SHAKING TABLE TESTS ON PERMANENT DISPLACEMENT OF CAISSON QUAY WALL DURING EARTHQUAKES

Takashi ORITA[1], Ikuo TOWHATA[2] And Abbas GHALANDARZADEH[3]

SUMMARY

Shaking table tests were carried out on seismic stability of a gravity quay wall in order to understand the causative mechanism of distortion during earthquakes. The stress-strain behaviour of the foundation sand beneath a quay wall model was reproduced by using experimental data. It was thereby shown that strong seismic inertia force combined with excess pore water pressure and softening of sand induced large deformation of the foundation. Discussion was further made of dynamic earth pressure as well as fluctuation of excess pore water pressure behind a quay wall.

2. INTRODUCTION

The seismic resistance of a harbor structure is one of the keys which support post-seismic emergency action as well as restoration of the affected area. Despite this importance, the function of a harbor is often affected by the distortion of quay wall which is induced by seismic action. Even if a total failure of a quay wall is avoided, significant distortion may reduce the function of a harbor by, for example, preventing a land transportation from reaching the quay or damaging the shape of cranes and other facilities.

1 Engineer, Taisei Corporation, Tokyo, Japan.
2 Professor of Civil Engineering, University of Tokyo, Japan. E-mail towhata@geot.t.-tokyo.ac.jp
3 Assistant Professor of Civil Engineering, University of Urmia, Iran

The present paper focuses its attention on a gravity quay wall which is made up of a heavy caisson wall with backfill soil behind it. The seismic deformation of a quay wall of this type highly relies on two factors; the one being the earth and pore water pressures exerted by the backfill, while the other being the resistance against distortion which is generated by the foundation. It should be recalled that the conventional soil dynamics have elementary theories or models which can deal with these factors. The dynamic earth pressure is often calculated by the Westergaard's approximate solution (Westergaard, 1931), the cyclic component of pore water pressure was assessed by Matsuo and O-hara (1965), and the shear resistance at the base is calculated by using the friction angle of the foundation soil. Moreover, the seismic stability of a retaining wall is normally analyzed by using a seismic earth pressure theory (Mononobe and Matsuo, 1929). This theory assumes a development of a definite failure plane in the backfill. When the backfill is composed of water-saturated loose sand as in the present case, the idea of failure plane needs to be examined. Noteworthy is that the classical theories so far mentioned did not pay attention to the accumulation of excess pore water pressure. With this viewpoint, the authors carried out a systematic series of shaking table model tests in order to monitor the excess pore water pressure and to investigate its role played in the large distortion of a quay wall. Furthermore, it was also an important aim to measure and compare with classical theories the dynamic earth pressure as well as cyclic components of pore water pressure. For details, refer to Ghalandarzadeh et al. (1998).

2. METHOD OF SHAKING TABLE TESTS

The configuration of models employed in the present study is manifested in Fig.1. The whole model was placed in a model container which measured 200 cm in length and 40 cm in width. Two ends of the container were made of rigid walls, while side walls were transparent in order to facilitate the direct observation of the distortion of the model. The model ground consisted of compacted sand at the base, foundation loose sand which is of a trapezoidal shape, a rubble mound made of gravel, loose backfill soil, and a caisson model. Replacement of seabed clay by foundation sand is frequently practiced in reality so that settlement and lack of bearing capacity in soft clay are avoided. Be noted that the compacted base sand is a replacement of unliquefiable seabed clay in reality. The use of clay in model tests was avoided due to its long consolidation time needed.

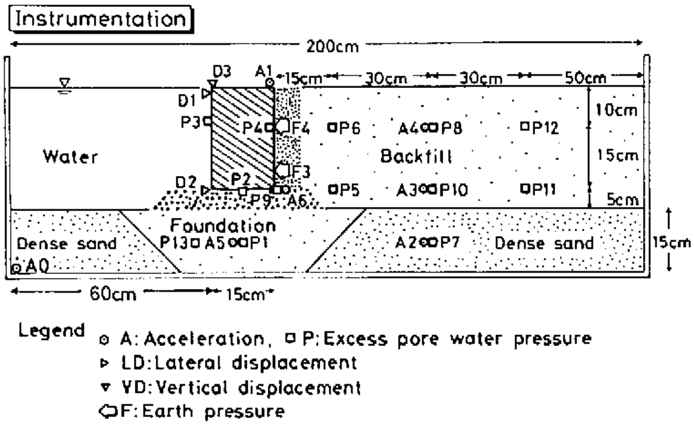

Fig.1 Cross section of model for shaking table tests

The model of a gravity quay wall was made of a wooden box which was filled with sand and water in the early stage of the program. This model produced only 1.5 kPa of effective base contact pressure which was lower than the effective overburden pressure at the same elevation in the backfill. It is, hence, called a light caisson model. In the later stage, iron weights were added to the sand filling in order to increase the effective contact pressure to 2.0 kPa. This model is called a heavy caisson model. A filter zone behind a caisson box was made of gravel. A variety of transducers were installed in a model in order to monitor pore water pressure and acceleration as well as quay wall displacement and earth pressure behind a caisson. Horizontal shaking took place in the longitudinal direction of the model in Fig.1.

3. DENSITY OF EMPLOYED SAND DEPOSIT

A special attention was paid to the density of employed Toyoura sand which had the maximum and minimum void ratios equal to 0.977 and 0.597, respectively. Loose Toyoura sand was placed by a technique of moist tamping in which sand with 5% water content was placed softly, followed by circulation of carbon dioxide gas and water from the bottom for higher degree of saturation. This technique was able to produce such a loose state of relative density as 0% or even less.

This loose state, which is hardly encountered in nature, was considered to be necessary in 1-g model tests. Since the research target was a large deformation and failure of water-saturated sand within a short period of shaking, the in-sit undrained behavior had to be reproduced in tests. It is well known that the undrained stress-strain behavior and dilatancy are affected by two important factors which are namely confining pressure and density. Since 1-g shaking table tests cannot produce the in-situ high level of stress, sand tends to be more dilatant and gain more shear resistance under undrained conditions. It was intended in the present study to cancel this undesirable feature by decreasing the density of sand.

4. DEFORMATION OF GRAVITY QUAY WALL

Experiments on Heavy Caisson Model

Figure 2 demonstrates the development of distortion of a heavy quay wall model in test TYP1-25. Void ratio was 1.02 in the foundation beneath the caisson, while 1.01 in the backfill. Shaking continued for around 10 seconds with its magnitude equal to 280 gal and frequency of 3 Hz. Embedded square grids which was made of colored sand helped one to observe through the transparent side wall the overall deformation of

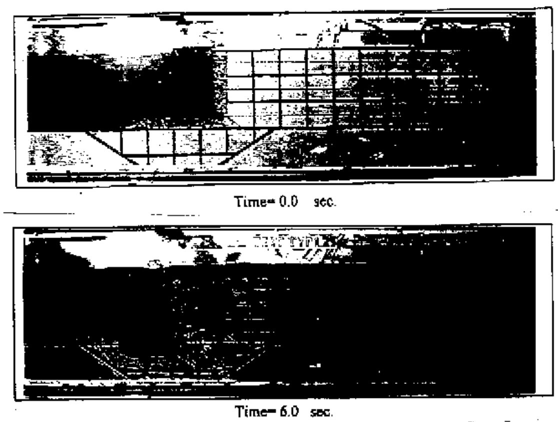

Fig.2 Development of distortion of heavy quay wall model (TYP1-25)

subsoil. In the figure, the caisson box overturned towards the water as occurred in most cases in reality, and large shear deformation in the foundation as well as in the backfill is noteworthy. There is no definite slip plane along which an overlying soil mass slid. Although a quay wall failure is frequently idealized for analysis by a slip failure of a rigid soil block, it does not match the experimental observation.

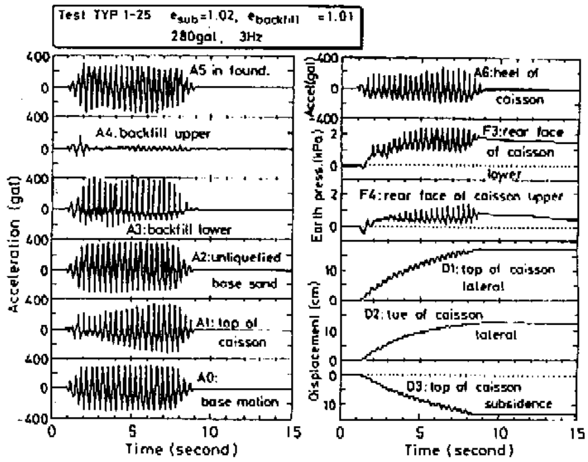

Fig.3 Time history of acceleration, earth pressure and displacement of caisson wall in test TYP1-25

Time history of response acceleration in the horizontal direction is illustrated in Fig.3. For the location of transducers, refer to Fig.1. While the shaking in the compacted base (A2) was regular and harmonic, the loose backfill (A3) exhibited an extremely irregular shape of response. This was apparently due to the loss of rigidity caused by liquefaction of sand. The increased level of earth pressure (F3 and F4), which is a total stress, was mainly due to the accumulated excess pore water pressure.

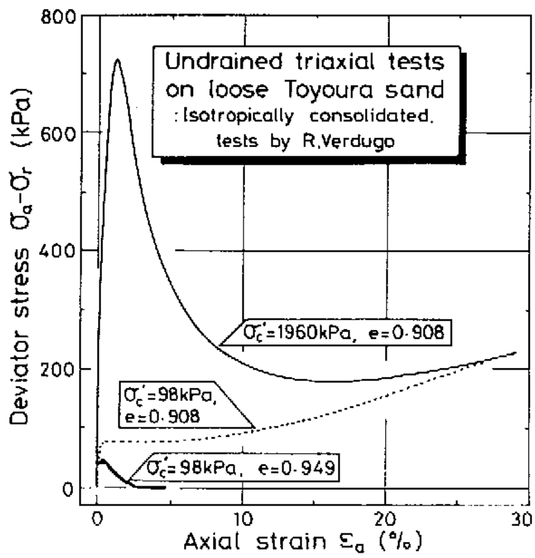

Fig.4 Undrained compression test of loose Toyoura sand (R.Verdugo, 1992)

In spite of the substantially loose density of foundation and backfill soils, the displacement of the caisson wall (D1, D2, and D3) ceased at the end of shaking. The fact that the caisson could not continue its motion after the end of shaking suggests that the liquefied foundation sand had sufficient shear strength against the static shear stress induced by gravity. This observation is in a good contrast with the results of triaxial undrained compression test on the same Toyoura sand in Fig.4. Be noted therein that even denser sand (void ratio = 0.949) exhibited negligible shear strength in a large strain range after the peak strength and softening that follows. One of the reasons for this discrepancy is the higher confining pressure in Fig.4 which induces more negative dilatancy. What is more important seems, however, the initial static stress in the model test which can affect the large deformation behavior of sand (Kato et al., 1999).

The time history of excess pore water pressure in test TYP1-25 is illustrated in Fig.5. Generally, the developed pore pressure behind the caisson (P4 and P6) was lower than the initial effective vertical stress, svo', while 100% development of pore pressure was the case in the backfill at a sufficient distance from the caisson (P10, P11, and

P12). The spatial distribution of excess pore water pressure thus observed was illustrated in Fig.6 where contour curves of the pore pressure near the end of shaking were drawn. Note that the numbers in the figure stand for the excess pore water pressure divided by the initial effective vertical stress, svo', in percentage. It is evident in the figure that the backfill soil near the quay wall developed lower magnitude of pore pressure. This observation is consistent with the experience during the 1995 earthquake in Kobe where no sand boiling was detected near quay walls of man-made islands which had liquefaction at many places inside (Towhata et al., 1996).

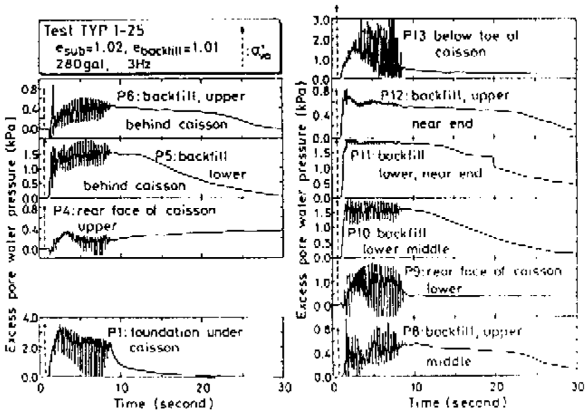

Fig.5 Time history of excess pore water pressure in test TYP1-25

The lack of high pore pressure near the quay wall model in experiments is attributed to the outward translation of the wall. In this regard, it should be recalled that extensive liquefaction in the backfill occurred when a caisson wall was sandwiched between two man-made islands and could not move laterally (Towhata et al., 1996). By assuming undrained condition and water saturation, the variation of excess pore water pressure, Δu, due to changes of vertical and horizontal stresses is expressed by

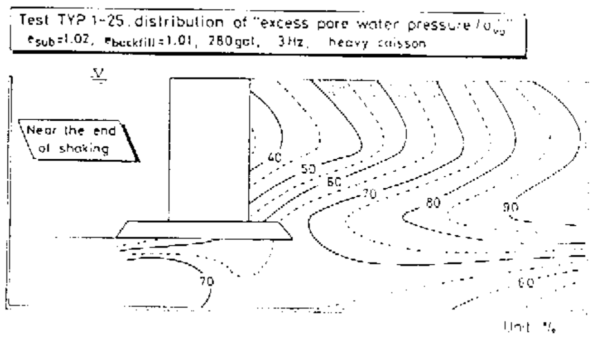

Fig.6 Pore pressure development ratio in backfill in test TYP1-25

$$\Delta u = \Delta\sigma_h + A(\Delta\sigma_v - \Delta\sigma_h) \quad (1)$$

where "A" is something similar to Skempton's pore pressure parameter. It was supposed initially that the horizontal earth pressure, $\Delta\sigma_h$, was reduced by the outward motion of the quay wall which in turn decreased the pore pressure as Eq.1 infers. This idea was, however, ruled out later because the measured lateral earth pressure, F3 and F4 in Fig.3, increased conversely. At present, accordingly, it is presumed

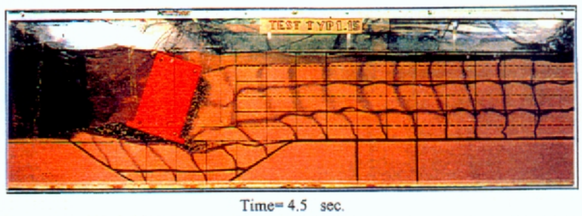

Fig.7 Configuration of test on light caisson model (TYP1-15)

that large shear deformation in the backfill (Fig.2) induced positive dilatancy and reduced pore pressure.

Experiments on Light Caisson Model

In the early stage of the research program, the importance of the weight of a quay wall model was not understood. Hence, the quay wall box was filled simply with sand (light caisson model). Fig.7 illustrates a typical test result (TYP1-15) in which void ratio was 0.81 in foundation and 1.00 in backfill, respectively. Shaking was of 300 gal and 3 Hz with duration of 10 seconds. It is interesting that the model quay wall rotated backwards, while translating towards the water. This direction of rotation is opposite from what was observed in many real damaged quay walls as well as in heavy caisson models.

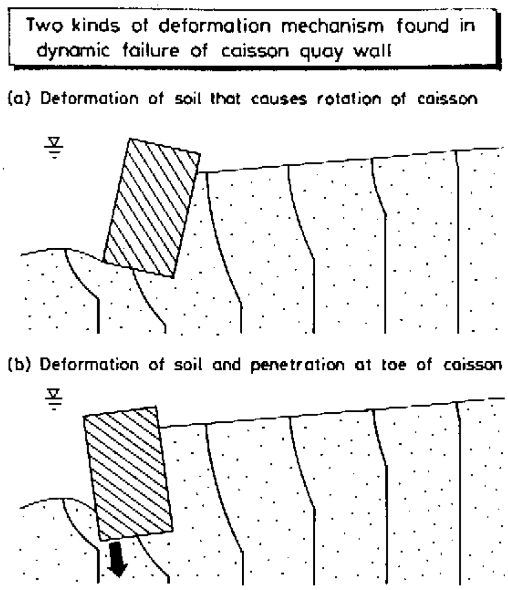

Fig.8 Different manners of deformation of heavy and light caisson models

Comparison is made schematically in Fig.8 of manners of deformation for both heavy and light quay wall models. Firstly, the magnitude of lateral displacement in the backfill soil is greater near the surface than at the bottom for both kinds of walls. By recalling that a conventional mechanism of rotational slip failure along a circular arc generates greater displacement at a lower elevation, in proportion to the distance from the center of rotation, it is reasonable to say that the slip of a rigid block is not adequate for the mechanism of quay wall distortion. Further, despite that both types of quay wall models were associated with large shear deformation of backfill soil, the direction of wall rotation was opposite. It seems that, when the heavy caisson translated towards the water due to the increased inertial force of the wall mass, the contact stress at its toe increased (Fig.8b), and the toe consequent penetrated into the foundation sand. When a quay wall was heavy (Fig.2), the significant penetration of the toe and the displacement of the foundation sand towards the bottom of the model sea was evident.

At the end of this section, a remark has to be made of Fig.9 in which a light caisson model was situated upon densified foundation sand (void ratio = 0.62). Shaking had an extremely strong amplitude of 880 gal and 10 Hz in frequency. Although the foundation sand did not deform substantially, the displacement of the caisson model was still significant. This infers

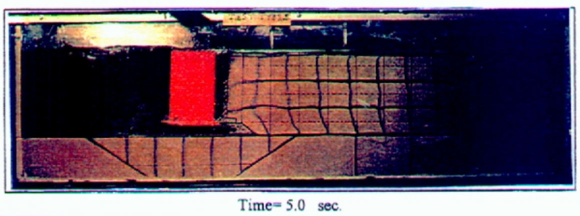

Fig.9 Configuration of light caisson model resting on densified foundation sand (TYP1-05)

that the limited magnitude of frictional resistance between a caisson and subsoil still allows damage to occur even when the foundation sand is compacted.

5. DYNAMIC EARTH PRESSURE ON QUAY WALL

In the following sections, study is going to be made of earth pressure and pore water pressure exerted on the caisson wall, both heavy and light ones, by backfill soil. Firstly, Fig.10 indicates the magnitude of cyclic component of earth pressure, which is total stress in definition, plotted against the initial void ratio of the backfill soil. In order to remove the effects of intensity of shaking, the measured amplitude of earth pressure was divided by $k_h \gamma z$ in which k_h designates the amplitude of base acceleration divided by the gravity acceleration, while g is the total unit weight of backfill sand, and z the depth measured from the surface. This normalized earth pressure amplitude at a shallow depth (z=10 cm) is greater than that at a greater depth (z=20 cm). This is probably because, behind the caisson wall, the magnitude of shaking was greater near the surface; compare A1 and A6 records in Fig.3, for example. It was not possible to find out a clear effects of the void ratio because its range of variation was limited.

The Mononobe-Okabe formula (Mononobe and Matsuo, 1929) has long been used for calculation of design seismic earth pressure. Being an extension of the original Coulomb formula, it assumes a development of slip surface and a rigid block movement of soil.

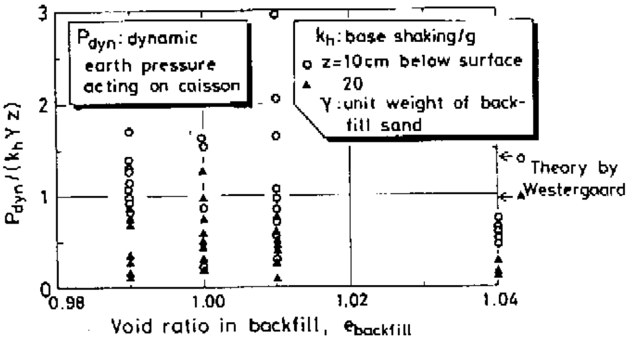

Fig.10 Magnitude of cyclic component of earth pressure behind caisson wall

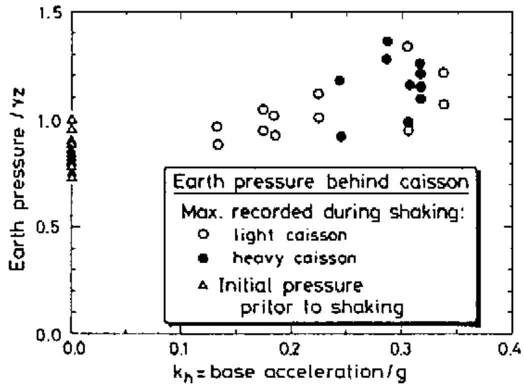

Fig.11 Magnitude of total earth pressure behind caisson wall

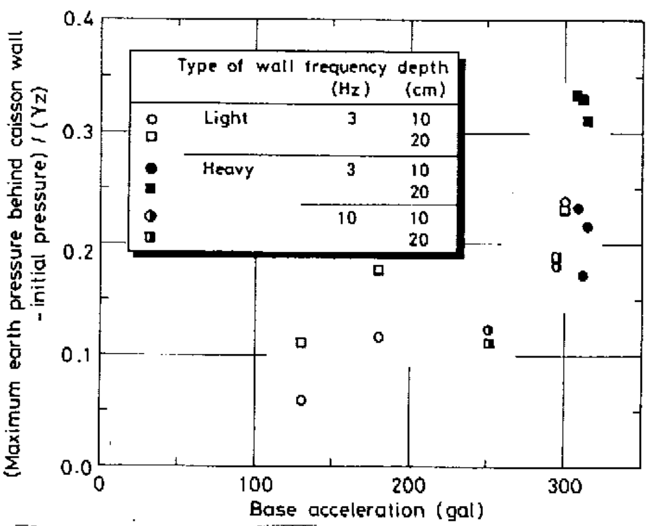

Fig.12 Increment of earth pressure during shaking

Since observation of model tests (Figs 2 and 9) do not support such a mechanism of failure, a different kind of earth pressure calculation is desired.

An approximate solution of dynamic liquid pressure acting on a rigid vertical wall was proposed by Westergaard (1931). By applying it to the present problem,

$$P_{dyn} = \frac{7}{8} k_h \gamma \sqrt{Hz} \tag{2}$$

where P_{dyn} is the cyclic amplitude of the earth pressure, and H designates the depth of liquefied layer. A prediction by this formula is compared with observation in Fig.10. Since the range of variation of experimental data is wide, it is difficult to discuss about the extent of agreement. It seems possible, however, to state that the Westergaard prediction gives a reasonable upper bound of the observed earth pressure amplitudes.

In the next discussion, the cyclic component of earth pressure as discussed above is added to the increment of the mean earth pressure. The maximum value of the earth pressure which was thus obtained was plotted in Fig.11 against the intensity of base shaking. Evidently, the earth pressure increased with the magnitude of base shaking. The weight of a caisson model did not affect the earth pressure. Fig.12 is concerned with the increment of the total pressure from the initial pressure prior to shaking. The initial earth pressure is the one measured by those pressure transducers behind the caisson. The maximum increment thus determined increases with the intensity of the base shaking. It appears possible to represent this relationship between the earth pressure increment and the base shaking by a linear equation, although being merely a rough approximation.

6. MAGNITUDE OF PORE WATER PRESSURE FLUCTUATION BEHIND QUAY WALL

It was already mentioned in a preceding section that the developed excess pore water pressure behind a caisson model was lower than the initial overburden pressure probably because of the large shear distortion of the soil. In the present section, the cyclic fluctuation of the excess pore water pressure is focused on. Since the wall-soil interaction is interested in, only those pore pressures measured behind the wall are studied.

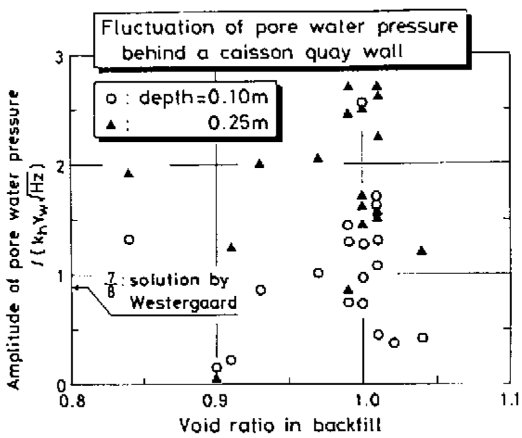

Fig.13 Magnitude of cyclic component of pore water pressure behind caisson wall

Matsuo and O-hara (1965) carried out shaking table tests to reveal that the amplitude of fluctuation of pore pressure is approximated by 70% of what a formula of Westergaard type gives. With this in mind, Fig.13 of the present study normalized the measured pressure fluctuation by a Westergaard-type factor. Firstly, the figure shows that the extent of fluctuation is greater when the backfill sand is looser. Secondly, the fluctuation is greater at lower elevation. This feature is opposite from what was observed for earth pressure (Fig.10). Accordingly, the measured fluctuation is much greater than the vertical coordinate of 7/8 which is suggested by Westergaard-type formula.

7. REPRODUCTION OF STRESS-STRAIN BEHAVIOR OF FOUNDATION SAND

It is an interesting attempt to reproduce the stress-strain behavior of soils which are subjected to shaking-table tests. Although a video camera could capture the time history of large shear strain through a transparent wall of a soil container, it is not able to directly measure the shear stress. An alternative idea was proposed by Koga and Matsuo (1990) who obtained shear stress, τ, in a level model deposit by integrating in the vertical direction the monitored horizontal acceleration;

$$\tau = \rho \int_0^z (acceleration) dz \qquad (3)$$

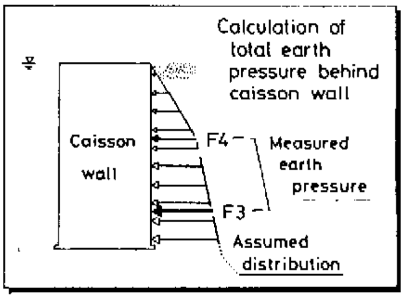

Fig.14 Monitoring of earth pressure behind caisson model

where ρ is the mass density of sand, and the integration is made from the surface downwards. A similar attempt was made by Sasaki et al. (1992) as well. The present study further extended the method as described above to a two-dimensional configuration of model ground. It was therein aimed to investigate the stress-strain and stress-path behaviors of the foundation sand which deformed substantially and induced distortion of the caisson model during shaking (Fig.2).

Figure 1 showed that the caisson model had many transducers which recorded its horizontal acceleration, displacement, earth pressure and pore pressure. Fig.14 illustrates how the total earth pressure behind a caisson model was determined by using two earth pressure data at F3 and F4. A linear distribution between the recorded pressure was assumed and the total horizontal thrust force was determined. The equation of motion of a caisson wall in the horizontal direction is given by

Mass × (acceleration) = (Earth pressure behind caisson)
- (Water pressure in front of caisson) - (Shear force at base) (4)

Since acceleration, earth pressure, and water pressure in Eq.4 were measured, it was possible to calculate the remaining shear force at the base. Because this shear force is simply a cyclic component, the initial static shear stress was calculated on the other hand by Boussinesq elastic theory and added to the cyclic component. Then, the action-reaction law allows that the obtained shear force is equal to the shear force in the foundation sand. The shear stress in the foundation sand was obtained by dividing the shear force by the base area of the rubble mound beneath the caisson.

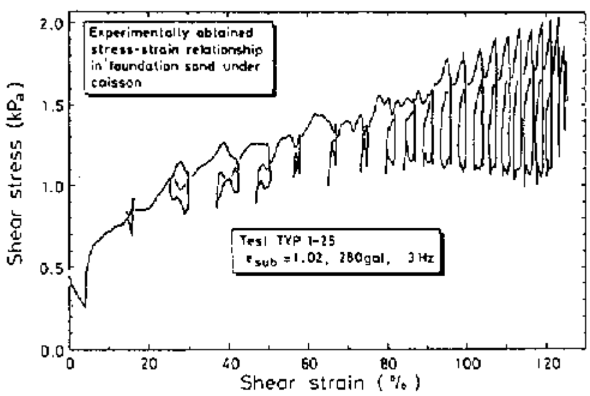

Fig.15 Reproduced stress-strain behavior of extremely loose foundation sand under heavy caisson

The time history of shear strain in the foundation was more easily obtained; the horizontal displacement at the toe of a caisson (D2 in Fig.1) was divided by the thickness of the foundation sand. Moreover, the time history of the vertical effective stress in the foundation was obtained by subtracting the recorded pore pressure (P1) from the initial total stress that was derived by Boussinesq theory as well. The effective stress was plotted against the shear stress to draw a stress-path diagram.

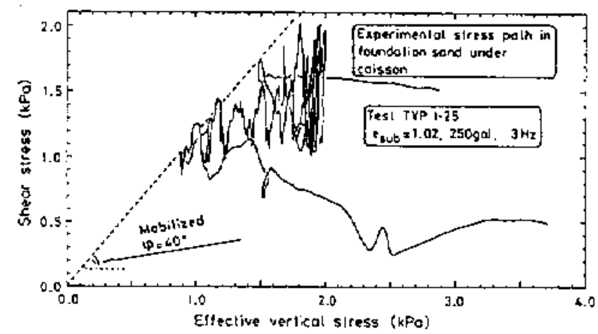

Fig.16 Reproduced stress-path behavior of extremely loose foundation sand under heavy caisson

8. DISCUSSION ON STRESS-STRAIN BEHAVIOR OF FOUNDATION SAND

The reproduced behavior is going to be presented for two tests. The first is the one in Fig.2 (TYP1-25) in which a remarkable distortion occurred in the very loose foundation sand. Fig.15 reveals the reproduced stress-strain behavior. Starting from the initial state of static shear, the shear stress increased due to the development of pore water pressure behind the caisson. Noteworthy is that shear strain increased when the shear stress was loaded, while it decreased upon unloading of stress. Each cycle of loading and unloading produced residual deformation, which accumulated to the significant strain and, consequently, the translation of the caisson model occurred. When shaking was switched off, the deformation of soil and displacement of the caisson stopped as shown in Fig.3. Thus, flow failure, which means a large deformation of sand under static force, did not occur even in this loose sand.

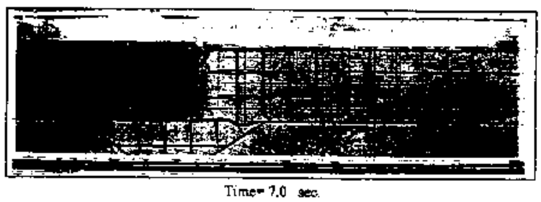

Fig.17 Configuration of test on heavy caisson model resting on dense foundation sand (TYP1-21)

The stress-path behavior is the same test in manifested in Fig.16. This figure demonstrates a line of mobilized friction angle equal to 40 degrees. This angle is probably not too far from the true friction angle of the tested sand. The state of effective stress reached the failure envelope immediately after the onset of shaking. Although the effective stress did not decrease to zero, the stress near the failure envelope made it very easy for shear strain to accumulate with loading cycles. In this regard, it is reasonable to state that the large displacement of the caisson model was mainly induced by the intense shaking, while pore pressure development in the loose foundation sand made deformation easy to increase by bringing about the stress state near the failure line.

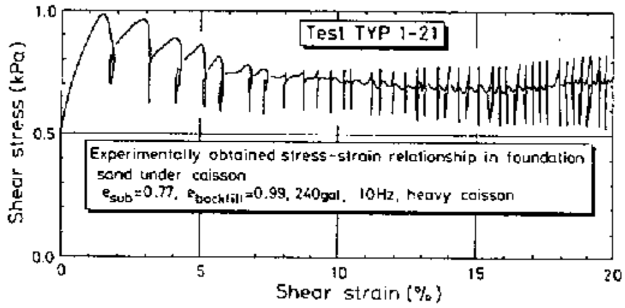

Fig.18 Reproduced stress-strain behavior of denser foundation sand under heavy caisson

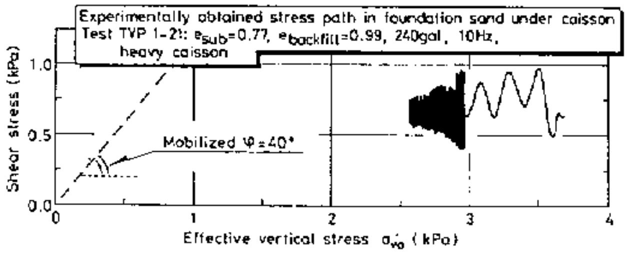

Fig.19 Reproduced stress-path behavior of denser foundation sand under heavy caisson

Figure 17 illustrates the shape of the model (test TYP1-21) which was situated upon denser foundation sand (void ratio = 0.77). In spite of the strong magnitude of shaking (240 gal at 10 Hz), distortion of the model was not significant. The stress-strain behavior which was reproduced from this test results is presented in Fig.18. Although displacement of the caisson wall accumulated with the number of loading cycles, the magnitude of strain was much smaller than what was observed for loose foundation sand in Fig.15. This observation is consistent with the stress-path diagram of the same test (Fig.19) in which the state of effective stress remained far from the failure line. Since the excess pore water pressure in the loose backfill sand (void ratio = 0.99) was high (Fig.20) similar to Fig.6, it is reasonable to state that densification of foundation sand is much more important than improvement of backfill sand in order to mitigate the seismic deformation of a gravity quay wall.

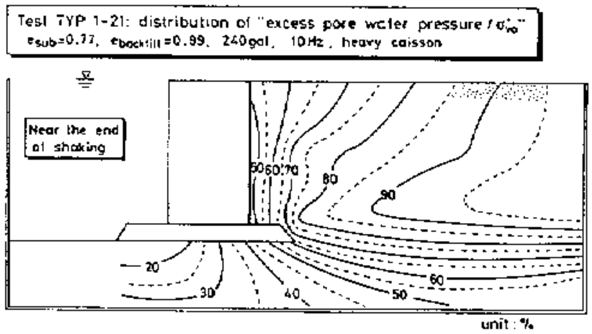

Fig.20 Distribution of excess pore water pressure in Test TYP1-21

9. CONCLUSION

A series of shaking table tests were carried out in 1-g gravity field in order to investigate the causative mechanism of significant displacement of a gravity quay wall during earthquakes. Another point of interest was the magnitude of seismic earth pressure and pore water pressure that occur behind a quay wall. A special care was taken to compensate for the bad effects of low confining pressure in 1-g model test by reducing the density of sand. The conclusions drawn from the tests are described below.

1) The large deformation of a quay wall is induced by substantial shear deformation of sand behind and beneath the wall. Rigid block movement of soil is not the case because no slip plane was observed.

2) Westergaard formula of seismic liquid pressure is not bad for assessing cyclic component of earth pressure behind a caisson wall.

3) The maximum earth pressure increases more or less linearly with the intensity of base motion.

4) Excess pore water pressure does not attain 100% liquefaction behind a caisson wall. This is because the concerned sand develops positive dilatancy while achieving significant shear deformation.

5) The cyclic component of excess pore water pressure is much greater than a formula of Westergaard type infers.

6) The large distortion of a caisson wall is caused by the combined effects of high pore water pressure in the foundation sand and the seismic inertia force acting on the caisson and the backfill.

7) For mitigation of the caisson displacement, it is more adequate to densify the foundation sand than to improve the backfill sand.

10. ACKNOWLEDGMENT

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