MODIFICATION OF THE SLIDING BLOCK CONCEPT FOR CAISSON WALLS

Masoud MOHAJERI1, Koji ICHII2, Tamotsu TAMURA3

1 Researcher, Port and Airport Research Institute, Japan, mohajeri@pari.go.jp
2 Researcher, Port and Airport Research Institute, Japan, ichii@pari.go.jp
3 Manager of R & D, Penta Ocean Construction, Japan, Tamotsu.Tamura@mail.penta-ocean.co.jp

ABSTRACT: Two series of shake table tests were performed in order to study the sliding displacement of caisson type quay walls and breakwaters due to earthquake motions. Model caissons were subjected to different input accelerations and the resultant sliding displacements were measured. Test results show that considering single yield acceleration during the displacement analysis may lead to erroneous results. The comparison of the test and analysis results suggests the use of two-level yield accelerations as static and dynamic levels. The outcomes of the test results also reveal the drastic effect of rocking on triggering the sliding.

Key Words: Shake table tests, Sliding block concept, Caisson wall, Breakwater, Yield acceleration, Rocking, Rotational natural frequency

INTRODUCTION

Simple straightforward methods have been developed for evaluating the permanent displacements of gravity walls during earthquakes (e.g., Newmark, 1965; Franklin and Chang, 1977; Richards and Elms, 1979; Whitman and Liao, 1985; Fujiwara, 2000). These methods are based on the sliding block model, adopting similar assumptions to those made for the pseudo-static analysis. In this method the threshold acceleration for a caisson wall on a rubble mound is determined by the value resulting in a factor of safety of unity for sliding of the soil-structure-water system. When the ground motion acceleration exceeds the threshold acceleration, the soil-structure-water system begins to move by translation along the base of the wall and the failure plane through the backfill. By two times integrating the area of the acceleration time history that exceeds the threshold acceleration and continuing the time integration until the sliding stops, the displacement of the wall relative to the firm base below the failure plane can be determined.

However, Iai (1998) and Okamura (2002) showed separately that actual seismic performance of gravity type walls during earthquakes often does not meet the assumptions inherent in sliding block analysis. Steedman and Zeng (1996) proved that the conventional design procedure might not be in conservative side where the foundation is firm but rocking type response of the wall is involved.

This paper reports the outcomes of two series of shake table tests as well as the results of the analytical study, which were aimed to highlight different aspects of the conventional sliding block concept, and it concludes with a series of practical suggestions for modification of the conventional method.

TEST METHOD

Two series of shake table tests were performed to provide the input data for the current research. The circular shake table in Penta Ocean Construction Institute of Technology (POINT) and Port and Airport Research Institute in Japan (PARI) was used separately to perform the first and second series of the experiments. The diameter of both tables is about 6 m and they are located in a pool, which can be filled with water to depth of 1.8 m.

In the first series of the experiments, a gravity type caisson wall was modeled and mounted over a very stiff layer of sand mound. A horizontal tension force of 200 Kgf was applied to the caisson in order to model the backfill and hydrodynamic pressure on the model. The size of the model caisson walls and the position of the measuring transducers are shown in Figure 1.

Figure 1. Model caisson walls in the first series of tests

Table 1. List of the first series of shake table tests

<table>
<thead>
<tr>
<th>CASE</th>
<th>Max. Acc.</th>
<th>Tension Force from Base</th>
<th>Tension</th>
</tr>
</thead>
<tbody>
<tr>
<td>1−1</td>
<td>100 Gal</td>
<td>10 cm</td>
<td>200 kgf</td>
</tr>
<tr>
<td>1−2</td>
<td>125 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1−3</td>
<td>150 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1−4</td>
<td>175 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1−5</td>
<td>200 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1−6</td>
<td>225 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1−7</td>
<td>250 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1−8</td>
<td>275 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2−1</td>
<td>150 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2−2</td>
<td>175 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2−3</td>
<td>200 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2−4</td>
<td>225 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2−5</td>
<td>250 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2−6</td>
<td>275 Gal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2−7</td>
<td>300 Gal</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Twenty cycles of sinusoidal waves were considered as the input acceleration and different amplitudes were tried during different cases. Table 1 shows the list of the tests, which were performed in the first part of the study.

In the second series of the experiments, three caisson type composite breakwaters were modeled and mounted over a layer of rubble mound. The size of the model caissons and the position of the measuring transducers are shown in Figure 2.

The first series of shake table tests on model caisson walls were performed without water and with three different water levels. Three earthquake records observed in Hachinohe, Ofunato and Kobe ports were used as input motion for each case. A sweep test was also performed to provide an idea about the resonance frequency of the system in dry condition. Shaking for each case was started with 100 gal maximum acceleration and repeated with higher peak accelerations of 200, 300, 400, 500 and 600 gal until a clear horizontal deformation was observed in the outputs.

SHAKE TABLE TEST RESULTS AND ANALYSIS OUTCOMES

The first series of shake table tests on model caisson walls

A relatively ideal condition was planned for the first series of the tests and harmonic sinusoidal acceleration with constant frequency of about 13 Hz was selected as an input motion. Figure 3 shows typical test results, including time history of the input acceleration, horizontal base displacement and tilting angle of the model caisson wall.

Based on the conventional sliding block concept, the critical earthquake coefficient (k<sub>c</sub>) and the corresponding yield acceleration (A<sub>y</sub>) were calculated. As shown in Figure 4, considering a resultant backfill and hydrodynamic force, may increase the contact pressure between the wall base and the mound. Consequently, the horizontal component of friction force may increase, which will lead to a rise in k<sub>c</sub> and A<sub>y</sub> values.

Figure 4. Conventional yield accelerations for the caisson walls with and without tension force

Confirming the above-mentioned discussion, the test results showed that shifting the location of the tension force to a higher level gives rise to a larger moments and smaller residual sliding displacements (Fig. 5).

Figure 5. Comparison of the residual displacements for cases with different tension force locations

On the other hand, yield acceleration was back calculated from the test results. Different yield accelerations were tried and the residual sliding displacement was calculated by means of two times integration. This trial and error procedure were continued to reach the sliding displacement obtained from the tests. Figure 6 shows "Input acceleration-Back calculated yield acceleration" for 8 tests of case-1. It seems that the back calculated yield accelerations increase with the peak input acceleration values, which is in contrast with the conventional sliding block concept.

Figure 6. Back-calculated yield accelerations- Case 1
As shown in Figure 7, the starting moment of sliding for each cycle of loading might clearly be distinguished. Therefore, it was attempted to find the threshold acceleration for each cycle of loading that coincides the beginning of sliding. These accelerations could be considered as threshold accelerations for each cycle of loading.

Figure 7. The starting moment of sliding for each cycle of loading and the coinciding threshold acceleration

Figure 8 illustrates these results and implies that as the peak sinusoidal input acceleration increases, the threshold acceleration value rises accordingly. However, “peak acceleration-threshold acceleration ratio” for seven cases was found to be in the range of about 0.7-0.9. The back calculated yield accelerations for all the cases showed smaller values than the threshold accelerations. They were about 50% of the threshold accelerations and about 40% of the peak acceleration values.

Figure 8. Comparison of the back-calculated yield accelerations and the threshold accelerations for seven shake table tests

These results indicate that the yield acceleration at the time that sliding tends to start and during the sliding is not a constant value and it seems that the yield acceleration decreases immediately after sliding. Therefore, two levels of yield accelerations were considered so as to find an analytical procedure. First level is called static yield acceleration, which is equal to the threshold acceleration at the initial point of the sliding. Then, immediately after the start of the sliding, a jump in the yield acceleration is assumed. Therefore, a dynamic yield acceleration as the secondary yield level is considered. This procedure was tried by considering the threshold acceleration as static level of yield acceleration. The dynamic level was found by trial and error in a way that the analyses result the same displacement value, which was obtained from the tests.

The results of these analyses are presented in Figure 8, as well. It can be seen that the dynamic yield acceleration is always smaller than the back-calculated yield acceleration. It is also smaller than the static yield acceleration (threshold acceleration) in a range between 25% and 50%.

The second series of shake table tests on model breakwaters

The second series of the experiments were planned to provide more realistic test conditions. Therefore, the acceleration records of the earthquakes observed in Kobe, Hachinohe and Ofunato ports were used as input motion. Rubble mound was placed and different water levels were tried.

Start of sliding

As shown in Figure 9, test results indicate that it is almost impossible to distinguish a specifically clear moment as a starting point of sliding. Therefore, a clear point as yield acceleration could not be recognized. These results showed that the initial shake caused vertical movements. As a result, differential vertical displacements and rocking motions of the caisson over the mound were triggered as horizontal fluctuations. Following to the initial fluctuations, the first major horizontal sliding displacement occurred. Comparison of the horizontal displacement and acceleration time histories showed that the first major sliding took place at the time that the larger acceleration records have already been experienced without any major sliding (peak-1 & 2 in Fig. 9A) and it only started when the rocking action of the caisson started.

Figure 9. Base displacement and horizontal acceleration time history for typical results of second shake table test series
Rotational resonance effect on sliding

The tilting angle, \( \theta \), could be derived from both vertical and horizontal displacement measurements. Comparisons showed that the peak horizontal displacements and peak rocking angles happened almost at the same time (Fig. 10).

![Figure 10. Comparison of the horizontal displacement and rocking angle peaks](image)

Therefore, the possibility of rotational resonance of the caisson was considered. In other words, it was thought that when the frequency of the input motion reaches to the rotational natural frequency of the caisson wall, the rocking behavior starts and consequently triggers the horizontal displacement.

Using a pseudoelastic approach (Zeng, 1998), the fundamental frequencies of the horizontal and rotational modes are given by

\[
 f_H = \frac{1}{2\pi} \sqrt{\frac{K_H}{M}} \tag{1}
\]

\[
 f_I = \frac{1}{2\pi} \sqrt{\frac{K_I}{I}} \tag{2}
\]

in which \( K_H \): stiffness of the foundation in horizontal direction; \( K_I \): stiffness of the foundation in rotation; \( M \): mass of the structure and \( I \): mass moment of inertia about the rotation center. Estimation for \( K_H \) and \( K_I \) is given by Wolf (1988) as

\[
 K_H = \frac{G B}{2(1-\mu)} \left[ 6.8 \left( \frac{L}{B} \right)^{0.65} + 0.24 \right] \tag{3}
\]

\[
 K_I = \frac{G B^2}{8(1-\mu)} \left[ 3.2L + 0.8B \right] \tag{4}
\]

in which \( B \): width of the foundation; \( L \): length of the foundation; \( G \): shear modulus of soil; and \( \mu \): Poisson ratio.

Since the exact values of shear wave velocity or shear modulus of the mound material were not available, the ratio of the rotational and sliding natural frequencies (\( \frac{f_I}{f_H} \)) was considered and it was calculated to be about 0.4.

On the other hand, sweep test results were used to find the horizontal resonance frequency of the system. The amplitude of the input motion in sweep test was almost constant and equal to about 100 gal. When the frequency reached to about 10-11 Hz, the resonance started. The acceleration records on top of the caisson started to increase up to level of 500 gal. At this moment, both sliding and rotational components started to show larger amplitudes, which confirmed the resonance (Fig. 11).

![Figure 11. Sweep test results and natural frequency of the caisson-mound system](image)

The results show that the sliding and rotational resonance started at the same time and the “rotational natural frequency-sliding natural frequency” ratio for this case could be considered as unity. The sweep test result was different from the ratio calculated from Eqs. (1) and (2) and it was decided to consider the experimental ratio.

Analytical study on the test results and modification of the conventional sliding block concept

The results suggest that a combination of at least two parameters may determine the starting time of the horizontal deformation. When the frequency of the table reaches to rotational resonance frequency of the system and at the same time the acceleration passes a certain limit, the horizontal deformation starts. This limit or yield acceleration is different from the conventional sliding block definition.

Therefore, to apply a correction factor (\( \alpha \)) on the conventional yield acceleration seems to be necessary, and this will lead to a new threshold acceleration, which will be referred as static yield acceleration.

On the other hand, one of the key parameters in estimation of the residual horizontal displacement is the friction force between caisson wall and rubble mound. In the conventional approach, the friction angle between the caisson and the gravel fill is selected and then a single value of frictional resistant force is derived.

However, the results showed that the idea of single-value yield acceleration could not be a proper assumption.
The dynamic yield acceleration should be smaller than the static yield acceleration, and might be evaluated by application of a correction factor ($\beta$) on the static yield acceleration.

According to this simplification, sliding will not happen until acceleration reaches the static yield acceleration and immediately after the sliding starts, the limit drops to a lower value of dynamic yield.

The conventional critical earthquake coefficient and yield accelerations were calculated for the second series of shake table tests with no water level, water level over mound, water level until half of the caisson and water level until top of the caisson (Table 2). The forces were calculated by means of the equations proposed by Westergaard (1933).

Table 2. The critical earthquake coefficient and yield accelerations for the second series of shake table tests

<table>
<thead>
<tr>
<th>Water Level</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_c$</td>
<td>0.6</td>
<td>0.6</td>
<td>0.37</td>
<td>0.13</td>
</tr>
<tr>
<td>Yield Acceleration (gal)</td>
<td>588</td>
<td>588</td>
<td>365</td>
<td>130</td>
</tr>
</tbody>
</table>

Two levels of yield accelerations i.e. static and dynamic yield accelerations were considered. Since in the second series of the shake table tests, there was an equal possibility for sliding toward both sides, the yield accelerations were considered in both directions. Several analyses were performed by means of a Fortran-77 code and different levels of static and dynamic yield accelerations were tried.

During the analyses procedure, if the ground motion acceleration exceeded the threshold acceleration in either side, the caisson began to move by translation along its base. The threshold acceleration level immediately jumped to dynamic level, once the sliding started. By two times integrating the area of the acceleration time history that exceeded the threshold acceleration until the end of the sliding, the relative displacement of the caisson and base could be determined analytically.

Figure 12 illustrates a typical sliding displacement time history, calculated by the modified sliding block analysis and compare it with the measured records during the shake table test. As shown in Table 2, the conventional yield acceleration for the case with water surface on top of the caisson (WL-4) is about 130 gal. In this analysis the $\alpha$ and $\beta$ correction factors were 0.7 and 0.9 that resulted the static and dynamic yield accelerations of about 90 and 80 gal, respectively.

Although the analysis outputs do not exactly fit the test results, but it seems that they can predict the experimental phenomenon and provide the residual sliding displacement to some extent.

This type of analysis was repeated for all the test cases and the different correction factors were tried by trial and error to investigate a range of values for these factors. The analyses resulted in the reported correction factors ($\alpha=0.7$ and $\beta=0.9$).

Figure 13 shows the induced inertia force, which was calculated by multiplying the mass of the central caisson (400 kg) to the measured acceleration records in the caisson.
Then, the friction force or the sliding yield force between the base of the caisson and the rubble mound was calculated by multiplying the friction factor \( \alpha = 0.6 \) to the weight of the caisson. Figure 13 confirms that analysis with the conventional concept could lead to zero displacements. However, test results showed 15 mm and 7.5 mm residual displacements for the cases with Hachinohe and Ofunato records, respectively.

Figure 14 shows the conventional and modified analysis results and compares them with the test measurements. In these analyses the correction factors of \( \alpha = 0.7 \) and \( \beta = 0.9 \) were also applied.

**CONCLUSIONS**

Based on the test and analysis results it could be concluded that:

1) As the peak sinusoidal input acceleration increased, the threshold acceleration value rose accordingly. However, “peak acceleration-threshold acceleration” ratio for seven cases was found to be in a relatively close range.

2) The yield acceleration at the time that sliding tended to start and during the sliding was not a constant value and it seemed that the yield acceleration decreased immediately after sliding. Therefore, two levels of yield accelerations called static and dynamic yield acceleration were considered.

3) A combination of at least two parameters could determine the starting time of the horizontal deformation. These parameters were frequency of input motion and amplitude of input acceleration.

4) The application of a correction factor \( \alpha \) on the conventional yield acceleration to find the static yield acceleration and a correction factor \( \beta \) on the static yield acceleration to find the dynamic yield acceleration was introduced.

5) Almost all the modified analysis results gave closer estimation of the measured residual displacements and the conventional analysis resulted to almost zero residual displacements confirming that the idea of single-value yield acceleration could lead to erroneous results.

**ACKNOWLEDGMENT**

The authors are indebted to Mr. Yoshinori NAKAYAMA, Mr. Hireki EDA and Mr. Tsuyoshi TANAKA for their great assistance during the experimental work in PARI. Mr. Takamitsu OSHIMA and Mr. Makoto YOSHIDA performed the experiments in POINT. Their contribution in the research work is highly appreciated. The help of Mr. Yosuke KAWAMATA during the experimental work in POINT is also acknowledged.

**REFERENCES**


