한국토목섬유학회논문집 제3권 1호 2004년 1월 pp. 27 ~ 36

# Geosynthetic-Reinforced Segmental Retaining Walls in Tiered Arrangement - Case Study and Field Trial Wall Instrumentation

## 다단식 보강토 옹벽의 설계 - 사례연구 및 시험시공

*1 Yoo, Chung-Sik 유충식 *2 Jung Hyuk-Sang 정혁상


### 요 지

본 논문에서는 다단식보강토옹벽의 해석과 시험 시공을 통해 안정성을 검토한 내용을 다루었다. 한계평형에 근간을 둔 현행의 설계기준에 근거하여 설계된 각기 다른 네 개의 현장 옹벽에 대해 내・외적 안정성 대한 안정성 검토를 실시하였으며, 국외 보강토옹벽의 설계법인 FHWA 및 NCMA의 설계에 의거한 다단식보강토옹벽의 시험시공을 통해 상・하단 옹벽의 상호작용에 의한 거동에 대해 분석을 실시하였다. 본 논문에서는 연구결과를 토대로 다단식보강토옹벽의 역학적 거동을 이해하고, 설계 및 시공의 주안점을 언급하였다.


**Keywords :** geosynthteic-reinforced segmental retaining walls, tiered wall, geogrid, field instrumentation


### Abstract

This paper presents the results of stability analyses on soil-reinforced segmental retaining walls in a tiered arrangement. Four different walls were examined to investigate the appropriateness of their designs within the context of the current design guidelines based on limit equilibrium. Slope stability analysis against the compound failure mode, which is frequently ignored during design, was also performed based on the method recommended by FHWA design guidelines. Also presented are the results of instrumentation on a full-scale field trial wall constructed as part of this study. The implications of the findings from this study are discussed.

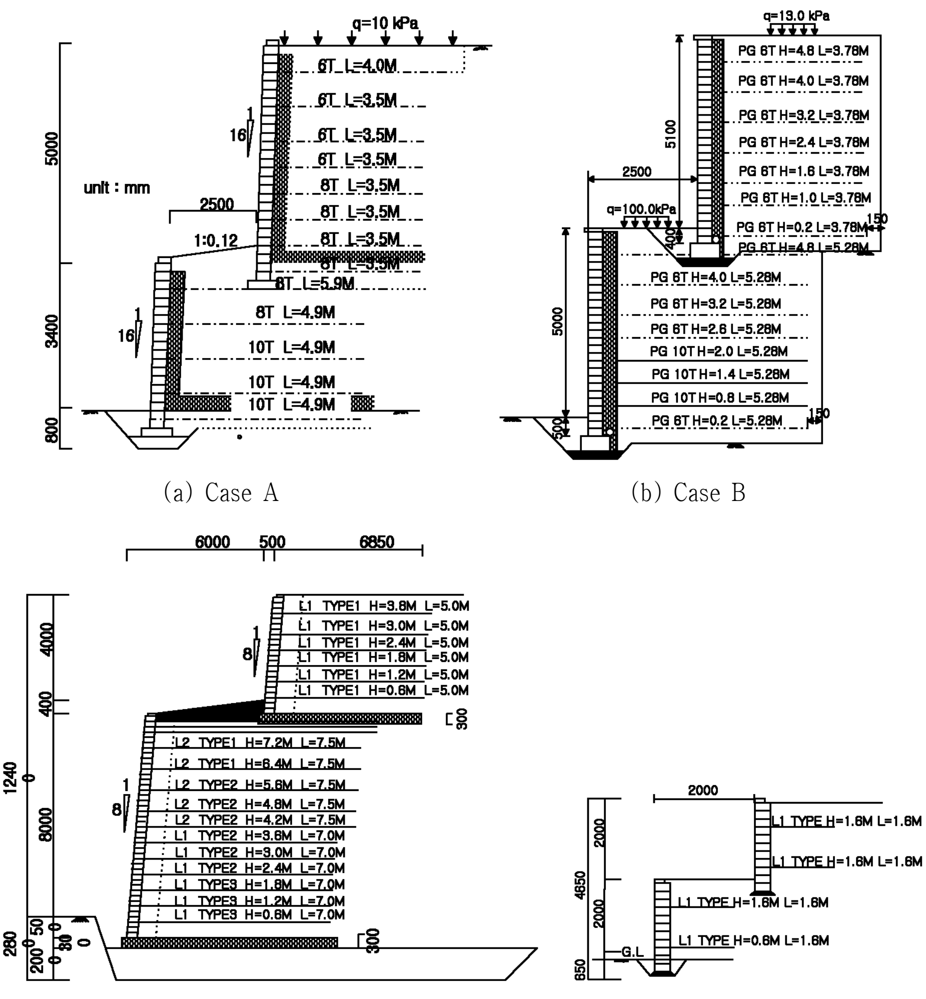

## 1.Introduction

The geosynthetic-reinforced segmental retaining wall systems have become increasingly popular worldwide since its first appearance in the early 1980's. Several benefits of the SRW systems include sound performance, aesthetics, cost and expediency of construction. The mortarless construction and small size of modular facing blocks provide great freedom in constructing walls with complex geometry under unfavorable site conditions. The flexible nature of the SRW systems has also been proven to be effective for sites under earthquake threat (Tatsuoka et al. 1997). Although the currently available limit equilibrium-based design approaches, i.e., NCMA design guideline (Collin 1997) and FHWA design

*1 성균관대학교 토목환경공학과 교수 (Professor, Dept. of Civil & Envir. Engrg. Sungkyunkwan Univ.)
*2 성균관대학교 토목공학과 석사과정 (Graduate Student. Dept. of Civil & Envir. Engrg. Sungkyunkwan Univ.)

guidline (Elias and Christopher 1997), are considered to be conservative on account of several assumptions regarding the wall behavior, much still needs to be investigated to bridge the gap between the theory and the practice. In addition, despite the fact that many geosynthetic-reinforced soil walls have been safely constructed and are performing well to date, there are many areas that need in-depth studies to develop a more generalized design approach that will help safely construct SRW systems under more aggressive and complex boundary conditions.

There are many situations where reinforced soil walls are constructed in a tiered configuration for a variety of reasons such as aesthetics, stability, and construction requirement, etc. Such a tiered configuration, however, tends to give design and construction engineers unnecessarily high confidence in terms of wall performance, especially for walls with an intermediate to large offset distance (D), D = 0.3 to 1.0 times lower tier height. A numerical investigation by Yoo and Kim (2002a), however, revealed that for cases with such a range of offset distance, the interaction between the upper and the lower tiers is not insignificant. Furthermore, as will be discussed in a subsequent section, the NCMA and the FHWA design guidelines tend to yield somewhat different results in the design calculations for a given condition on account of different design assumptions




(a) Case A (b) Case B

(c) Case C (d) Case D

Figure 1. Design cases analyzed

adopted in the two design approaches. In-depth studies are, therefore, required to improve the current design criteria for SRWs in a tiered configuration

In this study, a 5.6-m high full-scale SRW was constructed and instrumented with the aim of investigating the behavior of a tiered SRW at a full-scale level. This paper describes a review of design practice concerning this type of SRW systems, construction and instrumentation of the full-scale test wall, details of the observed performance, and finally design implication.

## 2. Review of Design Cases

For the purpose of making a direct comparison between the aforementioned two design approaches and of reviewing the design practice exercised in Korea, especially for SRWs in a tiered configuration, four tiered SRWs constructed in Korea are analyzed. Figure 1 shows the walls examined. As can be seen, the offset distance ranges 0.7～0.9 times the lower tier height (H1) with the reinforcement lengths of 0.7～1.3 times the respective H1. Table 1 summarizes the results of the external and the internal calculations. It should be noted that an internal friction angle of =30 and a unit weight of =18 kN/m$^3$ were used for the backfill soils as used in their original designs. This is justified since the purpose of the comparisons was not to examine the performance of the walls but to demonstrate the differences between the two

Table 1. Results of external and internal stability calculations for field eases

| Wall | NCMA | | | | FHWA | | | |
|---|---|---|---|---|---|---|---|---|
| | External | | Internal | | External | | Internal | |
| | $FS_{bsl}$ | $FS_{ot}$ | $T_{i,max}$(kN/m) | $L_e$ | $FS_{bsl}$ | $FS_{ot}$ | $T_{i,max}$ (kN/m) | $L_e$ |
| A | 3.13 | 8.87 | 19.7 | 3.36 | 1.27 | 2.13 | 30.5 | 4.05 |
| B | 2.19 | 4.53 | 19.8 | 1.50 | 1.23 | 1.76 | 36.9 | 2.51 |
| C | 2.79 | 6.09 | 16.0 | 2.43 | 2.02 | 5.01 | 37.5 | 3.87 |
| D | 1.28 | 3.54 | 9.9 | 0.05 | 1.67 | 1.65 | 19.7 | 0.3 |

Note: 1) $FS_{bsl}$ = factor of safety against base sliding
2) $FS_{ot}$ = factor of safety against overturning
3) $T_{i,max}$ = maximum reinforcement force within lower tier
4) $L_e$ = embedded length beyond active zone for top-most reinforcement in lower tier
5) For Wall D. FHWA design guideline assumes no interaction.

Table 2. Results of external and internal stability calculations for test well

| Layer | Elev (m) | Internal stability | | | | | |
|---|---|---|---|---|---|---|---|
| | | $FS_{to}$ | | $FS_{po}$ | | $FS_{isl}$ | |
| | | NCMA | FHWA | NCMA | FHWA | NCMA | FHWA |
| 1 | 0.2 | 0.91 | 0.68 | 47.63 | 35.19 | 2.88 | 1.23 |
| 2 | 0.6 | 0.95 | 0.71 | 45.66 | 35.85 | 2.98 | 1.27 |
| 3 | 1.0 | 0.98 | 0.73 | 43.69 | 34.16 | 3.09 | 1.31 |
| 4 | 1.4 | 1.03 | 0.76 | 41.71 | 33.14 | 3.21 | 1.36 |
| 5 | 1.8 | 1.07 | 0.79 | 39.73 | 32.13 | 3.34 | 1.41 |
| 6 | 2.2 | 0.90 | 0.66 | 30.73 | 25.00 | 3.48 | 1.46 |
| 7 | 2.8 | 0.80 | 0.58 | 23.19 | 19.69 | 3.71 | 1.55 |
| 8 | 3.4 | 0.75 | 0.54 | 18.29 | 16.10 | 3.98 | 1.65 |
| 9 | 4.2 | 0.73 | 0.51 | 13.90 | 12.91 | 4.41 | 1.81 |
| 10 | 5.0 | 0.74 | 0.51 | 10.67 | 10.57 | 4.94 | 1.99 |
| External stability | | $FS_{be}$=32.27 $FS_{st}$=3.15 $FS_{ot}$=3.34 | | | | | |
| | | $Fs_{be}$=29.68 $FS_{st}$=1.79 $FS_{ot}$=2.90 | | | | | |

design approaches.

As seen in Table 1, the factors of safety against direct sliding based on the FHWA design approach appear to be approximately 40 to 70 % smaller than those based on the NCMA design approach except for wall D. In fact, the walls A, B, and C did not satisfy the base sliding requirements specified by the FHWA design approach. A similar trend is observed for the overturning. The results of the internal stability calculations also show that the FHWA design approach gives larger maximum reinforcement forces than the NCMA design approach. In addition the FHWA design approach tends to yield larger embedment lengths beyond active failure surfaces than the NCMA design approach, which in turn result in larger pullout capacities. It should be noted that the maximum reinforcement force for the lower tier is presented instead of the factor of safety against tensile overstress failure to allow for a direct comparison between the two approaches. Likewise, the embedment length beyond the active zone for the top most layer of lower tier is used for the pullout check. It should be noted that the wall D, which does not satisfy the external and the internal stability requirements specified by NCMA design approach, actually failed sometime after the wall completion. A compound mode of failure was identified, in which a failure surface initiated at the toe of the lower tier and then propagated back behind the reinforced soil block of the upper tier (Park 2000).

The comparison of the results from the two design approaches indicates that the FHWA design approach tends to give rather smaller factors of safety for both the external and the internal stability calculations than the NCMA design approach. Apart from the different design earth pressures adopted in each of the design approaches, the differences in the calculation models (i.e., the way in which the upper tier is treated) adopted in the two design approaches may also be responsible for the discrepancies. On account of the limited number of cases considered in this study, general conclusions cannot be drawn from these comparisons. Further investigation is required to fill the gap between the two design approach

## 4. Field Instrumentation

### 4.1 Field wall

The full-scale field test wall was constructed at the Geotechnical Experimentation Site (GES) in Sungkyunkwan University in Korea. The ground at the site consists of approximately 3.0 m of miscellaneous fill material including sand and gravel. Underlying the fill layer is a 3.0 to 4.0-m-thick alluvial sandy clay deposit followed by a 6.0 to 8.0-m-thick weathered granite residual soil overlying a slightly weathered granite rock stratum. The incompetent foundation condition raised some concerns over possible excessive settlement of the test wall. The upper 3-m-thick foundation soil was therefore replaced and compacted with more competent weathered granite soil.

The 5.6-m-high test wall consists of two tiers, i.e.,

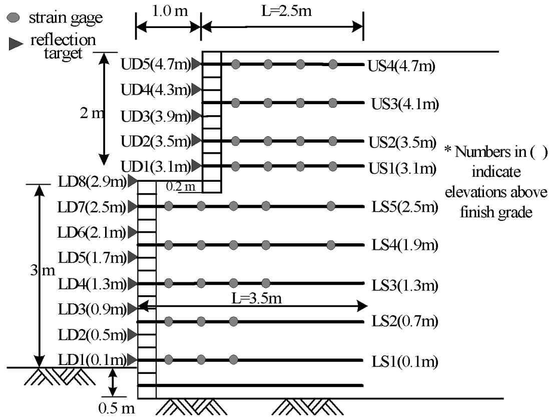



Figure 2. Instrumented wall

a 3.4-m-high lower tier and a 2.2-m-high upper tier as illustrated in Figure 2. The offset distance between the two tiers is 1.0 m with no pre-batter angle of the wall face. Eleven layers of reinforcement were placed at a maximum vertical spacing of 0.6 m. For each tier, the reinforcement length ratio with respect to the respective tier height including the embedded depth was kept constant approximately at 1.0. The reinforcement distribution was selected so as to satisfy all the performance requirements specified by both the NCMA and FHWA design guidelines but with rather marginal factors of safety against the internal stability requirements as shown in Table 2. This was intended to induce large strains in the reinforcement and the backfill soil so that the interaction between the two tiers can be more clearly addressed.

The wall was constructed using modular blocks (450～330 mm in plan 200 mm in height) having a compressive strength of 21 MPa with a maximum water absorption of 68% for standard weight aggregates. Shear keys are formed on each block to transfer shear between two blocks. Biaxial geogrid with a tensile strength of 55 kN/m at a maximum strain of 12.5% was used as reinforcement for the test wall. The geogrid has interwoven structures made of high modulus polyester yarns, covered by an additional protective layer of PVC. The nominal thickness and the aperture size are 1.0 mm and 20 mm, respectively. A series of wide width strip tensile strength test (ASTM D 4595) were conducted in an attempt to identify stress-strain-strength characteristics of the geogrid. Specimens of 0.2-m-wide and 1.2-m-long were used during the tests with a loading rate of 103%/min as specified in ASTM D 4595. The connection between the facing and the geogrid was ensured primarily though the shear keys with no mechanical connector. No connection strength properties were available.

Construction of the wall followed the general procedure for that of a typical geosynthetic-reinforced segmental retaining wall. Weathered granite soil available near the GES was used as the backfill material. Laboratory tests performed on the backfill material revealed that the percents passing the No. 4 and No. 200 sieves were approximately 93% and 9%, respectively, with the coefficients of uniformity and curvature of C*u*=12 and C*c*=1.4. Based on the Unified Soil Classification System (USCS) the soil was classified as SW-SM. According to the standard Proctor test (ASTM D 698) the maximum dry unit weight was $\gamma_{d,max}$=19 kN/m$^3$ with the optimum water content of $w_{opt}$=7%. A crushed gravel with the mean diameter D50=15 mm was used to infill the spaces between adjoining modular block units. The gravel was also extended to a distance of 300 mm behind the facing column to create a drainage layer without any filter layer.

Construction specification required that the backfill material be compacted to a minimum of 95% of standard Proctor. A walk-behind vibrating drum roller was used to compact the backfill material located within 1 m of the back of the facing column while a 10-ton drum roller was used elsewhere. After compaction, the field unit weight was checked using the sand cone method, and a compacted unit weight of 19 kN/m$^3$ was measured.

A series of consolidated-undrained triaxial compression tests with pore pressure measurements were performed to evaluate the shear strength properties of the compacted backfill soil. The soil was placed in a mold and compacted to a bulk unit weight of 19 kN/m$^3$, which corresponded to the field density. On account of the size of specimens (35 mm in diameter 75 mm in height) particles retained on the No. 4 sieve, if any, were removed during preparation of the test specimens. Considering the relatively large percent passing the No. 4 sieve of 93%, however, it was thought that the removal of particles retained on the No. 4 sieve did not have any significant influence on the test results. The results of the triaxial tests are presented in Figure 3. The estimated internal friction angle at a density corresponding to the as-compacted state was approximately $\phi'$=35 with a cohesion intercept of c'=5 kPa. The cohesion intercept was considered as apparent one since it was obtained by linear fitting to a curved failure envelope, and was disregarded in relevant design calculation

## 4.2 Instrumentation

The monitoring program was focused more on displacement measurements. Monitored items included horizontal deformation at the wall face and strains in the reinforcement. Three instrument arrays were installed in the wall at the locations shown in Figure 2. Table 3 gives the details of types and locations of the instruments. Measurements were collected during construction over a two-week period of June 25, July 7, 2002, and continued for more than six month after the wall completion.

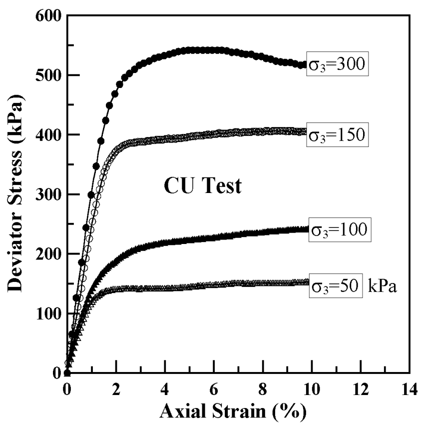
Horizontal deformation at the wall face was monitored using a 3D total station (MONMOS Model NEA2A) having an accuracy of 0.1 mm. Reflection targets were attached onto the wall facing units at a number of pre-selected locations (Fig. 4). A fixed benchmark was placed on a building located near the site. Readings were then taken during successive stages of wall construction.

Localized strains in the reinforcement were measured using high elongation bonded resistance

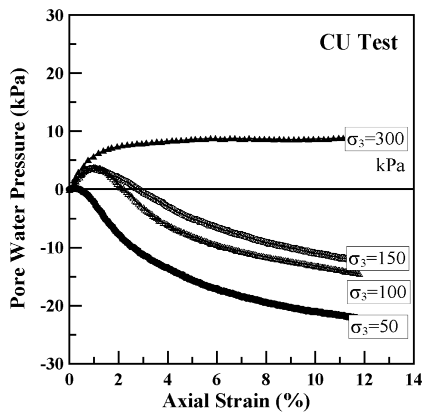
Table 3. −Location of instruments

| Array/Instrumentation | Location |
|---|---|
| Array A, B, C | −1.0, 0, +1.0 from wall center line (m) |
| Optical surver target on Wall facing column | 0.1, 0.5, 0.9, 1.3, 1.7, 2.1, 2.5, 2.9, 3.1, 3.5, 3.9, 4.3, 4.7m above wall base |
| Strain gage | 0.5, 1.0, 1.5m behind wall facing (LS1−LS3)<br>0.5, 1.0, 1.5, 2.5m behind wall facing (LS4−LS5)<br>0.5, 1.0, 1.5, 2.5, 3.0m behind wall facing (US1−US4) |

Note: LSn = strain gages installed on $n^{th}$ reinforcement layer in lower tier
USn = strain gages installed on $n^{th}$ reinforcement layer in upper tier




(a) stress−strain curve




(b) pore pressure−strain curver

Figure 3. Results of triaxial compression tests on backfill soil

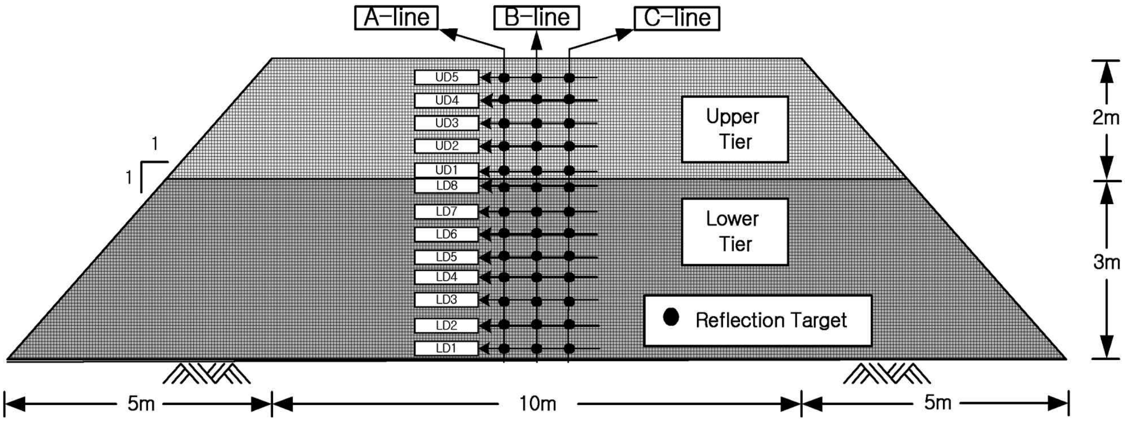



Figure 4. Front view of full-scale test wall

strain gages manufactured by Tokyo Sokki Kenyujo Company. (Model YFLA-5-1L) installed at 36 locations for each instrumentation array. These gauges have a gauge length and resistance of 5 mm and 120 Ohms, respectively. Strain gages were bonded directly to the polyester strands of the geogrid using a high elongation cyanoacrylate adhesive (Model CN manufactured by Tokyo Sokki Kenyujo Company ). N-1 coating material and VM tapes made of Chloroprene rubber and Butyl rubber, respectively, were additionally used for moisture-proofing. Strain gauge readings were collected using a readout unit (Vishay Intertechnology Model P-3500) having an application range of 19999with an accuracy of 3. Photographs of strain gauge installation are shown in Figure 5.

## 4.3 Horizontal deformation at wall face

### 1) Lower tier

Figure 6 illustrates the patterns of horizontal deformation of the lower tier during various stages of lower tier construction. It should be noted that the data recorded in all three target arrays are presented. As can be seen in Figure 6, the deformation profiles at the completion of the lower tier tend to follow a typical pattern for a soil reinforced segmental retaining wall with a maximum of approximately 1.0% of the wall height occurring at the mid-height. An important observation is that the wall base exhibited lateral movement as great as 1.0 cm. Although no detailed study on the geotechnical properties of the foundation soil was conducted, such a large horizontal movement at the wall base may

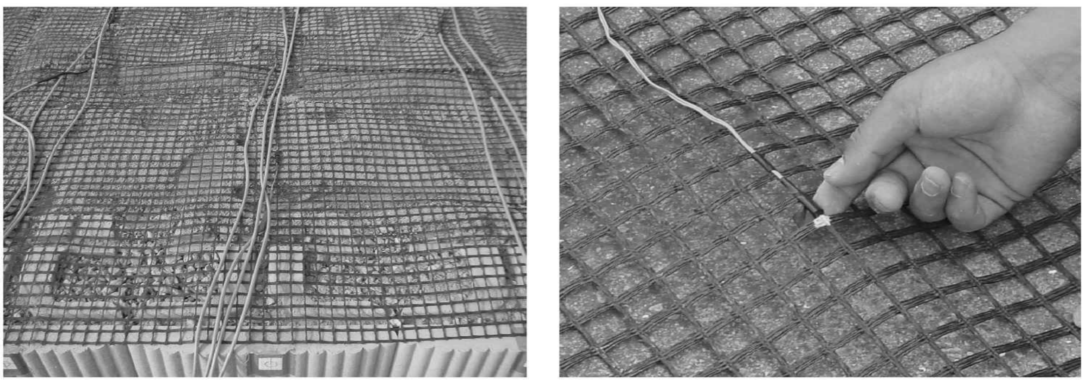
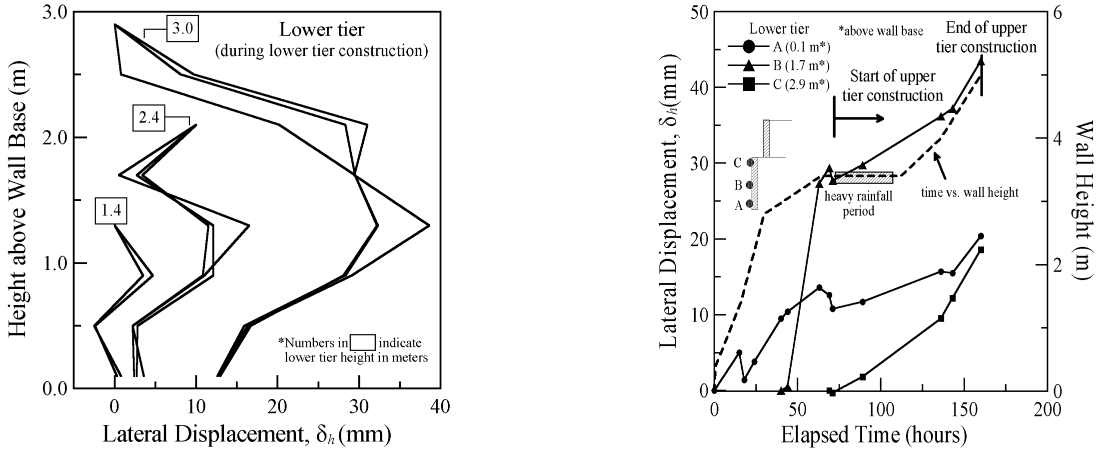


Figure 5. Photographs of strain gauge installation

have been related to insufficient toe resistance arising from the relatively flexible sandy clay soil deposit despite the ground improvement effort. Rowe and Skinner (2001) have reported significant toe movement for a reinforced soil wall constructed on a compressible foundation based on a numerical study. In addition, significant toe forces have been measured in full-scale test walls by Bathurst and Benjamin (1990), and its contribution to stability of reinforced soil walls has been demonstrated by Jewell et al. (1992). The results of the present study together with those from the aforementioned studies imply that larger horizontal wall deformation at the wall base may occur for reinforced soil walls situated on a less competent foundation.

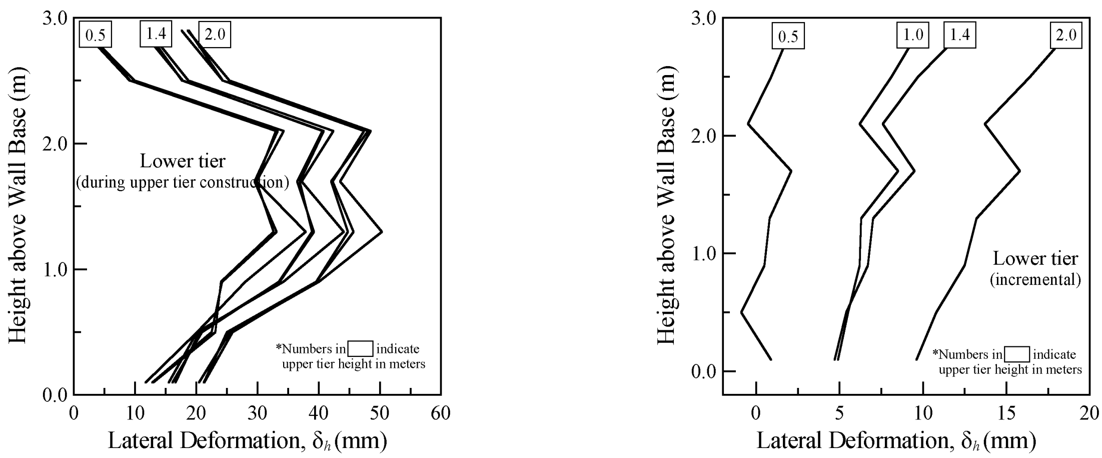
Illustrated in Figure 7(a) are the horizontal deformation profiles of the lower tier at various stages during the upper tier construction. As expected, the interaction between the two tiers resulted in significant increases in the wall deformation with the increase being concentrated more on the upper half of the wall. Upon completion of the upper tier, the maximum horizontal wall deformation increased by approximately 50%. The effect of the upper tier construction on the lower tier becomes more evident in the incremental deformation profiles. As seen in




(a) displacement profile

(b) evolution of wall displacement

Figure 6. Horizontal wall deformation upon lower tier completion




(a) profile

(b) h vs. time

Figure 7. Horizontal wall deformation profiles of lower tier during upper tier construction

Figure 7(b), the incremental profiles show larger movements at the top, due primarily to the additional earth load by the upper tier. A possible strategy to minimize the effect of upper tier on the lower tier may be to increase the reinforcement density as well as the length of reinforcement in the upper portion of the lower tier, as discussed by Yoo and Kim (2002a) and Yoo (2003). The rigid body translation of the entire wall observed in this figure may also have been caused by the rather compressible foundation.

### 2) Upper tier

The horizontal deformation profiles of the upper tier are presented in Figure 8. A salient feature shown in this figure is that significant movements occurred at the wall base due primarily to the lateral movements of the lower tier. This trend implies that the interaction between the two tiers not only affects the performance of the lower tier but also influences the upper tier performance. The current limit equilibrium-based design approaches, however, do not address such an issue as they suggest the upper tier be designed as if it were a single wall. Provision must therefore be provided for a upper tier to have adequate resistance against additional load arising

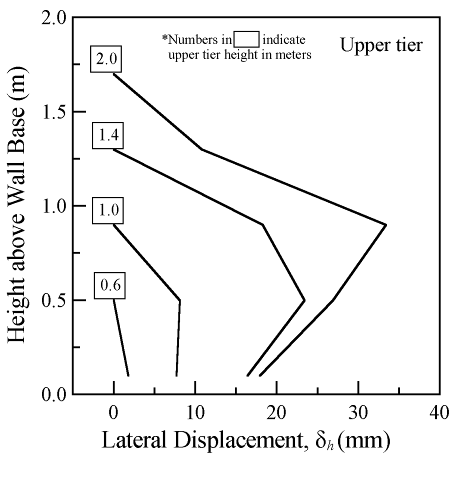



Figure 8. Horizontal deformation of upper tier during upper tier construction

from such an interaction. A numerical study by Yoo and Kim (2002a) has demonstrated that a denser and longer reinforcement distribution in the bottom 1/3 of the upper tier would help resist the additional load arising from the interaction between the upper and lower tiers.

## 5. Conclusions

This paper presents the performance of a geosynthetic-reinforced segmental retaining wall in a tiered arrangement. For the purpose of identifying fundamental principles involved in the mechanical behavior of SRWs in a tiered arrangement and to form a database for use in the improvement of the current design approaches, a 5.6-m-high full-scale field wall was constructed and instrumented.

The results indicated that the interaction between the upper and the lower tiers significantly increases not only the horizontal deformation of the lower tier but also that of the upper tier. Also measured was significant horizontal wall movement at the wall base due in large part to the relatively compressible nature of the foundation despite the foundation improvement effort. There exists some discrepancies between the measured reinforcement forces and the calculated ones based on the current design approaches, especially in the upper and bottom layers.

The wall exhibited significant post-construction deformation, as great as 100% of the construction-induced deformation, which may have been caused by the time-dependent settlement of the foundation. The wall deformation continued to increase during the heavy rainfall periods. The measured strains during the events of heavy rainfalls correlated fairly well with the data pertaining to the wall deformation during the heavy rainfall period, suggesting that there exist a strong correlation between the wall deformation and the heavy rainfall. On account of the limited data available, in-depth studies are required to draw general conclusions regarding the causes of post-construction deformation including the effect of rainfall.

## Acknowledgements

The author wishes to express sincere gratitude for Mr. Hyuk-Sang Jung's effort during field instrumentation.

## References

1. Bathurst, R.J. 1990. Instrumentation of Geogrid-Reinforced Soil Walls, *Transportation Research Record, No. 1277, Soils, Geology, and Foundations, Modern Geotechnical Methods: Instrumentation and Vibratory Hammers*, pp. 109～116.
2. Bathurst, R.J. 1992. Case study of a monitored propped pannel wall, *Proc. 5th Int. Symp. on Geosynthetic-Reinforced Soil Retaining Walls*, Balkema, Rotterndam, Brookfield, pp.159～166.
3. Bolton, M.D. 1986. The strength and dilatancy of sands, *Geotechnique*, London, U.K., 36(1), pp.65～78.
4. Chou, N. N. S. & Wu, J. T. H. 1993. Investigating performance of geosynthetics reinforced soil walls, *Rep. No. CDOT-UCD-R-93-21*, Colorado Dept. of Transportation.
5. Collin, J.G. & Berg, R.R. 1994, Permanent geogrid reinforced soil retaining walls US experience, *Proc. Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls*, Tatusoka and Leshchinsky (eds), pp.61～68.
6. Ho, S.K. & Rowe, R.K. 1996. Effect of Wall Geometry on the Behavior of Reinforced Soil Walls, *Geotextile and Geomembranes*, 14, pp.521～541.
7. Juran, I. & Christopher, B. 1989. Laboratory Model Study on Geosynthetic Reinforced Soil Retaining Walls, *Journal of Geotechnical Engineering*, ASCE, Vol.115, No.7, pp.905～926.
8. Kapurapu, R. & Bathurst, R.J. 1995. Behavior of Geosynthetic Reinforced Soil Retaining Walls Using the Finite Element Analysis, *Computers and Geotechnics*, Vol.17, pp.179～299.
9. Ochiai, Y. & Fukuda, N. 1996. Experimental Study on Geotextile-Reinforced Soil Walls with Different Facings, *Proc. 1st*, *European Geosynthetics Conf.*, Netherlands, pp.113～120.
10. Porbaha, A. & Goodings, D.J. 1997. Centrifuge Modeling of Geotextile-Reinforced Cohesive Soil Retaining Walls, *Journal of Geotechnical Engineering*, ASCE, Vol.122, No.10, pp.840～848.
11. Rowe, R.K. & Ho, S.K. 1997. Continuous Panel Reinforced Soil Walls on Rigid Foundations, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol.123, No.10, pp.912～920.
12. Simac, M.R., Christopher, B. R. & Bonczkiewicz, C. 1990. Instrumented field performance of a 6 m geogrid soil wall, *Proc. 5th Int. Conf. On Geotextiles, Geomembranes and Related Products*, Den Hoedt (ed.), pp.53～59.
13. NCMA, 1997. Design Manual for Segmental Retaining Walls, 2nd Ed., NCMA, Virginia.
14. FHWA, 1997. Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines, *FHWA Demonstration Project 82*, FHWA, Washington, DC, FHWA-SA-96-071
15. Yoo. C. S., & Lee, K. M. 1999. Investigation on Behavior of Reinforced Segmental Retaining Walls, *Journal of Korean Geotechnical Society*, KGS, Vol.15, No.1, pp. 53～62.