13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 702

FACTORS AFFECTING SHAKING INDUCED WATER PRESSURE BEHIND A QUAY WALL

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SUMMARY

Softening of saturated backfill soils behind quay walls is a direct effect of generation of excess pore water pressure caused by ground shaking. The movement of quay walls are also the consequence of the softening and liquefaction of backfill soils. On the other hand it seems that the movement of quay wall itself can affect the mechanism and also the amount of excess pore water pressure behind the wall. To study the mentioned problem two series of shaking table tests were conducted. In the first type of tests different intensities of input shaking were applied to the models and quay wall movement could occur as the consequence of the shaking. In the other type of tests the movement of model wall was controlled by a mechanical system. After applying the input shaking the wall was forced to move with different velocities to different maximum displacements. The response of water pressure both in term of hydrodynamic and excess pore pressure was measured behind the model walls. The results clearly show that the excess pore water pressure was affected by the velocity and also the maximum displacement of the model wall. The number of cycles generating the maximum pore water pressure increased when the velocity and the maximum displacement of the wall was increased. The mentioned effect was observed to be more pronounced when the input acceleration was smaller or the backfill soil was denser. Hydrodynamic water pressure also was observed to be affected by the wall movement. It is concluded that, considering the effect of liquefaction with no care about the effect of wall movement and also without considering the effect of liquefaction which is the case in most design codes, are both in two extreme sides of conservation.

INTRODUCTION

The design of quay walls is one of the most interesting and important concepts in geotechnical engineering. The relatively poor performance of this type of structures during some recent earthquakes has shown the need for better understanding of their behaviour (Towhata [1]). The great Hanshin-Awaji earthquake with M_s of 7.2 induced severe damages to several kilometres of the quay walls located in the Port Islands and Rokko Islands. The main damages were due to submergence and seaward movement of

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walls which destructed the structures and facilities established on them. The wall movement was accompanied by lateral displacements of the backfill soils causing cracks along the shorelines (Ishihara [2]). 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 fulfilment of the stability requirements. They consider a simple failure mechanism and then approximate the earth pressure using limit equilibrium or equivalent methods (Ghalandarzadeh [3]). Historically several experiences have been done on model quay walls using shaking tables and centrifuges. One of the most initial centrifuge tests carried out on quay walls are the tests conducted by Kutter [4]. Following these tests some other centrifuge tests were done such as model number 11 in the VELACS (Verification of Liquefaction Analysis using Centrifuge Studies) project in 1993. More recently some attempts have been done at Ports and Harbours Research Institute (PHRI) in Japan to model the behaviour of the quay walls during the 1995 Hanshin-Awaji earthquake. Most of the past researches have been focused on dry sandy backfill soils, and very little information is available regarding the dynamic earth pressure due to submerged backfill soils (Matsuzawa [5]). One of the recent researches conducted by Ghalandarzadeh [6] showed that displacement of the quay walls are mainly caused by the effects of dynamic earth pressure together with the seismic inertia force, although liquefaction increased the earth pressure and reduced the shear resistance of foundation sand. Also they noted that the pore pressure was not the main reason of the wall displacement, because the wall stopped its motion at the end of shaking and cyclic loading; however the pore pressure was still high.

In the current study factors effective in excess pore water pressure generation and dissipation behind quay walls and also their effect on the induced hydrodynamic pressure is studied. For this purpose some 1g shaking table tests were carried out on model quay walls and the hydrodynamic and hydrostatic pore pressures regarding the amount of wall movement and its velocity are investigated. Firuzkooh sand is used in these tests, which is quite similar to Toyoura sand and the below aims are followed:

1. Studying on the pore water pressure response.
2. Investigating the effect of wall movement parameters i.e. amount of movement, movement velocity and mode of movement on excess pore water pressure generation.

INSTRUMENTATION

The investigations described in this paper were conducted in a unique shaking table retaining wall test system which is shown schematically in figure 1. The table is designed to move in one direction. The entire system consist of five components: (1) shaking table and soil box, (2) wall driving system, (3) retaining wall, (4) transducers and (5) data acquisition system. The shaking table is made of two boards fixed together with three vertical steel plates. The excitation is induced manually by hand and the vertical plates make a uniform sinusoidal dynamic acceleration on the model. The soil box is 120cm long, 40cm wide and 50cm deep made of a 2cm thick transparent Plexiglas which is set over the shaking table. In order to isolate the saturation system so preventing any sand entering the water and CO_2 entrance paths, the bottom of the soil box is covered with a layer of No.100 screen mesh. In order to investigate the effects of wall movement a driving system was introduced so forcing the wall to move with different velocities and to different maximum displacements. The wall can undergo several types of movements: rotation about the base (overturning), rotation about the top and translation as a rigid body. The driving system works with four independent shafts which are spun with a link worked electrical motor. In order to prevent any vibrations translated to the model as well as the transducers due to the electromotor spin up, some anti-shock plastics were used.

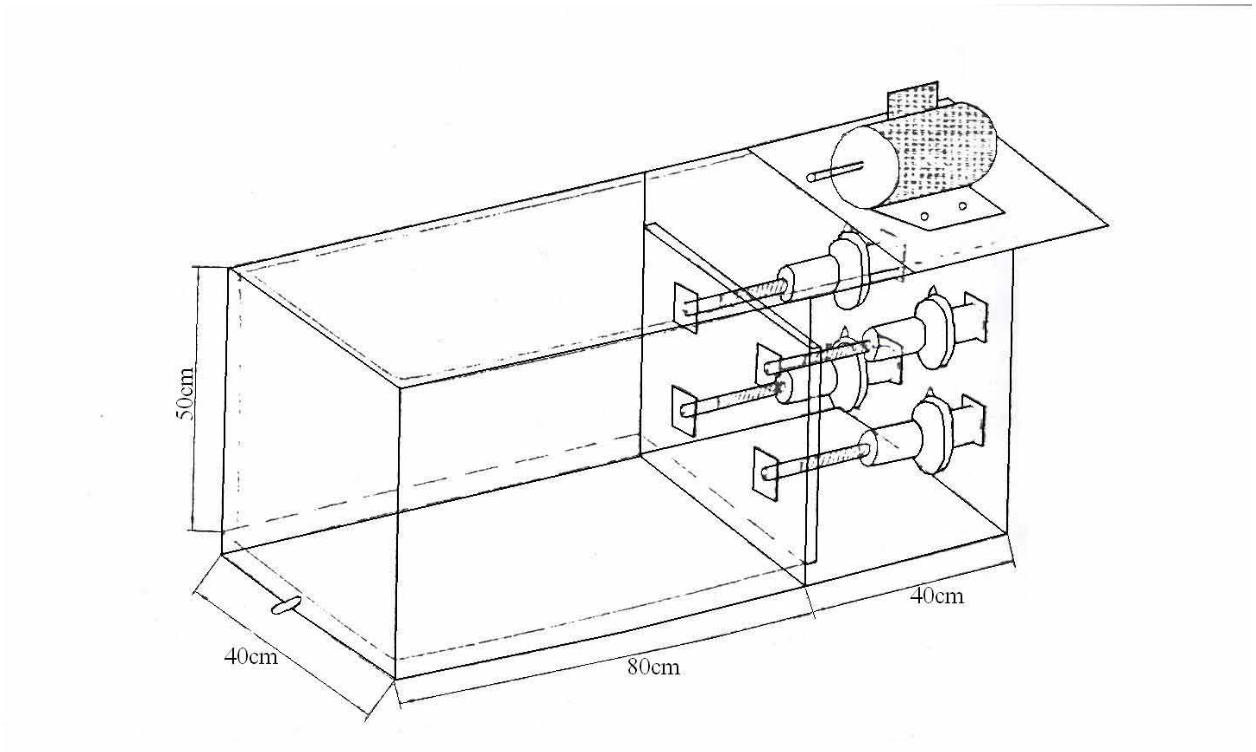


Figure 1: Schematic view of the shaking table and the driving system

As shown in figure 2, three types of transducers are used in this study to monitor the displacements, accelerations and pore pressures. The pore pressure transducers are fixed in place, but the acceleration transducers are free and can vibrate with the adjacent soil easily.

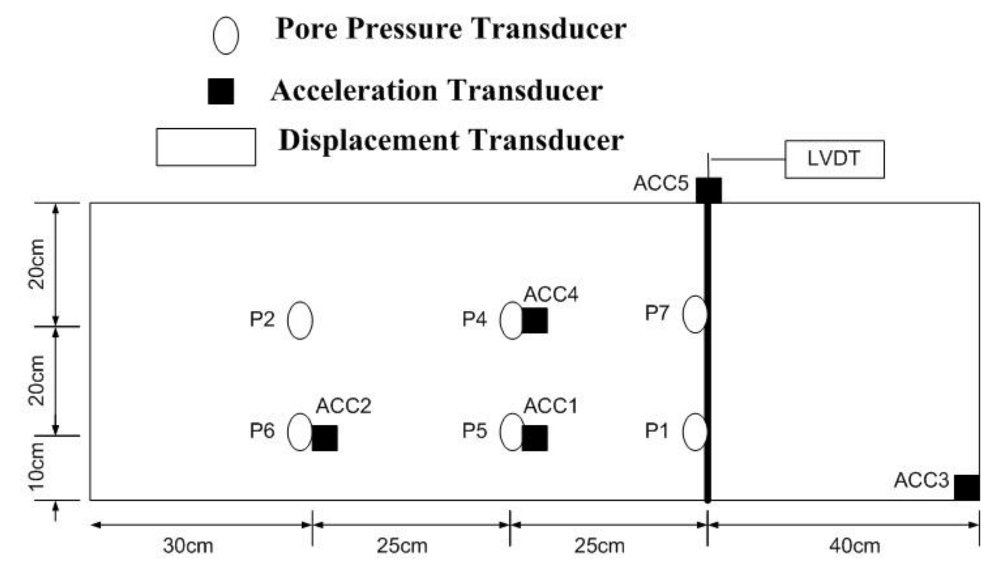


Figure 2: Schematic location of transducers in model tests

Due to the considerable amount of data that is generated from the shaking table experiments, a high-speed data acquisition system is used. The analog signal from the transducers is taken at predetermined sample rate of 250 samples per second for each channel, and then digitized by an analog-to-digital converter. The digital data are stored and processed by a personal computer.

SOIL PROPERTIES

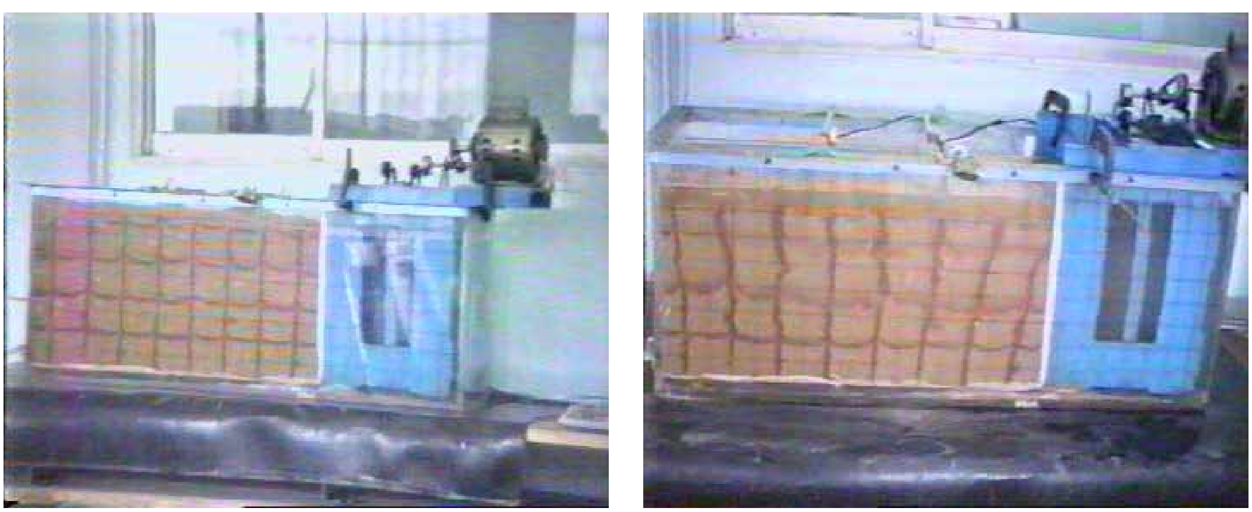
Oven-dry Firuzkooh silica sand was used throughout this study. The typical properties of the above sand are shown in table 1 which can be compared to those of Toyoura sand and Sengenyama sand. All tests during this study were carried out on the above sand with different average densities. The coefficient of permeability for the Firuzkooh sand obtained from constant head permeability experiments was 0.0125 cm/sec. Using the Hazen relation ($K=0.01D_{10}^2$) the coefficient of permeability is estimated as 0.019 cm/sec which agrees well with the constant head experimental values.

Table 1: Physical properties of Firuzkooh sand and two other well known sands

Type of sand	Gs	e_{max}	e_{min}	D_{50}(mm)	FC(%)	Cu	Cc
Firuzkooh	2.658	0.943	0.603	0.3	0	2.58	0.97
Toyoura	2.65	0.977	0.597	0.17	0	-	-
Sengenyama	2.72	0.911	0.55	0.27	2.3	-	-

MODEL PREPARATION

In figure 3 the model quay wall before and after shaking is shown. Making controlled loose to dense deposit is possible using the wet tamping method; hence the Firuzkooh sand was mixed with 5% water.



a) Before shaking **b) After shaking**

Figure 3: Model quay wall before and after shaking

Wet Firuzkooh sand was poured inside the container and carefully tamped to the target void ratio. To observe the overall deformation of the subsoil square grids of dyed sand were installed in the liquefiable sand. Since the grid consisted of the same material as the model subsoil, it easily moved together with the model without bending, floating, or subsidence. Then the soil models are percolated with carbon dioxide to help dissolve the air in the void space, in order to facilitate full saturation by water. The saturation process is performed by injecting water gradually from the bottom of the soil box. After preparing the model, it is shaken in a harmonic manner in the horizontal longitudinal direction with a variety of acceleration amplitudes. The base shaking frequency was 4.1 Hz and the model was shaken for 8 seconds in all tests. In the first series of tests the wall was free to easily move as the consequence of shaking. In the second series of the tests the wall movement was controlled by the driving system and the wall was forced to move with different velocities to different maximum displacements.

TEST RESULTS AND DISCUSSIONS

a) Free movement of wall

Figure 4 shows the test data obtained from the model tests where the wall was free to move during shaking.

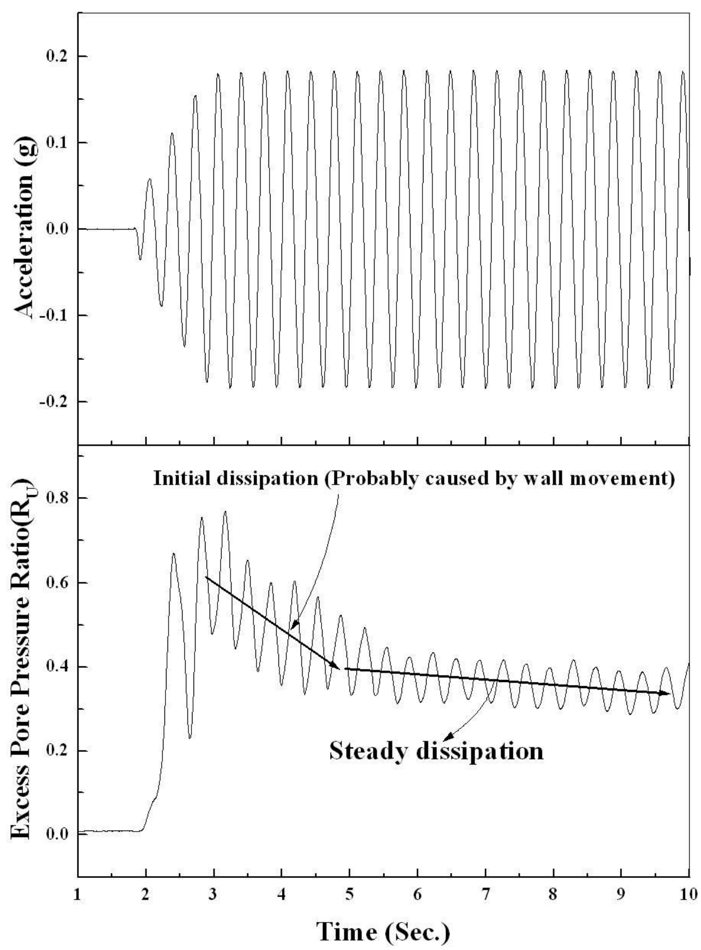


Figure 4: Behavior of excess pore pressure observed in model tests with free movement.

According to the above figure, the measured time 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 can be named as hydrostatic excess pore pressure and the other is the fluctuating part which will be referred to as cyclic water pressure. As it can be seen in figure 4, 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 coincide with the movement of the model quay wall whereas the slower reduction which is the steady dissipation, is probably due to drainage. Also the hydrostatic excess pore pressure was observed to be dependent on the velocity of the wall. When shaking starts and excess pore pressure reaches to its maximum value, the wall starts to move monotonically consequently. The recorded displacement of the model wall was used to calculate the monotonic velocity. As shown in figure 5, the rate of initial dissipation of excess pore pressure increased with increasing the maximum monotonic velocity of the model wall.

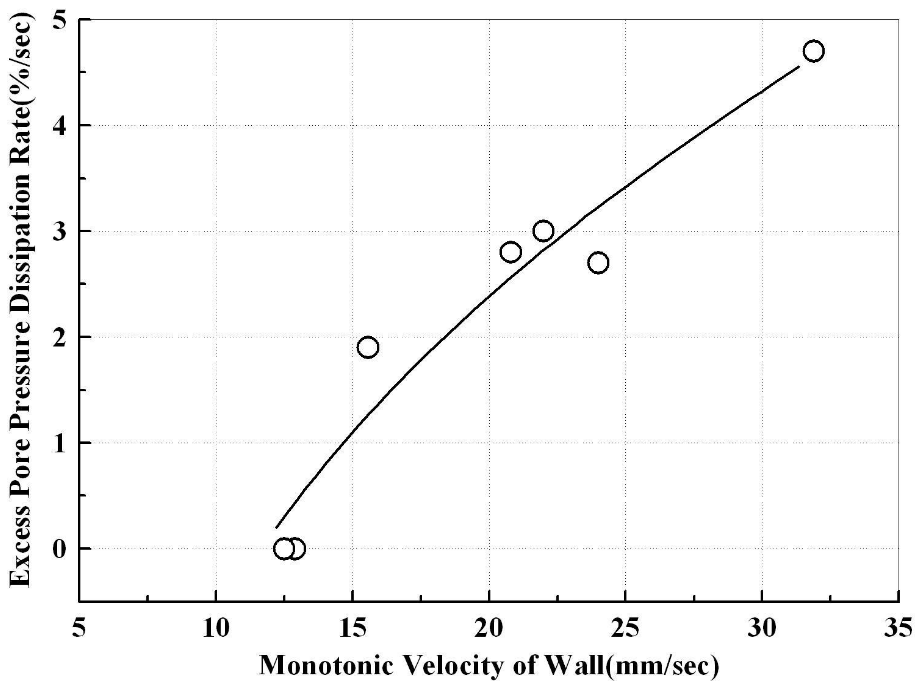


Figure 5: Effect of monotonic velocity of the wall on initial dissipation of hydrostatic excess pore pressure

It is interesting to notice that the higher base acceleration which caused faster monotonic movement of the wall (figure 6), 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. 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 7. 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.

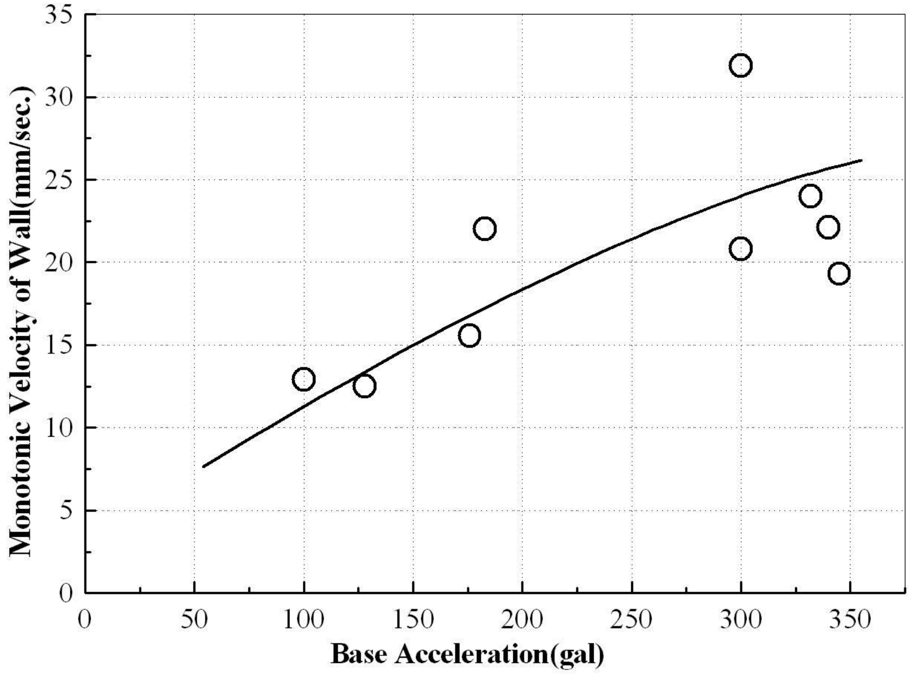


Figure 6: Effect of base acceleration on the initial monotonic velocity of model quay wall

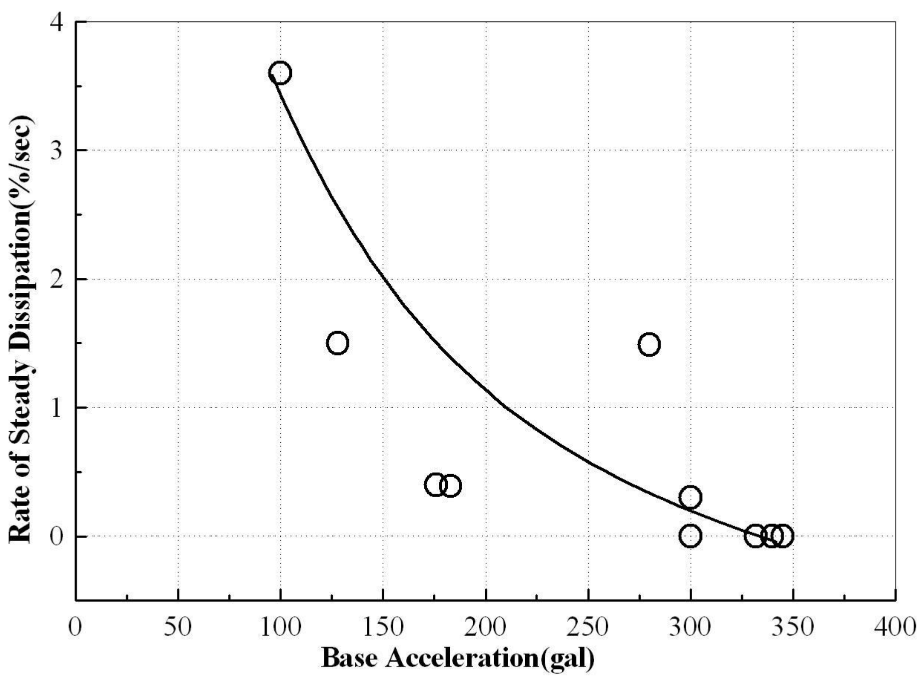


Figure 7: Effect of base acceleration on the steady dissipation rate of hydrostatic excess pore pressure

b) Forced movement of wall

In order to investigate the effects of wall movement more precisely, the wall is moved by a driving system. Figure 8 shows the effect of wall displacement on the excess pore pressure ratio (r_u) recorded at two depths. The velocity of the wall movement was 18.5 mm/sec. The wall movement has drastically reduced the excess pore pressure ratio, however after cessation of the wall movement it has increased.

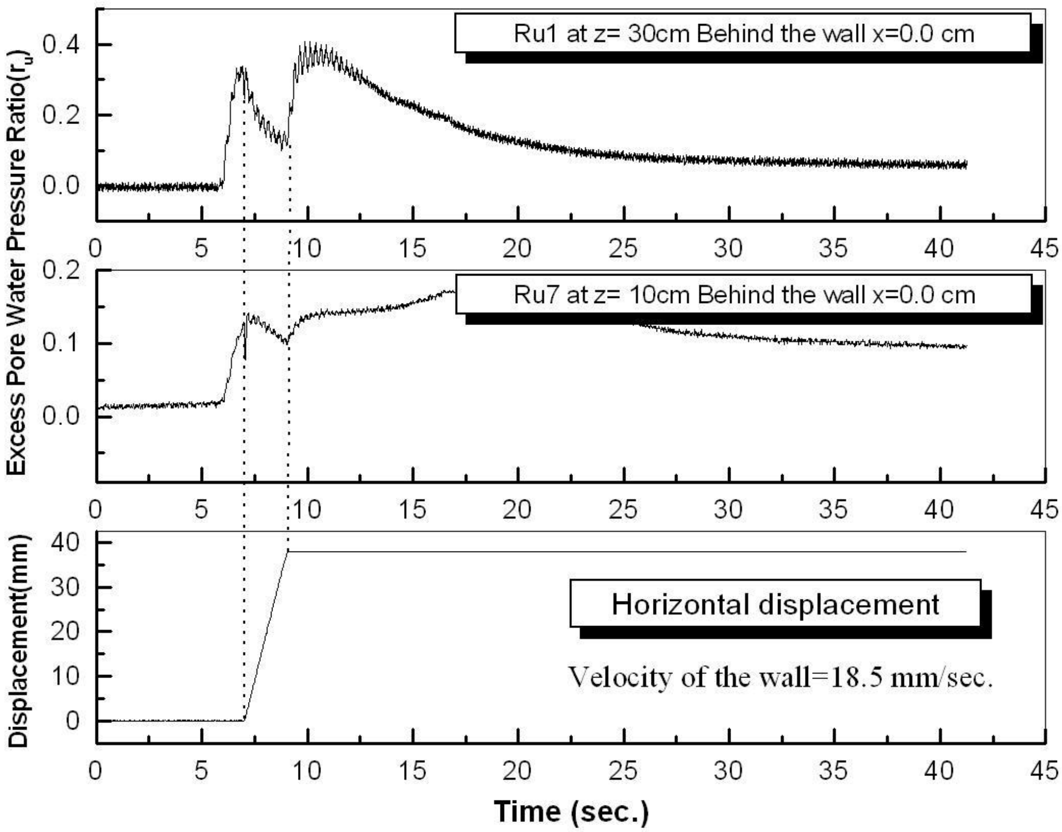


Figure 8: Effect of wall movement of excess pore pressure ratio

The effects of base acceleration, the amount of wall displacement as well as its velocity and the backfill void ratio are depicted in figure 9. As seen increasing the acceleration decreases the reduction in r_u regardless to the mode of wall movement. This is due to the draining nature of the r_u reduction which is similar to the behavior observed in the previous types of tests (free wall). More over this phenomenon is more significant in deeper elevations (P1) where the acceleration amplitude maybe larger. Figure 9 shows that wall displacement as well as its velocity has increased the amount of r_u reduction that is sand loses the chance to liquefy. Consequently the liquefaction potential of the backfill soil can be reduced if the quay wall monotonically moves more and faster during an earthquake. This can be an important concept in seismic designs of retaining structures. Nonetheless it can be masked by high base accelerations. Also looser backfill soil (larger void ratio), which means a soil with higher liquefaction potential, has decreased the reduction of r_u. So with lower base accelerations and a denser backfill the reduction of r_u can be increased.

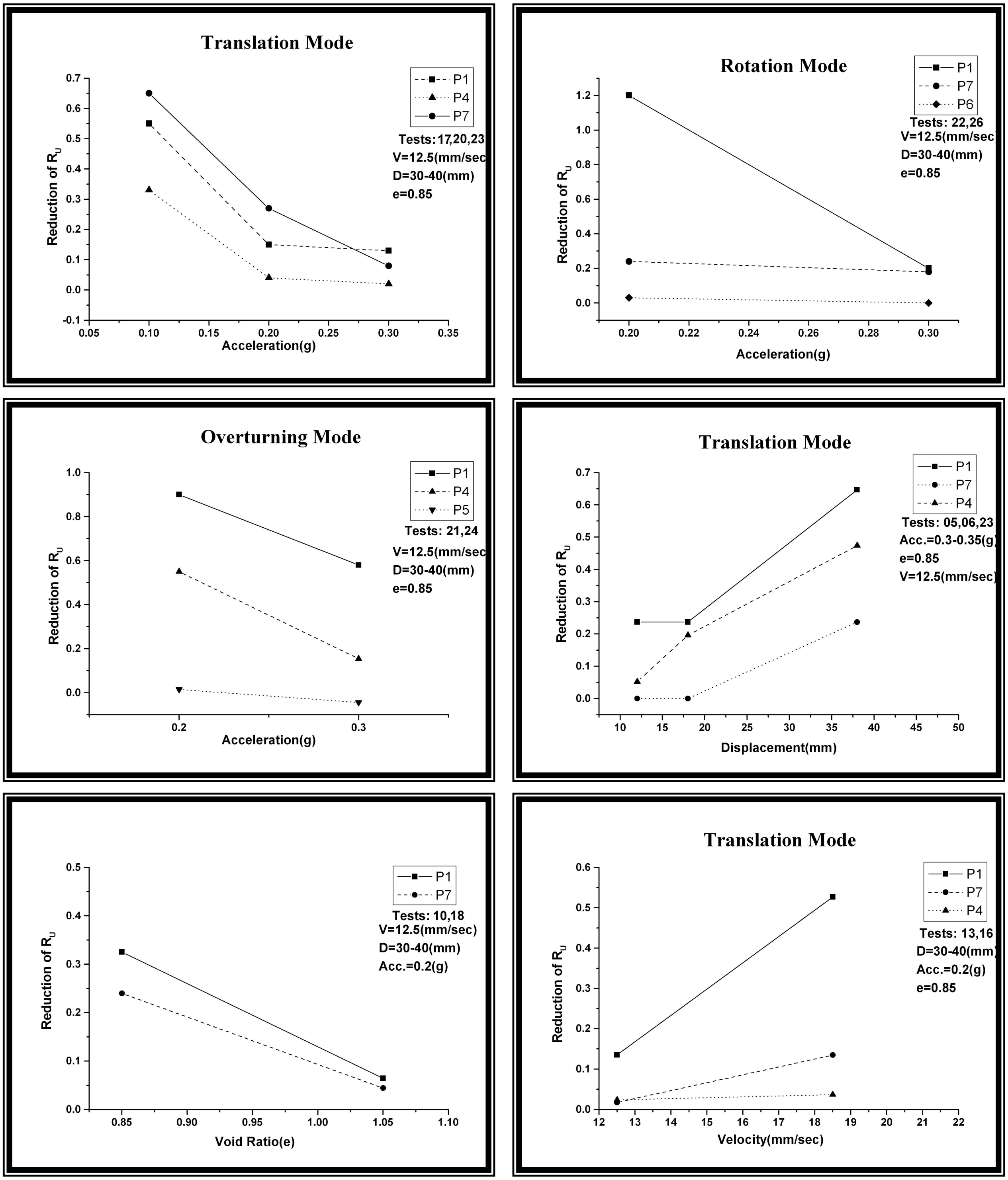


Figure 9: Factors affecting excess pore pressure ratio reduction

Number of cycles required to retrieve the reduced excess pore pressure due to wall movement is considered in figure 10.

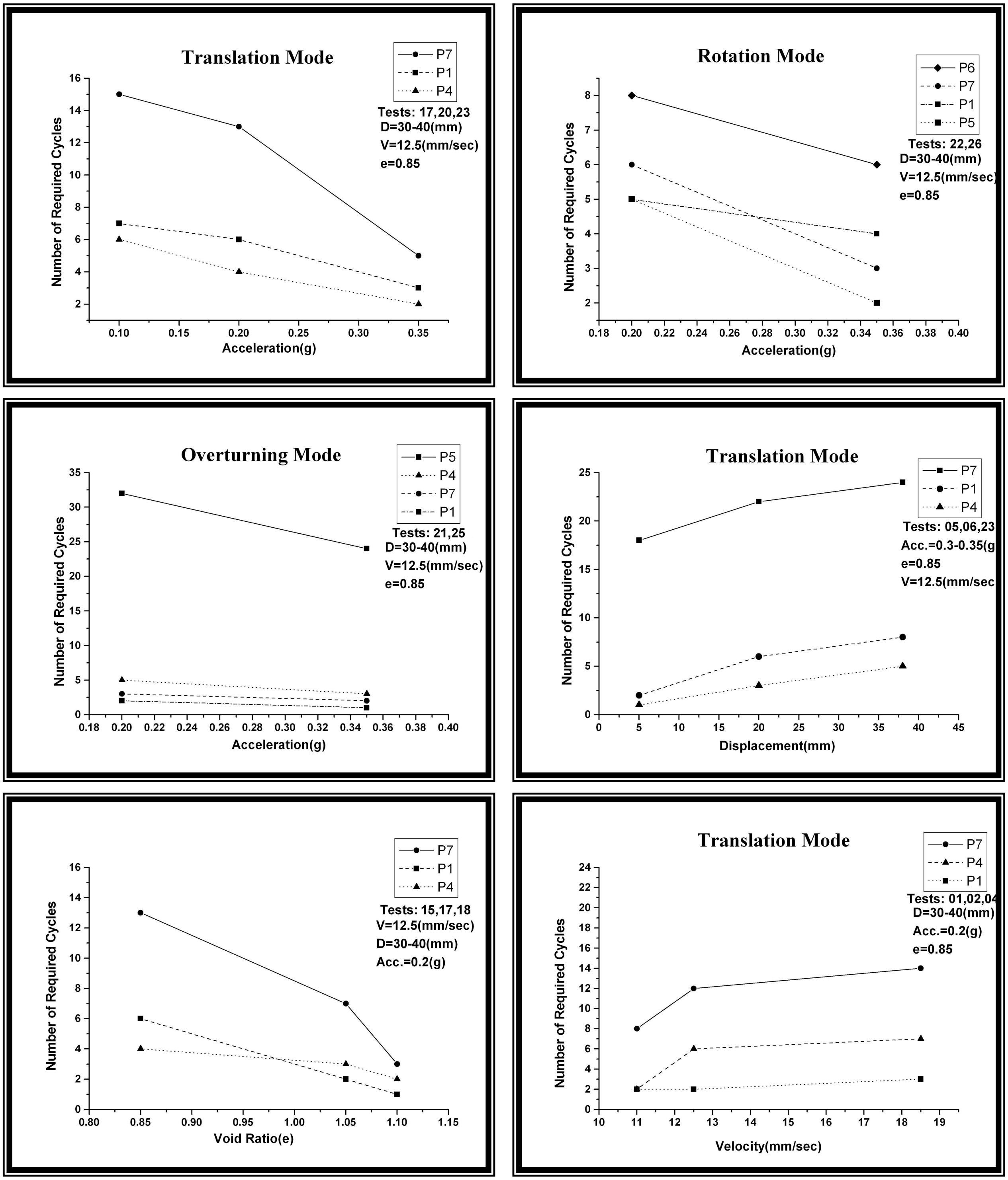


Figure 10: Factors affecting the number of cycles to reach the maximum pore pressure ratio after cessation of wall movement.

Regarding figure 10 increasing the base acceleration which induces higher excess pore pressures, as well as using looser backfill soils; will reduce the number of required cycles mentioned above. In other words higher base acceleration and looser backfill soil will hasten reaching the maximum pore pressure ratio. Also higher wall displacements and velocities will require more number of cycles to achieve the maximum excess pore pressure ratio. It can be concluded that relation 1 suggested by Lee [7] and De Alba [8] for excess pore pressure buildup is only applicable for fixed in place walls.

$$r_u = \frac{1}{2} + \frac{1}{\pi}\sin^{-1}\left[2\left(\frac{N}{N_L}\right)^{1/\alpha} - 1\right] \tag{1}$$

where r_u is the pore pressure ratio, N is the number of loading cycles, N_L is the number of cycles required to produce initial liquefaction (r_u=1.00) and α is a function of the soil properties and test conditions (Kramer [9]).

For movable walls another term considering the displacement and velocity of the wall should be introduced. This is done in the JOUN DELE software as below:

$$\Delta r_u \approx -3.273D^3 + 0.0248V^{0.5} - 0.0638Ln(V) \tag{2}$$

$$r_u = \frac{1}{2} + \frac{1}{\pi}\sin^{-1}\left[2\left(\frac{N}{N_L}\right)^{1/\alpha} - 1\right] + \Delta r_u \tag{3}$$

where Δr_u is the pore pressure ratio variation value, D is the amount of wall monotonic movement and V is its monotonic velocity. Relation 2 reveals that the amount of wall displacement reduces the excess pore pressure ratio more than the wall velocity. The following figure shows the normalized hydrodynamic pressure affected by the wall movement.

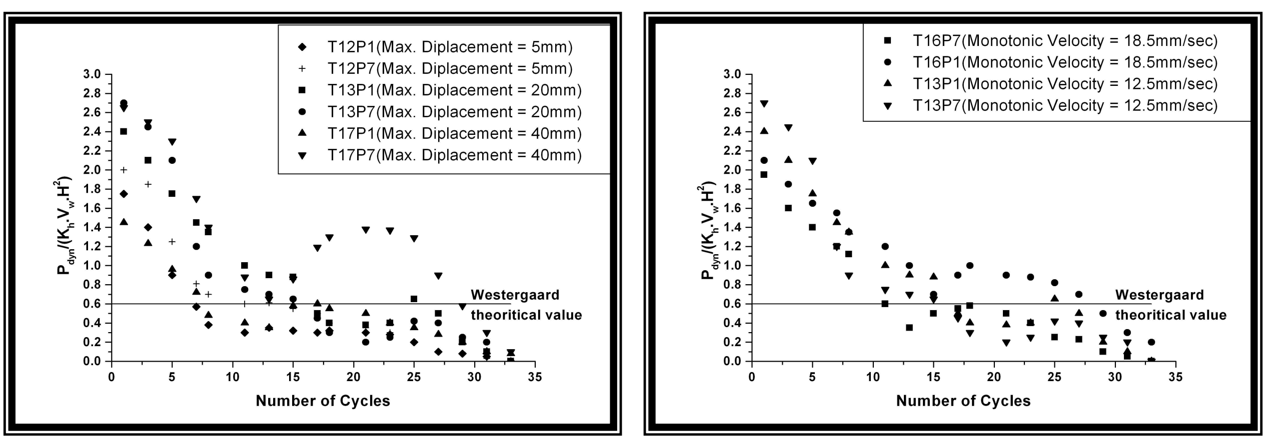


Figure 11: Normalized hydrodynamic pressure affected by wall movement and velocity.

It is obvious that in the tests which larger displacements were induced on the wall, the hydrodynamic pressure was lower. Moreover after the wall stops moving, the lost hydrodynamic pressure somehow restitutes; whereas there is an increase in the hydrodynamic pressure. However the effect of wall velocity on the hydrodynamic pressure is not obviously apparent. Also figure 11 reveals that during the initial cycles which liquefaction has not occurred the hydrodynamic pressure is more than the theoretical value estimated by Westergaard [10], but after some more cycles where liquefaction takes place the recorded hydrodynamic pressure gets lower than the Westergaard theoretical value.

It is worth notable that the above effects decrease by increasing the distance from the wall and are most significant in regions adjacent to the wall.

CONCLUSION

This study is mostly focused on the excess pore pressure generated behind the quay walls. The main gists which can be educed from the results are:

- High base accelerations and looser backfill soils can increase the excess pore pressure as well as decreasing the number of cycles required to reach maximum excess pore pressure ratio.
- Excess pore water pressure is drastically reduced by the quay wall net seaward displacement and its monotonic velocity. As a result they can increase the number of cycles required to reach maximum excess pore pressure ratio. These effects are more pronounced when the base motion acceleration is smaller or the backfill soil is denser.
- The hydrodynamic pressure exerted on the wall is mitigated by the amount of wall monotonic seaward movement. However the wall velocity is not observed to contribute obviously.
- Before liquefaction the cyclic water pressure is larger than the value estimated by the Westergaard formula, but after liquefaction it gets smaller than the Westergaard value.

Seismic designing of retaining structures without considering the backfill liquefaction which is the case in most design codes can be far from safety. Indeed including the effects of liquefaction should be done considering the wall movement. On the other hand omitting wall movement in liquefaction analyses may be a conservative design procedure.

REFERENCES

1. Towhata I, Ghalandarzadeh A, Prasad SK, Vargas-Monge W. "Dynamic failures observed in water front areas." Special Issue of Soils and Foundations 1996; 149-60.

2. Ishihara K, Yoshida K, Kato M. "Ground deformation characteristics caused by lateral spreading during the 1996 Hanshin-Awaji earthquake." Presented at 6th U.S.-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems, Tokyo, 1996.

3. Ghalandarzadeh A, Towhata I, Momeni SO. "Shaking induced water pressure acting on walls." Publications Committee of the XV ICSMGE, Editor. Proceedings of the 15th International Conference on Soil Mechanics and Geotechnical Engineering, Istanbul, Turkey. Lisse: Balkema, 2001; 3: 2229-32.

4. Kutter BL. "Centrifugal modeling of the response of clay embankments to earthquakes." PhD Thesis, Cambridge University, 1982.

5. Matsuzawa H, Ishibash I, Kawamura M. "Dynamic soil and water pressures of submerged soils." Journal of Geotechnical Engineering, ASCE 1985; 111(10): 1161-76.

6. Ghalandarzadeh A, Towhata I, Takashi O. "Shaking table tests on seismic behavior of quay walls subjected to backfill liquefaction." Geotechnical Earthquake Engineering and Soil Dynamics Conference ASCE, Seattle, May 1998.

7. Lee KL, Albaisa A. "Earthquake induced settlements in saturated sands." Journal of the Soil Mechanics and Foundations Division, ASCE 1974; 100(GT4).

8. De Alba P, Chan CK, Seed HB. "Determination of soil liquefaction characteristics by large-scale laboratory tests." Report EERC 75-14, Earthquake Engineering Research Center, University of California, Berkeley, 1975.

9. Kramer SL. "Geotechnical earthquake engineering." Prentice Hall, Inc., Upper Saddle River, New Jersey, 1996: 376-7.

10. Westergaard HM. "Water pressure on dams during earthquakes." Transactions of ASCE, Paper no. 1835, 1931: 418-33.