FAILURE OF SEGMENTAL RETAINING WALLS DUE TO THE INSUFFICIENCY OF BACKFILL PERMEABILITY

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Abstract: Segmental retaining walls (SRWs) (primarily those with precast concrete block facing) reinforced by geogrids or geotextiles are used extensively as retaining walls and bridge abutments in transportation networks in Iran. In spite of significant advantages of SRWs, like flexibility on soft foundations, seismic stability and their cost effectiveness, there are some cases of SRWs failure because of neglecting basic geotechnical engineering principles in wall details. In an attempt to identify possible causes for a 12 m height wall collapse, a comprehensive investigation was carried out including laboratory tests on the backfill, stability analyses on the as-built design based on the current design approaches and numerical slope stability analyses with pore pressure consideration. The investigation revealed that the inappropriate stability design and bad-quality backfill were mainly responsible for the collapse. Finally, practical significance of the findings from this study is discussed for future similar projects.

Keywords: Geogrid, reinforced earth retaining wall, failure, permeability, backfill

INTRODUCTION

Failure plays an important role in engineering practices. For the engineer, knowledge of engineering failure is just as important as knowledge of its successes. Through the forensic study of failures, engineers can learn to avoid similar technical errors, allowing them to build more efficient, safer structures. A 12 m height segmental retaining wall system using geosynthetic reinforcement failed in Tehran capital of Iran in late February 2006. This segmental retaining wall failure is analyzed with respect to the design, and construction to determine the causes of the failure. In an attempt to identify possible causes of the collapse, a comprehensive investigation was carried out, including soil mechanics laboratory tests for the SRW's backfill soil, stability analyses on the as-built wall based on the current design approaches and global slope stability with pore pressure analysis. Segmental retaining walls normally are designed to satisfy the internal, external, and local stability requirements of facing elements to ensure their long-term performance. The objectives of this paper are to identify the causes of the wall failure firstly and secondly to clarify the influence of insufficiency of backfill permeability on wall failure and finally illustrate consequences of neglecting fundamental principles of geotechnical engineering in the design and construction of geosynthetic reinforced soil retaining wall. To avoid any additional SRW failure, it is necessary to conduct a forensic study to explicate the common mistakes that have caused the collapse of the SRW and offer guidance for future engineering practices.

WALL DESCRIPTION

The wall was constructed for a newly developed urban highway in north part of Tehran. The landscaping and earthwork for the new highway required the construction of a 12-m high geosynthetic reinforced SRW. At this location, a 160-m-long retaining wall was required, ranging in height from 5 to 12 m and 1750 m^2 in facing area to retain a natural slope. The wall was situated on a slightly sloping ground, immediately next to a 36 m wide highway away from the wall face. The highway eventually joins a main junction that is located approximately 30 m away from the wall. A typical sectional view of the tallest section of the wall, inferred from the information gathered from the site, is shown in Figure 1.

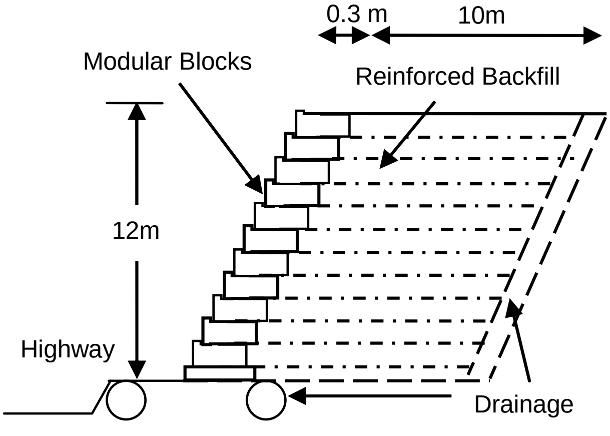

Figure 1. Wall geometry cross section

The safety of any structure is highly dependent on its material's stability and durability. Therefore, the selection of reinforcement and backfill soil for SRW's must consider the performance, service life, and the environmental conditions of the structure. Geologically, the original ground existing behind and under the embankment is composed of colluvial soil and weathered cemented soil and rock respectively. The colluvial layer is composed of clay and sandy gravel. The site investigation indicated that the reinforced backfill and retained soils were essentially the same being a completely decomposed sandy clayey soil available at the site. A number of laboratory tests were conducted to quantify relevant geotechnical properties of the select fill for use in a series of analyses using representative soil samples collected from different locations within the reinforced backfill and retained zones. As seen in the particle-size distribution curve in Figure 2, the completely decomposed backfill soil contained over 38% of fines passing the number 200 sieve. According to the Unified Soil Classification System ASTM D2487, the soil was classified as SC, clayey sand, with a plasticity index of 16. The standard Proctor test ASTM D698 yielded a maximum dry unit weight of 21 $kN \cdot m^3$ with an optimum water content of 15%. According to the physical tests, the backfill soil did not comply with the FHWA recommendation.

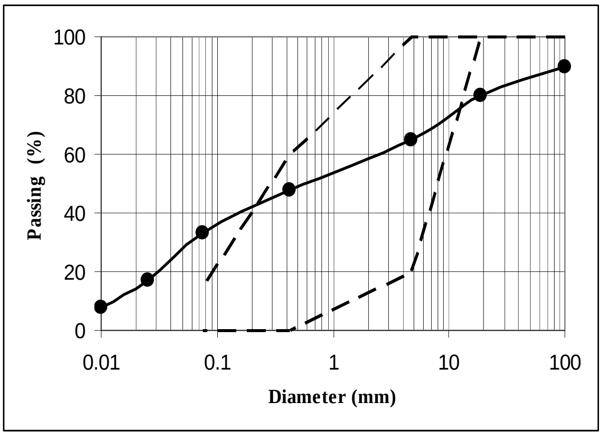

Figure 2. Particle size distribution curve of backfill

For determination of the saturated permeability and the shear strength parameters, specimens were compacted to the field density corresponding to 90% of its maximum dry unit weight obtained from the standard Proctor test. A series of falling head permeability tests yielded an average saturated hydraulic conductivity of $K_s=1.0\times10^{-6}$ m/s. In addition a series of large-scale direct shear tests using specimens of 300 x 300 mm in plan and 150 mm in height gave as compacted shear strength parameters of $\phi=27°$ and cohesion intercept of c=10 to 20 kPa. The natural soil classification and properties are shown in Table 1.

Table 1. Natural soil properties

Layer	USCS	c (kPa)	ϕ (Deg)	k (m/s)
Colluvial-soil (Backfill)	SC/SM	10≅20	27	1×10^{-6}
Weathered soil & rock	SC/GC	25≅30	35	4×10^{-7}

The geosynthetic reinforcements used in Iran are predominantly flexible geogrid made of a variety of polymeric materials. Because of its complexity, the quality verifications of geogrid on site are always problematic. To identify mechanical characteristics of geogrid reinforcement layers, a series of rib tensile strength tests on the specimens recovered at the site were conducted in accordance with the test procedure as specified in GRI-GG1, Test Method 1988. A loading rate of 10±3%/min was used as specified in GRI-GG1. According to the test results, the ultimate tensile strength and axial stiffness of approximately 90kN/m and 800kN/m, respectively, were estimated. Because the tests were performed on geogrids obtained at the site, the measured tensile strength of 90kN/m was thought to reflect possible decrease due to installation damage during construction.

The results of investigation indicated that 8 to 9 m long reinforcement layers were placed at a uniform vertical spacing of 0.6 m, thus satisfying the minimum reinforcement length 0.7H according to the currently available design approaches, i.e., National Concrete Masonry Association (NCMA -Collins 1997) and Federal Highway Administration (FHWA- Elias and Christopher 1997).

The wall facing was constructed using concrete modular blocks 450x450x200 mm (Length, Width and Height) with compressive strength between 18 to 20 Mpa. Visual investigation of the wall facing during the field investigation shows 5 cm set-back of the modular facing blocks was provided. A shear-key type connection between the modular blocks was used to transmit shear between the facing elements. Also crushed gravel was used to infill the spaces between reinforced backfill and retained soil. The crushed gravel was also extended to under the backfill to create a blanket drainage system without any filter layer. At this site the average ultimate bearing capacity of the foundation soil was approximately 800 kPa.

CAUSES OF FAILURE

Based on site observations and engineering studies for each failed case study, the causes of failure generally can be distinguished as natural influences and professional mistakes. The wall failure occurred at the end of February 1999. The highest section of wall (12 m) failed during a heavy rainstorm. Numerous worldwide studies have reported that intense rainfall has been the major factor responsible for many slope failures including reinforced earth structures, Huang (1994), Rahardjo et al. (2001) and Pando et al. (2005). As seen in the photos taken after the collapse by the authors in Figure 3, the collapsed portion of the wall extended approximately to 10 m, resulting in a total slid volume of soil over 350 m^3. In Figure 4 the geogrid reinforcements and cracked modular blocks near the sliding zone are shown.

Figure 3. Photo of collapse in the reinforced soil wall

Figure 4. Wall failure-geogrid layers visible

A review of the original design was performed using the NCMA, Design Guidelines. Internal, external, facing stability and global stability were evaluated. The original design was based on the assumption that no hydrostatic forces would be acting on the SRW. A check of the original design was performed. This design check used the NCMA procedure in its entirety. Connection strength properties between the SRW units and reinforcement were based on laboratory tests provided by the manufacturer. The design section presented here is for the tallest section of wall measuring 12m in height from levelling pad to top of the wall. For analysis, the available design/analysis program SRWall (Bathurst 2001) were used, which were developed based on the NCMA design approach.

The current limit equilibrium-based design approaches require allowable long-term design strengths (Ta) of the reinforcements; block interface and modular block/geogrid connection strength properties for analysis. The allowable long-term design strengths for the geogrids were estimated based on the vendor provided information equal to 35kN/m. Because of the absence of the block interface and the geogrid/block connection properties, typical values based on NCMA (Collins 1997) and Chungsik Yoo et.al. (2004) recommendations are used.

WALL STABILITY ANALYSIS

The results of internal stability calculations that were shown in Table 2, indicate that the lower three layers do not meet the NCMA design requirement for tensile overstress (FS_{to}), exhibiting factors of safety well below the required minimum value of $FS_{to(min)}$ =1.0. Also the upper two layers do not meet the NCMA design factor of safety for pullout (FS_{po}).

Table 2. Internal stability results

Layer	Elev. (m)	Internal Stability		
		FS_{to}	FS_{Po}	FS_{isl}
1	0.3	0.64	10.2	1.23
2	0.9	0.85	11.4	1.45
3	1.5	0.97	9.51	1.67
4	2.1	1.15	8.3	2.41
5	2.7	1.2	7.84	2.73
…	…	…	…	…
14	8.1	12.1	2.21	4.21
15	8.7	13.5	1.87	5.31
16	9.3	14.2	1.45	7.12
17	9.9	15.3	1.21	8.19
18	10.5	>16	1.11	13.52
19	11.1	>16	0.95	19.32
20	11.7	>16		

FS_{to} = tension overstress factor of safety
FS_{po} = pullout factor of safety
FS_{isl} = Internal sliding factor of safety

GLOBAL WALL STABILITY ANALYSIS

By considering the geometry of the SRW system and the failure mode, limit equilibrium based on global stability analyses were deemed necessary to identify causes of the failure. In order to simulate what actually occurred after the rainfall with a high degree of realism, the stability analyses were performed based on effective stresses with pore water pressures effects of the rainfall on wall. A series of steady state seepage analyses considering soil permeability, rainfall intensity and duration were first conducted assuming a non-deforming soil to determine a critical pore water pressure distribution during the event of the rainfall by Geo-Slope-SEEP/W Version 5 software. The results of this analysis are presented in Fig. 5. Then limit equilibrium-based slope stability analyses were carried out with Mohr-Coulomb failure criterion. This was done using a commercial software package GEO-SLOPE Version 5. The global factor of safety and failure surface of the wall are illustrated in Fig.6. The location of the failure surface was in fact in accordance with the traced failure surface based on the information gathered in the field.

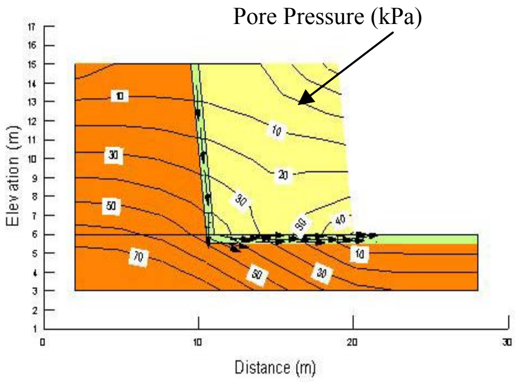

Figure 5. Pore-water pressure distribution used in slope stability analysis

The results of the slope stability analyses are presented in Table 3. The safety factor of SRW was larger than 1.2 in case of the dry case and was less than the required safety factor for wet case (after of the rainfall and backfill saturation).

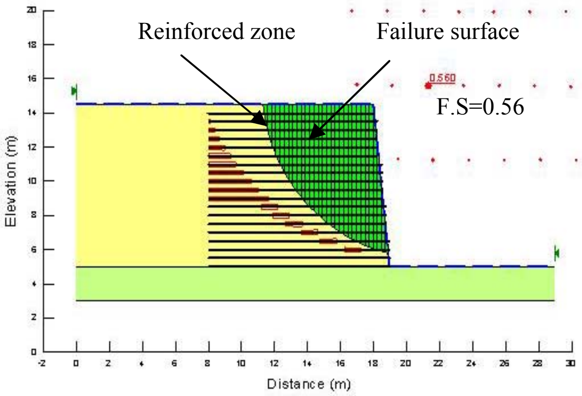

Figure 6. Failure surfaces obtained for wet case

Table 3. The results of Slope stability analysis

Method	Safety factor of SRW	
	Dry Case	Wet case
Bishop	1.90	0.56
Spencer	1.88	0.69

CONCLUSIONS

In this study a comprehensive site investigation and slope stability analyses were conducted to investigate the causes of the collapse of a 12m SRW. Based on the forensic diagnosis of the observed SRW failure, intense rainfall was the most important natural influence to causing the SRW failure. However, inadequate planning, inappropriate as-built design, the bad-quality backfill soil with a significant percentage of fines, available at the site and poor construction workmanship are responsible for the SRW failure.

The connection between the reinforcement and the facing of a SRW is an important component of the system that must be considered in both the design and construction of these systems. Also for tall SRWs or walls with complex geometry, global stability analyses should be carried out in design steps for various environmental conditions.

At last important lessons learned from the collapse is perhaps that the consequence of neglecting basic geotechnical engineering principles can result in a catastrophic collapse such as one that described in this paper.

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