

Lessons Learned from a Failure of Geosynthetics-Reinforced Segmental Retaining Wall

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ABSTRACT: This paper presents a case history of a geosynthetics-reinforced segmental retaining wall, which collapsed during a severe rainfall immediately after the completion of the wall construction. In an attempt to identify possible causes for the collapse, a comprehensive investigation was carried out including physical and strength tests on the backfill, stability analyses on the as-built design based on the current design approaches, and slope stability analyses with pore pressure consideration. The investigation revealed that the inappropriate as-built design and the bad-quality backfill were mainly responsible for the collapse. This paper describes the site condition including wall design, details of the results of investigation and finally, lessons learned. Practical significance of the findings from this study is also discussed.

1 INTRODUCTION

Geosynthetic-reinforced segmental retaining wall systems (GR-SRW) have been used in a variety of applications since its first appearance in the early 1990's. Although many geosynthetic reinforced soil walls have been safely constructed and are performing well to date, there are many areas that need in-depth studies in order to better understand the mechanical behavior of SRW systems under more aggressive and harsh environments. Despite the inherent margin of safety against both external and internal stability when using the currently available limit equilibrium-based design approaches (Collins 1997, Elias and Christopher 1997) numerous major and minor structural problems have been reported during and after construction, covering a range of minor structural damage to total collapse. Although limited, such problems have contributed to the current lack of acceptance for geosynthetic-reinforced soil wall systems by some practitioners and government agencies in Korea. Many areas need further studies in order to better understand the short- and long-term behavior and to construct safer and more economic geosynthetic-reinforced segmental retaining wall systems.

This paper focuses on a collapse of a 7.4-m-high GR-SRW constructed in a plant complex for landscaping and earthwork. The collapse occurred during a severe rainfall immediately after the completion of the wall construction. In an attempt to identify possible causes for the collapse, a comprehensive investigation was carried out including physical and mechanical laboratory tests on the backfill, stability analyses on the as-built design based on the current design approaches, and slope stability analyses with pore pressure consideration. The investigation revealed that the inappropriate as-built design and the bad-quality backfill soil together with the severe rainfall were mainly responsible for the collapse. This paper describes the site condition including wall design, details of the results of investigation and finally, lessons learned and finally, practical implications for design.

2 SITE CONDITION AND WALL DESIGN

2.1 Site condition and wall collapse

The wall constructed during the period of April ~ June 2003 at Chung-Nam province, South of Seoul, Korea. The landscaping and earthwork for a factory complex required the construction of several geosynthetic-reinforced segmental retaining walls. At one location, a 150-m-long retaining wall was required, ranging in height from 1 m to 7.4 m. As seen in Figure 2, the wall was constructed to retain the embankment for use as an approach road to the factory complex. The wall situated on a slightly sloping ground, immediately next to a 2-m-wide waterway existing approximately 4 m from the wall face. The waterway eventually joins an irrigation reservoir that is located approximately 10 m away from the wall.

The collapse occurred in late July 2004, during a sever rainfall that recorded a maximum rate of 39 mm/hr. A total precipitation during a two-month period of June and July was approximately 500 mm. In fact, this period is the annual monsoon season in Korea, with average precipitation of 400 mm. As seen in the photos taken after the collapse, most of the wall sections remained intact except one location where the wall was sheared off. Also noted in these photos is that a significant portion of the roadway in the retained slid down with the reinforced soil mass. Including the observed failure pattern, there are existed a number of evidences leading to a conclusion that the failure was closely related to global and external instability stability.



Figure 1. Photos taken after the collapse

2.2 Wall design and construction

A typical sectional view for the tallest section of the wall measuring 7.4 m in height from leveling pad to top of the wall crest is shown in Figure 2. On account of the difficulties in obtaining design documents a comprehensive site investigation was carried with the aim of obtaining relevant information as to the as-built design. The results of investigation indicated that 5 m-long reinforcement layers were placed at a uniform

spacing of 0.6 m, thus satisfying the minimum reinforcement length $0.7H$ according to the currently available design approaches, i.e., NCMA (Collins 1997) and FHWA (Elias and Christopher 1997). It should however be noted that the 5-m high broken back slope portion extending from the end of the reinforced zone might have produced a considerable magnitude of de-stabilizing force against the reinforced soil mass. According to the reconstructed design, such a factor appeared to have not been accounted for.

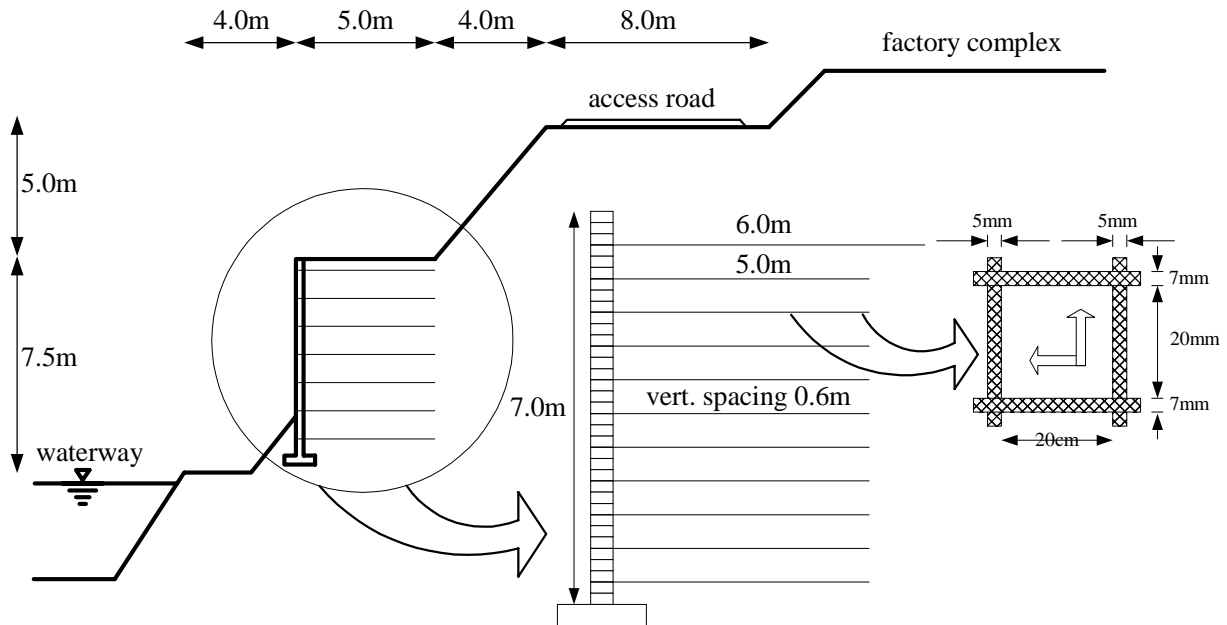


Figure 2. A typical section of a geosynthetic-reinforced segmental retaining wall

The wall facing was constructed using modular blocks 200 mm in height. The reinforcement was a high density polyethylene geogrid of which the detailed geometry is given in Figure 2. Visual inspection of the wall facing during the field investigation suggested that no 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 courses. A crushed gravel was used to infill the spaces between adjoining modular block units. The gravel was also extended to some distance, approximately, 300 mm behind the facing column to create a drainage layer without any filter layer.

As part of this investigation, an extensive laboratory testing program was designed and implemented to the reinforcement and the backfill. The results were then used in subsequent stability analyses. A series of rib tensile strength tests were conducted in accordance with the test procedure as specified in GRI-GG1 (GRI Test Method 1988) in an attempt to identify stress-strain-strength characteristics of the geogrids. Specimens of 10 cm in length were prepared from recovered geogrid layers from the site and used for the tests. A loading rate of $10 \pm 3\%/min$ was used as specified in GRI-GG1. The results are presented in Fig. 3 from which an ultimate tensile strength and an axial stiffness of approximately 65 kN/m and 500 kN/m, respectively, were estimated. Considering the ratio 0.25~0.35 of working stress stiffness (at 1,000 hours) to the stiffness obtained in a constant rate of strain test such as GRI-GG1, as suggested by Bathurst (2004), the actual stiffness of the reinforcement at the event of failure could have been much lower than 500 kN/m.

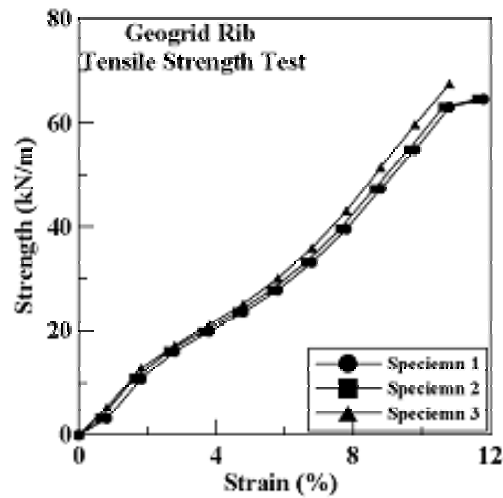


Figure 3. The result of rib tensile strength tests

Table 1 summarizes the engineering properties of the backfill soil. It is of worth noting that the backfill soil contained over 36% of fines passing the number 200 sieve. According to Unified Soil Classification System USCS, the soil was classified as clayey sand SC, exhibiting considerable plasticity. Permeability tests on the compacted specimens yielded a low hydraulic conductivity on the order of 10^{-6} cm/s on account of the rather high content of fines. 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. Also performed were large-scale direct shear tests on specimens (500×500 mm in plan \times 250 mm in height) compacted to 95% of its maximum dry unit weight obtained from the standard Proctor test (ASTM D 698). The estimated internal friction angle at a density corresponding to the as-compacted state was approximately $\phi' = 22^\circ$ with a cohesion intercept of $c' = 30$ kPa. The rather high cohesion value was thought to represent as apparent cohesion considering partially saturated nature of the compacted fill.

Table 1. The engineering properties of the backfill soil

Specific gravity (Gs)	2.65	Unified Soil Classification System (USCS)	SC
Liquid limit (LL, %)	35.5	Hydraulic conductivity (cm/sec)	4.12×10^{-6}
Plastic limit (PL, %)	26.0	Maximum dry unit weight (kN/m^3)	18.4
Percentage passing No.200 sieve (%)	36.8	Optimum moisture content (%)	13.7

3 STABILITY ANALYSIS

3.1 NCMA and FHWA design approaches

Limit equilibrium-based stability analyses were conducted based on the NCMA and the FHWA design approaches. For analysis, two available design/analysis programs were used; SRWall (Bathurst 1999) and MSEW (Leshchinsky 1999), which were developed based on the NCMA and the FHWA design approaches, respectively. The stability analyses were aimed at gaining insight into margins of safety for the as-built design.

1) Selection of material properties

The results of the limit equilibrium-based stability analysis are significantly influenced by the way in which soil shear strength properties for the backfill material are selected. An internal friction angle of 25° together with a unit weight of 19 kN/m³ was used for analysis as tested. The apparent cohesion was disregarded as the current design guideline require. As the wall appeared to have been situated on a harder soil stratum, internal friction angles of 35 degrees with cohesion of 50 kPa were rather arbitrarily assigned.

The current limit equilibrium-based design approaches require allowable long-term design strengths (T_a) of the reinforcements, block interface and block/geogrid connection strength properties for analysis. The allowable long-term design strengths for the reinforcements were estimated based on the vendor provided information as tabulated in Table 2. Due to the absence of the block interface and the geogrid/block connection properties, typical values reported in a literature (Bathurst 1997) were used, as listed in Table 3.

Table 2. Reduction factors and allowable tensile strength

Reduction Factor	Allowable Tensile Strength (kN/m) $T_a = T_{ult}/(RF_{CR} \times RF_D \times RF_{ID} \times FS)$
$RF_{CR} = 1.5, RF_D = 1.1, RF_{ID} = 1.1, FS = 1.5$	26

Table 3. Block/block interface and block/geogrid connection properties (After Bathurst 1997)

Block / block interface shear properties ^a			Block /geogrid connection properties ^b		
Min. (a_u) (kN/m)	Angle (λ_u) (deg.)	Max. ($V_{u,max}$) (kN/m)	Min. (a_{cs}) (kN/m)	Angle (λ_{cs}) (deg.)	Max. ($V_{ac,max}$) (kN/m)
7	45	29	18	22	27

Note) ^a Interface shear capacity is defined as $V_u = a_u + \Delta W_h(z) \tan \lambda_u$, where $\Delta W_h(z)$ =weight of facing column above sliding surface under consideration

^b Connection strength is defined as $T_{cs} = a_{cs} + \Delta W_h(z_i) \tan \lambda_{cs}$, where $\Delta W_h(z_i)$ =weight of facing column above ith block

2) Results

The results of the stability analyses on the reconstructed as-built design are summarized in Table 4. As noticed, the external stability calculations based on the NCMA design approach indicated that the stability against the direct sliding mode of failure could not be assured, giving the factor of safety value well below the required minimum of 2.0. The factors of safety against the over-turning and the bearing capacity modes of failure were also marginal. The results of internal stability calculations indicate that the lower two layers do not meet the NCMA design requirement for tensile overstress, exhibiting factors of safety well below the required minimum value of $FS_{to(min)} = 1.0$. The results of analyses based on the FHWA design approach yielded even lower factors of safety for all modes of failure, thus supporting that the as-built design was not properly designed to meet the current design guidelines.

Table 4. Results of stability analysis on north wall

Layer	Elev. (m)	Internal stability					
		FS_{to}		FS_{po}		FS_{isl}	
		NCMA	FHWA	NCMA	FHWA	NCMA	FHWA
1	0.4	0.75	0.59	8.76	7.63	1.21	0.95
2	1.0	0.97	0.75	9.13	8.20	1.35	1.04
3	1.6	1.08	0.83	7.97	7.43	1.52	1.14
4	2.2	1.21	0.93	6.81	6.66	1.74	1.30
5	2.8	1.38	1.05	5.65	5.89	2.03	1.44
6	3.4	1.62	1.21	4.49	5.13	2.44	1.66
7	4.0	1.94	1.42	3.33	4.36	2.99	1.95
8	4.6	2.42	1.72	2.17	3.59	3.79	2.37
9	5.2	3.23	2.19	1.01	2.83	5.17	3.02
10	5.8	4.85	3.01	2.25	2.06	8.12	4.15
11	6.4	8.62	4.82	0.97	1.29	18.38	6.63
12	7.0		11.80		2.37		19.90
External stability		NCMA	$FS_{bc}=5.16$ $FS_{bsl}=1.14$ $FS_{or}=3.08$				
		FHWA	$FS_{bc}=2.15$ $FS_{bsl}=0.927$ $FS_{or}=2.33$				

FS_{bc} = Factor of safety for bearing capacity,

FS_{bsl} = Factor of safety for base sliding,

FS_{or} = Factor of safety for overturning

FS_{to} = Factor of safety for tensile overstress

FS_{po} = Factor of safety pullout,

FS_{isl} = Factor of internal sliding

These low factors of safety, well below the minimum requirements, can in fact be argued to have been related to two causes. First, the low factors of safety values against external stability are direct consequences of neglecting the additional surcharge load due to the 5-m high broken back slope portion in the retained zone. Second, the destabilizing forces for the internal stability were significantly greater than expected during design as it appeared that an internal friction angle of 30° was arbitrarily assumed without conducting any laboratory tests. The lateral active earth pressure used to compute reinforcement loads would therefore have been far from realistic.

3.2 Global stability analysis

Considering the geometry of the wall system and the failure mode, limit equilibrium-based global stability analyses were deemed necessary to identify causes of the failure. In order to simulate what actually occurred before, during and after the rainfall with a high degree of realism, the stability analyses were performed based on effective stresses with pore water pressures considering the rainfall. Also of importance was to consider the unsaturated nature of the less permeable material comprising the reinforced and the retained fill.

A series of transient 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. Limit equilibrium-based slope stability analyses were then carried out with modified Mohr-Coulomb failure criterion to allow for shear strength variation due to the presence of matric suction arising from the antecedent unsaturated nature of the reinforced and retained fill. This was done using a commercial software package GEO-SLOPE Ver. 5. The results are presented in subsequent paragraphs.

1) Transient seepage analysis

The transient seepage analyses purported to gain insights into the transient water pressure distributions in the reinforced and retained soils, and to determine the pore water pressure distribution for use in subsequent slope stability analyses. Considering the rather low soil permeability, the transient water pressure distributions during the event of heavy rainfall appeared to be critical in identifying the causes of the failure as high water pressures might have developed within the reinforced zone whilst no water pressure was considered in the design assuming full drainage. The fact that a steady state seepage analysis is unable to determine the transient response of the ground water table to a specific rainfall event necessitated a transient analysis as opposed to a steady state analysis. A commercially available finite-element package SEEP/W was used for analysis. SEEP/W simulates both saturated and unsaturated flows under steady state and transient conditions.

The first step toward the seepage analysis was to generate a dynamic steady state initial condition which is used as a starting point for a subsequent seepage analysis. The importance of modeling an antecedent hydrology condition on any subsequent hydrology modeling has been fully discussed by Blake et al. (2003). The initial condition was generated considering the rainfall during the monsoon season of a magnitude of 400 mm over a two-month period in which the total rainfall was equally partitioned over the two-month period. Upon generation of the initial condition, a subsequent transient analysis was then carried out with due consideration of the heavy rainfall event with a magnitude of 200 mm. The total simulation length was 4-days for the transient analysis. The rainfall event was simulated for the first 48 hours by specifying a flux of 2.3×10^{-6} m/s on the surface boundary AB. For the remaining post-rainfall 48 hours, the surface boundary AB was set to zero flux.

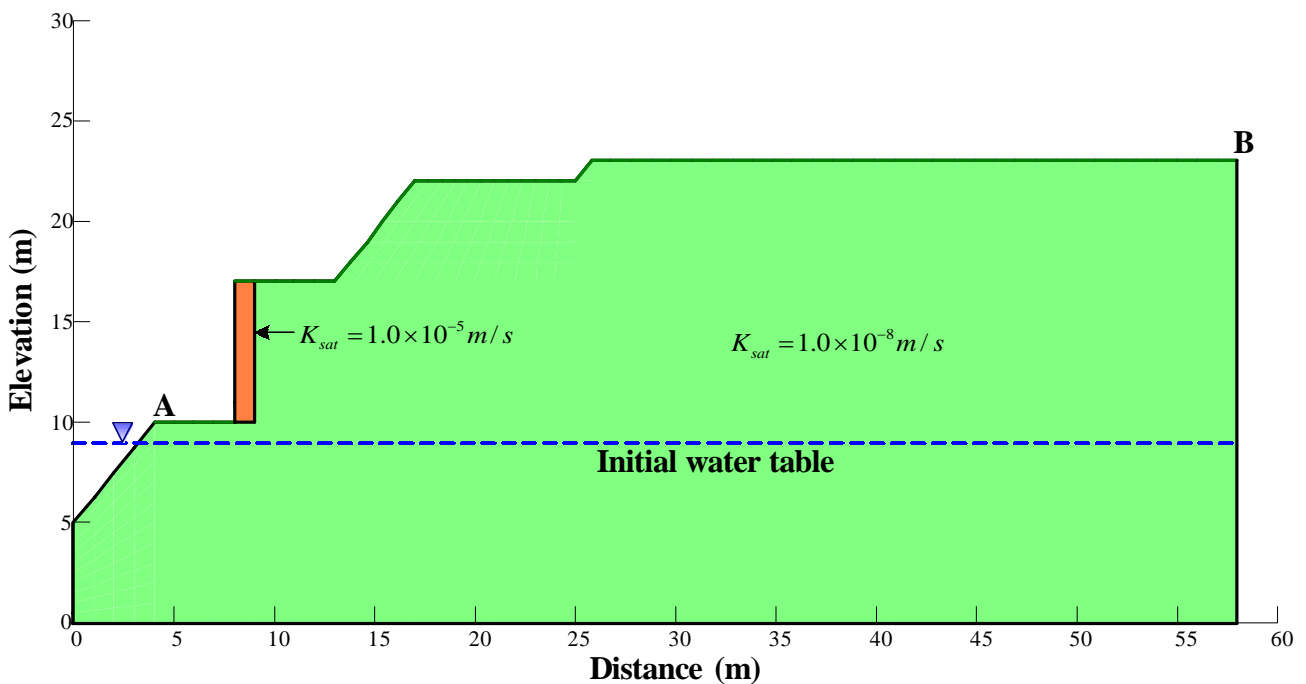


Figure 4. Model domain used 2D seepage analysis

Figure 5 shows an evolution of porewater pressure distribution with time for the two-day period. Also shown in Figure 6 is the porewater pressure distribution at the end of the heavy rainfall that was used as input for a subsequent slope stability analysis.

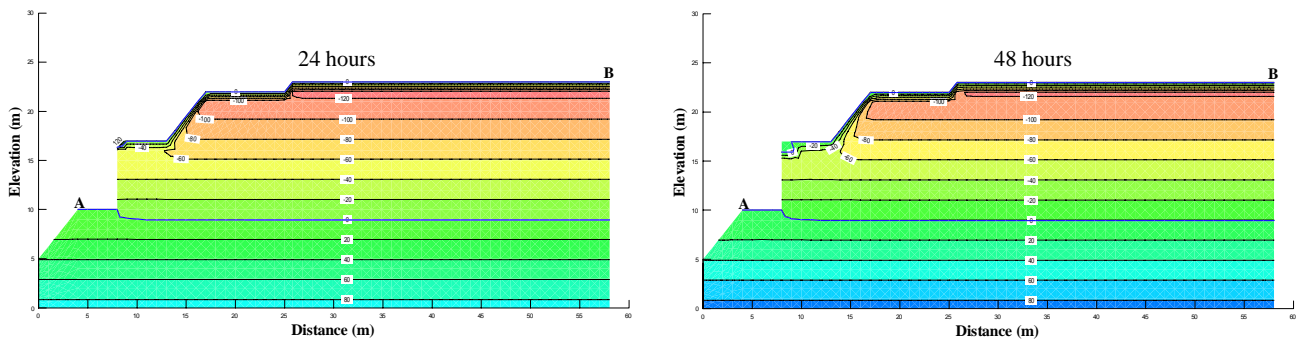


Figure 5. Porewater pressure distribution with time for 48 hours after rainfall

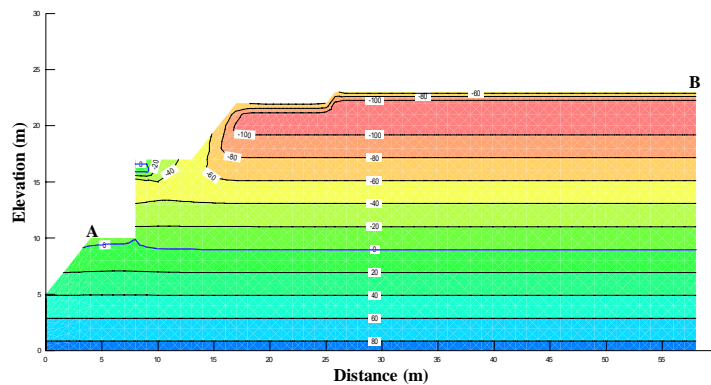


Figure 6. Porewater pressure distribution used in slope stability analysis

2) Limit equilibrium stability analysis

A series of limit equilibrium stability analyses were conducted with full consideration of the pore pressure distribution obtained from the transient seepage analysis. This was accomplished using a slope stability analysis program SLOPE/W. A salient feature of SLOPE/W is an integrated groundwater analysis capability for the purpose of calculating pore pressures for a subsequent slope stability analysis. Also an additional important feature is an ability to incorporate negative pore pressures (commonly referred to as matric suction) in the unsaturated zone above the water table in a slope stability analysis. The slope stability analyses were aimed at gaining insights into the failure mechanism and identifying the roles of pore water pressures on the failure. In selecting appropriate strength parameters for the select fill and the retained soil, the results of the CU triaxial tests were used including the cohesion value of 40 kPa. Such cohesion appeared to represent the unsaturated nature of the compacted rolled fill, and therefore rather unconservative results were expected from the stability analyses. Both positive and negative pore water pressures predicted by SLOPE/W were subsequently used as input ground water conditions for slope stability analyses. Factors of safety were calculated using Bishop's simplified method.

Figure 7 illustrate traces of failure surfaces obtained for two cases; one for immediately after construction (Dry case) and the other for at the end of the rainfall (Wet case). Also shown are the calculated factors of safety. As seen for Dry case in Figure 7(a), the failure circle initiates at the toe of the foundation soil and then propagates back into the reinforced as well as the retained zones, suggesting a typical compound mode of failure. The corresponding factor of safety for Dry case is close to being equal to 1.28 suggesting that the wall was only marginally safe. The location of the failure surface for Wet case is rather shallow compared to that for Dry case but with a significantly low factor of safety of 0.7 supporting the collapse of the wall. Had the tendency of strength reduction of the unsaturated nature of the compacted fill upon saturation been taken into consideration, the actual factor of safety would have been lower than the calculated one. The location of the

failure surface was in fact in accordance with the traced failure surface based on the information gathered in the field.

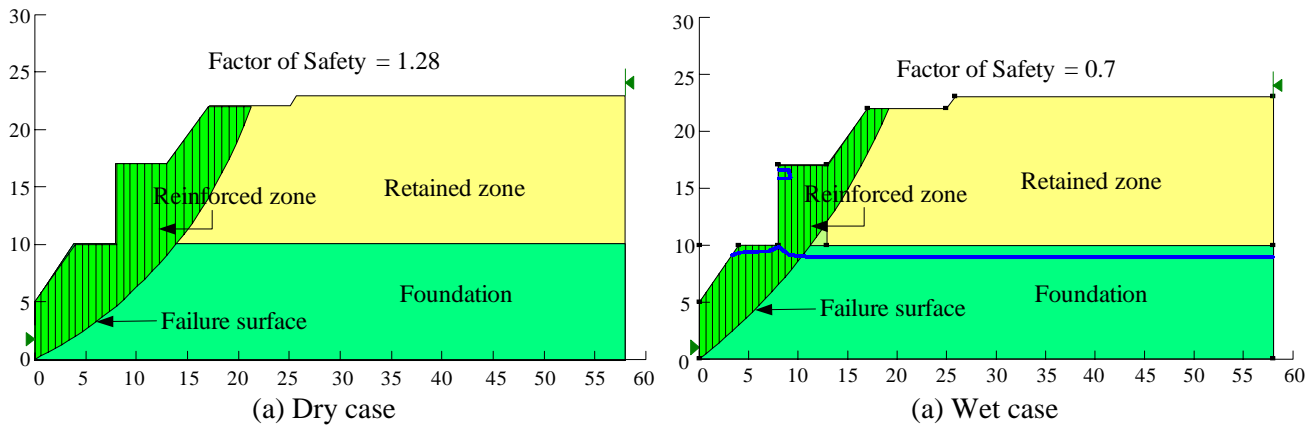


Figure 7. Failure surfaces obtained for dry case and wet case

4 IMPLICATIONS FOR DESIGN

Soil-reinforced segmental retaining walls are used as an integral part of earth structures that must maintain their integrity to satisfy serviceability and ultimate limit state requirements of the earth structures to which the SRWs are used. It is therefore essential to consider not only the stability of wall itself but that of total system including the adjacent topography. There are, however, many instances, at least in Korea, that not enough consideration is given to the overall stability of the systems in which SRWs are used. Notwithstanding such an inappropriate design practice, most of the soil-reinforced walls are being survived well, perhaps, on account of the inherent conservatism in the current design approaches.

Of importance design implications from this case history are three-fold. First, all backfill material used in reinforced walls must be evaluated for their appropriateness as select fill. It is a common practice, at least in Korea, to use residual soils available at site with no appropriate physical and strength tests. Recent laboratory tests, conducted at Sungkyunkwan University, on backfill materials collected from a number of reinforced wall construction sites revealed that 50% of the backfill materials did not conform to the specifications by FHWA design guidelines. Second, a slope stability analysis for the global or compound failure mode should be included in a routine design, especially when an external surcharge load in terms of additional fill is expected in the retained zone. Third, when performing the slope stability analysis, pore water pressures in the reinforced and retained zones should be considered with a high degree of realism for meaningful results.

The conventional limit equilibrium-based design approaches only specifies the length and distribution of reinforcement that are required to fulfill the internal and external stability requirements with an assumption the backfill material being a free draining material. Considering current situations of which rather low quality materials are frequently used as the select fill, the above assumption is far from the state of practice. In addition, most of commercially available design/analysis software cannot sometimes fully account for complex wall geometry and therefore some engineering judgment is required to transform a given geometry to an equivalent system. An improper modeling can yield of significance consequence such as the one discussed in this study. It is essential that any final design of non-routine design cases should be checked by a qualified engineer with appropriate geotechnical engineering background.

5 SUMMARY AND CONCLUSIONS

The results of a collapse of a 7.4-m-high GR-SRW constructed in a plant complex for landscaping and earthwork were presented. The collapse occurred during a sever rainfall immediately after the completion of the wall construction. In an attempt to identify possible causes for the collapse, a comprehensive investiga-

tion was carried out including physical and mechanical laboratory tests on the backfill, stability analyses on the as-built design based on the current design approaches, and slope stability analyses with pore pressure consideration. The investigation revealed that the inappropriate as-built design and the bad-quality backfill soil together with the severe rainfall were mainly responsible for the collapse.

Of importance lessons learned from the collapse are two fold. First, the global stability analysis with pore-water consideration should be included in a non-routine design of geosynthetic-reinforced segmental retaining walls. Second, the quality of backfill soil should be correctly evaluated in order to avoid its potential adverse effects. Last but not least important lesson 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. Therefore, reinforced soil wall design engineers should be able to bring basic geotechnical engineering principles into their designs with great care.

REFERENCES

- ASTM D 4595-86, Standard Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method, pp. 809~819.
- Brian, D.C and Dobroslav, Z., "Stability Analyses of Rainfall Induced Landslides", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. No 4., pp. 362~372(2004).
- Blake, J.R. and Renu, J.P., "Prediction of rainfall-induced transient water pressure head behind a retaining wall using a high-resolution finite element model", *Computers and Geotechnics* 30, pp. 431~442(2003).
- Elias, V. and Swanson, P., "Cautions of Reinforced Earth Residual Soils", *Transportation Research Record* 919, pp.21~26(1983).
- FHWA, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines, *FHWA Demonstration Project 82*, FHWA, Washington, DC, FHWA-SA-96-071(1997).
- GRI Test Method GG1, "Geogrid Rib Tensile Strength", *Geosynthetic Research Institute*.
- Yoo, C.S., Jung, H.S., Lee, S.W., "Reinforced Earth Retaining Wall of The Collapsed-A Case Study", *KGS Spring Conference*(2004).
- Mitchell, J.K. and Zornberg, J.G., "Reinforced Soil Structures with Poorly Draining Backfills Part : Case Histories and Applications", *Geosynthetics International*, Vol 2, No.1, pp.265~307(1995).
- NCMA, *Design Manual for Segmental Retaining Walls*, Second Edition, pp. 20~115(1997).