Geosynthetics - 7th ICG - Delmas, Gourc & Girard (eds) © 2002 Swets & Zeitlinger, Lisse ISBN 90 5809 523 1

Lessons learned from a 6-year-old geosynthetic reinforced segmental retaining wall

C. S. Yoo, Sungkyunkwan University, Suwon, Kyong-Gi Do, Korea

ABSTRACT: This paper presents the results of an investigation on a 6-year-old geosynthetic reinforced segmental retaining wall, which showed a sign of distress through excessive lateral wall movements. In an attempt to identify possible causes and to provide mitigation measures, a comprehensive investigation was carried out. The results indicated that the wall was not designed to satisfy current design criteria, and that significant movements had occurred during and after construction. Several remedial measures for the wall were recommended with due consideration of the site condition. The implications and the findings of this study are discussed in great detail.

1 INTRODUCTION

Geosynthetic reinforced segmental retaining wall (SRW) systems have become popular since its first appearance in the early 1980's for reasons of performance, aesthetics, cost and expediency of construction. Especially at sites with poor foundation conditions, the SRW systems offer significant technical and cost advantages over conventional concrete retaining walls. In addition, the mortarless construction and small size of modular blocks allow more rapid construction. Economic benefit due to several advantages is that geosynthetic reinforced segmental retaining walls in excess of 1 m in height typically offer a 25 to 45% cost saving over comparable conventional reinforced cast-in-place concrete retaining walls (Bathurst & Simac 1994). Although the currently available limit equilibrium-based design approaches (NCMA 1997, FHWA 1997) are considered to be conservative due to several assumptions adopted regarding the wall behavior, numerous major and minor structural problems have been reported during and after construction, covering a range of minor structural damage to total collapse. Much still needs to be investigated to fill the gap between the theory and the practice.

A 6-year-old geosynthetic reinforced segmental retaining wall exhibited a sign of distress through unexpected large movements. A comprehensive investigation was carried out in an attempt to identify possible causes and to provide mitigation measures to ensure the short and long-term stability. The investigation program was comprised of wall profiling using an optical surveying device and stability analysis of the original design. The results of the wall profiling and the stability analysis provided sound basis as to the wall stability. Based on the results of investigation, possible causes for the excessive wall movements were identified and several remedial measures for the wall were recommended with due consideration of the site condition.

2 SITE CONDITION AND WALL DESIGN

2.1 *Site condition*

The wall was constructed during late 1994 through early 1995 at a site for a chemical processing plant located in the southern part of Korea. Extensive earthwork was involved in preparing the site. The total wall is approximately more than 300-m-long with variable wall height ranging from 3.6 m to 9.8 m. The plan view is shown in Figure 1. Upon completion of the wall construction, two and four-lane roads were constructed immediately behind the north and the east walls. Various facilities were then constructed along the roads.

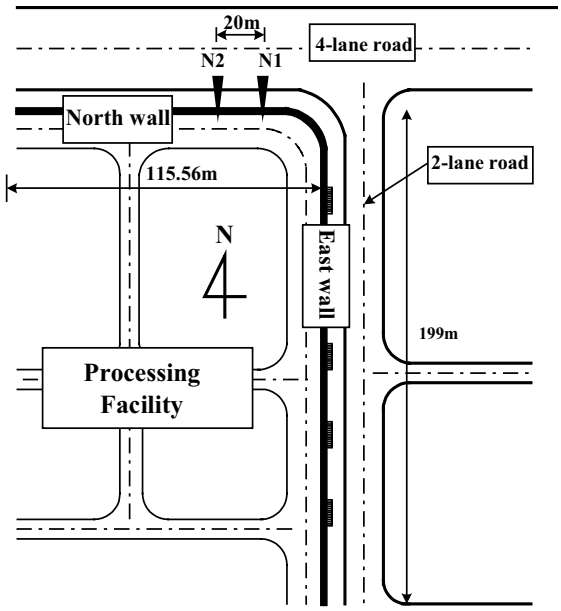
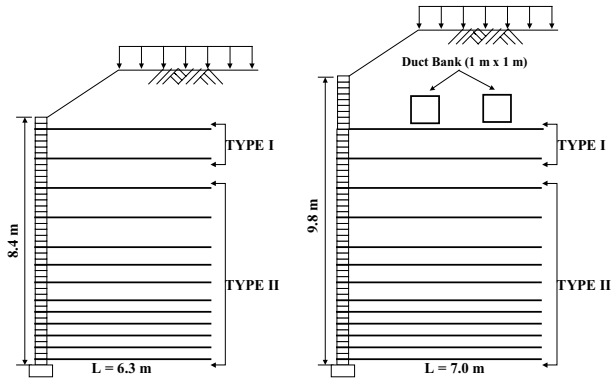


Figure 1. Plan view of site

2.2 *Wall design and construction*

The wall was constructed using modular blocks (520 × 460 mm in plan and 200 mm in height) having a compressive strength of 21 MPa and geogrids available on the market. Although no detailed design documents were available at the time of investigation, available construction documents suggested that the original wall design required a pre-batter of 1/16 (3.5 degrees) through the automatic set-back of the incremental facing blocks. Fiberglass alignment pins (13 mm in diameter and 135 mm in length) were used to transmit shear between the facing courses. Figure 2 illustrates two typical sections of the original design, each for the north and the east walls. Note that the east wall has alternating 8.4-m-high and 9.8-m-high sections in order to accommodate access stairs. As seen in this figure, the primary reinforcement layers of a polyester geogrid (TYPE-I and TYPE-II) with a uniform length of 0.7~0.75 times the design wall height (H) were installed. TYPE-I and TYPE-II reinforcements have an ultimate tensile strength of 55 and 80 kN/m, respectively. A salient feature of the original design for the east wall is that a significant upper portion of the east wall was left unreinforced in

order to accommodate 1 × 1 m duct banks for utility lines running parallel to the wall. No provisions were made to protect the unreinforced zone by any means in the original design. After encountering problems during construction described later in this section, the original design of the east wall was modified.



(a) north wall (b) east wall

Figure 2. Typical sections of north and east wall

No detailed information was available regarding the backfill soil and the compaction procedure adopted during construction. Laboratory tests performed on the soil samples taken from the slope portion revealed that the percent passing the No. 200 sieve is approximately 2% with the coefficients of uniformity and curvature of C_u=12 and C_c=1.4, respectively, and that the maximum dry unit weight is $\gamma_{d,max}$=1.8 t/m^3 with the optimum water content of w_{opt}=8%. Based on the Unified Soil Classification System (USCS) the soil was classified as SP. Available information on the wall construction, however, suggested the backfill soil being sandy clay with gravels. A crushed uniform size gravel was to infill the spaces between adjoining modular block units. A drainage layer was created by extending the gravel to a distance of 500 mm behind the facing column.

Construction records indicated a number of structural problems during and after the east wall construction ranging from excessive lateral bulging of the wall face to a partial collapse. Sever rainfall during the construction period and various construction activities including installation of duct banks after the wall completion were blamed for the problems. The weak performance of the wall during construction prompted the original design to be modified, especially in the upper unreinforced zone of the east wall, and most of the upper 1/3 of the east wall was reportedly to have been reconstructed. The modification to the original design mainly involved placing short geogrid layers and tying them to duck banks in an attempt to form deadman anchors. Upon completion of the wall construction, the east wall was monitored at several stations for a period more than one year using inclinometers attached to the wall face. No appreciable movements were reported and the monitoring program was terminated.

3 WALL PERFORMANCE

3.1 *Excessive wall movements of north wall*

In August 2000, several years after the wall completion, the owner noted significant wall movements along a 20-m-long section of the north wall, for which no significant problems were reported during and after construction. Two sections, N1 and N2 on the north wall shown in Figure 1, were therefore closely monitored using plum bobs. Figures 3 and 4 show the patterns of wall movements and the records of maximum wall movements with respect to time after the start of monitoring, respectively. Also shown in Figure 4 are the possible geogrid strain levels inferred from the measured wall displacements assuming that no slip between the geogrids and the backfill had occurred. Note that these movements were measured over a three-month period until the movements appeared to have been ceased and do not include displacements that had occurred during construction. The actual strains developed in the reinforcements might therefore be much higher than those indicated in Figure 4.

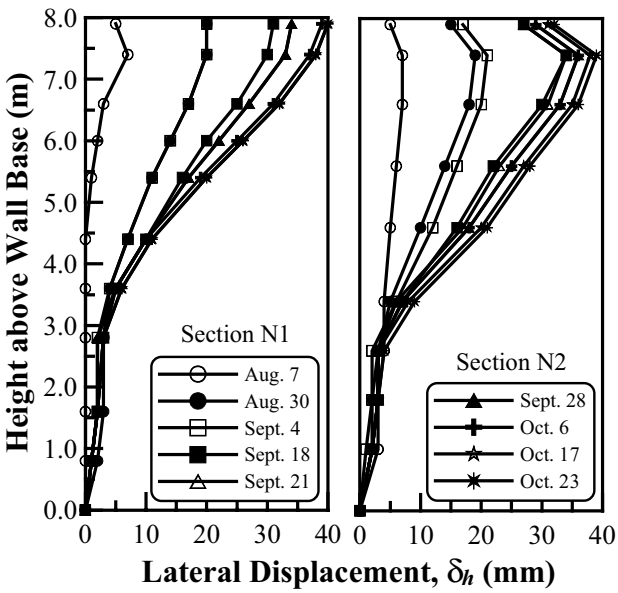


Figure 3. Lateral wall movement profiles of north wall

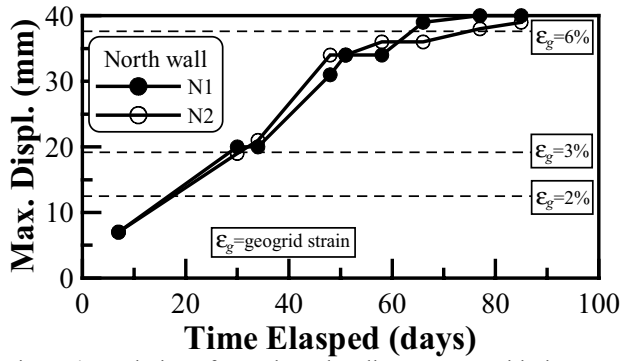


Figure 4. Evolution of max. lateral wall movement with time

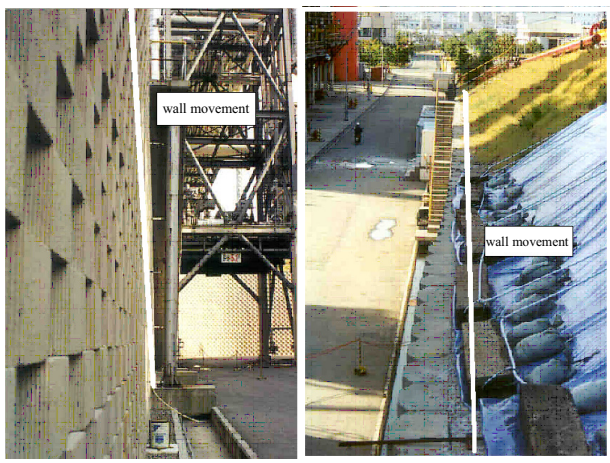


Figure 5. Photographs of wall movements (north wall)

As seen in Figure 3, the lateral wall movement profiles essentially follow a cantilever type with a maximum value of approximately 40 mm occurring at the top. The wall movements increased almost linearly with time for a 1.5-month period, after

which the movements leveled off, as illustrated in Figure 4. These movements can also be visually seen from the photographs taken at the site shown in Figure 5. As one can notice, significant movements at the top are manifested by the out-of-alignment of the column of the access-stair attached to the wall. Although no additional movements were recorded approximately three month after the start of monitoring, the prospect of further movement of the wall and the long term stability was a concern to the owner, prompting this investigation.

Although no wall movements were detected in the east wall during this period, a visual inspection of the wall was carried out during initial stage of this investigation. It was observed that some vertical cracks on the facing units had developed, which appeared to have been formed after the wall completion.

3.2 *Wall profiling*

The write as well as the owner felt that the recent north wall movements measured during the monitoring period did not represent the total movements that had occurred during the service period, which were the critical information as to the states of stress-strain of the components comprising the wall system, such as the backfill and the geogirds. A complete profiling of the entire north and east walls was therefore conducted in order to obtain the information on the order of magnitude of the movements that might have occurred after the wall completion.

The wall profiling was performed using a 3D total station (TOPCON Model GTS-700). Reflection targets were attached onto the wall facing units at a number of pre-selected locations. A total of 8 sections were selected for profiling; two for the north wall and six for the east wall. Figure 6 shows a schematic view of the location of the monitoring sections. For each target array, a fixed benchmark was placed at approximately 5 m away from the array and readings were taken accordingly.

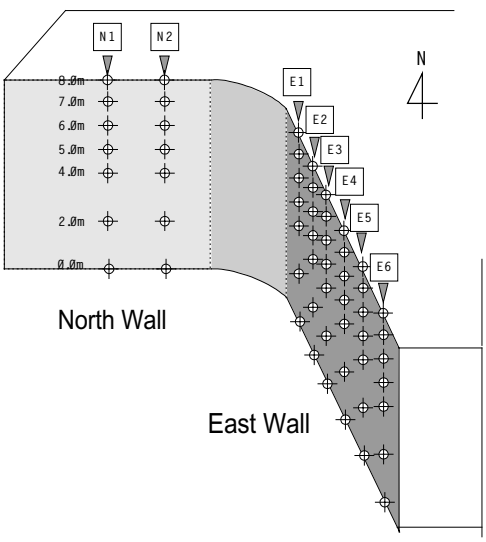


Figure 6. Schematic view of profiling program

The results of the wall profiling were carefully analyzed to estimate the total wall movements that had occurred during and after the wall construction. The design pre-batter angle of the wall face was a critical parameter in inferring the total wall movements from the profiling results. As mentioned previously, a pre-batter angle of 3.5 degrees was assumed based on the information available. Cumulative wall movements during and after the wall construction were then deduced from the profiling results.

Figure 7(a) illustrates the inferred wall movements for the north wall sections. Also shown in this figure is the wall face with a batter angle of 3.5 degrees. As seen in this figure, it appears that these two sections, which showed a sign of distress most recently through the excess wall movements, moved approximately 500 mm at the top, assuming a fixed condition at the base of the wall. A maximum geogrid strain on the order of 8% would be anticipated for that magnitude of wall movements. Considering the strain at rupture of 12% for the geogrids used in the wall construction, the inferred levels of strains appear to be excessive even though possible slip between the geogrid layers and the backfill are taken into consideration.

Figure 7(b) illustrates the inferred wall movements of the various sections for the east wall. As seen in this figure, the results of the wall profiling suggest that significant wall movements, as great as 400 mm, occurred in section E2. Sections E3 E6 exhibited relatively smaller wall movements compared to other sections. Although the east wall sections did not show significant post-construction movements, the stain levels in the geogrids were also believed to be high.

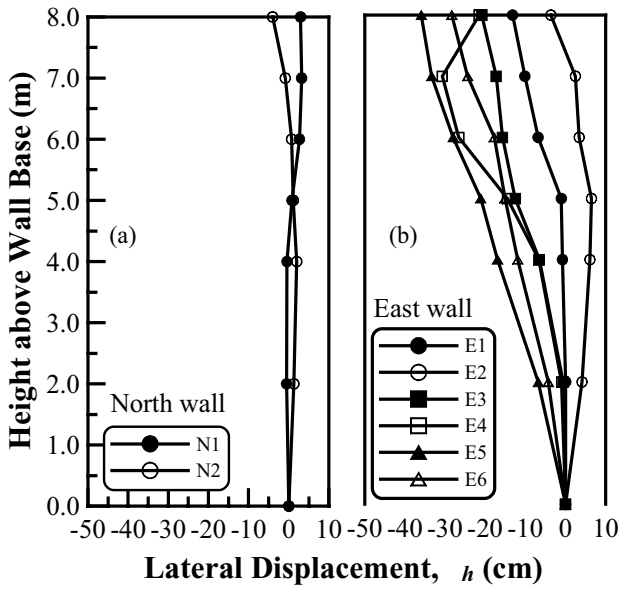


Figure 7. Out-of-alignment of north and east wall sections

The profiling results presented above may not exactly represent the actual wall movements due to possible modifications to the original design during construction and errors involved in profiling. Nevertheless, given the circumstances, it appeared that significant wall movements had occurred during and after construction in the entire wall sections, and that strain levels in the geogrids were undesirably high. And therefore, the likelihood of additional wall movements was considered to be quite high due possibly to any changes in environmental conditions, such as creep of geogrids, water pressure due to seasonal rainfalls, among others.

4 ANALYSIS OF ORIGINAL DESIGN

In an attempt to evaluate the adequacy of the original design, two typical sections, shown in Figure 2, each for the north and the east walls were analyzed based on the limit equilibrium-based design/analysis approaches. The results are presented under subsequent headings.

4.1 *Selection of material properties*

As mentioned, available construction documents and other information indicated the soil samples taken from the slope portion might not have been the same as the backfill soil. An internal friction angle of 30 degrees together with a unit weight of 19 kN/m^3 was therefore assumed for the backfill soil in the subsequent stability analysis with due consideration of a common

practice exercised in Korea. This value, however, was considered to be unconservative when considering the strain levels already existed in the backfill soil. Since the wall appears to be situated on a competent rock stratum, high shear strength properties were assumed for the foundation soil.

No data on the mechanical properties of the connection between the polymeric reinforcement and the modular facing units were available at the time of the investigation, and therefore the connection strengths were assumed to be the same as the allowable strengths of the geogrids. A value of 0.9 was chosen as the direct sliding and pullout coefficients required for analysis.

Results of current design approaches are significantly influenced by the reduction factors and the safety factor for uncertainties required when calculating the long-term design strength (LTDS) of geogrids. The reduction factors and the safety factor were chosen with due consideration of recommended values in NCMA and FHWA design guidelines, as shown in Table 1.

Table 1. Reduction factors and long term design strength

Reduction Factor	Long Term Design Strength (kN/m) $T_a = T_u/(RF_{CR} \times RF_D \times RF_{ID} \times FS)$	
	TYPE-I	TYPE-II
RF_{CR} =2.15, RF_D =1.1 RF_{ID} =1.1, FS=1.1	15	21

4.2 *Limit equilibrium-based stability analysis*

A series of limit-equilibrium based stability analyses were performed using computer programs SRWall ver 2.1 (SRWall 1997) and MSEW ver 1.1 (MSEW 1998). Note that SRWall and MSEW are developed based on the NCMA and FHWA design guidelines, respectively.

The results of external stability analysis indicated that both the north and east walls satisfy all limit-equilibrium based performance criteria in accordance with NCMA and FHWA design guidelines. As summarized in Table 2, however, all the sections analyzed did not satisfy the internal stability requirement for tensile overbreak, showing factor of safety values well below the minimum required value of $FS_{to(min)}$=1.0, supporting the problems encountered during construction. Although not presented here, the original design did not meet the local stability requirements such as local overturning and connection strength.

Table 2. Results of internal stability calculation

Layer	Elev. (m)	North wall		East wall	
		FS_{to} (SRWall)	FS_{to} (MSEW)	FS_{to} (SRWall)	FS_{to} (MSEW)
1	0.2	1.02	0.80	0.89	0.70
2	0.6	1.07	0.83	0.92	0.72
3	1.0	1.12	0.86	0.96	0.75
4	1.4	1.18	0.90	1.00	0.78
5	1.8	1.24	0.94	1.05	0.81
6	2.2	1.05	0.79	0.88	0.68
7	2.8	0.95	0.71	0.79	0.60
8	3.4	0.90	0.66	0.73	0.55
9	4.2	0.90	0.65	0.71	0.53
10	5.0	0.96	0.66	0.73	0.53
11	6.0	1.10	0.72	0.78	0.55
12	7.0	1.07	0.61	0.68	0.44
13	8.0	1.02	0.92	0.52	0.28

The results of stability analysis strongly suggest that most of the wall sections were not adequately designed to satisfy the currently available design criteria, and that the recent movement at the north wall, to a large extent, might have been triggered by the inadequate design.

5 POSSIBLE CAUSES AND REMEDIAL MEASURES

Considering all the circumstances and the results of stability analysis, the recent north wall movements may be due in large part to the inadequate design. In addition, recognizing the possibility of the backfill soil being clayey soil, the possibility of volume expansion of the backfill soil upon saturation cannot be ruled out. No mineralogical study on the backfill soil, however, was made due to inability to obtain representative samples. Furthermore, considering that the recent movements occurred after a rainy season, additional water pressure due to malfunction of the drainage system could also be one of the causes.

Although this investigation was initiated in response to the recent north wall movements, the rest of the wall sections also appeared to have experienced significant wall movements. Since the entire wall sections appeared to be unstable according to the current design criteria, the recent excessive movements, and the profiling results, remedial measures were provided to ensure both the short-term and long-term stability. Selection of remedial measures was highly influenced by the site condition in which a number of utilities and pipelines are buried at shallow depths. Suggested remedial measures include reconstruction of upper 1/3 of the wall using either a segmental retaining wall or a reversed L-shape concrete retaining wall. Also suggested is to provide a means of external support onto the entire wall sections with a reversed L-shape concrete retaining wall.

6 CONCLUSIONS

The results of an investigation on a 6-year-old geosynthetic reinforced segmental retaining wall are presented. The stability of the entire wall sections was evaluated based on the results of stability analysis and wall profiling, among others, and remedial measures were suggested to ensure both the short-term and long-term stability.

Although no definite causes can be identified, inadequate design appeared to be the prime cause for the recent north wall movements. Possible volume expansion of the backfill soil upon saturation and additional water pressure could also be the causes for the movements.

The results of this investigation indicate that large post-construction movements can occur for walls not adequately designed to meet the current design criteria. Also confirmed is the ability of geosynthetic reinforced segmental retaining walls to accommodate large movements before failure. Neglecting fundamental principles involved in the reinforced earth wall design and construction may result in substantial cost for repair and/or reconstruction.

REFERENCES

Bathurst, R.J. & Simac, M.R., 1994. Geosynthetic reinforced segmental retaining wall structures in North America. *Proceeding 5th International Conference On Geotextiles, Geomembranes and Related Products*, Singapore, 24 p.

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

MSEW 1.1 1998. Mechanically Stabilized Earth Walls Software, ADAMA Engineering, Inc.

NCMA. 1997. Design Manual for Segmental Retaining Walls, 2nd Ed., NCMA, Virginia.

SRWall 2.1 1997. Design Software for Segmental Retaining Wall s, NCMA, Virginia.