Bull. ERS, No. 33(2000)

Preliminary Report on Damage to Retaining Walls Caused by the 1999 Chi-Chi Earthquake

by

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ABSTRACT

Performance of conventional type retaining walls, reinforced-soil retaining walls and reinforced soil embankments during the 1999 Chi-Chi earthquake in Taiwan is reported. It is argued that damage to conventional type retaining walls was caused by permanent ground displacement along surface faults, large-scale slope movement, loss of bearing capacity in the subsoils, excessive inertia force of the wall, and/or insufficient compaction of the backfill. A good performance of anchored retaining walls compared to that of gravity-type retaining walls is reported as well. It is shown that extent of damage to reinforced-soil retaining walls using keystones as a facing was affected by the amount of vertical spacings of the reinforcements under otherwise similar conditions. It is also shown that damage to wrapped-around type reinforced-soil embankment may have been triggered by local failure due to both an excessive vertical spacing of the reinforcements and an insufficient anchoring length of the wrapping reinforcements.

INTRODUCTION

The Chi-Chi earthquake of September 21, 1999, caused serious damage to a number of soil structures in the central part of Taiwan. The authors visited the affected area on December 13 and 14, 1999, in order to investigate damaged/undamaged soil structures, including conventional type retaining walls, reinforced-soil retaining walls and a reinforced-soil embankment, while excluding conventional type embankments and earth dams. This paper reports preliminary results from the investigation, focusing on possible causes for damage to conventional type retaining walls, effects of vertical spacing of reinforcements on the different performance of reinforced-soil retaining walls using keystones as a facing, and possible mechanism of collapse of wrapped-around type reinforced soil embankments. Refer to Fig. 1 for the location of sites reported in this paper.

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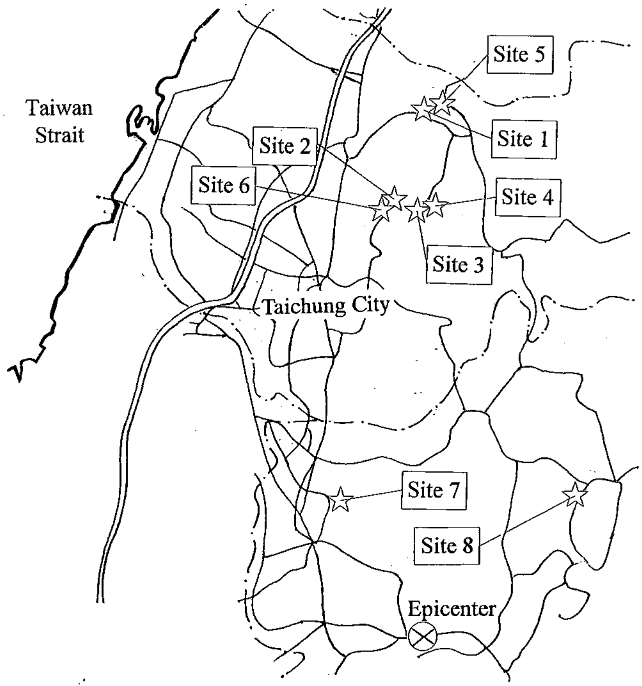

Figure 1. Location of sites reported in this paper

PERFORMANCE OF CONVENTIONAL TYPE RETAINING WALLS

Plate 1 shows collapse of a reinforced concrete (RC) retaining wall with a height of about 7 m at site 1 in Fig. 1, located along a road which leads to the southern side of the Chang-Geng Bridge. Interestingly, adjacent RC walls with similar dimensions survived the earthquake. Since uneven vertical displacement of about 30 cm, possibly caused along a surface fault, was observed in a field located in the neighborhood of the damaged wall, it was estimated that the damage to the wall was caused by this uneven displacement. It should be noted that extensive damage was also observed to houses which were located along the extended line of the boundary of the uneven displacement.

Plate 2 shows damage to an anchored RC retaining wall with a height of about 4 m at site 2 in Fig. 1, located near the top of an embankment for possible housing lots along District Road No. 129. It suffered an opening of about 30 cm at one of the construction joints. Since continuous cracking in the ground surface and other retaining walls located in the lower part of the embankment was observed, a large-scale slope movement, which was induced by the earthquake, was estimated to have caused the damage to the wall shown in Plate 2.

Plates 3a and 3b show damage to a gravity-type concrete retaining wall at site 3 in Fig. 1, located along District Road No. 129. It had been constructed on a small valley, and its height was about 4 m at its tallest section. The whole wall suffered outward permanent displacement and settelement. In particular, the tallest section suffered the largest settlement, which caused a large cracking at the lower part of this section. At the time of the investigation, free ground water level could be observed in the crack (Plate 3b). Therefore, it was estimated that the bearing capacity of subsoils, possibly under saturated conditon, was not enough to support the load from the wall during the earthquake. It seems that, although a transversal drainage pipe to evacuate surface water on the road through a side ditch had been installed in the adjacent backfill at a higher elevation, it may not have been effective in lowering the ground water level.

Plates 4a and 4b show damage to another gravity-type concrete retaining wall at site 4 in Fig. 1, located along District Road No. 129. It had been constructed on a small valley, and its height was about 4 m. The concrete wall partly suffered outward displacement, while an old masonry wall that had been located at the back of the gravity type wall was found to be free from any damage. Since the amount of the backfill between the concrete wall and the old masonry wall was limited, the earth pressure exerted on the concrete wall from the backfill may not have been very large during the earthquake. It was, therefore, estimated that the damage to this concrete wall was due to excessive inertia force of the wall and/or loss of bearing capacity in the subsoils. In relation to this, it should be noted that undamaged portion of the concrete wall had been supported by an H-shaped steel rod (Plate 4b), which may have resulted into such a good performance.

Plate 5 shows extensive tilting of a RC retaining wall with a height of about 5 m at site 5 in Fig. 1, located at the downstream side of Shih-Wui Bridge. Since the backfill was found to be under relatively loose state at the time of the investigation, an excessive seismic earth pressure due to insufficient compaction of the backfill was estimated to have caused the damage. Loss of bearing capacity in the subsoils may have caused the damage as well. Further, it is reported that thrust up of the south bank of Shar-Lian Creek and the instability of the river bank caused collapse of Shih-Wui Bridge (SGRDF, 1999). This may also have caused the damage to the retaining wall although no evidence of permanent ground displacement could be observed at the time of the investigation.

Regarding different performances of different types of retaining walls, a comparison between that of a gravity-type wall and that of an anchored wall could be made at site 2 in Fig. 1. As shown in Plate 6, the gravity-type wall tilted largely, while the anchored wall suffered no damage. It should be noted that these walls were located outside of the area where the afforementioned large-scale slope movement took place. It was, therefore, estimated that the damage to the gravity wall was due to excessive inertia force of the wall and/or loss of bearing capacity in the subsoils, similarly to the case with the gravity wall shown in Plate 4. On the other hand, the anchored wall behaved well, demonstrating the effectiveness of anchoring against seismic loads.

PERFORMANCE OF REINFORCED-SOIL RETAINING WALLS

All the reinforced-soil retaining walls reported herein had a facing consisting of stacked concrete blocks, called as keystones. Each keystone had a height of 20 cm. As reinforcements, geodrids were placed in the backfill and connected to keystones with pins (Fig. 2).

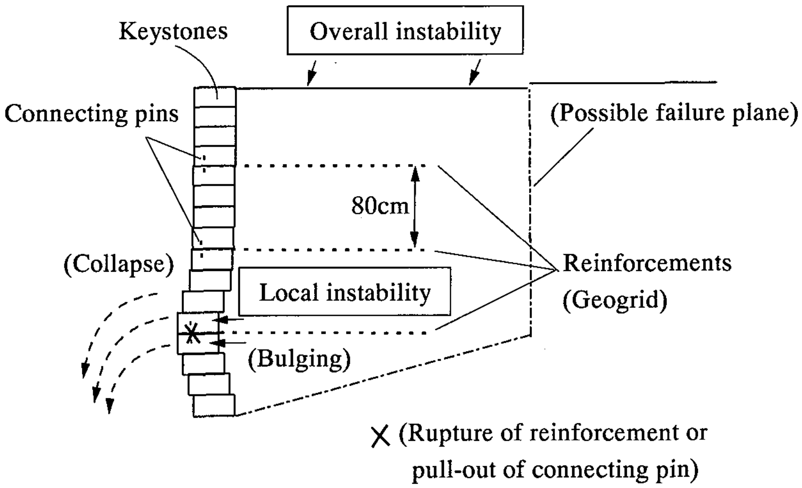

Fig. 2. Schematic figure on damage to reinforced-soil retaining walls using keystone blocks for facing

Plates 7a and 7b show partial collapse of a reinforced-soil retaining wall located along District Road No. 129 at site 2 in Fig. 1, which was located outside of the area where the large-scale slope movement as described in the previous chapter took place (Plate 2). The total height of the wall was about 3 m. Adjacent to this wall, another reinforced-soil retaining wall having a total height of about 3.5 m also partly collapsed in a similar manner. As schmatically shown in Fig. 2, reinforcements were placed in the backfill at a vertical spacing of 80 cm, and it was estimated that the stiffness of the facing and the vertical spacing of reinforcements were not sufficient to resist against seismic earth pressures. The estimated failure mechanism is as follows (Fig. 2):

1) Since the lower part of the facing was subjected to larger earth pressures, this part suffered bulging deformation.
2) The bulging deformation caused rupture of reinforcements and/or pull-out of connecting pins, resulting into local instability by loss of connection between reinforcements and facing.
3) Collapse of the lower part of the facing due to the local instability triggered complete failure of facing.

It should be noted that differential vertical settlement of about 30 cm was observed at the surface of backfill, located at a horizontal distance of about 6 m from the facing. Note also that, at the adjacent wall which suffered partial collapse as described above, cracking with an opening of about 30 cm was observed in the surface of backfill at a horizontal distance of about 5.4 m from the facing. It was, therefore, estimated that overall instability, inducing larger failure zone in the backfill with forming a failure plane, may have been triggered by the complete failure of facing as described in 3). Since the length of the reinforcements is reported to be 2.7 to 3.0m (Huang, 2000), the location of the possible failure plane may be in the unreinforced zone as shown in Fig. 2.

Plates 8a and 8b show, respectively, an undamaged reinforced-soil retaining wall with a height of about 3 m and a masonry wall with a height of about 2.5 m, which were located at site 6 in Fig. 1 along District Road No.129 near the damaged reiforced-soil retaining walls as mentioned above. Since the undamaged reinforced soil retaining wall had reinforcements at a vertical spacing of 60 cm, it was estimated that the different performances of these reinforced-soil retaining walls shown in Plates 7 and 8 are due mainly to different vertical spacings of reinforcements. If these walls were under otherwise similar conditions, the critical value of the vertical spacing to induce a marginal condition of failure was possibly between 60 and 80 cm. It can be also inferred that the seismic stability of these reinforced-soil retaining walls was almost comparable to that of the masonry wall shown in Plate 8b, which suffered only minor cracking.

Plates 9a through 9d show reinforced-soil retaining walls at Chang-Chiun Park in Chung-Hsin New Village, site 7 in Fig. 1. Each section consisted of three facings with an individual height of about 3 m. At one section, part of the middle facing collapsed (Plate 9b). At an adjoining section, the middle facing suffered residual deformation due to bulging at its lower part (Plate 9c). Adjacent to this, there existed an undamaged section (Plate 9d). The conditions of reinforcements at these sections were as follows:

1) At the back of the collapsed facing (Plate 9b), no reinforcement could be found. On the other hand, H-shaped steel rods were found in the backfill, which were estimated to have been used as temporary supports during construction of the facing.
2) At the back of the bulged facing (Plate 9c), reinforcements were placed at a height of 120 cm from the bottom of the wall; i.e., the vertical spacing of reinforcements was 120 cm. One of the reinforcement was estimated not to have been connected to keystones, where the extent of facing deformation was relatively large. On the other hand, the adjoining reinforcement was found to have been connected to keystones, where the extent of the facing deformation was relatively small. In addition, at the latter section, another reinforcement was found to have been placed partly at a height of 200 cm from the bottom of the wall and was connected to keystones.
3) At the back of the undamaged facing (Plate 9d), reinforcements were placed at a height of 40, 120, and 200 cm from the bottom of the wall and were all connected to keystones; i.e., the vertical spacing of reinforcements was 80 cm.

Note also that at another section which was located opposite to the collapsed facing, reinforcements were placed with a vertical spacing of 60 cm. It was, therefore, inferred that the extent of damage was related with the amount of the vertical spacing of reinforcements, similarly to the cases shown in Plates 7 and 8a.

DAMAGE TO REINFORCED-SOIL EMBANKMENT

Plates 10a through 10d show damage to a reinforced-soil embankment at National Chi-Nan University. It consisted of four 10m-high reinforced-soil slopes with three 2m-wide berms (Huang, 2000). As reinforcemnts, geogrids were placed at a vertical spacing of about 80 to 100 cm. To form the surface of slope, about ten sand bags were stacked and wrapped in the reinforcements as schematically shown in Fig. 3.

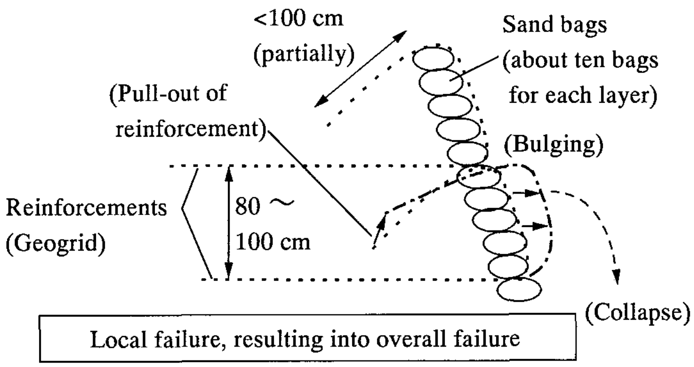

Fig. 3. Schematic figure on damage to wrapped-around type reinforced-soil embankments

At a section that suffered extensive failure (Plate 10a), the anchoring length of wrapping reinforcements was partially smaller than 100 cm (Plate 10b). On the other hand, at another section that suffered limited amount of deformation, being located adjacent to the undamaged one, the anchoring length of all the wrapping reinforcements that were investigated by the authors was larger than 100 cm (Plate 10c).

The estimated failure mechanism is as follows (Fig. 3):

1) Since the lower part of the stacked sand bags was subjected to larger earth pressures, this part suffered bulging deformation. Resistance of this part against bulging was low, due to excessive vertical spacing of reinforcements. In addition, the degree of compaction of backfill during construction process may have been low, due to low contribution of these reinforcements in reducing the horizontal tensile strain of backfill.

2) At a section with insufficient anchoring length of wrapping reinforcements, the bulging deformation caused pull-out of reinforcements, resulting into local failure by loss of anchoring resistance.
3) The local failure triggered collapse of upper embankment, resulting into overall failure including collapse of lower embankment due to extensive loads applied from the collapsed upper embankment.

Further, it should be noted that the cut slope failured during the construction of lower most reinforced slope and that the failure zone was filled with on-site soils, as described in detail and analyzed by Huang (2000). Therefore, the overall failure may have been triggered by local failure in the filled zone. Future studies are required on this issue.

CONCLUSIONS

The preliminary result from the investigation on the performance of could be summarized as follows.

Damage to conventional retaining walls may have been caused by (a) permanent ground displacement along surface faults, (b) large-scale slope movement, (c) loss of bearing capacity in the subsoils, (d) excessive inertia force of the wall, and/or (e) insufficient compaction of the backfill. At one site, good performance of anchored retaining walls was observed, when compared to damage to adjacent gravity-type retaining walls possibly caused by the factors (c) and (d) as listed above.

Extent of damage to reinforced-soil retaining walls using keystones as a facing was affected by the amount of vertical spacing of the reinforcements under otherwise similar conditions. At one site, walls with the vertical spacing of 60 and 80 cm survived the earthquake, while those with the vertical spacing of 120 cm suffered residual deformation due to bulging at their lower part. As an extreme case, walls with no reinforcement collapsed completely. At another site, even a wall with the vertical spacing of 80 cm collapsed partly.

Damage to wrapped-around type reinforced-soil embankment may have been triggered by local failure due to both an excessive vertical spacing of the reinforcements and an insufficient anchoring length of the wrapping reinforcements. Future studies are, however, required on the effects of filling of cut slope to restore failure which had occurred during construction of the embankment.

ACKNOWLEDGEMENTS

The authors wish to thank Dr. Huang, C.C. for his support in planning and conducting the investigation, and Prof. Tatsuoka, F., Dr. Tateyama, M. and Messrs. Matsuo O., Nakamura, S., Sawada, R., Uchimura, T. and Watanabe, K. for their help in providing and assembling information on the earthquake damage.

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Plate 1. Collapse of RC retaining wall due to permanent ground displacement along fault (along a road on the south of Chang-Geng Bridge; site 1 in Fig. 1)

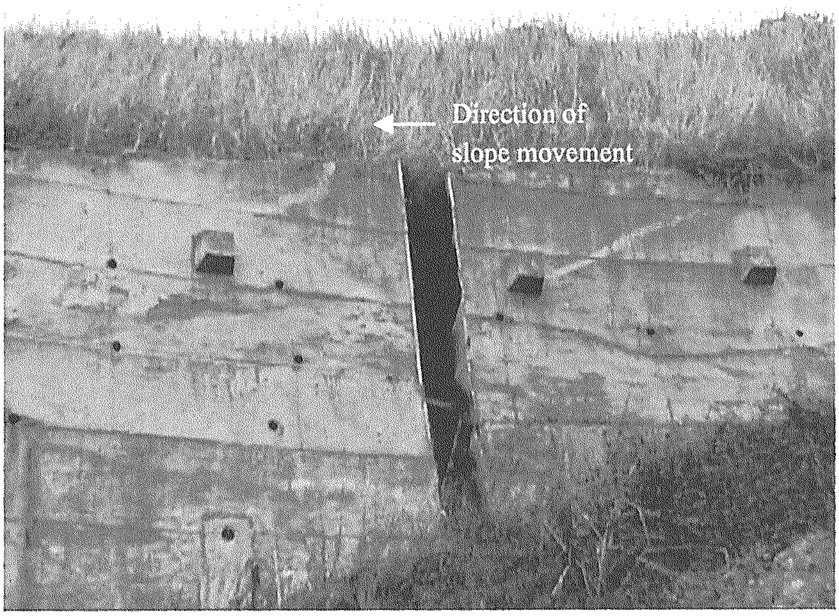

Plate 2. Opening at construction joint of anchored RC retaining wall due to large-scale slope movement (at an embankment along District Road No. 129; site 2 in Fig. 1)

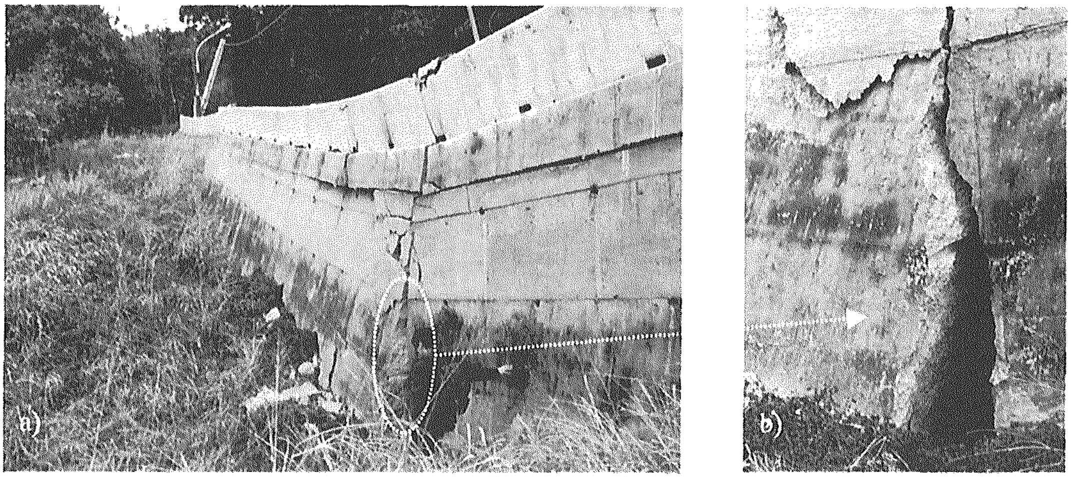

Plate 3. Damage to gravity type retaining wall due to failure of subsoil layers (along District Road No. 129; site 3 in Fig.1)

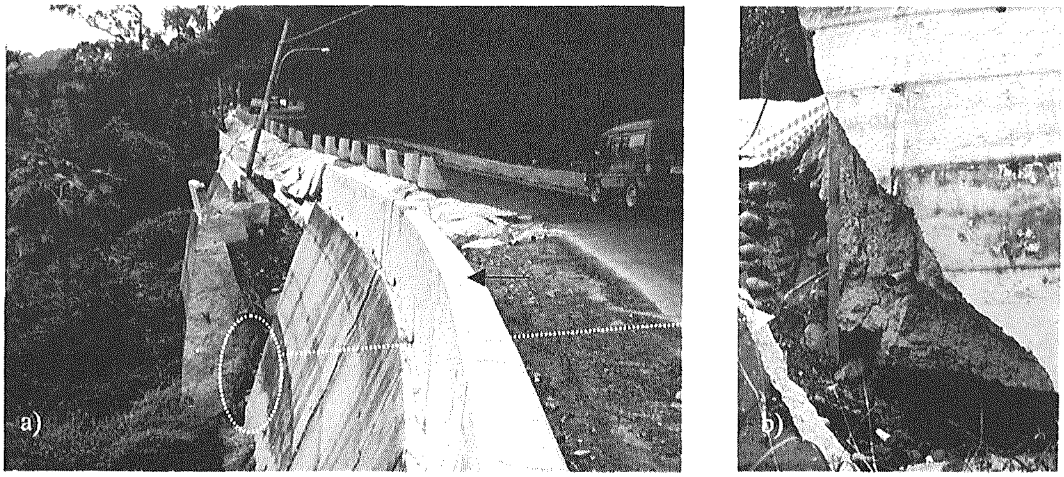

Plate 4. Damage to gravity type retaining wall due to excessive inertia force and/or loss of bearing capacity (along District Road No.129; site 4 in Fig.1)

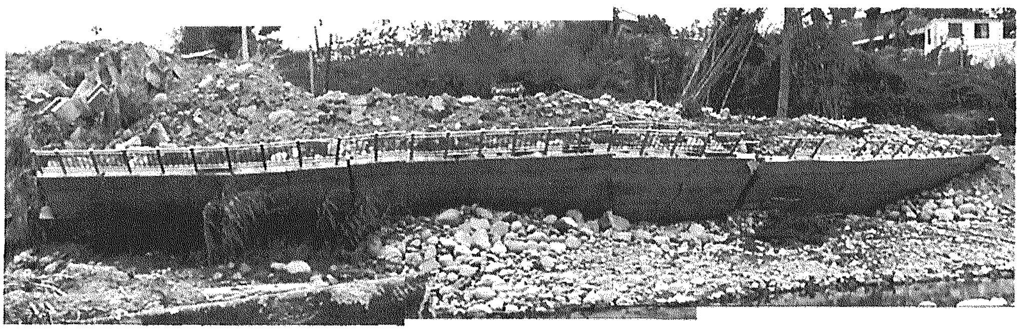
Plate 5. Large tilting of RC retaining wall (near Shih-Wui Bridge; site 5 in Fig.1)

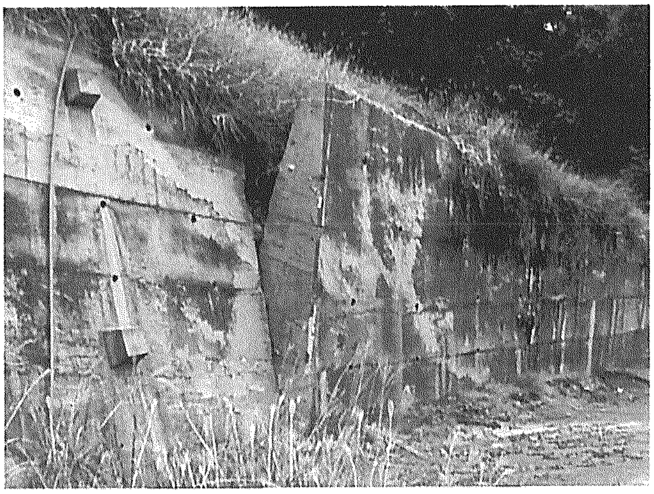

Plate 6. Tilting of gravity type retaining wall neighboring undamaged anchored retaining wall (at an embankment along District Road No. 129; site 2 in Fig. 1)

Plate 7. Damage to reinforced-soil retaining wall using keystones as a facing (along District Road No. 129; site 2 in Fig. 1)

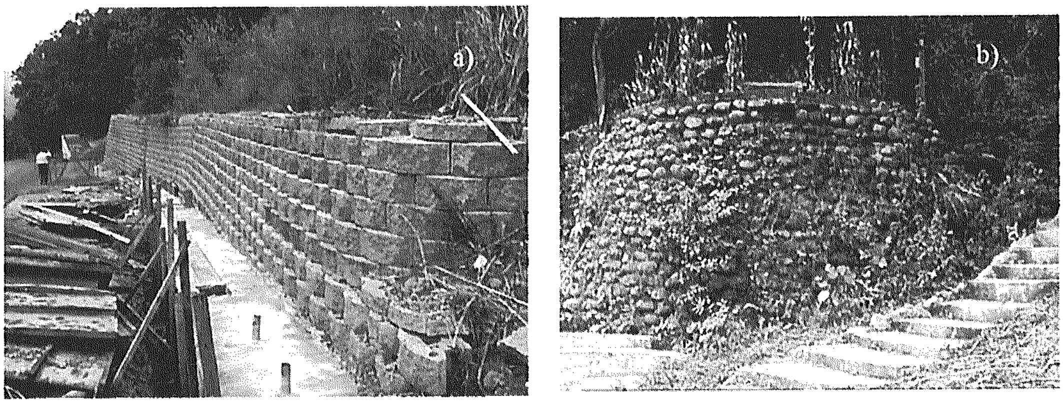

Plate 8. Undamaged reiforced-soil retaining wall and adjacent masonry wall (along District Road No. 129; site 6 in Fig. 1)

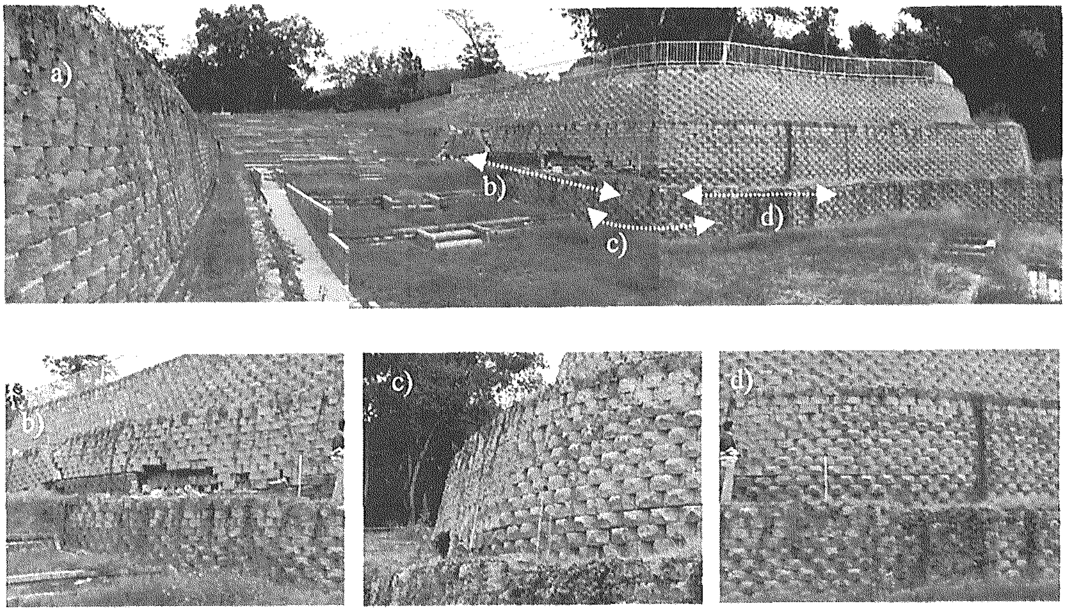

Plate 9. Different performances of reinforced-soil retaining walls
(at Chang-Chiun Park in Chung-Hsin New Village; site 7 in Fig. 1)

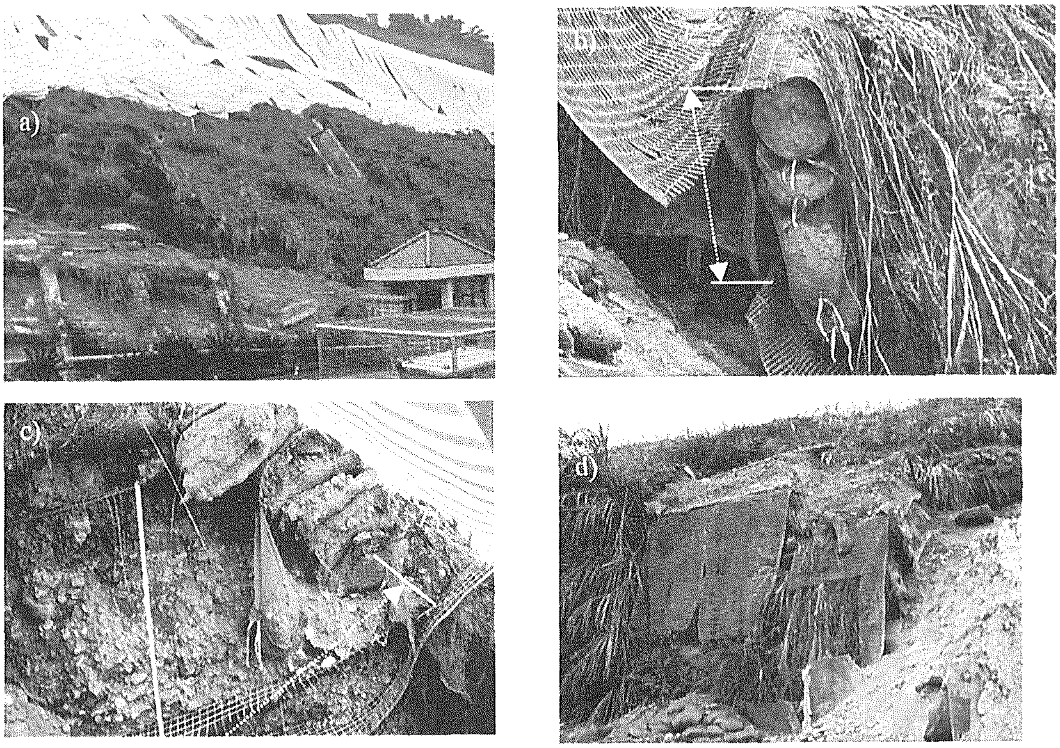

Plate 10. Damage to wrapped-around type reinforced-soil embankment
(at National Chi-Nan University; site 8 in Fig. 1)