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Global Stability of Geosynthetic Reinforced Segmental Retaining Walls in Tiered Configuration

계단식 블록식 보강토 옹벽의 전체 안정성

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요 지

본 논문에서는 계단식 형태로 시공되는 블록식 보강토 옹벽의 전체 안정성이 고려된 설계에 관한 내용을 다루었다. 다양한 제원과 이격거리로 설계된 네 가지 설계사례에 대해 현재 통용되고 있는 FHWA 및 NCMA 설계기준에 근거하여 내·외적 안정해석을 수행하고 그 결과를 토대로 두 설계기준의 차이점을 검토하였다. 아울러 대상 옹벽에 대해 한계평형해석에 근거한 사면안정해석과 연속체역학 기반의 강도감소기법 해석을 수행하여 계단식 옹벽의 설계를 지배하는 파괴 메카니즘을 고찰하였다. 그 결과 내·외적 안정성 공히 FHWA에서 채택하고 있는 설계기준이 NCMA 보다 보수적인 결과(낮은 안전율)를 주는 것으로 나타났다. 또한 계단식 옹벽의 보강재의 소요 포설길이는 전반적으로 전체 안정성에 좌우되는 것으로 검토되었으며 상부 옹벽의 보강재의 길이는 현 설계기준보다 현저히 증가시켜야 하는 것으로 검토되었다.

Abstract

This paper presents the global stability of geosynthetic reinforced segmental retaining walls in tiered configuration. Four design cases of walls with different geometries and offset distances were analyzed based on the FHWA and NCMA design guidelines and the discrepancies between the different guidelines were identified. A series of global slope stability analyses were conducted using the limit-equilibrium analysis and the continuum mechanics based shear strength reduction method with the aim of identifying failure patterns and the associated factors of safety. The results indicated among other things that the FHWA design approach yields conservative results both in the external and internal stability calculations, i.e., lower factors of safety, than the NCMA design approach. It was also found that required reinforcement lengths are usually governed by the global slope stability requirement rather than the external stability calculations. Also shown is that the required reinforcement lengths for the upper tiers are much longer than those based on the current design guidelines.

Keywords : Geosynthetic-reinforced segmental retaining Wall, Geosynthetics, Global stability, Limit equilibrium, Finite element analysis, Shear strength reduction

1. Introduction

Geosynthetic reinforced segmental retaining walls (GR-SRWs) have been well accepted in practice as alternatives to conventional retaining wall systems due to several benefits such as sound performance, aesthetics, cost and

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expediency of construction. This is especially true in Korea since its first appearance in the early 1990's. Recently the application of the GR-SRWs has been extended to public sectors such as roadway and railway constructions, especially in Japan as well as north America. For example, GR-SRWs are frequently adopted in bridge construction in public sectors, as the form of geosynthetic-reinforced soil (GRS) abutments in bridge applications (Lee and Wu, 2004).

There are many situations where GR-SRWs are constructed in tiered configuration for a variety of reasons such as aesthetics, stability, and construction constraints, etc. Yoo and Kim (2002), however, reported that the interaction between the upper and lower tiers is not insignificant for walls with an intermediate offset distance as per the FHWA design guideline (Elias and Christopher 1997), thus yielding larger wall deformation and reinforcements forces than what might be anticipated. In addition, the currently available design guidelines such as the NCMA (Collins, 1997) and FHWA design guidelines are somewhat empirical and geometrically derived.

Surprisingly, studies concerning GR-SRWs in tiered configuration are scarce. For example, Yoo (2003), Yoo and Jung (2004) reported the instrumentation results of a full-scale, 5 m high two tier segmental retaining wall that was constructed to investigate the short and long term performance of the segmental retaining wall. Leshchinsky and Han (2004) performed a series of finite difference analyses on multi-tiered segmental retaining walls in order to examine the failure mechanisms and to assess the required tensile strength as a function of reinforcement length, stiffness, offset distance, among others. Later, Yoo and Kim (2006) conducted a numerical investigation on two-tier segmental retaining walls with different offset distances. More recently, Yoo et al. (2005) investigated the deformation behavior of two-tier segmental retaining walls on competent foundation having different wall geometries as well as reinforcement layouts. Yoo and Song (2006) later extended the work by Yoo et al. (2005) for cases constructed on yielding foundation. Although these studies provided valuable information as the subject relevant to this study, in-depth studies are warranted in order to accumulate required data for improving the currently available design guidelines.

In this study four design case histories of geosynthetic reinforced segmental retaining walls in tiered configuration were considered, intending; 1) to highlight inherent differences between the currently available design guidelines, 2) to demonstrate the governing failure mechanism that yields the smallest factor of safety is the global failure, 3) to highlight the importance of carrying out the global stability analysis as part of design. This study presents the results of a series of analyses conducted in parallel using two independent type of analyses: one based on limiting equilibrium (LE) and the other based on continuum mechanics. This paper is intended to be an extension of the previous work done by Yoo and Kim (2006).

2. Review of Design Guidelines

2.1 NCMA (National Concrete Masonry Association)

The NCMA design approach basically replaces the

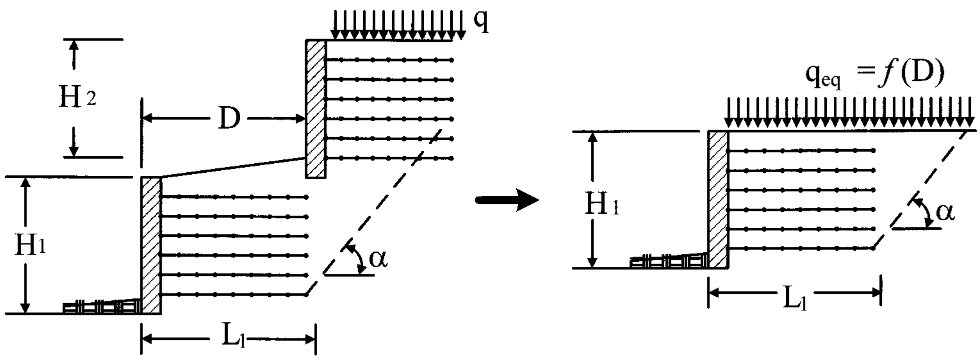


Fig. 1. Equivalent surcharge model (NCMA)

upper tier with an equivalent surcharge of which the magnitude is determined according to the offset distance D (Figure 1). External and internal stability calculations of the lower tier are performed assuming the lower tier being a single wall under the equivalent surcharge (q_{eq}). The upper wall is designed as if it were a single wall without taking into consideration of the possible interaction between the upper and the lower tiers. As for a single wall, the local stability calculations for the connection failure, local overturning, and internal sliding should be performed for both tiers. Details of the design procedure are available in Collin (1997).

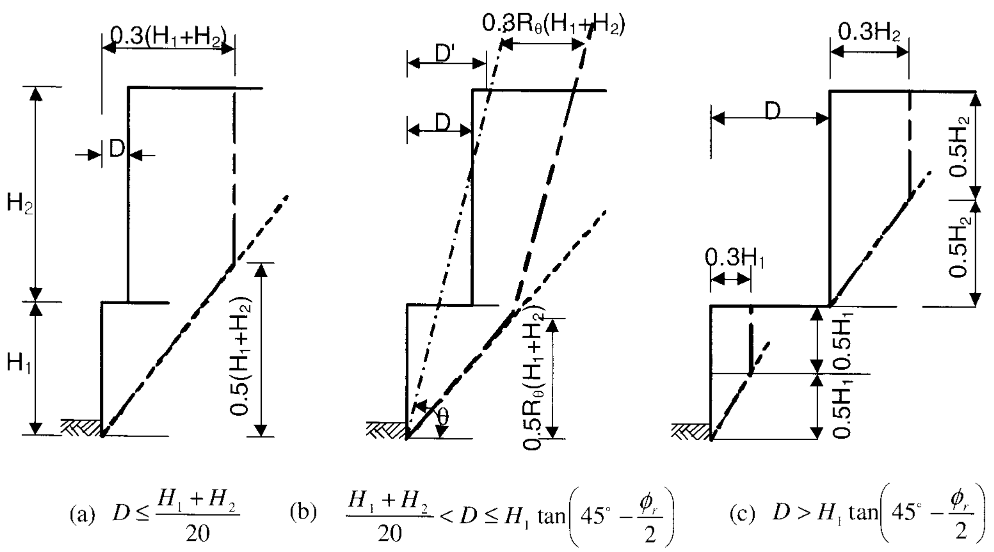
2.2 FHWA (Federal Highway Association)

In the FHWA design guideline, the required reinforcement lengths for the upper and lower tiers are determined based on the maximum tension line criteria given in Figure 2. For example, no interaction is assumed and each tier is designed independently when D > H_1 tan(90 − ϕ). When D ≤ 1 / 20(H_1 + H_2), on the other hand, the wall is designed as if it were a single wall with a height of $H = H_1 + H_2$. For walls with an intermediate offset distance of 1 / 20(H_1 + H_2) < D ≤ H_1 tan(90 − ϕ), the lower and upper tier reinforcement lengths are taken as $L_1 \geq 0.6H_1$ and $L_2 \geq 0.6H_2$, respectively. Where, H_1 = lower tier height, H_2 = upper wall height, L_1 and L_2 = reinforcement length of lower and upper tier, respectively, and ϕ = internal friction angle of backfill.

For internal stability calculations, additional vertical stresses at depths due to the upper tier are computed based on the criteria shown in Figure 3. Note, however, that these criteria are geometrically derived and empirical in nature. As for the NCMA approach, no provision is made to take into account the possible interaction between the upper and the lower tiers when designing the upper tier. The connection failure should also be checked for both tiers as part of internal stability check based on the procedure for a single wall (Elias and Christopher, 1997).

As discussed, the external and internal stability calculation models adopted in the two design guidelines are somewhat different, thus yielding different stability calculation results in terms of the factors of safety for most of the cases. In addition, although the two aforementioned design guidelines require to perform a global stability analysis to ensure overall stability, it is general practice that no global stability analysis is usually carried out in routine designs. Further study is warranted to fill the gap between the two design guidelines.

Note that in the FHWA and NCMA design guidelines



(a) $D \leq \frac{H_1+H_2}{20}$ (b) $\frac{H_1+H_2}{20} < D \leq H_1 \tan\left(45^\circ - \frac{\phi_r}{2}\right)$ (c) $D > H_1 \tan\left(45^\circ - \frac{\phi_r}{2}\right)$

-·-·-· inextensible
- - - - extensible

$D' = 2D\frac{H_2}{H_1+H_2}$ $R_\theta = \frac{\theta - \phi}{90^\circ - \phi}$

Fig. 2. Maximum tension line (FHWA)

outlined above, same minimum factors of safety for internal and external failure modes for a single wall are applicable for a multi-tiered wall. In addition, for the minimum factor of safety for global slope stability, a typical value used in a geotechnical project can be used.

3. Field Walls Considered

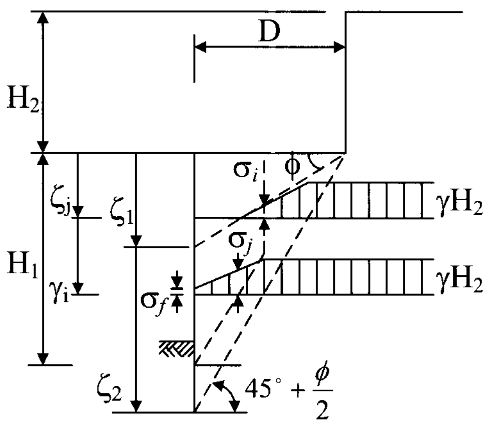
Figure 4 shows four field walls considered in this study. As summarized in Table 1, the total exposed wall heights range from 4 to 12 m with the offset distance ranging 0.23~0.45 times the total wall height (H). The reinforcement lengths vary as (0.38~0.56)H. Note that the walls are designed based on either NCMA or FHWA design approaches with the design parameters given in Table 2.

4. Stability Analysis

4.1 Internal and External Stability Analysis

The above field walls were re-analyzed by NCMA and FHWA design approaches with the aiming of demonstrating the inherent differences in the stability calculations. Table 2 summarizes the design parameters for the backfill and the reinforcement used in the stability analyses. Note that these parameters reflect the practice adopted in Korea.

The results of the external and the internal calculations are summarized in Table 3. Importance findings can



- $D \le H_1 \tan\left(45^\circ - \frac{\phi}{2}\right)$ $\quad \sigma_i = \gamma H_2$
- $D > H_1 \tan(90^\circ - \phi)$ $\quad \sigma_i = 0$
- $H_1 \tan\left(45^\circ - \frac{\phi}{2}\right) < D \le H_1 \tan(90^\circ - \phi)$

$$\sigma_f = \frac{\varsigma_j - \varsigma_1}{\varsigma_2 - \varsigma_1}\gamma H_2$$

where: $\varsigma_1 = D\tan\phi$, $\varsigma_2 = D\tan\left(45^\circ + \frac{\phi}{2}\right)$

Fig. 3. Calculation model for vertical stress increase due to upper tier (FHWA)

Table 1. Summary of wall geometry and reinforcement length

Wall	Height (m)			Offset distance D (m)	Reinforcement length (m)	
	Lower Tier, H_1	Upper Tier, H_2	Total H^1		Lower tier L_1	Upper tier L_2
A	3.8	5.4	8.8	2.5(0.34H)	4.9(0.56H)	3.5(0.7H_2)
B	5.6	5.6	10.5	2.5(0.23H)	5.3(0.50H)	3.8(0.8H_2)
C	8.8	4.4	12.4	5.0(0.40H)	7.0(0.56H)	5.0(1.3H_2)
D	2.6	2.2	4.6	2.0(0.45H)	1.6(0.38H)	1.6(0.8H_2)

Note) [1]exposed height

Table 2. Design parameters for backfill and reinforcement used in stability analysis

Wall	Backfill	Reinforcement				
		Reduction factor[1]				Tall (kN/m)[2]
A	c=0, $\phi=30^\circ$ $\gamma=18kN/m^3$	RF_D	RF_{ID}	RF_{CR}	FS	6T=16, 8T=21.5, 10T=27
B		1.05	1.1	2.15	1.5	6T=16, 10T=27
C						TYPE1=15, TYPE2=22 TYPE3=30
D						N/A

Note) [1]Reduction factors represent general practice; [2]T_{all}=allowable strength

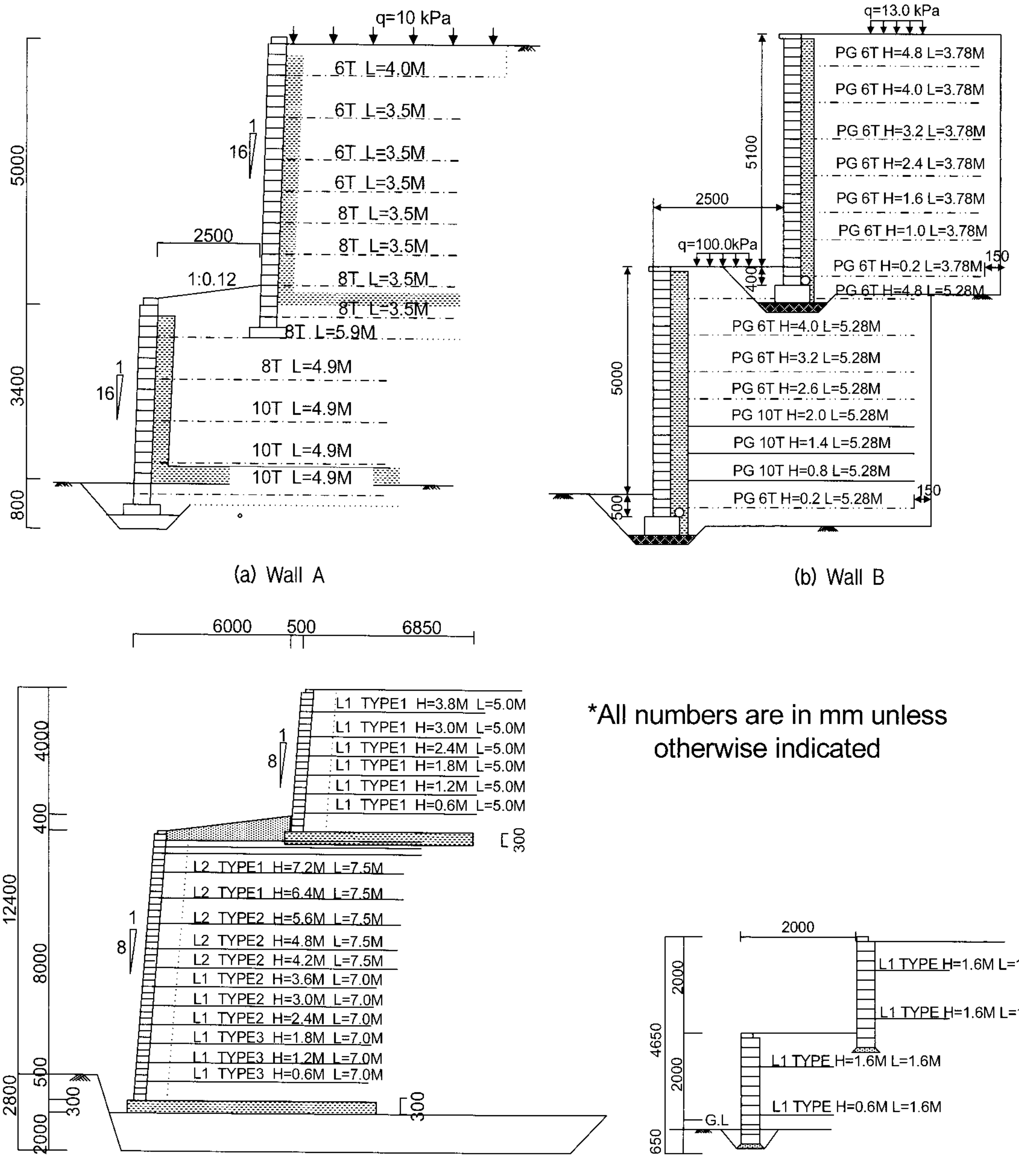


Fig. 4. Field walls considered

Table 3. Results of external and internal stability calculations for field walls

Wall	External				Internal			
	FS_{bsl}		FS_{ot}		$T_{i,max}$ (kN/m)		L_e (m)	
	NCMA	FHWA	NCMA	FHWA	NCMA	FHWA	NCMA	FHWA
A	3.13	1.27	8.87	2.13	19.7	30.5	3.4	4.1
B	2.19	1.23	4.53	1.76	19.8	36.9	1.5	2.5
C	2.79	2.02	6.09	5.01	16.0	37.5	2.4	3.9
D	1.28	1.67	3.54	1.65	9.9	19.7	0.3	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.

be summarized as follow. As seen in Table 3, the FHWA design guideline tends to give smaller factors of safety in the external analysis except for the wall D. For example, according to the NCMA design approaches walls A, B, and C satisfy the requirement for base sliding while the opposite is true according to the FHWA design approach. Additional global stability analyses in fact support the instability against base sliding as the factors of safety against global/compound stability for all the walls are less than the minimum of 1.2 (to be shown later), which suggests that the global/compound stability analysis should be conducted as required in the two design approaches.

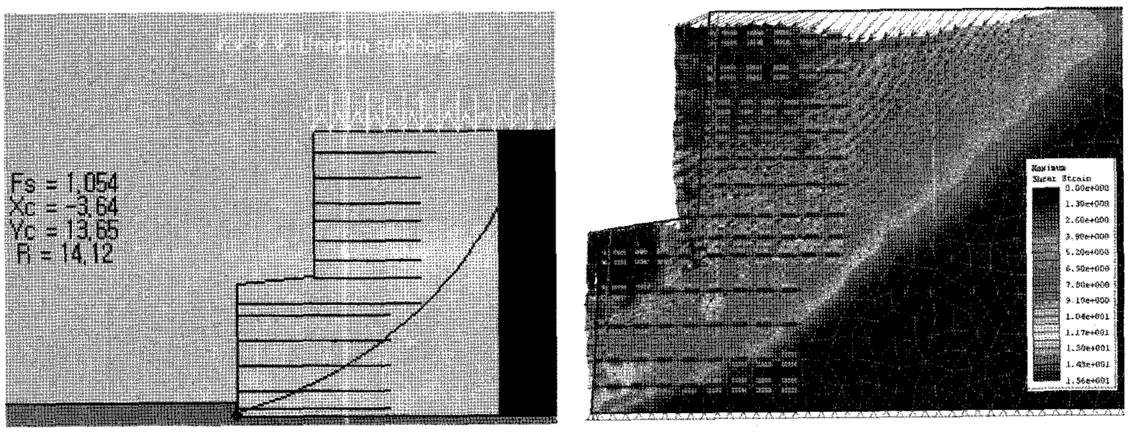
In terms of the internal stability calculations, the FHWA design approach gives significantly larger maximum reinforcement loads and the embedment lengths beyond active failure line than the NCMA design approach giving larger pullout capacities. Apart from the different design earth pressures adopted in these 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. Note that the NCMA and the FHWA design guidelines adopt the Coulomb and Rankine active earth pressures, respectively. Such differences may give designers confusion to some extent in selecting proper reinforcements in terms of strength. Further study is warranted to fill the gap between the two design approaches.

4.2 Global Slope Stability Analysis

A series of global slope stability analyses were additionally performed on the field walls, aiming at examining if the reinforcement layouts of the walls also satisfy the global stability requirement. The limit-equilibrium (LE) as well as the continuum mechanics based slope stability analyses were performed using, MSEW ver. 1.0 (Leshchinsky 1999) and Phase^2 (Rocscience, 2005), respectively. Note that the finite element analysis in conjunction with the shear strength reduction method (Griffths and Lane 1999) was employed as the continuum mechanics based approach. Two different approaches were adopted in this study to see if the two independent types of analyses would yield similar results so that an acceptable level of confidence in the results can be afforded. One of the advantages of the finite element analysis with the shear strength reduction (FEM-SSR) over traditional limit equilibrium approach is that no assumption needs to be made a priori regarding the shape or location of the failure surface.

In the finite element analysis with the shear strength reduction method (FE-SSR), the factor of safety (*FS*) of a slope can be defined as the number by which the original shear strength parameters must be divided in order to bring the slope to the point of failure (Griffths and Lane, 1999) so that the factored shear strength parameters (c_f', ϕ_f') can be defined as:

$$c_f' = \frac{c'}{FS} \tag{1}$$



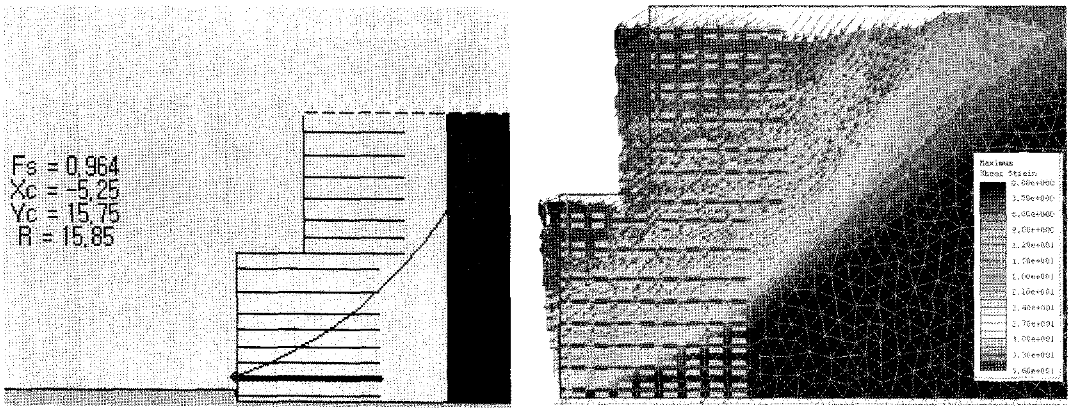
(a) LEM, FS=1.05 (b) FE-SSR, FS=0.92

Fig. 4. Global stability analysis : Wall A

$$\phi_f' = \frac{\phi'}{FS} \tag{2}$$

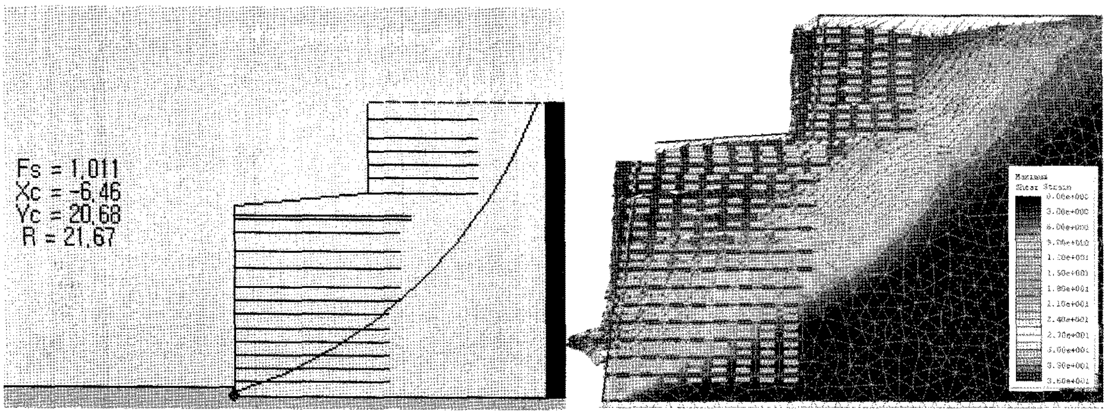
Note here that this definition of FS is the same as that adopted in the traditional LE methods. When adopting the shear strength reduction approach, there are several possible definitions of failure, e.g., non-convergence of the solution (Zienkiewicz and Talyor, 1989) or acceleration of slope displacement, etc. Details of the FE-SSR can be found in Griffths and Lane (1999).

The results of the global stability analyses, in terms of the minimum factors of safety and the corresponding failure surfaces, are given in Figures 4~7. The factors of safety values for each wall are summarized in Table 4. Note that the LE slope stability analyses were conducted based on the modified Bishop method. Salient features that can be observed in these figures are two-fold. First, for a given wall, the minimum factors of safety computed by the LE and the FE-SSR analyses are in good agreement, although the factors of safety from the FE-SSR are somewhat smaller (less than 10%) than those from the LE approach. Second, the potential failure surfaces from the two approaches are also similar in shape. These results demonstrate that the FE-SSR approach can also be effectively used in the global stability analysis of reinforced earth structures with an acceptable level of confidence. Another important observation is that for all the walls investigated, the minimum global factor of safety is smaller than those of the external stability calculations for the base sliding and over turning failure modes. Such a trend implies that the governing failure mechanism in terms of external stability is the global slope failure for walls in tiered configuration with an intermediate offset distance. A global stability check must be performed in addition



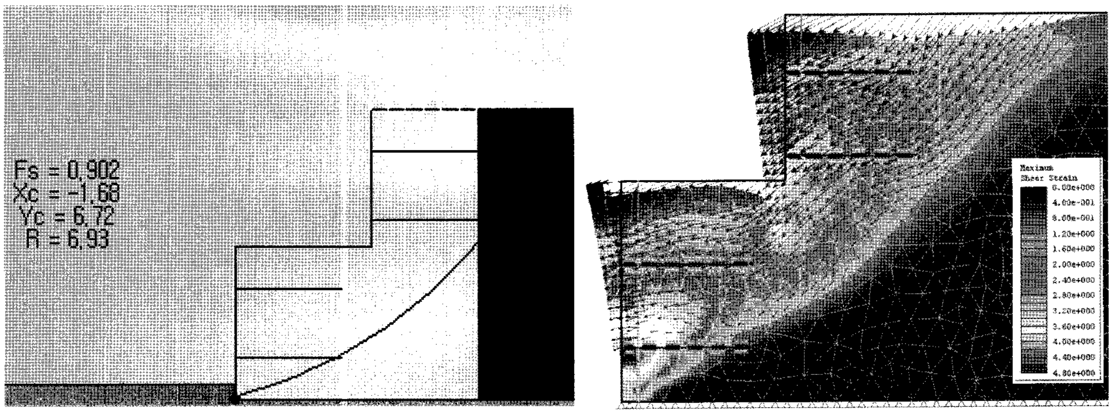
(a) LEM, FS=0.96 (b) FE-SSR, FS=0.93

Fig. 5. Global stability analysis : Wall B



(a) LEM, FS=1.01 (b) FE-SSR, FS=0.98

Fig. 6. Global stability analysis : Wall C



(a) LEM, FS=0.90 (b) FE-SSR, FS=0.82

Fig. 7. Global stability analysis : Wall D

Table 4. Summary of global stability analysis

	Factor of Safety			
	Wall A	Wall B	Wall C	Wall D
LE	1.05	0.96	1.01	0.90
FEM-SSR	0.92	0.93	0.98	0.82

to the external stability check when determining the reinforcement lengths.

4.5 Reinforcement Distribution to Meet Global Stability Requirement

Another series of global stability analyses were performed to determine the reinforcement distributions that meet the global stability requirement, taking the required minimum factor of safety as $FS_{\min}$ = 1.20. The results are given in Table 5 and Figure 8.

The results indicate that both the upper and lower tier reinforcement lengths need to be increased as great as by 50% to meet the global stability requirement. The results also show that the lower and upper parts of the upper and lower tiers, respectively, require much longer reinforcement lengths than those satisfying the external stability. Such a trend stresses that the global stability analysis is not an option but a requirement when designing GR-SRWs in tiered configuration with an intermediate offset distance. Another important observation is that the revised reinforcement lengths for the upper tiers in all walls are significantly longer than those required by the design guideline in which the upper tier is designed as an independent wall. The fact that both tiers' reinforcement lengths need to be increased to ensure the global stability requirement suggests that the interaction between the upper and lower tiers can be explicitly accounted for by performing the global stability analysis.

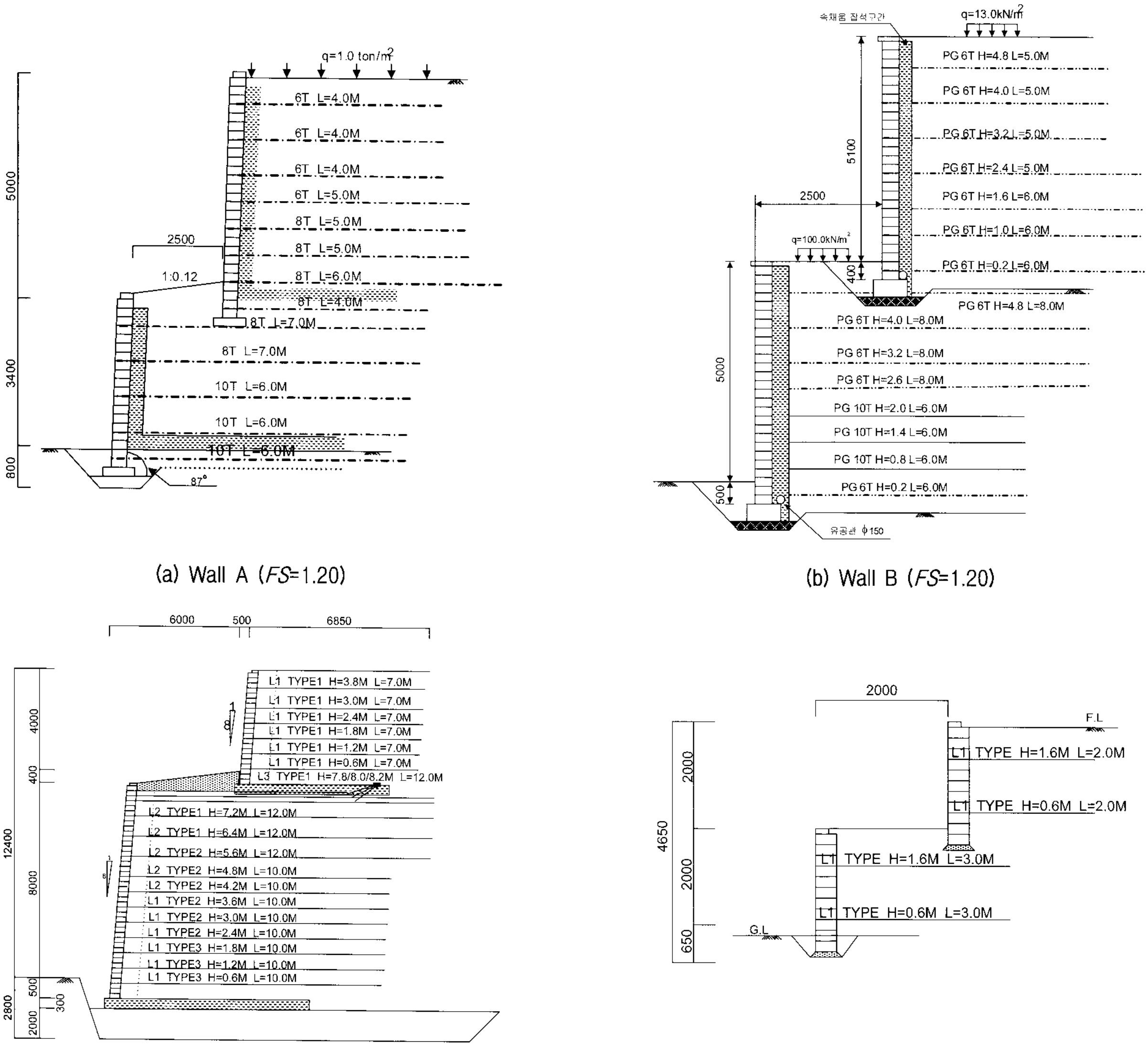
5. Summary and Conclusions

This paper presents the results of stability analyses on geosynthetic reinforced segmental retaining walls in tiered

Table 5. Summary of revised reinforcement lengths to meet global stability

Wall	Offset distance D (m)	FS		Reinforcement length (m)			
				Lower tier (L_1)		Upper tier (L_2)	
		as-designed[1]	revised	as-designed	revised[2]	as designed	revised[2]
A	2.5(0.34H)	0.92	1.20	4.9(0.55H)	7(0.80H)	3.5(0.65H_2)	6(1.11H_2)
B	2.5(0.23H)	0.93	1.20	5.3(0.50H)	8(0.76H)	3.8(0.68H_2)	6(1.07H_2)
C	5.0(0.40H)	0.98	1.20	7.0(0.56H)	12(0.97H)	5.0(1.13H_2)	7(1.49H_2)
D	2.0(0.45H)	0.82	1.26	1.6(0.35H)	3(0.65H)	1.6(0.73H_2)	2(0.91H_2)

Note) [1]based on FE-SSR; [2]maximum length



(a) Wall A (FS=1.20)

(b) Wall B (FS=1.20)

(c) Wall C (FS=1.20)

(d) Wall D (FS=1.26)

Note) All numbers are in 'mm' unless otherwise indicated.

Fig. 8. Reinforcement distributions to meet global stability requirement

configuration. Four design cases of walls with different geometries and offset distances were considered. Based on the results of stability analyses using the FHWA and NCMA design guidelines, the discrepancies between the two different guidelines were identified. A series of global slope stability analyses were conducted using the limit-equilibrium analysis and the continuum mechanics based shear strength reduction method aiming at investigating governing mode of failure for walls with an intermediate offset distance.

The results indicated among other things that the FHWA design approach yields conservative results both in the external and internal stability calculations, i.e, lower factors of safety, than the NCMA design approach. In addition to the different design earth pressures, 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. Also found is that required reinforcement lengths are usually governed by the global slope stability requirement rather than the external stability calculations, thus demonstrating the global stability analysis should be part of design calculations in addition to the internal and external stability checks. It is shown that when considering the global stability requirement, the required reinforcement lengths for the upper tiers are much longer than those based on the current design guidelines in which the upper tier is treated as an independent wall. These results warrant that a global stability based design approach needs to be developed for geosynthetic reinforced segmental retaining walls in tiered configuration.

Acknowledgements

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