New Horizons in Earth Reinforcement – Otani, Miyata & Mukunoki (eds)
© 2008 Taylor & Francis Group, London, ISBN 978-0-415-45775-0

Shaking table model tests on retaining walls reinforced with soil nailings

S. Nakajima
University of Tokyo, Tokyo, Japan

J. Koseki
Institute of Industrial Science, Tokyo, Japan

K. Watanabe & M. Tateyama
Railway Technical Research Institute, Tokyo, Japan

ABSTRACT: Based on results from shaking table model tests on retaining walls reinforced with soil nailings, effects of the soil nailings as an aseismic countermeasure for existing retaining wall is discussed. It is also attempted to apply Newmark's sliding block method with the introduction of the effect of the nailings so as to develop a procedure to predict the residual displacement of the walls with the nailings. Computed sliding displacements are in good agreement with the measured ones, while computed tilting displacements are much larger than the measured ones due possibly to overestimation of overturning moment used in the analysis.

1 INTRODUCTION

In Japan, use of geosynthethic-reinforced soil retaining wall with full height rigid facing for new permanent structures is continuously increasing because of their higher seismic performance than conventional type retaining walls (e.g. gravity, leaning and cantilever type retaining wall), which suffered severe damages in recent earthquakes (Koseki et. al. 2006). However, there exist large numbers of the conventional walls which support road or railway embankments. Installations of aseismic reinforcements to these walls are required especially in case such walls support important embankments.

Currently, large diameter soil nailing, which is called as LDN hereafter, is used for railway structures in Japan as an aseismic countermeasures for the existing retaining walls. It consists of a column of improved soil and a tension bar that is located at the center of the column. Based on shaking table model tests results (Kato et. al. 2002), it was reported that seismic performance of the retaining wall on sloped subsoil could be effectively improved even though such walls without nailings were severely damaged during the Chi-Chi earthquake in Taiwan (Huang et. al. 2005). On the other hand, it is essential to evaluate the effects of the aseismic counetermeasures in terms of the amount of reduced residual displacements in performance based design.

In view of the above background, in this study, the results from a series of shaking table model tests on retaining walls reinforced with LDNs (Nakajima et. al. 2007) are analyzed, and it is also attempted to develop a procedure to predict the residual displacements of the walls after the earthquake.

2 MODEL TEST PROCEDURES

After a wooden leaning type retaining wall model having a height of about 500 mm was placed on a sloped subsoil layer, a backfill layer was prepared. Both the backfill and the sloped subsoil layers were made of Toyoura sand having a relative density of about 90%, which were prepared by air pluviation using a sand hopper.

Cement-treated sand and phosphor bronze bar having a thickness of 0.8 mm and a width of 5 mm were used to model the improved column and the tension bar of the LDN, respectively. Strain gages were pasted on the surface of the column to measure mobilized tensile resistance. Sand particles were also glued on the surface of the column to achieve enough frictional resistance. As shown in Fig.1, the LDN models were installed at the bottom of the footing and wall facing. It should be noted that former ones did not restrict tilting of the wall because they were fixed with the footing by hinges. The schematic view of the model without the LDNs, which would be refereed to in this study, is also shown in Fig.1. The subsoil thickness in this study was increased as compared with the previous study by Kato et.al. (2002) so as not to restrict

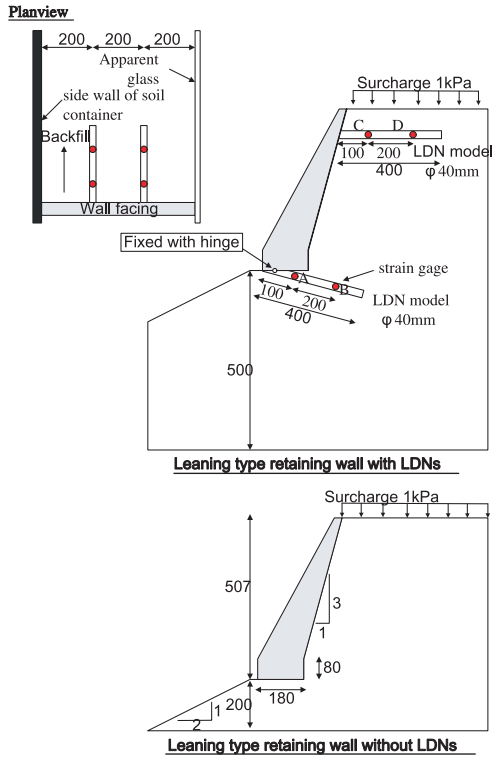

Figure 1. Schematic diagram of models (unit in mm).

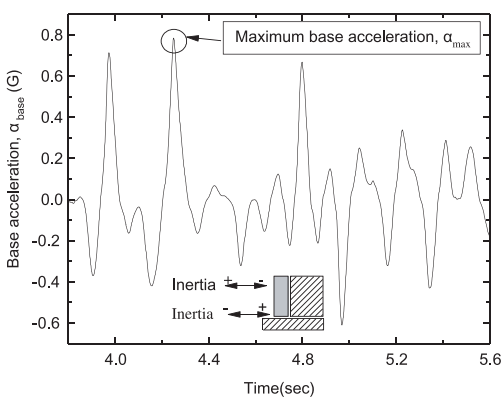

Figure 2. Typical time history of irregular excitation.

formation of failure plane passing through both the backfill and subsoil layers, which indicates the global instability of this model.

Seismic load was applied by shaking the soil container horizontally by using irregular excitations as typically shown in Fig. 2, while the maximum acceleration was gradually increased at an increment of about 100 gals until the wall displaced largely.

3 MODEL TEST RESULTS

3.1 *Failure process and residual displacements*

Figs.3a to 3d show the failure processes of the models. The sums of the residual displacements at each shaking step are plotted versus the maximum base acceleration α_{max} in Fig.4. Sliding displacement d_s, and tilting angle θ of the wall, and settlement of the backfill d_v are concerned in this study. The values of d_v were measured at a surface of the backfill having a horizontal distance of 200 mm from the wall facing as shown in Fig.1.

In case without the LDNs, increment of the residual displacements was accumulated largely after the formation of the failure plane in the backfill layers during the shaking at the α_{max} of 578 gals (Fig.3a). Residual wall displacements increased drastically after appearance of failure plane in the subsoil layer which indicated the bearing capacity failure during the shaking at the α_{max} of 636 gals (Fig.3b). In Fig.4, the value of permissible differential settlement of the backfill in the Level 2 earthquake for bridge abutment (damage level 3, which requires a minor retrofit work, in RTRI 1999) is also indicated. It was evaluated by reducing the actual value in prototype scale to one-tenth (i.e $200\,\text{mm} \times 1/10 = 20\,\text{mm}$), considering the difference in the model and prototype scales. As shown in Fig.4, the value of d_v in the case without the LDNs exceeded the permissible one during shaking at the α_{max} of 578 gals.

On the other hand, as also clearly shown in Fig.4, residual displacements decreased effectively by adding the LDNs. In case with the LDNs, failure plane in the backfill layers wasn't observed even after the shaking at the α_{max} of 640 gals (Fig.3c). The value of d_v was also reduced to about 5.7 mm, which was much smaller than the permissible value as evaluated above (i.e. 20 mm). With the increase of the α_{max}, residual displacements increased gradually, while no drastic increase of the wall displacements was observed during the whole shaking steps. However, after the formation of failure plane in the backfill and subsoil layers, wall displacements accumulated largely during the shaking step at the α_{max} of 1038 gals (Fig.3d). As also indicated in Fig.3d, the failure plane in the backfill layer was formed at a position just outside of the LDNs at the wall facing.

3.2 *Mobilized resistance by LDNs*

Mobilized tensile resistances by the LDNs at the wall facing (T_{top}) and bottom of the footing (T_{bottom}) at the timing of the α_{max} (i.e. the maximum inertia force was applied to the wall) are plotted versus the α_{max} in Fig.5. It should be noted that the values of T_{top} and T_{bottom} are converted to those per unit width considering the horizontal spacing of the LDNs as shown in Fig.1.

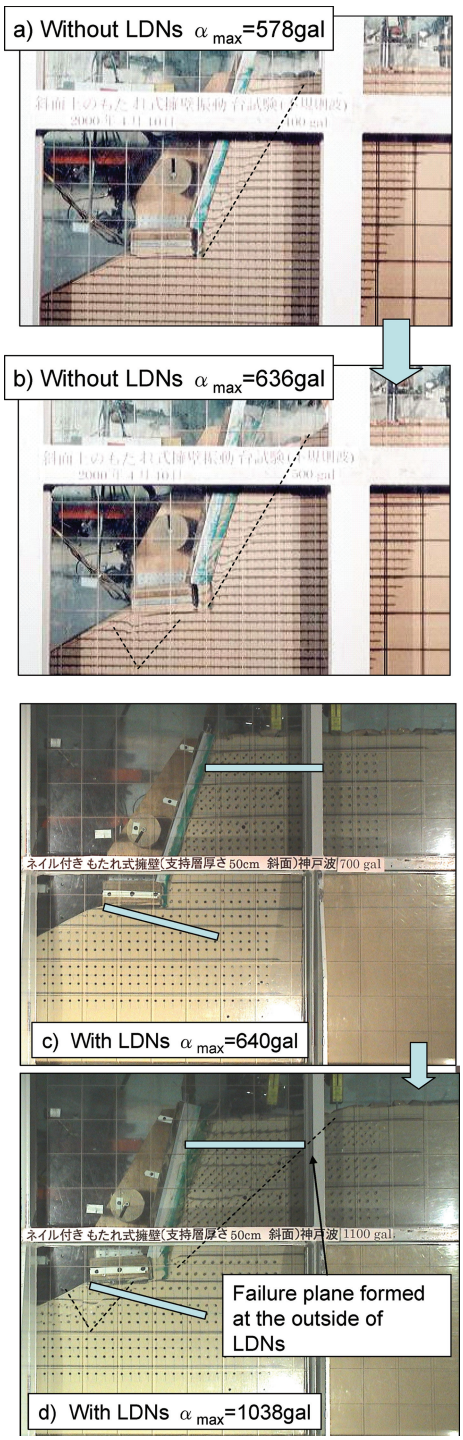

Figure 3. Failure processes of models.

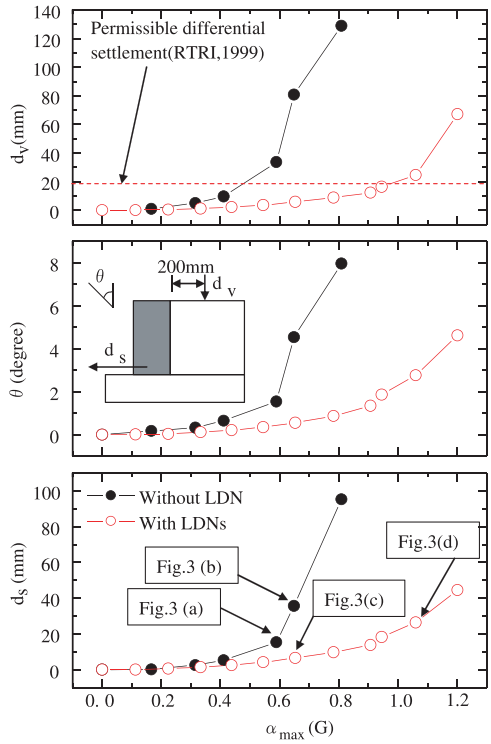

Figure 4. Comparison of residual displacements.

The values of T_{top} by gage C and gage D were almost equal to each other, while the T_{bottom} value by gage A, which is mobilized just beneath the footing, was much larger than the one by gage B which was pasted at a far position from the footing. This behaveior implies that the LDNs at the bottom could work effectively to restrict formation of failure plane in the subsoil layer that is associated with bearing capacity failure. As indicated in Fig.5, it was also observed that the T_{top} and T_{bottom} values were accumulated largely after the formation of failure plane in the backfill and subsoil layers during the shaking step at the α_{max} of 1038 gals. The levels of computed peak and residual tensile resistances as discussed later, are also indicated in Fig.5.

4 DISPLACEMENT ANALYSIS

4.1 *Newmark's sliding block method*

In order to evaluate the sliding displacement of gravity type retaining walls during earthquakes, Richards and Elms (1979) proposed to employ the Newmark's sliding block method. Relative displacement of the wall can be computed using the concept of threshold acceleration and the double integration proposed by Newmark (1965).

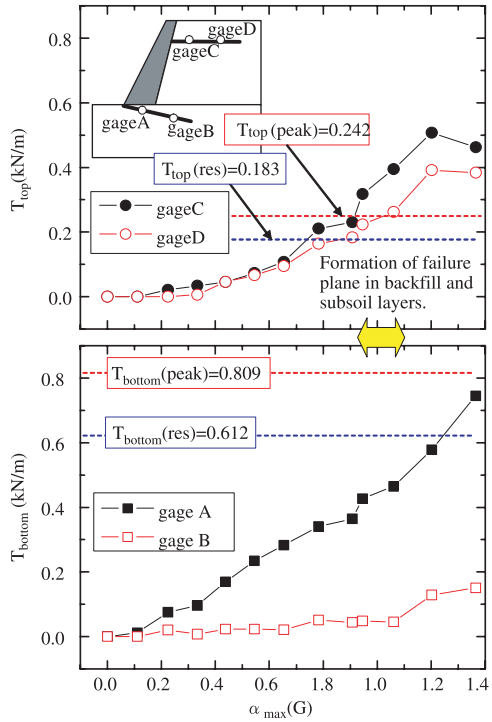

Figure 5. Mobilized tensile resistances by the LDNs.

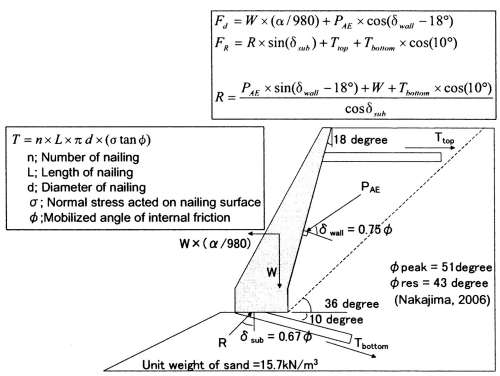

Figure 6. Pseudo static analyses against sliding.

As summarized in Fig.6, the threshold acceleration in this study was determined from a pseudo static analysis using the soil strength obtained from the relevant plane strain compression tests on dense Toyoura sand and the values of peak and residual resistances by the LDNs. The peak and residual mobilized angle of internal friction were set equal to 51 and 43 degrees in this analysis base on a result from relevant plane strain compression tests on dense Toyoura sand (Nakajima, 2006).

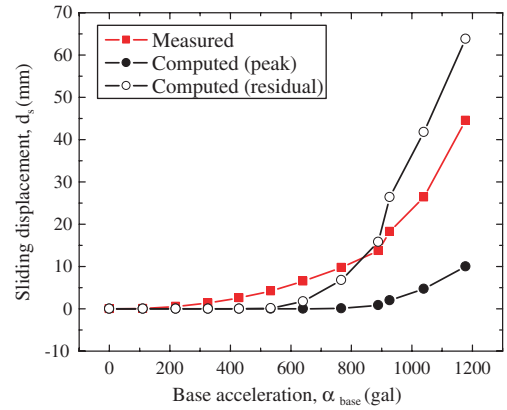

Figure 7. Comparison of sliding displacements.

It was assumed in the analysis that a driving force, which would induce the sliding of the wall, was the sum of the horizontal component of the earth pressure acted on the wall facing and the inertia force of the wall, while a resistant force against the sliding was assumed to be the sum of the horizontal component of the resistances by the LDNs and the frictional resistance at the bottom of footing.

The dynamic earth pressure was computed using a pseudo static approach proposed by Koseki et. al. (1998), while the angle of the failure plane in the backfill was set equal to 36 degrees based on the observation of shaking table model test as indicated in Fig.6. The resistances by LDNs were evaluated using the equation indicated in Fig.6. It should be noted that constant resistance was assumed to be mobilized in the relevant design guideline (RTRI, 1999) irrespective of the induced displacement. However, the measured resistance showed displacement-dependant behavior as shown in Fig.5.

Based on the results from the pseudo static analysis, the threshold acceleration was determined as the acceleration when the factor of safety against the sliding became unity. In this analysis, it was 711 gals when the peak soil strength was employed and 472 gals when the residual soil strength was employed. Computed sliding displacements using the above threshold accelerations and the Newmark's method are plotted versus the base acceleration in Fig.7. The computed displacement with threshold acceleration using the peak soil strength was smaller than the measured one, while the computed displacement using the residual soil strength corresponded well with the measured one. It should be noted that no displacement was computed until the base acceleration exceeded threshold acceleration, although a certain extent of the displacement was observed in the corresponding model test results.

$$M_d = W \times (\alpha/980) \times Yg + P_{AE} \times \cos(\delta_{wall} - 18°) \times (H/3)$$
$$M_R = R \times \cos(\delta_{sub}) \times 120 + T_{top} \times 400 + W \times Xg + P_{AE} \sin(\delta_{wall} - 18°) \times d$$
$$R = \frac{P_{AE} \times \sin(\delta_{wall} - 18°) + W + T_{bottom} \times \cos(10°)}{\cos\delta_{sub}}$$

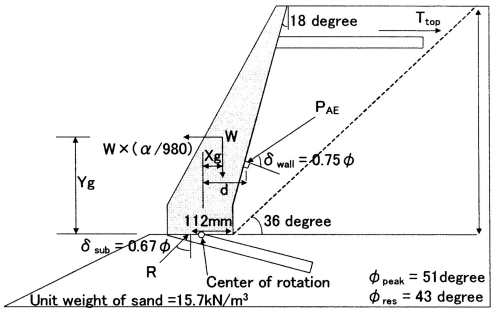

Figure 8. Pseudo static analyses against tilting.

One of the reasons causing such difference may be overestimation of the resistance by LDNs especially at lower acceleration levels. As shown in Fig.5, resistances by LDNs were not fully mobilized from the beginning of shaking. They increased gradually with the increase of the wall displacement. This behavior implies that displacement-resistance relationship of the LDNs as well as the maximum resistance, which is only focused in the current design procedure, should be taken into account in conducting displacement analysis like the Newmark's method.

In addition, the sliding displacement increment due to shear deformation of the subsoil (Koseki et. al. 2004) which was observed in the model test was also neglected in the above analysis. Therefore, further investigation on these issues is required.

4.2 *Newmark's rotating block approach*

Zeng and Steedman (2000) proposed to apply the Newmark's method to evaluate the overturning displacement of the walls. Following the same concept, evaluation of the threshold overturning moment is made in this study to compute overturning displacement of the wall.

Based on the results from the pseudo static approach as summarized in Fig.8, the threshold moment was evaluated. Horizontal component of the earth pressure and inertia force of the wall were taken into account to evaluate the driving moment, while the subsoil reaction force, the weight of the wall, vertical component of the earth pressure and the mobilized resistance by the top nailings were considered to evaluate the resisting moment against overturning.

It was assumed in the analysis that the center of the rotation of the wall was fixed at the center of the footing, and sum of the earth pressure would act at the middle third of the wall height. Based on the analysis

Table 1. Summary of threshold moment.

Case name	Soil strength	Mobilized resistance by top nailing	Threshold moment (N/m)
peak1	ϕ_{peak}	0.242 kN/m (Evaluated)	331.8
res1	ϕ_{res}	0.183 kN/m (Evaluated)	162.5
peak2	ϕ_{peak}	0.400 kN/m (Measured)	277.55
res2	ϕ_{res}	0.400 kN/m (Measured)	263.55

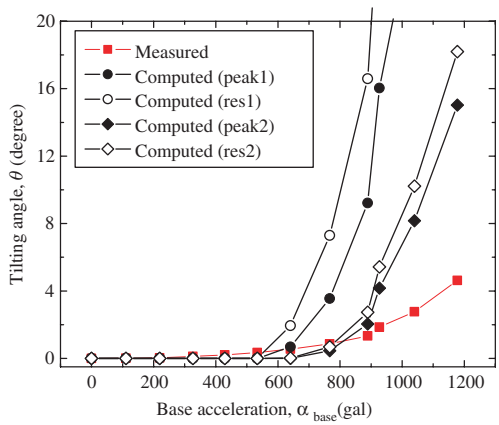

Figure 9. Comparison of tilting displacements.

of the model test in case without nailing, the resultant subsoil reaction was assumed to act at the point having the horizontal distance of 112 mm from the heel of the footing. In this analysis, measured value of the maximum mobilized resistance by top nailings, which was indicated in Fig.5, was also used because the computed ones using the equation in Fig.6 were different from the measured ones. Results and conditions of the pseudo static analysis are summarized in Table.1.

The computed tilting displacements of the wall are compared with the measured one in Fig.9. All the computed displacements were much larger than the measured one. This difference was possibly caused by overestimation of the driving moment. In this analysis, dynamic response of the retaining wall and backfill layer was not taken into account (i.e. the wall and backfill acceleration were set equal to the base acceleration). However, actual responses of the wall and the backfill were decelerated with the wall movement. In this point of view, further improvement on the evaluation of the driving moment is required.

5 CONCLUSIONS

Based on the results and analysis of the model test on retaining wall reinforced with large diameter soil nailings(LDNs), following conclusions were achieved;

1) Sliding, tilting displacement and settlement of the backfill in case with LDNs were effectively reduced compared with the case without LDNs.
2) Based on the measurement of the mobilized resistances by LDNs, it was found that the top nailings worked effectively to restrict the formation of the failure plane in the backfill layers, whilethe bottom ones resisted against the bearing capacity failure in the subsoil.
3) Newmark's method with the introduction of the effect of the LDNs was adopted to simulate the model test result. Computed sliding displacements corresponded well with the measured ones, while computed tilting displacements were much larger than the measured ones possibly because of the overestimation of the overturning moment.

REFERENCES

Huang, C.C., 2005, Seismic displacements of soil retaining walls situated on slope, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol.131, No.9, pp.1108–1117

Kato, N., Huang, C.C., Tateyama, M., Watanabe, K., Tatsuoka, F., and Koseki, J. 2002, Seismic stability of several types of retaining walls on sand slopes, *Proc. of 7th International Conference on Geosynthetics*, Vol.1, pp.237–240

Koseki J., Tatsuoka F., Munuf Y., Tateyama M., and Kojima K.,1998, A modified procedure to evaluate active earthpressure at high seismic load, *Special issue of Soils and Foundations*, Japanese Geotechnical Society, pp.209–216

Koseki, J., Kato, N., Watanabe, K., and Tateyama, M., 2004, Effects of subsoil and backfill conditions on seismic displacement of gravity type retaining walls, *Cyclic Behavior of Soils and Liquefaction Phenomena (Triantafyllidis, ed.),Belkema*, pp.665–671

Koseki J., Barthurst, R.J., Guiler, E., Kuwano, J. and Maugeri, M., 2006, Seismic stability of reinforced soil walls, *Proc. of 8th International Conference on Geosynthetics*, Vol.1, pp.51–77

Nakajima, S., Koseki, J., Watanabe, K., and Tateyama, M., 2006, Evaluation of allowable displacements of retaining walls by shaking table model tests, International Conference on Physical Modeling in Geotechnics, Hong-kong, Vol.2, pp.1101–1106

Nakajima, S., Koseki, J., Watanabe, K., and Tateyama, M., 2007, Shaking table model tests on retaining walls with aseismic counter measures, *13th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering*, Kolkata, (accepted)

Newmark, N.M., 1965, Effects of earthquake on dams and embankments, *Geotechnique*, Vol.15, No.2, pp.139–160

Railway Technical Research Institute, 1999, *Design guideline of railway structures Aseismic design*, Maruzen, pp.326 (in Japanese)

Richards, R. Jr., and Elms, D. G., 1979, Seismic behavior of gravity type retaining walls, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol.105, No.4, pp.449–469

Zeng, X., and Steedman., R.S., 2000, Rotating block method for seismic displacement of gravity walls, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol.126, No.8, pp.709–717