acceleration of 71 gal. The estimated values of pore water pressure at the depths of 6 m and 14 m were shown to be somewhat larger than that registered during the earthquake.

Key words : earthquake, in-situ test, liquefaction, pore pressure, sandy soil (IGC : D7)

INTRODUCTION

To gain in-depth understanding for the mechanism of liquefaction, it is necessary to obtain exact time histories of accelerations and pore water pressures recorded in the field deposit during actual earthquakes. An early attempt to this goal was made during the period of 1975 to 1979 at a site in the Ohgi-

shima man-made island in Japan, but this undertaking came to an end in vain when the instrumentation was completely devastated by flooding due to torrential rainfall (Ishihara, 1981). Another attempt of field instrumentation was undertaken in 1977 under the sponsorship of the Tokyo Metropolitan office at a site in the Owi No.1 Island in Tokyo Bay. After awaiting for three

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years, a fairly large tremor was felt over the area of Tokyo Bay on September 25, 1980 at the time of an earthquake which took place in Chiba with a magnitude 6.1. At that time, the ground shaking was great enough to trigger the instruments and to monitor increases in pore water pressure in the sandy deposit together with the accelerations on the ground surface. The detailed account of instrumentation, soil investigations and acquisition of in-situ data is reported in the previous paper by Ishihara et al. (1981). The measurements at this site were continued for some time following this earthquake, but because of malfunctioning of the piezometers, the observation was discontinued in the summer of 1981. For continued operation of the instruments, budget was provided and four pieces of piezometers were newly embedded at approximately the same place in the fall of 1982. On this occasion, an additional piece of seismograph was embedded at a depth of 10 m to monitor accelerations under the ground. While awaiting another event to come, a fairly large earthquake with a magnitude of 6.2 took place on October 4, 1985 with the epicenter located north of the Tokyo Bay. This earthquake produced ground motions which are large enough to build up pore water pressures in the deposit. In the following pages of this paper, detailed account on the soil investigations, processing of recorded data and their interpretations will be described.

Apart from the attempts as above, efforts have been made by Harp et al. (1984) to simultaneously record accelerations and pore water pressures during aftershocks of major earthquakes by means of a portable instrument unit composed of an accelerometer and a piezometer. Immediately after the main shock of the Mammoth Lakes earthquake of May, 1980, the portable unit was carried to the east shore of Convict Lake where several ground cracks by lateral spreading were observed due to liquefaction developing in the lake sediment consisting of gravel-containing coarse sand. The piezometer tip

was pushed into the sediment about 5 cm below the bottom of a 1.3 m deep hole drilled with an auger. The hole was backfilled with sand and gravel. Harp et al. (1984) reported that there was no excess pore water pressure observed during the series of aftershocks, despite the maximum acceleration registered was as high as 148 gal. Although monitoring of elevated pore water pressures has not yet been made, this technique appears to have a great potential as a useful tool for measuring pore water pressures at a variety of sites at a minimal cost.

TEST SITE

The Owi No.1 Island is located on the west side of Tokyo Bay as shown in Fig.1. This island was reclaimed from 1961 to 1969 with soils dredged from the nearby seabed to open a route for navigation. The depth to the seabed before filling was about 10 m. Soils were transported in slurry through pipes and discharged in the area being filled. When the soil surface reached the sea water level, waste soils from other construction

Fig. 1. Locations of epicenters of the 1980 and 1985 earthquakes and recording station

Fig. 2. Location of the recording station in Tokyo Bay

sites were brought in and dumped over the dredged fills. Near the south end of the island as shown in Fig.2, a lot of about 200 m² was provided by the Tokyo Metropolitan office as a test site for the present investigation.

TEST PROGRAM AND INSTRUMENTATION

Two sets of piezometers were embedded at two depths as shown in Fig.3. One set, designated as A1 and A2, was installed 5 m apart at depths of 7 m and 14 m, respectively. Another set, B1 and B2, was embedded at depths of 6 m and 14 m, respectively. Each of the new arrays of piezometer installments was at a distance of 2 m from the array of the previously installed piezometers. The method of burying the piezometers in the bored holes was the same as that employed in the 1977 installation. The transducer in each piezometer probe was of the strain gage

Fig. 3. Positions of instrumentation and in-situ sounding tests

type with a full capacity of 200 kN/m². The structure of the probe encasing the transducer is the same as that of the old transducers embedded in 1977.

The two-component seismograph mounted on the ground surface in 1977 was still effective for the continued measurements. A new piece of two-component servo-type seismograph was embedded in the sand deposit at a depth of 10 m. The location of this underground seismograph is indicated by C2 in Fig.3. The seismograph is capable of monitoring accelerations up to 1000 gal (approximately equal to the gravity acceleration) within the frequency range between zero and 400 Hz. The acceleration pickup is encased in a cylindrical capsule 9 cm in diameter and 64 cm in length. It weighs about 15 kg and is equipped with four legs 5 cm long to facilitate anchoring of the capsule in the upright position. The procedure for installing the seismograph is as follows. First of all, a hole 10 cm in diameter was drilled to a depth of 10.2 m using the drilling mud and after washing the bottom of the hole, the seismograph capsule was lowered using a steel rod as a guide. To position the capsule in the right azimuth, a lid with a direction indicator was fixed to the top of the bore hole casing and a guide rod coming out of a small hole through the lid was rotated to adjust the direction of the seismograph in the hole. After fixing the right azimuth, the guide rod was pushed to make the legs

penetrate firmly into the soil at the bottom of the bored hole. The guide rod was then lifted. The sand mixed with water was poured to backfill the hole. To keep the capsule from a long-term sinking due to its weight, wire ropes were connected to a horizontally buried anchor bar near the ground surface as shown in Fig. 4. The final layout of the seismograph capsule is schematically shown in Fig. 4. The electrical cable was led to the ground surface and to the amplifier in the recording house. The signals from this underground seismograph are transmitted to the oscillograph to be recorded simultaneously with those of the on-the-ground seismograph and four piezometers. Electrical power supply for the amplifiers and the oscillograph is put on all the year round. Whenever a tremor is felt at this site, a starter circuit triggers the oscillograph and a device of time stamp. It was then possible to obtain eight simultaneous outputs, four from the two seismographs and four from the piezometers to be recorded on a running strip chart paper. The duration of time for one continuous recording was set at one minute. After finishing a

Fig. 4. Schematic picture showing the installation of the underground seismograph

one-minute recording, the whole system can be automatically reset ready for triggering the subsequent shock.

SUBSURFACE SOIL CONDITIONS

The soil conditions at the test site had been investigated by the standard penetration test, Dutch cone test and also by means of a series of laboratory tests on undisturbed soil samples obtained by the large diameter samples and Osterberg samples. The positions of these in-situ investigations performed in 1977 are indicated in the plan of Fig. 3. In the 1983 investigation, two holes indicated by C1 and C2 in Fig. 3 were additionally drilled. The bored hole C1 was drilled to obtain an exact picture of soil profile in close proximity to the new location of pore water pressure measurements. The soil profile and blow count data at the location C1 are presented in Fig. 5, where it may be seen that there exists a surface layer composed of dumped waste soils to a depth of 4.2 m, underlaid by a layer of silty fine

Fig. 5. Soil profile at Boring C1

sand. The soils above the depth of 10 m are regarded as having been placed there when the landfilling was conducted in this area in 1960's. The original seabed deposit underlying the reclaimed soils is of alluvial origin and has nearly constant blow count values of 3 to 10 down to a depth of 20 m. It is also noted that the general picture of soil profile at this location is approximately the same as that obtained nearby at the time of the 1977 investigation (see Fig. 8 of the previous paper by Ishihara et al, 1981). The bored hole C2 was drilled to embed the underground seismograph. The in-situ measurements of shear wave and longitudinal wave velocities were also conducted in this bored hole by means of what is called the downhole method. The soil profile at C2 location is presented in Fig. 6, together with the results of velocity loggings. Fig. 6 shows a soil stratigraphy which is approximately identical to that at location C1 and also at the place of SPT studied in the 1977 investigation. It can be seen in Figs. 5 and

6 that the key soil deposits at depths of 6 m and 14 m where the piezometers were embedded do indeed exist commonly in all three soil profiles described above. The blow count values are shown to be equally on the order of 3 to 10 for the reclaimed deposit at depth 6 m and on the order of 2 to 5 for the alluvial deposit at depth 14 m.

SAMPLING AND LABORATORY TESTING

When the holes were drilled in 1982 for embedding the piezometers, undisturbed soil samples were procured by means of the Osterberg type piston sampler from three of the four bored holes used for the piezometer embeddments, as indicated in the plan view of Fig. 3. At the hole A 1 two samples each 7.6 cm in diameter and about 60 cm long were recovered from the silty fine sand deposit around the depth of 6 m. They are labelled as A 1-1 and A 1-2 sample as indicated in Fig. 5. Similarly, two samples were obtained at the hole B 1 from the same deposit. They are labelled as B 1-1 and B 1-2 sample as indicated in Fig. 5. From the silty fine sand deposit around the depth of 14 m, two samples were recovered only at the hole B 2, as indicated by B 2-1 and B 2-2 in Fig. 5. The steel pistons containing intact soil samples inside were cut into three to five small pieces each 15 cm long. The small specimens were sealed with paraffin and after being capped by steel lids at both ends, they were wrapped with vinyl sheets. Fifteen pieces of the small specimens were stored in the Geotechnical Engineering Laboratory of the University of Tokyo, and ten pieces were transported by an air cargo to the laboratory of Woodward-Clyde Co. in New Jersey upon request of Professor R. Dobry of the Rensselaer Polytechnique Institute in New York. Those samples left at the University of Tokyo were tested in the resonant column device and also in the cyclic triaxial test apparatus to determine the shear moduli, damping ratio and the resistance to liquefaction. The samples

Fig. 6. Soil profile at Boring C2

shipped to the United States were tested by Ladd (1983) to determine the key parameters of the soils under both static and dynamic loading conditions.

RESULTS OF FIELD TESTS

A new type of tests performed in the 1982 investigation was the in-situ measurements of the velocity of shear wave propagation, V_s , and the velocity of longitudinal wave propagation, V_p , by means of the down-hole method. This method consists of applying a shock to the ground surface and detecting the front of shock wave propagation in a bored hole at different depths. When measuring the shear wave velocity, the side of a wooden plate fixed on the ground surface is hit with a hammer to generate a horizontally polarized shear wave travelling downwards through soil deposits. When measuring the longitudinal wave velocity, the wooden plate is vertically hit to produce a wave propagating downwards in the vertical direction. In either case, the time of arrival of wave front is monitored by a signal receiver placed in the bored hole. The measurement as above is successively made at varying depths through the bore hole. Fig. 7 shows the time of wave travel thus obtained at the bored hole C2, which is plotted versus depth for each of the shear wave and longitudinal wave velocity measurements. In the case of shear wave logging the plot of travel time versus

distance is subdivided into seven sections having approximately identical characteristics, and the shear wave velocity is determined for each of these subdivided layers as shown in Fig. 7. Similarly, the longitudinal wave velocity is estimated as indicated in Fig. 7. It is well known that the longitudinal wave propagation through saturated soil deposits below the ground water table is governed mainly by the compressibility of water itself and, therefore, the velocity of propagation in such conditions becomes approximately equal to that of the compressive wave through water (Ishihara, 1971). In view of this the evaluation of the longitudinal wave velocity is made more roughly in Fig. 7 for only three subdivided sections through the 20 m depth of the deposit being studied. The velocity profiles thus established are demonstrated in Fig. 6, where it can be seen that the shear wave velocity at depths of 6 m and 14 m is 155 m/sec. and 195 m/sec., respectively. The Poisson's ratio computed from the velocities, V_s , and V_p for each layer is also shown in Fig. 6, where it can be seen that the ratio takes a value approximately equal to 0.5 because the soil is fully saturated (Ishihara, 1971).

RESULTS OF LABORATORY TESTS

(1) Index Properties

Results of index property tests on the soil from the deposit at depth 6 m indicate that the material is variable in the gradation but it consists mainly of two parts, viz.,

Fig. 7. Records of velocity logging at Boring C2

Fig. 8. Grain size distribution curves of soils from the key deposits

Table 1. Comparison of shear wave velocity and shear modulus at small strains

Depth	Items	In-situ velocity logging	Resonant column test at Univ. of Tokyo	Resonant column test at Woodward-Clyde
6m	V_s^* (m/sec)	155	147~194	165
	G_0^{**} (kN/m ²)	41,700	40,500~69,000	46,500
14m	V_s (m/sec)	195	179	230
	G_0 (kN/m ²)	66,000	59,000	94,000

* Shear wave velocity

** Shear modulus at small strains

low plastic silt and silty sand. The grain size distribution curves of these two materials are shown in Fig. 8. The variability of soils in this deposit appears to come from the fact that the samples were recovered from the upper portion of the deposit where different materials had been placed when the land was artificially filled. The void ratio of the soils in this layer ranges between 0.80 and 1.20 and the specific gravity is about 2.72.

The material at the depth of 14 m is quite uniform and basically a silty fine sand containing shell fragments. As indicated by the gradation curve in Fig. 8, the fines content (percent of material passing the No. 200 sieve) is about 35% and the void ratio of the soil is between 0.75 and 0.85. The specific gravity is 2.74.

(2) Resonant Column Tests

The resonant column test device used in the investigation at the University of Tokyo is equipped with an exciter atop the solid cylindrical test specimen. This test device also permits cyclic triaxial tests to be performed on an identical specimen by applying low-frequency cyclic axial stress through the vertical piston, after the resonant column test has been finished. The size of the sample is 6 cm in diameter and about 12 cm in height. The resonant column tests were also performed at Woodward-Clyde Soil Mechanics Laboratory by Ladd (1983) to determine the shear modulus at small strains, G_0 , and strain dependent moduli and damping characteristics of soils. The values of the maximum shear modulus, G_0 , and corresponding shear wave velocity, V_s , obtained

Fig. 9. Strain-dependent shear modulus reduction curves obtained at the University of Tokyo

from three different sources are presented together in Table 1. It may be seen that all the measured values are in reasonably good agreement with each other, except for the fact that the modulus value obtained in Woodward-Clyde laboratory is somewhat larger than that measured elsewhere.

The strain-dependent shear modulus reduction curves obtained at the University of Tokyo are demonstrated in Fig. 9. The shear moduli at strain levels in excess of about 6×10^{-4} were determined in the cyclic triaxial phase of tests performed on identical specimens. The resonant column tests performed at Woodward-Clyde laboratory showed practically the same modulus reduction curve as the average curve obtained at the University of Tokyo as indicated in Fig. 9.

The strain-dependent damping ratio curves obtained at the University of Tokyo laboratory are shown in Fig. 10, where it may be seen that there is practically no difference among the samples taken from the depths

Fig. 10. Strain-dependent damping ratio curves obtained at the University of Tokyo

Fig. 11. Cyclic stress ratio versus number of cycles obtained at the University of Tokyo

Fig. 12. Accelerations and pore water pressures recorded during the October 4, 1985 earthquake

of 6 m and 14 m. Almost the same results were obtained from the tests in Woodward-Clyde laboratory.

(3) Cyclic Triaxial Tests

The results of cyclic triaxial tests obtained in the University of Tokyo laboratory are shown in Fig.11 where it may be seen that the cyclic stress ratio causing 5% double-amplitude axial strain in 20 cycles is on the order of 0.25 for the soil at the depth of 14 m. The samples secured from the depth of 6 m were not tested at the University of Tokyo. The cyclic triaxial tests performed at Woodward-Clyde laboratory were strain-controlled tests aimed at determining the threshold strain and not directly comparable to the results shown in Fig.11, but their results are useful to obtain pore pressure buildup characteristics of the soil, as described in the following pages.

PORE WATER PRESSURES AND ACCELERATIONS RECORDED DURING THE CHIBA-IBARAGI EARTHQUAKE

An earthquake with a magnitude 6.2 shook the northern area of Tokyo Bay at 9:26 p.m. on October 4, 1985. The earthquake, named Chiba-Ibaragi-Kenkyo earthquake, had a focal depth of about 80 km, and its epicenter was located about 25 km northeast of the city of Chiba, as indicated in Fig.1. The intensity of shaking in the area surrounding Tokyo Bay near the epicenter was V in terms of the Japanese Meteorological Agency scale. This event produced the second greatest intensity of shaking in the area of Tokyo Bay following the Mid-Chiba earthquake of 1980 since the great Kanto earthquake of 1923. Among many seismographs that have monitored motions of this earthquake, the one installed on Owi Island

Fig. 13. Fourier spectra of the acceleration records

No.1 was of particular interest because it was combined with the simultaneous recording of pore water pressures by means of the piezometers embedded in the loose deposits of sand, as described in the foregoing pages. Of the four piezometers installed in 1982, piezometer A1 and A2 had been out of gear due probably to insufficient watertightness in the electrical system at the piezometer probe. Piezometer B1 and B2 were operated properly and monitored the pore water pressures as shown in Fig.12. The rise in pore water pressure was 0.17 m in the sand deposit at a depth of 6 m and 0.40 m at a depth of 14.0 m. The accelerations in two directions monitored simultaneously on the ground surface and by the seismograph at a depth of 10 m are also demonstrated in Fig.12. The Fourier spectra of the acceleration records, shown in Fig.13, indicate that the predominant periods of motions on the ground surface and at the depth of 10 m in NS-direction are 0.4, 0.65 and 0.9 seconds. The predominant period in EW-direction is shown to be about 0.5 sec. both on the ground surface and at the depth of 10 m.

COMPARISON BETWEEN RECORDED AND COMPUTED DATA

The recorded pore water pressures shown in Fig.12 indicate that during the initial seven seconds after triggering, there were small fluctuations in the pore water pressure due presumably to changes in total stress during the small initial motions. About seven seconds after triggering, the strong shaking began with the maximum acceleration of 71 gal in N-S direction on the ground surface and pore water pressure rose suddenly. At the depth of 14 m, the pore water pressure rose to a value 40 cm in water height or 5.3% of the mean effective confining pressure and then started to decay fairly rapidly. About 15 seconds after the occurrence of the peak, the pore pressure decreased to a value of about 10 cm whereupon the rate of decreases became small. The manner of

pore pressure dissipation as above is almost the same as that observed in the same deposit at the time of the 1980 earthquake (Ishihara et al, 1981). With respect to the pore water pressure at the depth of 6 m, it rose to a value of 17 cm almost simultaneously with the occurrence of peak acceleration but it did not decay significantly during the 30 seconds of recording time following the peak. The reason for the absence of significant dissipation is not clearly known, but it may be speculated that the pore water pressure rise was so small that the pressure gradient was not sufficiently large to initiate immediate dissipation. In view of the variability in soil composition at this depth as visualized in the gradation curves in Fig.8, it may as well be speculated that the piezometer tip embedded at this depth happened to be surrounded by rather impermeable silty soil leading to the difficulty for dissipating pore water pressures.

In order to visualize the feature of pore water pressure changes more in detail, the main portion of the records is reproduced in Fig.14 with an enlarged time scale. It may be seen in this figure that the pore water pressures at the two depths started to build up at the same time and reached their peaks simultaneously, fluctuating almost in unison to each other. The small fluctuations appear to have resulted from the propagation of longitudinal waves.

To examine the pore pressure buildup characteristics more in detail, a trajectory in plan view of the two components of accelerations on the ground surface is depicted on the diagram of Fig.15 with the time inscribed as a parameter. Similar trajectory in plan view is also shown in Fig.16 for the two components of accelerations recorded at the depth of 10 m. Figs.15 and 16 indicate that the pore pressures began to rise 7.39 seconds after triggering and reached their peaks at 8.45 second. It is to be noted in Fig.15 that during this period of 1.06 seconds, the acceleration trajectory on the ground surface made an excursion of about one cycle mainly in the north-south direc-

tion. Fig.16 also indicates that the major part of the accelerations recorded at the depth of 10 m made a one-cycle excursion mainly in the north-south direction during the 1.06 second period in which the pore pressures built up simultaneously at the two depths. Therefore, it may be mentioned that the pore pressure buildup during the shaking of the 1985 earthquake was caused by approximately one-cycle of shear stress application in the north-south direction during the period of about 1.06 second.

(1) Comparison between Recorded and Computed Accelerations

Seismic response analysis was made for a representative soil profile at the Owi Island site by means of the computer program "SHAKE" developed by Schnabel et al. (1972). This computer program is based on the one-dimensional wave propagation theory and permits the effects of non-linearity of soils to be taken into account by incorporating the equivalent shear moduli and damping ratio into the analysis. In the present study, the strain-dependent shear modulus and damping ratio curves for sandy soils were established based on the test results

Fig. 14. Recorded data with enlarged time scale

Fig. 15. Trajectory in plan of the accelerations recorded on the ground surface

Fig. 16. Trajectory in plan of the accelerations recorded at the depth of 10 m

Fig. 17. Strain-dependent shear modulus reduction curves used for the response analyses

Fig. 18. Strain-dependent damping ratio curves used for the response analyses

Fig. 19. Model of the deposit used for the response analyses and their results

Fig. 20. Model of the deposit used for the response analyses and their results

shown in Figs. 9 and 10, but the curves for cohesive soils were assumed based on the test results on other cohesive soils. The shear modulus and damping ratio curves used in the present analysis are shown in Figs. 17 and 18. The soil profile at boring C1 in Fig. 5 was used to model the conditions of soil stratification for the computational

purpose. The model of the soil profile used for the analysis is shown in Figs. 19 and 20. The time histories of accelerations recorded on the ground surface were used as input motions and the motions at the depth of 10 m were computed to be compared with those recorded at that depth during the earthquake. The comparison as above was

Fig. 21. Comparison of the accelerations computed and monitored at the depth of 10 m

Fig. 22. Comparison of the accelerations computed and monitored at the depth of 10 m

made for each of the two-components of recorded accelerations. The time histories of acceleration at the depth of 10 m computed by using the N-S component of recorded acceleration on the ground surface is shown with dashed lines in Fig. 21, together with the N-S component of acceleration recorded at the depth of 10 m during the earthquake. It may be seen that the overall shape of computed time history is in reasonable agreement with the acceleration time history re-

corded during the earthquake. The similar comparison is presented in Fig. 22 for the E-W component of acceleration, where it may as well be seen that the agreement between the computed and recorded motions at depth 10 m is overall satisfactory.

Peak values of accelerations and shear strains read off from the computed time histories are plotted versus depth in Figs. 19 and 20, together with the peak values of accelerations recorded on the ground surface

and at the depth of 10 m during the earthquake. It may be seen that the computed values are approximately equal to those monitored during the earthquake.

(2) Comparison between Recorded and Estimated Pore Water Pressures

Estimate of pore water pressures was made based on the results of response analysis and the laboratory tests on undisturbed samples. First of all, the magnitude of shear stresses that must have been induced in the soil deposit during the earthquake was determined by means of the seismic response analysis in which the time history of the accelerations monitored on the ground surface was used as input motions. The outcomes of such analyses have already been presented in Figs. 21 and 22. Peak values of computed time histories of shear stress at each depth of the deposit were divided by the mean effective confining pressure, σ'_0 , to determine the maximum shear stress ratio, $\tau_{\max}/\sigma'_0$, where σ'_0 is defined as $(1 + 2K_0)\sigma'_{v0}/3$ with the earth pressure coefficient at rest $K_0=0.5$ and the effective overburden

pressure, σ'_{v0} . The value of $\tau_{\max}/\sigma'_0$ obtained by using the N-S component of the ground surface acceleration is plotted versus depth in Fig. 23. The next step is to provide the relationship between the residual pore water pressure and the maximum shear stress on the basis of the laboratory test results. It is well known that this kind of correlation is strongly dependent on the number of cycles of shear stress application. In view of the fact that the major part of shaking during the 1985 October 4 earthquake consisted of one cycle of load application as demonstrated in Figs. 15 and 16, the relationship as above was established by reading off, from the triaxial test data, the amount of pore pressure generated during the first one cycle of loading application. The pore water pressures thus obtained are normalized to the initially applied confining stress and plotted versus the cyclic stress ratio, $\sigma_d/(2\sigma'_0)$, which was employed in each run of the triaxial tests, where σ_d is the amplitude of cyclic axial stress and σ'_0 is the confining stress initially applied to the triaxial samples. The relationships between the cyclic stress ratio and the residual pore pressure ratio established for the samples from the depths of 6 m and 14 m are shown in Figs. 24 and 25, respectively. The above relationships are considered to be indicative of the pore pressure buildup characteristics of in-situ soils at two depths under a single cycle of load application. With respect to the curve for the depth of 6 m, the test data from the silty sand portion alone were used to establish the relationship in Fig. 24. In view of the variability of the soil composition at this depth, the mere use of this relationship might be open to question.

Having known the pore pressure buildup characteristics as above, it becomes possible, as follows, to estimate the pore water pressures which must have developed in the in-situ deposit in the Owi Island during the 1985 October earthquake. The maximum shear stress ratio is estimated, from the diagram of Fig. 23, to have been on the order of 0.17 and 0.16 in the deposits at

Fig. 23. Maximum shear stress ratio obtained from the response analysis

depths of 6 m and 14 m, respectively. Entering with these values into the curves in Figs. 24 and 25, one can obtain the residual pore pressure ratio of 0.07 and 0.09 for the deposits at depths of 6 m and 14 m, respectively. The pore water pressure ratios thus estimated are shown in Fig. 26 versus depth. Also shown in Fig. 26 for comparison are the pore water pressures actually measured during the 1985 event. The actually measured pore pressures are shown by normalizing them by the mean effective confining pressure, $\sigma'_0 = (1 + 2K_0)\sigma_{v0}'/3$. It may be seen that the estimated pore water pressures are about 40% larger than the measured values both at the depths of 6 m and 14 m. As mentioned in the foregoing, the soil composition at depth 6 m is quite variable and the use of the relationship in Fig. 24 alone may not reflect the true picture of pore pressure response having occurred in this deposit. This may be the reason why coincidence is not good for this deposit.

Fig. 24. Cyclic stress ratio versus pore pressure ratio relationship for the soil at depth 6 m

Fig. 25. Cyclic stress ratio versus pore pressure ratio relationship for the soil at depth 14 m

Fig. 26. Comparison between the predicted and observed pore water pressures

The difference for the deposit at 14 m depth is not clearly known. It must be noticed, however, that a slight change in the curves correlating the cyclic stress ratio and pore pressure ratio does exert a large influence on the values of estimated pore water pressures, and for the small magnitude of pore water pressure as measured in the present investigation, the high degree of accuracy in the estimate appears difficult to be achieved by the present state-of-the-art method.

For comparison sake, the pore water pressures estimated and measured for the event of 1980 September 25 are quoted from the previous study (Ishihara et al, 1981) and presented in Fig. 26. It is apparent that the pore water pressures monitored during the 1985 event was much smaller than those measured during the 1980 earthquake.

CONCLUSIONS

After the in-situ measurements of accelerations and pore water pressures at the time of 1980 October 25 earthquake, the piezometers embedded in the ground on the Owi No.1 Island in Tokyo Bay became out of gear in 1982 because of the breakdown in the electrical system. In 1982 a new set of piezometers was embedded at the same place as continued efforts to monitor pore water pressures in the field. On this occasion, an additional piece of seismograph was installed under the ground at a depth of 10 m. Under the instrumentation as above, an earthquake of magnitude 6.2 shook the area of Tokyo Bay on October 4, 1985 and triggered a set of instruments to monitor accelerations and pore water pressures. The maximum horizontal acceleration recorded on the ground surface was 71 gal and the pore water pressures monitored in the deposits at depths of 6 m and 14 m were 17 cm and 40 cm in water height, respectively, each corresponding to 4.5% and 5.3% of the mean effective confining pressure. The recorded pore water pressures indicate that they had built up in the course of main shaking of the ground mo-

tions consisting of one full cycle of shear stress application. It was also observed that in the deposit of silty sand at depth 14 m containing fines of about 35%, the developed pore pressure dropped to about one-third of its peak value within 10 seconds, whereas the pore water pressure at a depth of 6 m remained almost undissipated at least for the period of 60 seconds. The silty sand deposit at depth 6 m is quite variable in the grain size composition, and therefore something unpredictable must have taken place in that deposit during the 1985 October earthquake.

When the new set of instruments was installed in 1982, detailed studies were made on in-situ soil conditions by means of the velocity logging and also by recovering undisturbed samples using the Osterberg type piston sampler. Part of the samples was air-shipped to the laboratory of Woodward-Clyde Consultants in New Jersey, U.S.A. and the rest was carried to the laboratory of the Department of Civil Engineering, University of Tokyo. Undisturbed samples were tested in these two laboratories to determine the strain-dependent shear moduli and damping ratio as well as to assess the liquefaction resistance. On the basis of the results of tests in the laboratory and in the field, seismic response analyses were conducted using the computer program SHAKE whereby the recorded ground surface accelerations were used as input motions. The motions thus computed for the deposit at the depth of 10 m were compared with those recorded there by the underground seismograph. The agreement between the computed and measured accelerations at the depth of 10 m was shown to be satisfactory. The seismic response analyses provided the information of the magnitude of shear stress that must have been induced in the soil deposits during the 1985 event. The cyclic shear stress ratio obtained therefrom was entered into the experimentally established correlation between the cyclic shear stress ratio and the residual pore pressure ratio in order to assess the pore water pressures that must have developed in the deposits during the 1985 earthquake. The

comparison of the pore water pressures thus estimated with those actually measured showed, however, some deviation for both the deposits at the depths of 6 m and 14 m.

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Around the time when the in-situ measurements had been started in Owi Island in 1977, various kinds of migrating wild birds began to find the place pleasant to settle in winter season. Their number has increased so many in recent times that the Tokyo Metropolitan office decided eventually to convert a section of the Island into a park for the bird sanctuary. Since the recording station was located in that part of the island, the instrumentation was abandoned and the in-situ observation at Owi No. 1 Island came to an end in February 1986 after 9 year's uninterrupted operation since 1977.

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