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Shaking table tests on a large geosynthetic-reinforced soil retaining wall model

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ABSTRACT: The results of large scale shaking table tests on a 2.5-m high scaled model of geosynthetic-reinforced soil retaining wall with sand backfill and a full-height rigid facing, are reported. The distributions of acceleration, tensile force in reinforcement layers and displacement are represented. It is shown that the model wall was very stable, behaving as a monolith during large dynamic loading, reaching a maximum acceleration of 500 gals. The wall itself was also stable in a special test in which the supporting sand deposit was liquefied and the bearing capacity for the facing was lost.

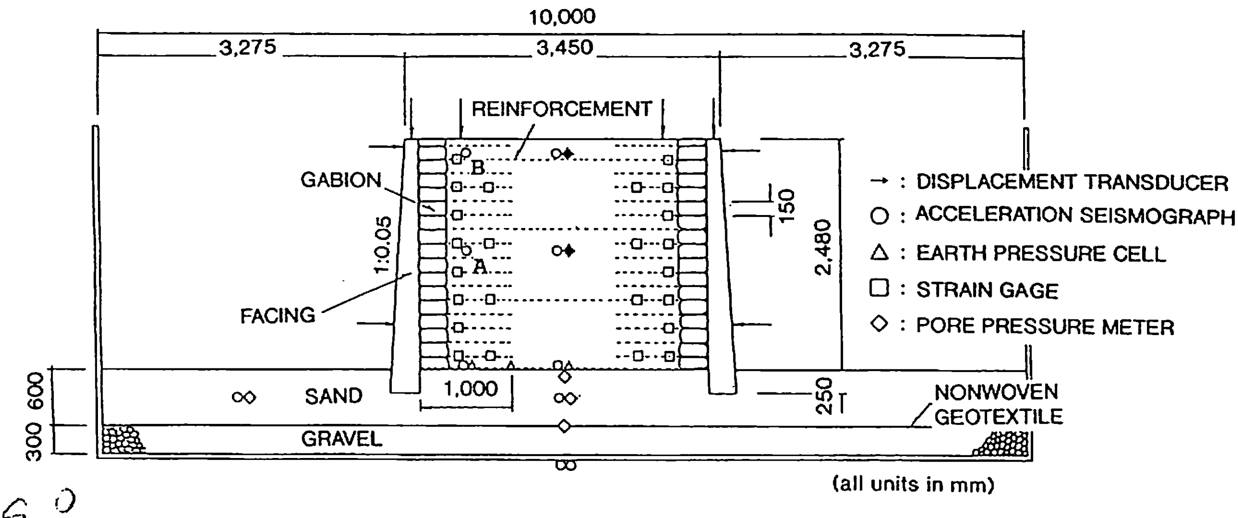
1. INTRODUCTION

The authors proposed a reinforced earth method using relatively short geosynthetic sheets and a continuous rigid facing (Murata et al.,1991, 1992, Tatsuoka et al., 1991, 1992). This type of facing is a cast--in-place, lightly reinforced, concrete layer placed directly on a wrapped-around wall face. To verify its applicability to actual construction projects, the authors constructed two full-scale test embankments, observed their long-term behavior and finally performed loading tests, including lateral loading tests, bringing them to failure (Tamura et al., 1993 and Tateyama et al., 1993 in this volume). Furthermore, a series of shaking table tests of five 100 cm-high scaled models (1/5) were performed to ascertain a seismic design (Murata et al., 1992). In these tests, the effects of facing rigidity, length of reinforcement, number of reinforcement layers and inclination of facing under dynamic loading conditions were investigated.

After these small scale shaking table tests, a 248 cm-high model, of which the scale is a half of the field prototype, was constructed on a large shaking table. Then, a series of shaking tests were performed in order to confirm the resistance capacity against earthquake load. From these test results, the authors found that with a relatively short length of planar grid reinforcement (40 % of the wall height), the use of a continuous rigid facing is very effective in stabilizing the wall and in reducing its deformation under dynamic loading conditions as well. The total length of geosynthetic-reinforced soil retaining walls (GRS-RWs) of this type, supporting railway tracks, now amounts to more than 10 km. This report describes the results of the large scale shaking table tests on the GRS-RW system model.

2. TEST MODEL

The wall height in the model was 248 cm and its width was 345 cm (**Fig. 1**). It was constructed on a dry sand layer (Hamaoka sand) having a mean grain diameter of 0.29 mm and a fines content of 0.8 % in a large sand box, placed in a large shaking table. The layout, the backfill (Inagi sand) and reinforcement material were essentially the same as those used for JR No.1 test embankment with sand backfill. However, the scale was reduced to a half of the field prototype.



5.0

Fig. 1 Configuration of large scale model and instrumentation

Table 1 Index properties of soils

5.0

	Wet density ρ_t(g/cm³)	Dry density ρ_d(g/cm³)	Water content w(%)	Degree of compaction(%)
Backfill	1.82	1.60	13.7	90.4
Ground	1.57	1.54	2.1	58.4*

* : Relative density

The model wall was constructed in a large steel sand box with a width of 3.0 m, as follows:

1) The base ground was placed using appropriate compaction (see **Table 1**). The relative density was 58.4 % (relatively loose state).

2) The wall was constructed in the same way as the prototype (also as the actual walls) (see Fig. 1 of Kanazawa et al., 1994 of this volume). The backfill sand had a mean grain diameter of 0.2 mm and a fines content of 16 %. The sand was compacted by using a small compactor at lifts of 15 cm utilizing gabions at the edge of each soil layer (see **Table** 1). The grid reinforcement used had a tensile rupture strength of 1.0 tf/m, one third of that of the grid used for the full-scale test wall. When the similitude rule is applied, the strength of the grid should be about one fourth of the strength of the prototype grid. However, such a grid was not available. The aperture of the grid was 20 mm by 20 mm, and the length was 1.0 m, about 40 % of the wall height, except for three full-width layers (see Fig. 1).

3) A continuous rigid facing of cast-in-place unreinforced concrete layer was constructed directly over the wrapped-around wall face. The slope of the completed wall face was 1.0:0.05 (V:H). The sides of the facing were about 1 cm from the steel sand box.

The shaking table tests had the following three phases:

(1) **Vibration test A:** Nine stages of horizontal sinusoidal motion, at a frequency of 3.4 Hz, were applied to the shaking table for a duration of 20 seconds. At each step, the acceleration was increased by 50 gals (= cm/sec^2), from 100 gals up to 500 gals.

(2) **Vibration test** B: A time history of earthquake motion (horizontal component), recorded at the ground surface during a major earthquake in the past, was applied after having being adjusted to a predominant frequency of 2.45 Hz (considering the similitude rule) and a maximum acceleration of 500 gals. The application period was 2 minutes.

(3) **Vibration test** C (Liquefaction test): After saturating the supporting sand layer, two steps of sinusoidal motion, at a frequency of 2.0 Hz and with a maximum acceleration of 200 and 400 gals, were applied to the shaking table for a duration of three minutes. In this test, the

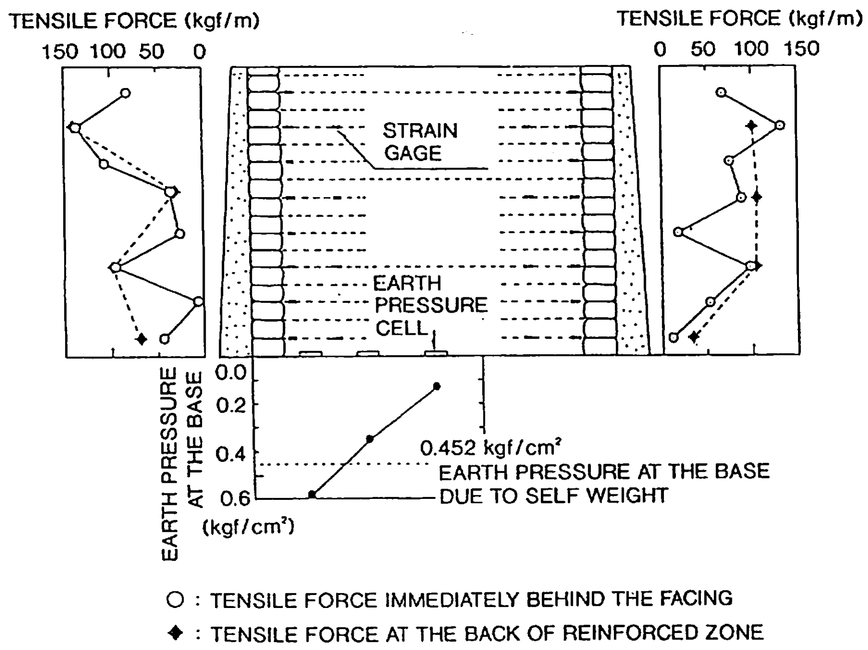


Fig. 2 Earth pressure and reinforcement tensile force after construction

supporting sand layer liquefied.

Many measuring instruments located in the backfill and the supporting ground, were set to observe the behavior of the model during the vibration tests (see Fig. 1).

3. TEST RESULTS

Fig. 2 shows vertical earth pressure at the base of the test wall and the reinforcement axial tensile force (per unit width) measured immediately after construction. It may be seen that the smallest earth pressure, which was only 30 % of the average overburden pressure, was observed at the middle of the backfill zone. The earth pressure increased towards the facing. This is partly due to the wall tendency to overturn about its toe. This view is supported by the distribution pattern of tensile force in the reinforcement layers. This result also indicates that at low stress levels in the wall, the soil immediately behind the facing resisted this tendency effectively due to its confinement by the rigid facing. It may also be noted that the integrated value of the observed vertical earth pressure was less than the weight of backfill since some of the weight of the backfill had been transferred to the bottom of the facing. It may also be noted that the tensile force in the full-width reinforcement layers, which were connected to the facings at both sides, were not particularly large when compared to that in the other layers. Namely, these full-width layers did not develop particularly large tensile force to resist the overturning of the wall (n.b., this was also the case under dynamic loading conditions as shown below).

Fig. 3 shows typical time histories of horizontal acceleration during Vibration test A, recorded at the base of shaking table, and at Points A and B in the wall (see Fig. 1). **Fig. 4** shows the amplification ratio of the maximum acceleration along the vertical direction in the reinforced zone and at the center of the wall during Vibration tests A and C. In Vibration test A, almost no amplification was observed up to the mid-height of the wall, while it was still less than 1.5 times at the crest. Further, the behavior was very similar for the reinforced zone and the central unreinforced zone. These results suggest that the wall exhibited an approximately monolith behaviour. The behaviour during Vibration test C (Liquefaction test) was similar, but the amplification was slightly larger. All these test results indicate

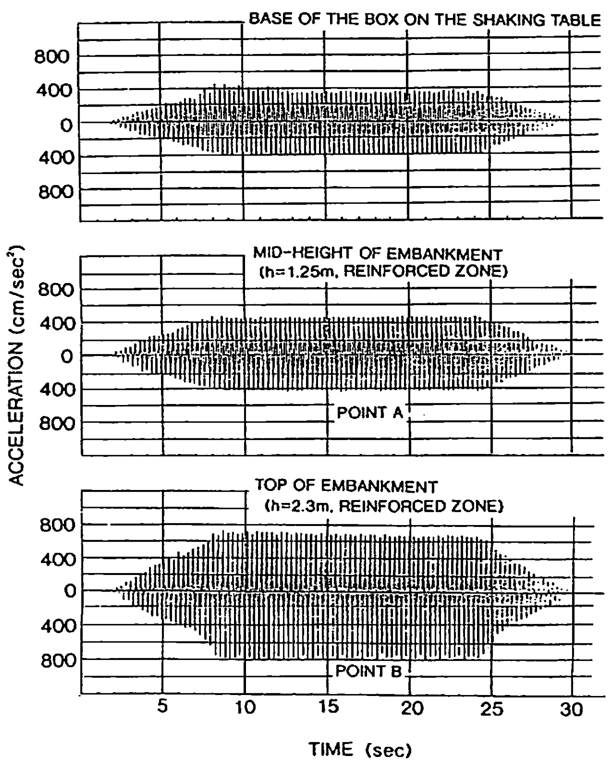


Fig. 3 Measured acceleration versus time (Vibration Test A)

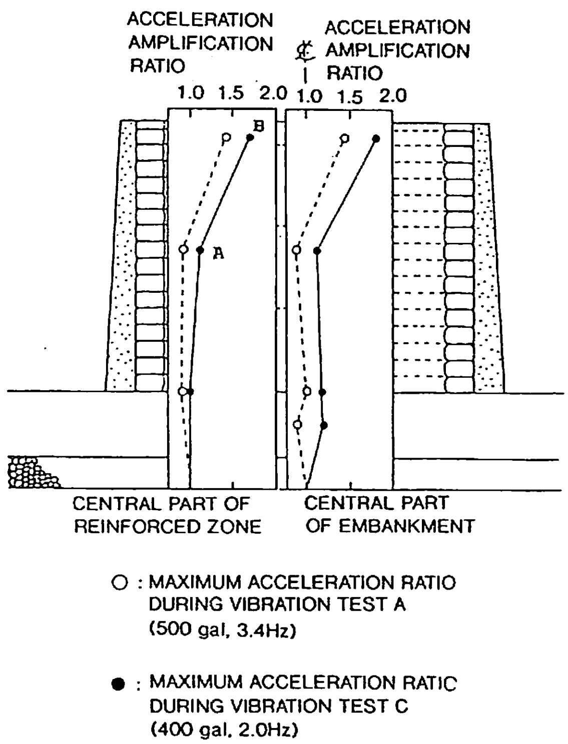


Fig. 4 Acceleration amplification ratio in the embankment

positive effects of reinforcement combined with rigid facing.

Fig. 5 shows the outward lateral displacement at the wall face and the settlement at the wall crest observed after the loading at 500 gals in Vibration test **A** and the loading at 400 gals in Vibration test **C**. It may be seen that even after very large dynamic load was applied, while failure did not occur in the supporting ground (Vibration test **A**), the deformation of the wall was very small (the outward lateral displacement at the top of facing was only 1.0 mm). **Fig. 6** shows the maximum reinforcement tensile forces during vibration compared with those before vibration. It may be seen that the increase during vibration was very small compared to the tensile force under the static condition. The maximum value during vibration was much smaller than the rupture strength of the reinforcement.

The increase in the deformation of the wall by Vibration test **B** using an amplified earthquake motion was smaller than that occurring by Vibration test **A**. The behavior of the model wall during Vibration tests **A** and **B** indicates that GRS retaining walls could be very stable even during strong earthquake motions when properly designed and constructed.

Vibration test **C** was a very special test, since prototype walls should never be constructed on a liquefiable soil deposit. That is, such a soil deposit will first be improved by some means before constructing a GRS retaining wall. This test was performed to verify if the wall will not fail catastropically even when the bearing capacity of the facing is lost. It may be seen from Fig. 5 that the maximum settlement observed at the crest after the test was about 11 mm and 28 mm at the center of the crest and the facing, respectively. The maximum outward lateral displacement of the facing was about 8 mm near the wall bottom. This rather large deformation occurred due to the lost of the bearing capacity of the supporting ground during liquefaction. It is to be noted that in spite of this relatively large deformation, the entire wall moved just as

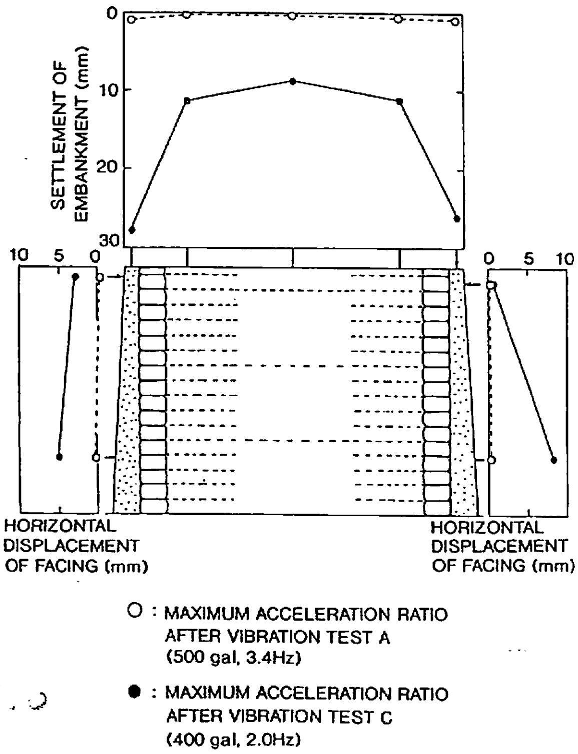


Fig. 5 Maximum deformation of the embankment after vibration

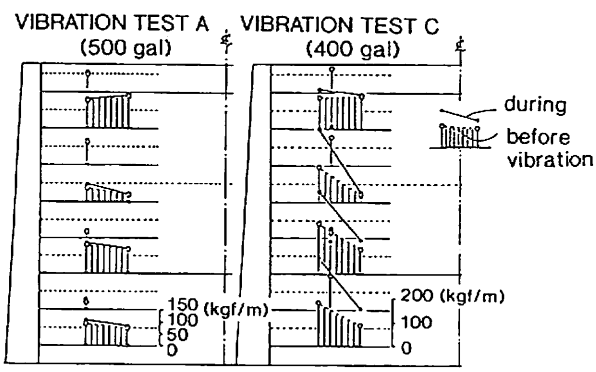


Fig. 6 Maximum reinforcement tensile force during vibration

a monolith and was very stable. It was confirmed after taking the wall apart that the connection between the facing and the reinforcement was not damaged at all. This is despite a rather large relative movement between the facing and the backfill (about 20 mm). It is likely that the stack of gabions acted as a buffer zone. It may be seen from Fig. 6 that the increase in the tensile force in the reinforcement layers during dynamic loading was slightly larger than that during Vibration test A. It seems that this relatively large increase was mainly due to the relative settlement of the facing (see Fig.5).

It should be noted that the tensile force in the three full-width reinforcement layers during Vibration tests A and C (also test B) was not particularly large when compared with those in the other layers (see Fig. 6). These results show that connecting both faces with these three layers did not contribute much to increase the stability of the wall during the dynamic loading tests.

3. CONCLUSIONS

Theoretical analyses of the results of the dynamic loading tests has not been performed so far. Yet, the test results indicate that GRS retaining walls with a continuous rigid facing could be very stable against severe earthquake loading, including foundation liquefaction.

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