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Comparative study on the seismic displacements of soil retaining walls situated on slope

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ABSTRACT: Seismic stability of a conventional leaning soil retaing wall and a reinforced soil retaining wall both situated on slope was investigated using shaking table tests and displacement analysis. In modeled environment the ductile behavior of the reinforced soil retaining wall was observed, while the ground was excited by strongly accelerating input. In contrast, brittle failure of the leaning retaining wall under medium level input ground acceleration was observed. The results of displacement analysis showed that residual friction angle of backfilled sand is responsible for the large displacement of leaning wall under medium level of ground acceleration. On the other hand, peak (or slightly deteriorated) friction angle of backfilled sand is responsible for the coherent behavior of the reinforced soil wall, even under strong input ground acceleration.

1 INTRODUCTION

A large number of leaning-type retaining walls (LRW) for the highway embankments on hillside were severely damaged during the 1999 Chi-Chi Earthquake in Taiwan (See Figures 1(a) and 1(b), Huang 2000). Traffic were paralyzed because of large differential settlement and intensive cracking along the highway system induced by the heavy movement of the LRWs.

To investigate the seismic stability of reinforced retaining wall situated on slope, we used the result of shaking table tests reported by Huang et al. (2000) and a new psuedo-static displacement calculation method.

Figure 1(a). Soil retaining wall and highway embankment damaged during the Chi-Chi earthquake in Taiwan.

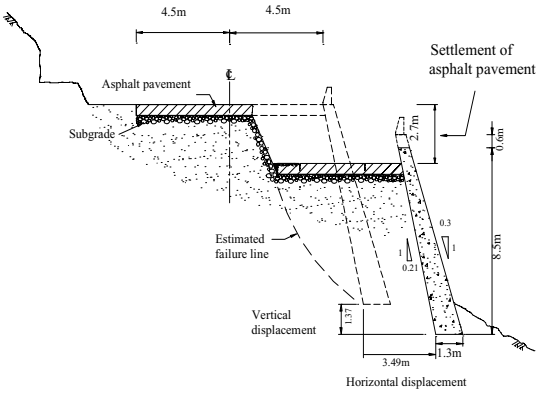

Figure 1(b). Cross section of the damaged soil retaing wall situated on slope.

2 SHAKING TABLE TESTS ON THE RETAINING WALL

The shaking tests on model retaing walls were performed using a shaking table at the Railway Technical Research Institute in Japan. A plane strain steel-framed sand box with length of 2600 mm, width of 600 mm, and height of 1400 mm was rigidly mounted on a displacement-controlled shaking table. For the role of subsoil medium and backfill, Toyoura send was used, which is a uniform sand having sub-rounded particles, with e_{max}= 0.977, e_{min}= 0.605, G_s= 2.64, D_{10}= 0.11 mm, D_{50}= 0.23 mm. Air-dried sand was discharged from a hopper located 0.8 m high above the sand surface at a constant rate to achieve a relative density of 90 % (dry unit weight of 15.6kN/m^3). A surcharge of 1 kPa was applied on the crest of the backfill using lead shots. To observe failure planes formed in the backfill, 2 mm-thick colored, horizontal sand layers were placed with a vertical spacing of 50 mm in the subsoil and the backfill, and a 50 mm-thick transparent glass plate was used to form the front-side of the sand box. The backside of the sand box was a steel plate, lubricated by attaching a 0.2 mm-thick Teflon sheet, see Huang et al. (2000) for the details of test set-up and results.

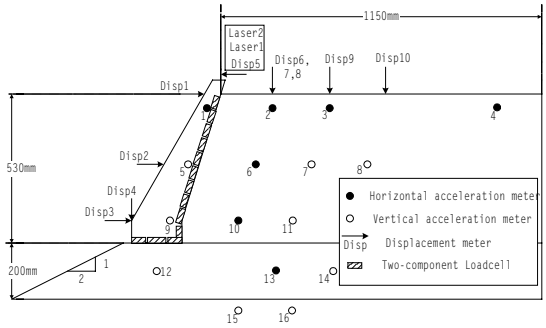

Figure 2a. Model test set-up for leaning-type soil retaing wall situated on slope (after Huang et al., 2000)

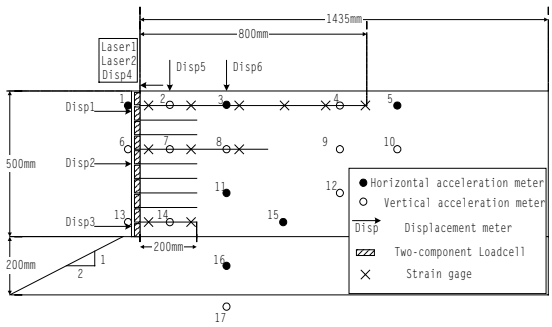

Figure 2b. Model test set-up for reinforced soil retaining wall situated on slope (after Huang et al., 2000)

Three types of retaining wall (RW) models were tested: leaning type RW (LRW, Figure 2a), grid-reinforced soil RW with a full-height rigid facing (RWR; Tatsuoka et al. 1998, see Figure 2b), and leaning type RW with reinforced backfill and foundation using large-diameter soil nails (R-LRW). The present study focuses on the former two types of walls (i.e., LRW and RWR). The NS component of horizontal ground acceleration recorded at the Kobe Ocean Meteorological Observatory during the 1995 Hyogoken-Nanbu Earthquake in Japan was used as the input base motion. The original wave was modified for having a dominant frequency of 5 Hz to match the geometry of the 1/10 scaled-down wall. In each test, the maximum acceleration at the shaking table, a_{max}, was increased with an increment of about 100 gals between two successive stages of shaking, until the model wall became severely displaced or tilted.

Figures 3(a) and 3(b) show the time histories of the base acceleration (a_{base}) and the outward displacement at the wall top ($_h$) of LRW and RWR. In these figures, the durations between two successive stages of shaking have been reduced for clarity. At the 4th stage of shaking (a_{max} 550 gals for LRW), $_h$ increased rapidly in association with the shear bands in the backfill. At the 6th stage (a_{max} 750 gals), lateral displacement of the wall became very large (152 mm or 0.304H), in association with the distinct shallow shear bands in backfill and subsoil (Figure 4a). Figures 3(b) and 4(b) show the results of shaking tests on RWR. A clear shear band developed in the backfill, passing through the reinforced zone and the slope at the 6th stage (a_{max} 750 gals). $_h$ attained 46 mm (0.092H) at the 8th stage (a_{max} 900 gals) without showing any sign of catastrophic failure. This behaviour indicates the so-called 'ductile' or 'coherent' dynamic behavior of the geosynthetically reinforced soil retaing wall.

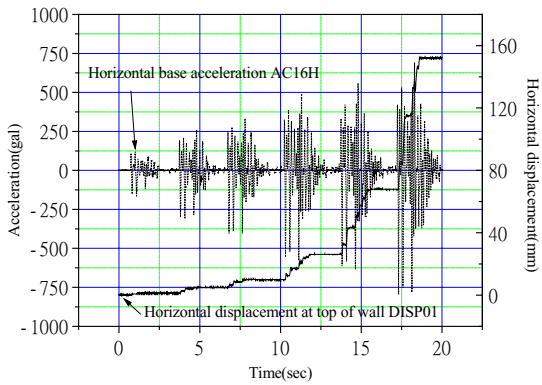

Figure 3(a). Time histories of input base acceleration and horizontal displacement of leaning-type wall (after Huang et al, 2000)

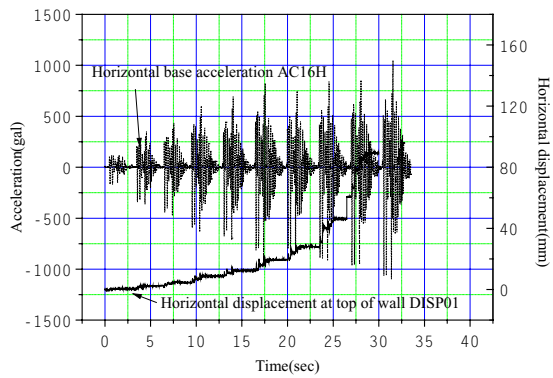

Figure 3(b). Time histories of input base acceleration and horizontal displacement of reinforced soil wall (after Huang et al., 2000)

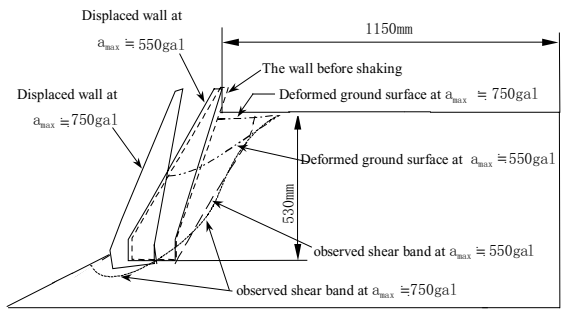

Figure 4(a) Displacements and shear bands observed in the shaking test of LRW.

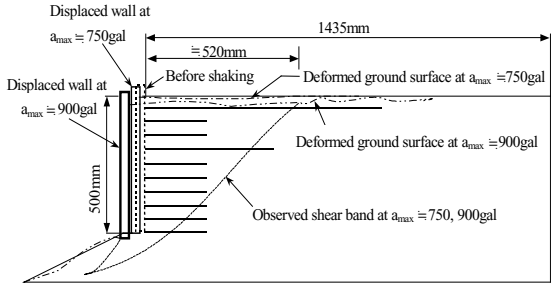

Figure 4(b) Displacements and shear bands observed in the shaking test of RWR.

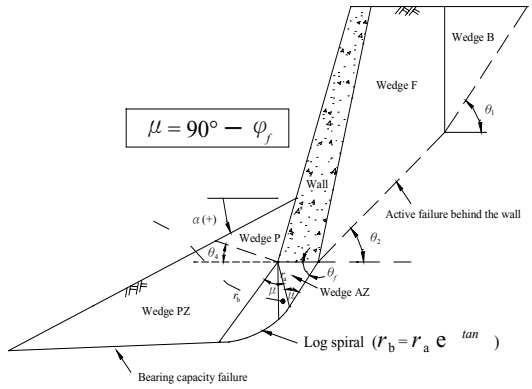

Figure 5 Observed failure mechanisms of soil retaing wall situated on slope.

3 PSEUDO-STATIC ANALYSIS

The experimental evidence shown above suggests that three failure mechanisms are responsible for the displacement of the soil retaining wall situated on slope. They are (see Figure 5):

1. Sliding failure along the wall base. The failure lines consist of active failure lines behind the wall and a shear line along the wall base.
2. Insufficient bearing capacity results in a triangular active wedge, a transitional zone characterized by a logarithmic spiral, and a passive wedge. This failure mechanism is similar to that used by Meyerhof (1963), Graham et al. (1988), and Huang and Tatsuoka (1994).
3. Tilting of the wall caused by local yielding of foundation soil. This mode of displacement was observed by Tatsuoka et al. (1997) in the 1995 Hyogo-Ken Nanbu earthquake in Japan, for gravity and cantilever retaining wall situated on flat ground.

It is seen in Figures 1(b), 4(a), and 4(b) that tilting of the wall are relatively minor. Therefore, the tilting mode of displacement was not considered in the present study. The accuracy of the ultimate bearing capacity obtained in the present study for the cases of vertical and inclined loading, active and passive resistance of soil were verified using various theoretical solutions, and is under preparation for publication (Huang and Chen, 2002). A personal computer program "SD-RWALL" was developed to calculate the safety factors of the wall, and also to calculate the critical seismic coefficient, k_{hcr}, (or yielding acceleration), and the seismic displacements of the walls. Note that the tensile stress of reinforcement in RWR model was not measured during the shaking table tests. Therefore, the pull-out resistance of the reinforcement located outside of the failure wedges was calculated from the overburden stress on the reinforcement and a reduced soil-reinforcement friction coefficient (i.e., 0.8 tan).

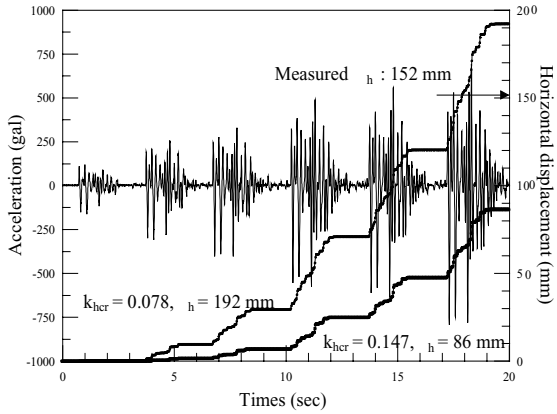

Figure 6 Time histories of input ground acceleration and calculated horizontal displacement of LRW.

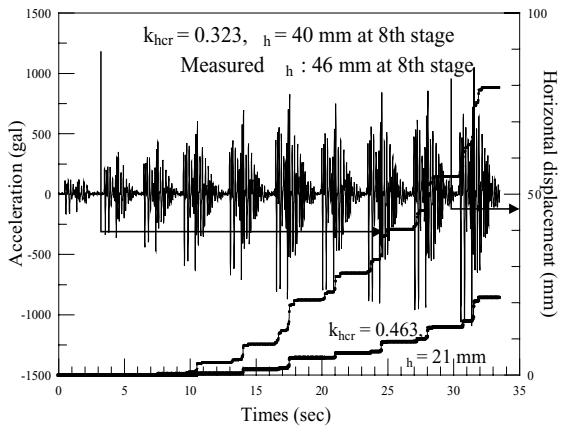

Figure 7 Time histories of input ground acceleration and calculated horizontal displacement of RWR.

4 SEISMIC DISPLACEMENT OF MODEL WALLS SITUATED ON SLOPE

We assume that Newmark's sliding block theory (Newmark, 1965) is applicable for calculating the horizontal displacements of the wall. The calculated $_h$ vs. time relationships for LRW and RWR are shown in Figures 6 and 7 respectively. It can be seen in Figure 6 that the measured value of $_h$ for LRW (152mm) is comparable to the calculated one when 32 36 . These values are relevant to the so-called 'residual strength' of dense Toyoura sand. The $_h$ vs. time relationship shown in Figure 7 shows that the measured

values of δ_h (=46mm) is close to the line for ϕ=45°. These values are relevant to the peak, or slightly deteriorated, friction angle of dense Toyoura sand. The above results suggest that:

1. Strain localization at the shear band, as discussed by Tatsuoka et al. (1998), may occurred to various extents for these two cases. The non-reinforced LRW experienced more reduction in ϕ because of the higher degree of strain localization.
2. Seismic stability of RWR may be increased by having higher values of ϕ in the shear bands.

5 CONCLUSIONS

Behavior of a conventional leaning retaining wall and a reinforced soil retaining wall situated on hillside under strong ground acceleration was investigated using shaking table tests and displacement analysis. Brittle failure of the leaning retaining wall under relatively smally ground acceleration was observed. Coherent behavior of the reinforced wall showing small tilting and displacement of the wall under relatively strong ground acceleration was also observed. A pseudo-static method was developed to calculate the safety factor and/or critical seismic coefficient for the horizontal sliding and the bearing capacity failure of the wall. Horizontal displacement of the wall was calculated using the critical seismic coefficient and Newmark's sliding-block theory. Results of horizontal displacement analysis suggest that relatively large displacement of leaning retaining wall is resulted from mobilization of residual friction angle along the slip surface. On the other hand, the relatively ductile performance of the reinforced soil wall is due to the lower degree of strain localization along the failure surface. Results of analysis imply the applicability of pseudo-static method in dealing with various ductilities of soil retaining walls under strong base acceleration. More studies on vertical displacement of various soil retaining walls should be performed to verify the above conclusions.

6 ACKNOWLEDGMENT

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