Proceeding of the 4th Asian Regional Conference on Geosynthetics
June 17 - 20, 2008 Shanghai, China

STUDY ON SEISMIC PERFORMANCE OF GEOGRID REINFORCED SOIL RETAINING WALLS AND DEFORMATION CHARACTERISTICS OF BACKFILL SOIL

Nakajima, S.[1], Hong, K.[2], Mulmi, S.[3], Koseki, J.[4], Watanabe, K.[5] and Tateyama, M.[6]

ABSTRACT: Based on results from a series of shaking table model tests on geogrid reinforced soil retaining walls, effects of material properties (i.e. pullout resistances, rupture strength and tensile rigidities) on seismic performance of reinforced soil retaining walls are discussed. Although the material properties of two geogrid models used in this study were largely different, residual displacements of wall facing were almost equal to each other. It is also attempted to obtain deformation characteristics of reinforced backfill soil from its dynamic responses and displacements of wall facing by assuming that the reinforced backfill would behave as one macro element.

Keywords: shaking table model tests, material properties, deformation characteristics of reinforced backfill, geogrid

BACKGROUND

Geogrid reinforced soil retaining walls (RRW) having a full-height rigid wall facing showed higher seismic performance than conventional concrete retaining walls during recent earthquakes (Koseki et.al. 2006). Due to ductile seismic performance of the reinforced soil retaining walls, it is requested to shift design procedure to performance-based design from conventional limit-equilibrium methods. In the performance-based design, seismic performance of retaining walls would be typically verified by confirming that the residual displacements do not exceed the allowable ones.

A series of shaking table model tests on the RRW has been conducted so as to investigate into their seismic behaviors (Watanabe et. al. 2003). Based on an observation during the previous model tests, a procedure to predict residual displacements of the RRW, which considers shear deformation of the reinforced backfill and subsoil beneath the reinforced backfill, was developed (Koseki et. al. 2004). In these model tests, however, a phosphor bronze strip was used as geo-grid model without referring to any similitude. Because use of the phosphor bronze as the reinforcement model made it possible to measure the tensile force during shaking. In addition to this, internal stabilities of the RRW were rather highlighted at the previous series of the model tests.

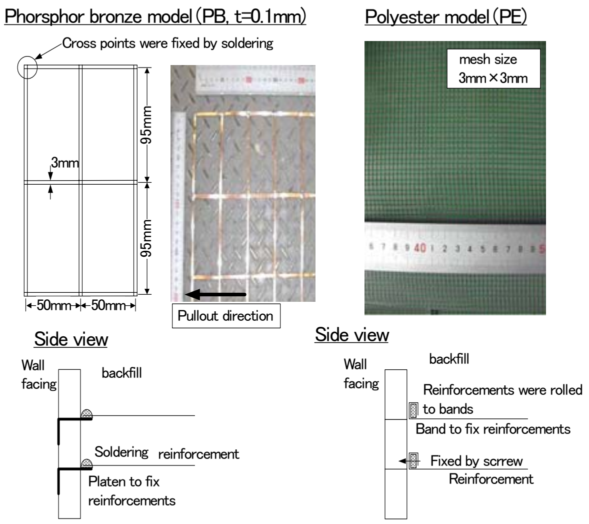

Fig.1 Geogrid models

Based on the above background, effects of material properties of the geogrid models on seismic performance of reinforced soil retaining walls were firstly investigated in this study. Secondly, it is also attempted to introduce the deformation characteristics of the reinforced backfill into the proposed procedure to predict residual displacements of the reinforced soil retaining walls.

[1] Student, Department of Civil Engineering, University of Tokyo, Japan, Email:snakaji@iis.u-tokyo.ac.jp
[2] Student, Department of Civil Engineering, University of Tokyo, Japan, Email:hkimhor@iis.u-tokyo.ac.jp
[3] Student, Department of Civil Engineering, University of Tokyo, Japan, Email:smulmi@iis.u-tokyo.ac.jp
[4] Professor, Institute of Industrial Science, University of Tokyo, Japan, Email:koseki@iis.u-tokyo.ac.jp
[5] Assistant Senior Researcher, Foundation and Geotechnical Engineering, Railway Technical Research Institute, Japan, Email:nabeken@rtri.or.jp
[6] Head, Foundation and Geotechnical Engineering , Railway Technical Research Institute, Japan, Email:tate@rtri.or.jp

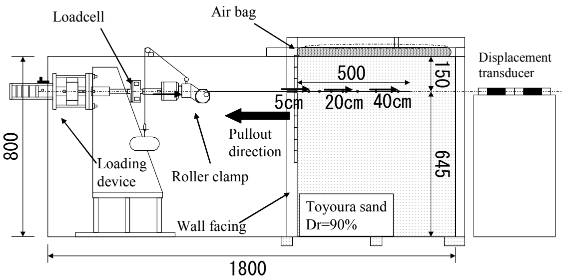

Fig.2 Apparatuses for pullout test (unit in mm)

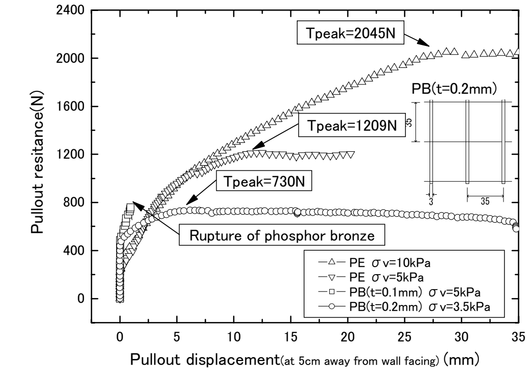

Fig.3 Comparison of pullout resistances

GEOGRID MODEL

Two types of geogrid models were used in this study. For the one type of the model, which will be called as PB hereafter, phosphor bronze strips having a width of 3 mm and a thickness of 0.1 mm were prepared in a lattice shape, and their cross points were fixed by soldering as shown in Fig.1. Sand particles were pasted on the surface of the reinforcement PB so as to mobilize the frictional resistances effectively. For the other type of model as also shown in Fig.1, a polyester mesh sheet having a thickness of 0.6 mm and a mesh size of 3 mm was used. The latter model will be called as PE hereafter.

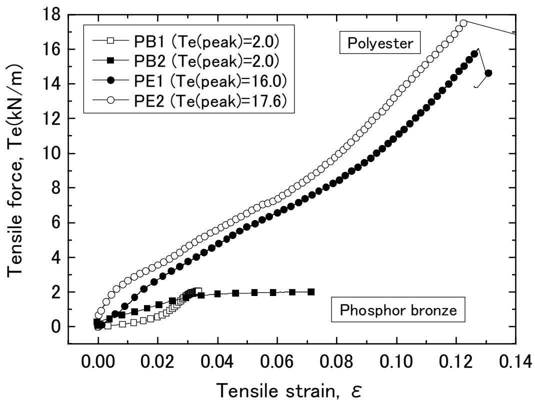

Fig. 4 Results from direct tension tests

MATERIAL PROPERTIES OF GEOGRID MODELS

Pullout tests and direct tension tests were carried out (Nakajima et. al. 2007) so as to compare the material properties of the geogrid models. Test apparatuses for pullout tests are shown in Fig.2. A specimen of the reinforcement model having a width of about 27 cm and a length of 50 cm (the total length including the portions in air was about 90 cm) was placed on a model ground consisting of air dried dense Toyoura sand having a relative density of about 90 %. Density of the model ground in pullout tests was set equal to the backfill layers in the shaking table model tests which will be discussed later.

So as to simulate the stress state of the backfill layers in shaking table model tests, overburden pressures of the pullout tests were set equal to 5 kPa (middle height of the backfill) and 10 kPa (bottom of the backfill). In Fig.3, pullout resistances are plotted versus the pullout displacement measured at a distance of 5cm from the wall facing. With increase in the overburden pressure, the peak resistance T_{peak} increased in case of the model PE. Rupture of the phosphor bronze strips occurred at the pullout resistance of about 800 N in case of the model PB (t=0.1 mm). Test result on the model PB (t=0.2 mm) having a different shape and thickness

Table1 Comparison of tensile rigidities

	Te(peak)/2 (kN/m)	Te(peak) (kN/m)	Te(peak)/2 (kN/strip)	Te(peak) (kN/strip)
	Per unit width		Per single strip	
PE1	120.5	143.1	0.36	0.43
PE2	105.1	130.0	0.31	0.39
PB1	65.7	60.5	65.7	5.25
PB2	65.7	29.0	65.7	2.52

(Nojiri et. al. 2007) is also plotted in Fig.3. The peak pullout resistance of the model PB (t=0.1 mm) was thought to be lower than that of the model PE by referring to the test results on the model PB (t=0.1 mm) although the shapes and the test conditions were slightly different.

To evaluate the rupture strength and tensile rigidity of the geogrid models, direct tension tests of the geogrid models were conducted. The width of the geogrid model in the direct tension tests was set equal to 5 cm in case of the model PE, while one strip having a width of 3 mm was tested in case of the model PB for the convenience of the test preparation. Constant strain rate loading of 1%/min was applied in both cases. Tensile loads that are converted to the values per unit width are plotted versus the tensile strain in Fig.4. Rupture strength Te(peak) are also indicated in Fig.4. The rupture strength of the model

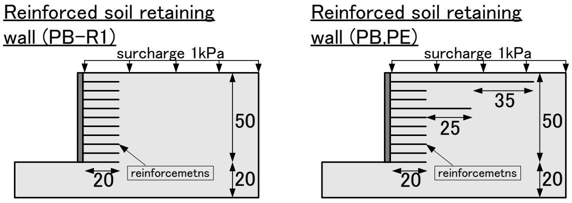

Fig.5 Cross sections of models (unit in cm)

PE was about eight times larger than the ones of the model PB because the number of the strips parallel to the tensile direction of the model PE was much larger than that of the PB.

Equivalent tensile rigidities per unit width and single strip of the two models, when the Te(peak) values and half of them were mobilized, are summarized in Table 1. The values of tensile rigidity of single strip of the model PB were larger than those of the model PE made of polyester, while the values per unit width of the model PE were larger than those of the model PB due to larger number of the strips of the model PE.

MODEL TEST PROCEDURE

As shown in Fig.5, two models of the geogrid reinforced soil retaining walls PE and PB were tested in this study. They had a height of 500mm and they were placed on a horizontal subsoil layer having a thickness of 200 mm. During preparation of the backfill layers, the two geogrid models were fixed with the wall facing at a vertical spacing of 50 mm as shown in Fig.1. The cross section of the reinforced soil retaining wall model with different arrangements of reinforcements (R1, Watanabe et. al. 2003) is also shown in Fig.5. Deformation characteristics of the reinforced backfill of the test R1 will be compared with the ones from this study later. Both the subsoil and backfill layers consisting of air dried Toyoura sand having a relative density of about 90 % were prepared by air pluviation using a sand hopper.

Seismic loads were applied by shaking the soil container horizontally with irregular waves as typically shown in Fig.6, while the maximum base acceleration was gradually increased from 0.9 G to 1.5 G at an increment of about 0.3 G. The final shaking step having the maximum acceleration of about 1.5 G was applied twice in both tests.

RELATIONSHIP OF RESIDUAL DISPLACEMENTS AND MATERIAL PROPERTIES

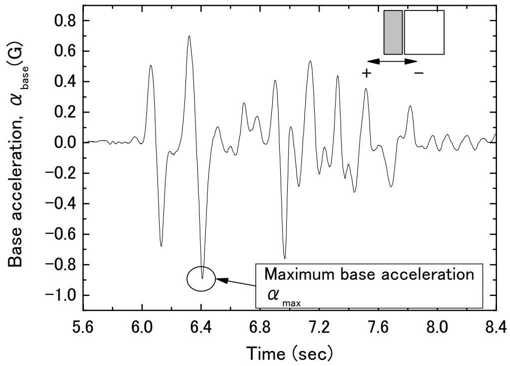

Fig. 6 Typical time history of base acceleration

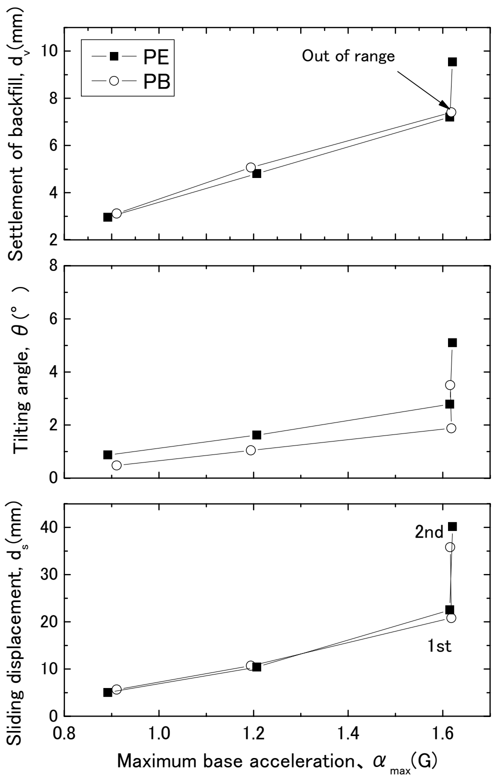

Fig.7 Comparison of residual displacement

Residual displacements after each shaking step are plotted versus the maximum base acceleration in Fig.7. Sliding displacement, tilting angle of the wall facing, and settlement of the backfill layers at the horizontal distances of 100 mm from the wall facing are concerned as the representative residual displacements in this study.

Seismic performances of the two reinforced soil retaining walls were similar to each other even though the material properties like pullout resistances, rupture strength and tensile rigidity were largely different from each other. Tilting angle of the wall facing of the model PB was slightly smaller than that of the model PE, while larger mobilized resistances could be expected with the model PE according to the aforementioned test results from the pullout tests and direct tension tests.

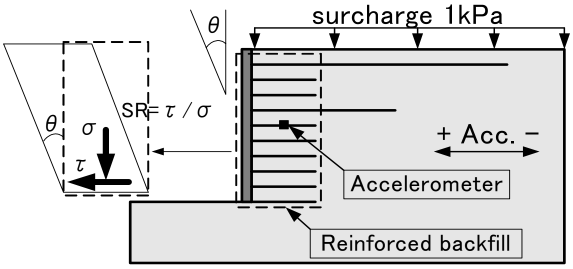

Fig. 8 Schematic diagram of shear deformation of reinforced backfill

It was observed in previous model tests on reinforced soil retaining walls that the sliding displacement of the wall was induced by the shear deformation of the subsoil beneath the reinforced backfill, and tilting of the wall was induced by the shear deformation of the reinforced backfill. Sliding displacements of the two model tests were almost similar to each other because the subsoil conditions were the same in this study.

On the other hand, tilting angle of the model PB was smaller than that of the model PE. Two factors might have affected this behavior. Firstly, the shear deformation of the reinforced backfill itself of the model PB was larger than that of the model PE. Possibly because the rigidity of the reinforced backfill of the model PB would be lower than that of the model PE by referring to the tensile rigidity as compared in Table 1. Secondly, tilting of the wall facing due to the shear deformation of the reinforced backfill was restricted by the pullout resistances of the reinforcement having the length of 45 cm and 80 cm. Pullout resistance of the model PB before its rapture was higher than those of the model PE as shown in Fig.3. Based on the observation that tilting angle of the model PB was lower than that of the model PE, the latter factor seems to overcome the former factor in this study,.

DEFROMATION CHARACTERISTICS OF REINFORCED BACKFILL

As discussed above, deformation characteristics of the reinforced backfill are important in evaluating the seismic performance of the reinforced soil retaining walls. Based on the model test results, shear stress - strain responses of the reinforced backfill were computed so as to develop a procedure to evaluate residual displacement of the reinforced soil retaining walls.

Shear deformation of the reinforced backfill considered in this study is schematically illustrated in Fig.8. Shear forces acting on the reinforced backfill can be computed as a sum of inertia force from dynamic

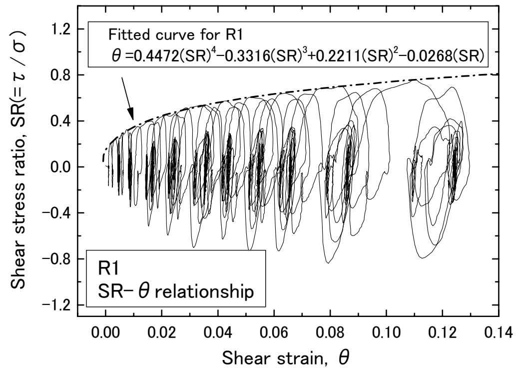

Fig.9 Stress-strain relationship in test R1

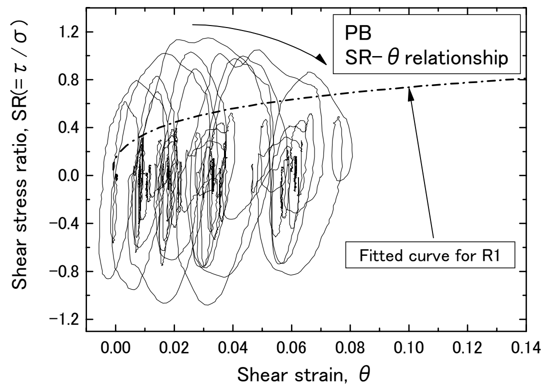

Fig.10 Stress-strain relation ship in test PB

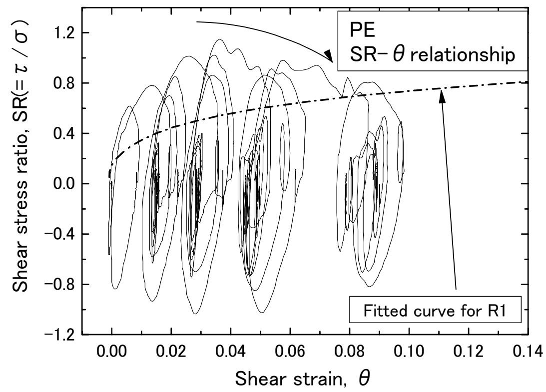

Fig.11 Stress-strain relation ship in test PE

responses of the reinforced backfill measured by accelerometers installed in the backfill layers. Shear stress of the reinforced backfill was computed by normalizing the shear force computed at the bottom of the reinforced backfill with its area of the reinforced backfill as shown in Fig.8. Acceleration records at the middle height of the reinforced backfill were used to evaluate the shear force in this computation. As also illustrated in Fig.8, averaged shear strain of the reinforced backfill can be regarded as the tilting angle of

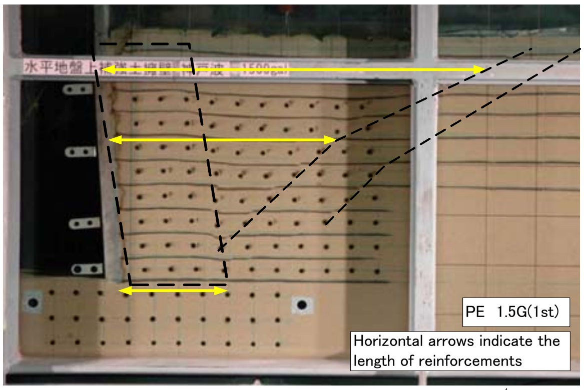

Fig.12 Photo taken after the shaking at 1.5G(1st) in the test PE

the wall facing by assuming that the reinforced backfill would deform uniformly as one macro element.

Shear stress ratio, which is defined as the shear stress normalized by the overburden pressure at the bottom of the reinforced backfill, is plotted versus the shear strain in Figs.9 to 11. The overburden pressure was assumed to be constant by neglecting the effect of vertical inertia forces for simplicity in this computation. As shown in Fig.8, mobilization of the positive shear stress ratio means that the reinforced backfill was subjected to the inertia force to the active direction. Fig.9 shows stress-strain relationship of the reinforced backfill in case of the test R1 which didn't have extended reinforcements. The cross section of the model ground in the test R1 was the same as those of the tests PB and PE while the shaking by using the irregular waves, which are typically shown in Fig.6, was cyclically applied at the maximum acceleration of about 100 gals to 1100 gals with its increment of about 100 gals. Figs. 10 and 11 show the stress-strain relation ships of the test PB and PE respectively.

It was found from Figs.9 to 11 that the shear strain was accumulated while the positive shear stress ratio (i.e. inertia force toward the active direction applied to the backfill) was mobilized. As also indicated in Figs.10 and 11, the mobilized values of SR were reduced after the peak value was exhibited in case of the tests PB and PE. Strain softening behavior might have affected these behaviors because shaking in the tests PB and PE was continued even after failure plane formed at just outside of the uppermost reinforcement in un-reinforced backfill as typically shown in Fig.12. It was also observed in Fig.12 that the reinforced backfill exhibited the uniform shear deformation as assumed in this computation.

Upper bound of the stress-strain relationship of the test R1 was fitted by using polynomial equation as shown in Fig.9. This fitted curve of the test R1 is also shown in Figs.10 and 11. As clearly shown in Fig.10 and 11, mobilized shear stress ratio by the tests PB and PE exceeded the fitted curve of the test R1. Pullout resistances by extended reinforcements were thought to be mobilized due to shear deformation of the backfill in these tests. In further study, the effects of the extended reinforcements should be properly taken into account.

SUMMARY

Results of this study can be summarized as follows

1) Seismic performance of the reinforced soil retaining walls by using different two geo-grid models was almost equal to each other although the material properties of the geogrid models were largely different.
2) Tilting angle of wall facing in the test PB was smaller than that of the test PE although larger resistances could be expected by the test PE.
3) Larger pullout resistances mobilized by the model PB at relatively small pullout displacements range would work effectively to reduce the tilting angle of wall facing although the peak pullout resistance of the model PB was smaller than that of the model PE.
4) Stress-strain relationships of the reinforced backfill can be computed based on measurements of dynamic response of the reinforced backfill and tilting angle of the wall facing, while effect of the pullout resistances of extended reinforcements should be properly introduced in further study.

REFERENCES

Koseki, J., Bathurst, R.J., Guler, E., Kuwano, J. and Maugeri, M. (2006), Seismic stability of reinforced soil walls, Keynote lecture, Proc. of the 8th International Conference of Geosynthethics, Proc. of the 8ICG, Vol.1, pp.51-78.

Koseki, J., Kato, N., Watanabe, K. and Tateyama, M. (2004), Evaluation of seismic displacement of Reinforced walls, Proc. of the 3rd Asian Regional Conference on Geosynthetics, pp.217-224.

Nakajima, S., Hong, K., Mulmi, S., Koseki, J., Watanabe, K., and Tateyama, M. (2007), Model tests on seismic performance of geo-grid soil reinforced soil retaining walls by using different geo-grids, International Workshop on Earthquake Hazards and Mitigations (EHAM2007).

Nojiri, M., Aizawa, H., Nishikiori, D., Sasada, Y., Hirakawa, D. and Tatusoka, F. (2007), Effects of pullout resistance on performance of geosynthethics reinforced-soil structure (in Japanese), 42nd Japan National Conference on Geotechnical Engineering, pp.1573-1574.

Watanabe, K., Munuf, Y., Koseki, J., Tateyama, M. and Kojima, K. (2003), Behaviors of several types of model retaining walls subjected to irregular excitation, Soils and Foundations, Vol.43, No.5, pp.13-27.