Shaking induced water pressure acting on quay walls

Pression d'eau induite per le seisme sur le quai

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ABSTRACT: Response of water pressure was measured behind a model quay wall and inside the fully saturated backfill. Both hydrodynamic and hydrostatic excess pore pressures were observed to be affected by the movement of the model wall. The amplitude and the velocity of the wall movement affected the water pressure. Results obtained from these tests show that the nature of water pressure is more complicated than what it is assumed in simplified theories.

RÉSUMÉ: La re'ponse de la pression interstitielle a e'te' mesure'e derrie`re un modele de mur de quai, a` l'inte'rieur du massif sature. On observe que les deux composantes hydrodynamique et hydrostatique de la pression interstitielle sont affecte'es par le mouvement du mur. L'amplitude et la vitesse du mouvement influencent la pression induite. Les re'sultats de ces essais montrent que la nature de la ge'ne'ration de la pression est plus. Sophistique' par rapport a` celle suppose'e par les the'ories simplifie'es.

1 INTRODUCTION

The seismic behavior of water front quay walls has attracted a lot of research interests due to its complexity. The relatively poor performance of this type of structures during some recent earthquakes has shown the need for better understanding of their behavior, Towhata et al. (1996). The soil behind and beneath such structures is often granular saturated material, susceptible to increase pore water pressure when subjected to seismic loads. The existing design regulations are mostly based on the fulfillment of the stability requirements. They consider a simple failure mechanism and then approximate the earth pressure using limit equilibrium or equivalent methods. Hydrodynamic pressure is evaluated by the simplified Westergaard (1931) method. To take into account of the permeability of the backfill soil, these methods treat the problem only for two extreme cases of very high permeable and very low permeable soils. Although there are some methods to modify the estimation of hydrodynamic pressure in these cases, e.g. Matsuo and O'hara (1965), Matsuzawa et al (1985), their methods do not simulate the real situation. For instance, they ignored the deformabilty of the soil skeleton and moreover, the possibility of increase of pore water pressure in the saturated granular material due to cyclic shearing. This latter phenomenon can seriously affect the stabi - ity of the quay wall and the basic mechanism of failure. Complexity of the behavior of the model quay wall observed in the present study may suggest a complicated behavior in the real cases. Some of the observations indicating such complication, will be explained in this paper.

2 TEST PROCEDURE

Figure(1) shows the schematic view of the model used in this study. A variety of models with different density of subsoil and backfill were tested using a shaking table facility. The input motion in all cases was harmonic but with different acceleration amplitudes and frequencies. In two tests the gravelly zones were removed and a membrane system was used to prevent dissipation of excess pore water pressure due to drainage. This was done to understand the effect of filter zone on the behavior of the model. The earth pressures were measured in two different depths of 10 and 20 cm. To understand the response of pore water a number of pore pressure transducers were embedded in different points. Accelerations of different points were also measured using a series of accelerometers as shown in Figure(1).Three displacement transducers provided sufficient data to analyze the displacement of model wall.

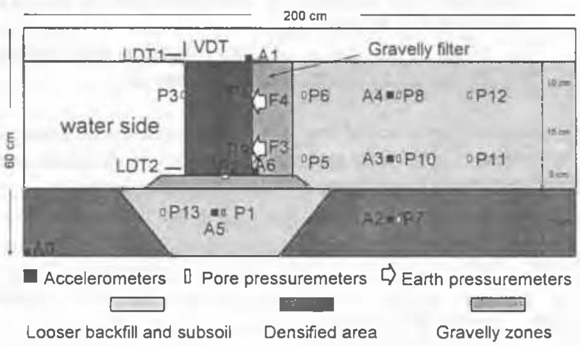


Figure 1 Schematic view of the model and instrumentation

3 CONVENTIONAL METHODS OF ESTIMATION OF DYNAMIC PRESSURES ACTING ON RETAINING STRUCTURES

Seismic earth pressure conventionally is calculated by the Mononobe-Okabe equation. This method basically is an extension of Coulomb theory, which assumes failure behavior of a rigid wall with a dry backfill. According to the Mononobe-Okabe equation active earth pressure can be calculated as follows:

$$P_{ac} = \frac{1}{2}\gamma H^2(1 - k_v)K_{ae} \tag{1}$$

where γ is the unit weight of backfill soil, H is the height of wall, k_v is the vertical seismic coefficient, and K_{ae} is active dynamic earth pressure coefficient. K_{ae} is a function of the geometry of the wall, the strength characteristics of backfill and also the horizontal and vertical seismic coefficients which comes in terms of $\psi = tg^{-1}k_h/(1-k_v)$. Seed and Whitman (1976) proposed an approximate value for K_{ae} by separating it into two static active earth pressure and a dynamic increment equal to 3/4 of horizontal seismic coefficient k_h. In the case of waterfront quay walls, where water is present in backfill soil, the hydrodynamic pressure is approximated by Westergaards' equation as below:

$$\Delta U_w = -\frac{7}{12} k_h \gamma_w H^2 \qquad (2)$$

This equation approximates the total thrust induced by free water, on a vertical surface. In reality the presence of soil skel - ton restricts the movement of the water and hence can influence the hydrodynamic pressure. Depending on the permeability of backfill soil, the presence of the water is taken into account by modifying seismic coefficient ψ to calculate the soil skeleton pressure. The hydrodynamic pressure calculated by the Westergaards' formula, is then added accordingly. Early work by Matsuo and O'hara (1965) was an attempt to consider the effect of soil skeleton and the period of base motion on the hydrodynamic pressure. Based on Matsuo-O'hara's method Matsuzawa et al. (1985) proposed a simplified equation to calculate hydrod - namic pressure as well as to correct the seismic coefficient ψ for evaluating dynamic earth pressure. The followings are the most important shortcomings of these methods:

- the wall is assumed to be non-failed
- Backfill soil is considered to be a rigid porous media
- Static build up of pore water pressure which can soften the backfill soil is ignored.
- Only inertial force and hence the relative movement of water and soil skeleton is considered as the source of hydrodynamic pressure.
- The effect of dilatancy of backfill soil in cyclic deformation caused by seismic loads is ignored.

To consider the effect of the hydrostatic excess pore pressure on the seismic coefficient ψ and hence on the dynamic earth pressure, Kramer (1996) proposed the following formula:

$$\gamma = \gamma_b(1 - r_u)$$
$$\psi = \tan^{-1}\left[\frac{\gamma_{sat} k_h}{\gamma_b(1 - r_u)(1 - k_v)}\right] \qquad (3)$$

Where r_u is pore water pressure ratio, $r_u = u_{excess}/\sigma'_0$, γ_{sat} and γ_b are saturated and submerged unit weight of backfill soil, respectively. Although the effect of hydrostatic excess pore pre - sure is taken into account in this equation, there exists the question that how much excess pore pressure could be generated behind a failed quay wall. Towhata et al. (1996) reported a clear lack of liquefaction just behind the failed quay walls during the great Hanshin earthquake which could be considered as the evidence of different liquefaction potential in the areas adjacent to quay walls.

The effect of interaction between the wall and backfill soil and therefore different time lags of dynamic earth and water pressures are also ignored in the methods described earlier.

4 TYPICAL TEST RESULTS

Figure(2) illustrates the typical test data obtained from the model tests during the current study. The measured history of excess pore pressure appeared to have two components. One is the accumulative residual pressure which can be supposed as the average of the measured data and will be named as hydrostatic excess pore pressure in this text and the other is the fluctuating part which will be referred to as cyclic water pressure. As it can be seen in this figure the excess pore pressure was generated immediately after the base shaking was started. After a few cycles the excess pore pressure reached to its maximum and decreased with an almost a fast rate. This sudden reduction was followed by an almost slower rate within the following cycles. This trend was observed in most of tests in the current study. The fast reduction of hydrostatic excess pore pressure which is referred to as initial dissipation here is almost coincides with the movement of the model quay wall whereas the slower reduction which is called steady dissipation in this text is probably due to drainage.

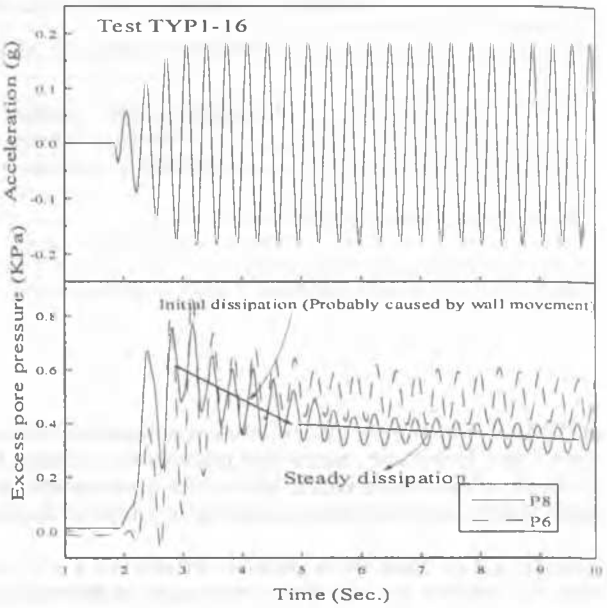


Figure (2) Typical behavior of excess pore pressure observed in model tests

5 HYDROSTATIC EXCESS PORE PRESSURE

Propagation of shear wave in saturated cohesionless backfill can create a residual water pressure called excess pore pressure. In the worst case, it can cause liquefaction and consequently full loss of strength of soil. To analyze the Hydrostatic and cyclic part of measured water pressures, the high and low frequency components are separated using F.F.T. technique. The maximum hydrostatic excess pore pressures are used to calculate r_u which shows the excess pore pressure ratio. Figure(3) shows the typical distribution of the maximum static excess pore pressure in the backfill area. As it is shown in this figure, 100% of water pre - sure ratio was achieved only at far distance from the wall. The transducers near the quay wall recorded the maximum excess pore pressure less than 100%.

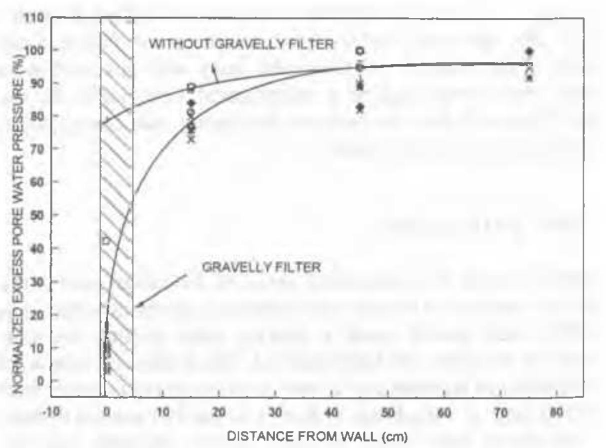
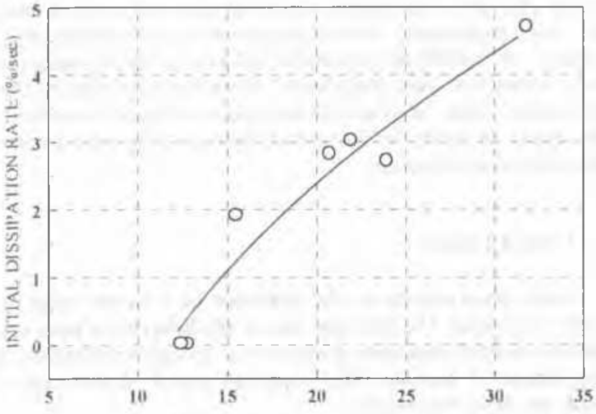
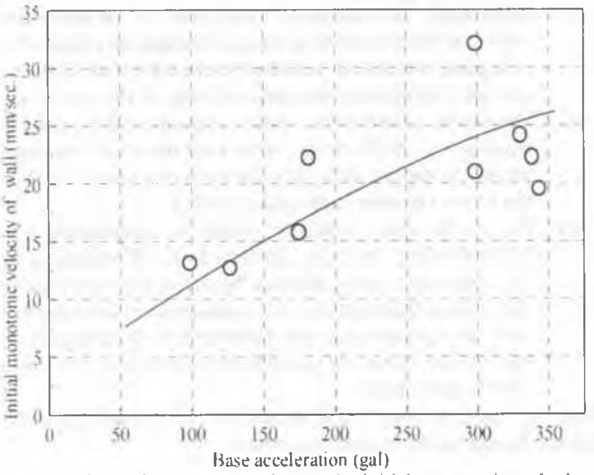


Figure3. Distribution of maximum residual excess pore water pressure in backfill area



Figure(4)Effect of the monotonic velocity of the wall on initial dissipation of hydrostatic excess pore pressure

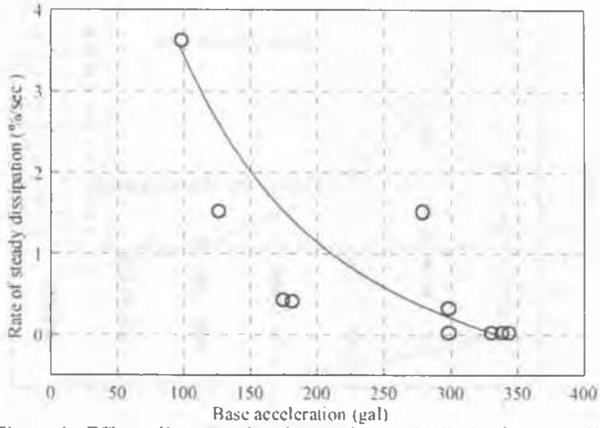


Figure(5) Effect of base acceleration on the initial monotonic velocity of model quay wall

Just behind the wall and particularly inside the gravely filter zone the excess pore pressure was almost zero. This reduction to some extent there existed, even if the filter zone was removed. It could be understood that the presence of the vertical filter and outward movement of the wall were the reasons for the reduction of the water pressure in near field.

Another observation about the hydrostatic excess pore pressure was its dependency to the velocity of the wall. When shaking starts and excess pore pressure reaches to its maximum value, the wall starts to move monotonically as a consequent. The recorded displacement of the model wall was used to calculate the monotonic velocity. As it is shown in figure(4) the rate of initial dissipation of excess pore pressure increased with increasing of maximum monotonic velocity of the model wall. It is interesting to notice that the higher base acceleration which caused faster monotonic movement of the wall (figure(5)), could create higher possibility of liquefaction. Interestingly the initial dissipation was faster in cases of higher velocity that means when the base acceleration was greater. It suggests that the initial rapid drop of the excess pore pressure was strongly affected by the wall movement.

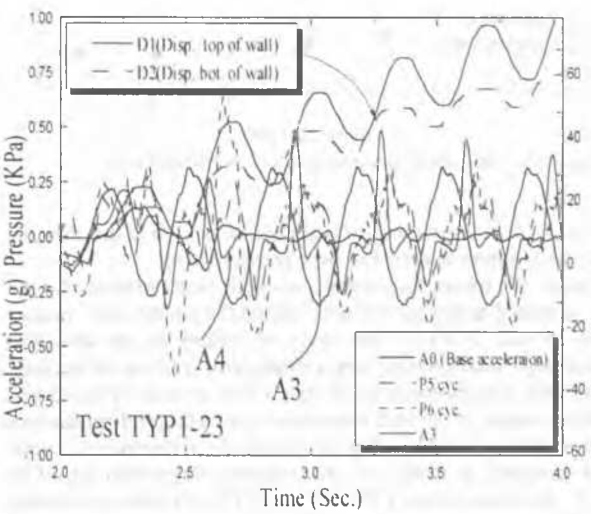
In contrast the base acceleration had significant effect on the steady rate of dissipation of hydrostatic excess pore pressure. This fact is shown in figure(6). As it can be seen in this figure the dissipation rate has reduced when the base acceleration was increased. This may suggest that the later steady dissipation could be due to drainage rather than the wall movement. The effect of the wall movement which was obvious in all tests of the present study has to be taken into account in seismic stability of quay walls.



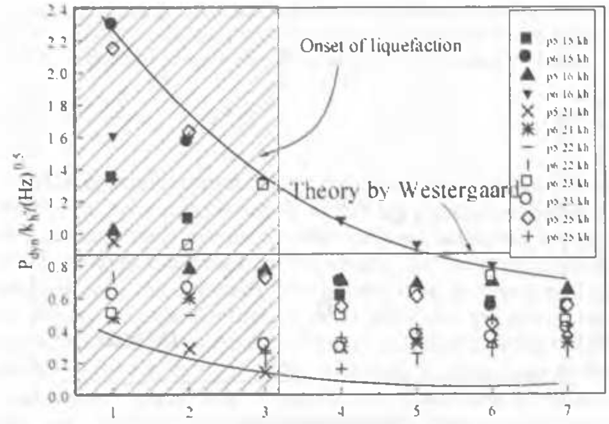
Figure(6) Effect of base acceleration on the steady dissipation rate of hydrostatic excess pore pressure

6 CYCLIC WATER PRESSURE BEHIND THE MODEL QUAY WALL

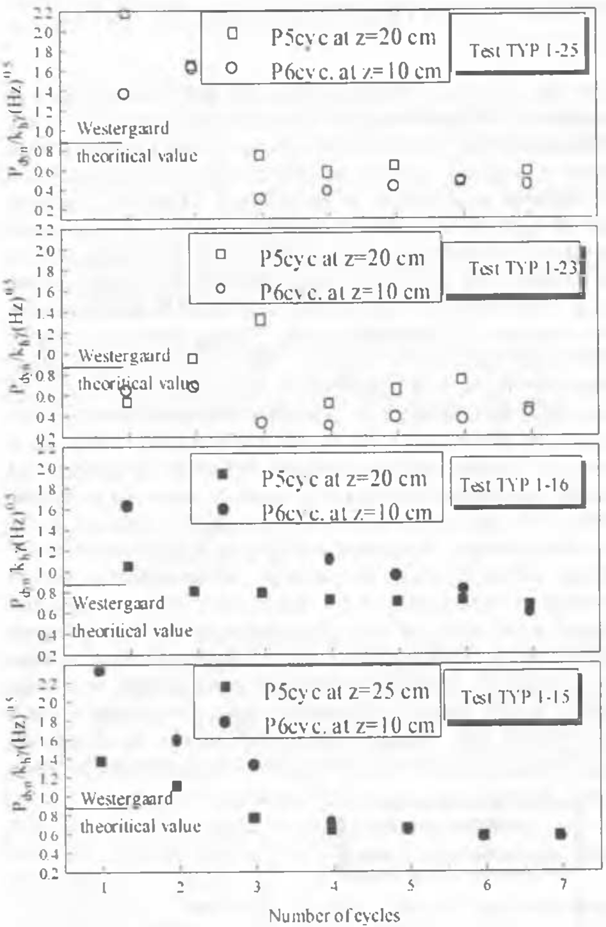
As it was shown in figure(2) the excess pore pressure had two components of static and cyclic nature. In this section the nature of fluctuating part of the measured excess pore pressure will be discussed. Figure (7) shows the time history of base acceleration A0, response acceleration in backfill soil A3 and A4, displacement of wall D1 and D2 along with cyclic part of excess pore pressures P5 and P6 measured behind the model quay wall. It can be seen that the cyclic water pressure has a larger value during first two cycles and its value decreased to an almost constant value in the continuing cycles. During first two cycles the cyclic water pressure is almost 180 degree out of phase comparing to the base acceleration A0. After the third cycle phase difference reduced to a smaller value and remained constant. It is interesting to notice that the cyclic water pressure is almost in phase with acceleration measured in position A3 probably indicating that the cyclic water pressure has an inertial nature. The measured displacement is roughly in phase with cyclic water pressure. Recorded acceleration at position A4 shows a drastic reduction from the second cycle indicating the soil softening or liquefaction. Very large value of cyclic pressure belongs to this period of time. It seems the value of cyclic water pressure before liquefaction is larger than that of after liquefaction. Except very later cycles which have sometimes very large negative cyclic pressure, in most of tests the explained trend was observed. This fact can be seen in figure(8). In this figure reduction of cyclic water pressure after onset of liquefaction is



Figure(7) Typical time history of accelerations, displacements and cyclic water pressure



Figure(8) Variation of cyclic water pressure during different cycles of shaking(before and after onset of liquefaction)



Figure(9) Cyclic water pressure observed in different tests

obvious. The term onset of liquefaction is used to describe the state of maximum hydrostatic pore pressure here.

Figure (9) shows the normalized cyclic water pressure for the data obtained at different depths behind the model wall. In tests TYP1-25 and TYP1-23 the mode of failure of the wall was overturning which means larger displacement of top of the wall. In contrast the failure mode in tests TYP1-16 and TYP1-15 was rotation which is the wall movement with a larger displacement at its bottom. It seems that the amplitude of normalized cyclic water pressure at depth 20 cm is slightly larger than that of 10 cm in the case of tests TYP1-25 and TYP1-23 with overturning failure. Whereas cyclic water pressure is smaller at the deeper point in the case of tests TYP1-16 and TYP1-15 with rotation mode of failure showing the effect of failure mode on the cyclic water pressure. Although it is difficult to see this type of data in all cases particularly when the cyclic shear deformation and dilatancy of backfill soil makes the behavior of cyclic water pressure sometimes very complicated, there are some other test data of present study similar to those shown in figure(9) confirming the effect of mode of movement (failure) of the model wall on the cyclic water pressure.

7 CONCLUSION

Some observations on the behavior of a model quay wall were explained. The attention was on the behavior of pore water which inserted excessive pressure on the quay wall during the application of shaking. The important points drawn from this study are described below:

a) Excess pore pressure generated right behind the model wall had two components of residual hydrostatic and fluctuating cyclic parts.

b) Hydrostatic pore pressure appeared to be affected by wall movement showing a rapid dissipation (drop).There is a good correlation between the rapid initial dissipation and the maximum monotonic velocity of the model wall.

c) Maximum hydrostatic pore pressure ratio did not reached to 100% in the areas near the wall. This observation is supported by the real behavior observed during the Great Hanshin earthquake (1995).

d) The cyclic water pressure which is comparable to the hydrodynamic pressure predicted by Westergaard theory, appeared to be affected by the softening of backfill soil due to liquefaction. The measured cyclic water pressure was greater than that predicted by Westergaard theory before onset of liquefaction whereas it was smaller after liquefaction

There are some evidences that shows the cyclic water pressure is affected by the mode of wall movement.

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