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Behavior of a Geosynthetic Reinforced Two-tier Segmental Retaining Wall on a Yielding Foundation

압축성이 큰 지반 위에 시공되는 계단형 블록식 보강토 옹벽의 거동

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요 지

본 논문에서는 압축성이 큰 지반 위에 시공되는 지오그리드 보강 계단형 블록식 보강토 옹벽의 거동 특성에 관한 내용을 다루었다. 대상옹벽으로는 2단으로 시공되는 높이 10m의 블록식 보강토 옹벽을 고려하였으며 압축성이 큰 지반위에 시공되는 경우를 고려하였다. 본 연구는 검증된 유한요소해석 모델을 사용하여 수행되었으며 기초지반의 침하가 옹벽의 변위 및 보강재 유발 인장력에 미치는 영향을 집중적으로 다루었다. 연구결과 시공중 발생하는 기초지반의 과다 침하는 옹벽의 변위 뿐 아니라 보강재 유발 인장력 또한 현저히 증가시켜 내·외적 안정성에 지대한 영향을 미치는 것으로 나타났다. 본 논문에서는 연구결과와 아울러 실무적 관점에서의 고려사항을 기술하였다.

Abstract

This paper presents the results of a numerical investigation on the behavior of a geosynthetic reinforced two-tier segmental retaining wall (GR-SRW) on a yielding foundation. A hypothetical 10 m high two tier GR-SRW to be constructed on an incompetent foundation containing a layer of relative soft soil deposit was considered. A verified finite-element procedure was employed to get insights into the effect of foundation yielding on the wall behavior including the wall deformation and the reinforcement load. It is shown that the effect of foundation yielding is to increase the wall deformation as well as the reinforcement load, thus influencing both the internal as well as the external stability of the wall. Practical implications of the findings obtained from this study are highlighted in this paper.

Keywords : Finite element analysis, Foundation yielding, Reinforced earth, Segmental retaining wall, Tiered wall

1. Introduction

Geosynthetic-reinforced segmental retaining wall systems have become increasingly popular in Korea since its first appearance in the early 1990'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. Although many geosynthetic reinforced soil walls have been safely constructed and are performing well to date, there are

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many areas that need in-depth studies in order to better understand the mechanical behavior of SRW systems under more aggressive and harsh environments. 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 constraints, 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), as defined in Figure 1, i.e., D = 0.3 to 1.0 times lower tier height.

Studies concerning multitiered reinforced earth walls are scarce. Yoo and Kim (2002) performed a numerical and experimental investigation on the general behavior of two-tier GR-SRW. They reported that the interaction between the upper and lower tiers has a considerable influence on the overall wall behavior, and the equivalent surcharge approaches adopted in the current design approaches (Collins 1997, Elias and Christopher 1997) may yield unconservative results in some cases. More recently, Yoo and Jung (2004) reported the measured behavior of a 5 m high two level GR-SRW constructed as a full scale test wall. They presented the wall deformation and reinforcement tensile strains measured during construction and highlighted the importance of considering the interaction between the upper and lower tiers in the design of geosynthetic wall in a tiered configuration. Another relevant study pertaining to the multitiered reinforced earth walls is perhaps the one by Leshchinsky and Han (2004) in which the results of limit equilibrium and continuum-based analyses on the multitiered reinforced earth walls are presented.

Although the aforementioned studies successfully identified the fundamentals of the behavior of multitiered reinforced earth walls, they are limited to walls on a non-yielding competent foundation. Reflecting that the construction environments for reinforced earth walls are increasingly becoming harsh, there are many situations where GR-SRWs are constructed on a yielding foundation. The current design approaches however do not address design issues related to the behavior of multitiered reinforced earth walls constructed on a yielding foundation. Recognizing the need for a further study into this subject, a calibrated FE model was employed to examine the effect of foundation yielding on the behavior of a two-tier GR-SRW. This paper describes the finite-element model used, the results of a parametric study, and finally, practical implications for design.

2. Review of Design Methods for Tiered GR-SRW

The NCMA and the FHWA design guidelines adopt different assumptions regarding the wall behavior. Fundamental principles of each design approach are discussed under the subsequent subheadings.

2.1 NCMA Design Approach

The NCMA design approach (Collin 1997) basically replaces the upper tier with an equivalent surcharge of which its magnitude is determined according to the offset

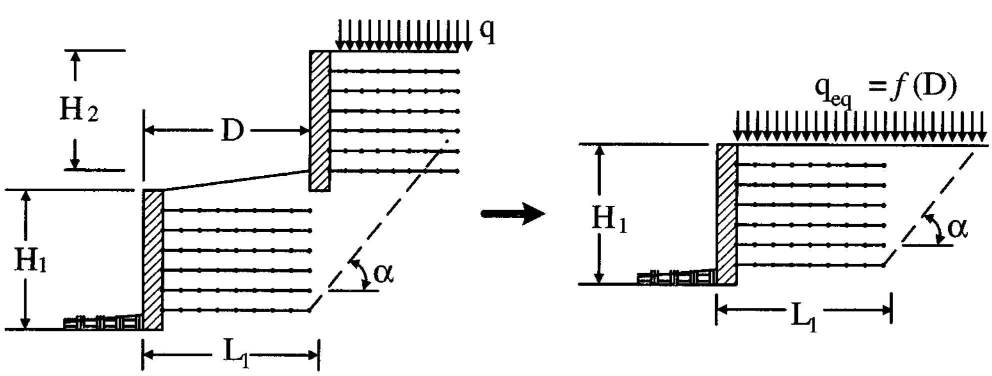


Fig. 1. Equivalent surcharge model (NCMA)

distance D (Figure 1). External and internal stability calculations for the lower tier are performed assuming the lower tier being a single wall under the equivalent surcharge. The upper tier 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 are required to be checked for both tiers. Details of the design procedure are available elsewhere (Collin 1997).

2.2 FHWA Design Approach

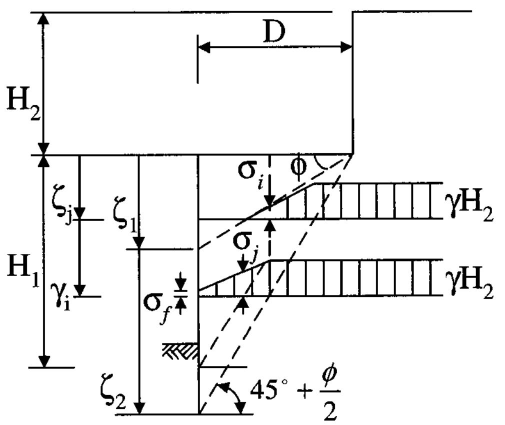
The FHWA design guideline (Elias and Christopher 1997) requires determining the lower and the upper reinforcement lengths L1 and L2, respectively, that satisfy external stability requirements based on the offset distance D together with the lower and upper tier height, H_1 and H_2, respectively. When $D > H_1 \tan(90° - \phi)$, no interaction is assumed and each tier is independently designed. On the other hand, for cases with $D \le \frac{1}{20}(H_1 + H_2)$, a single wall with a height of H = H_1+H_2 is assumed. When $D > \frac{1}{20}(H_1 + H_2)$, the interaction between the two tiers is assumed and the reinforcement length for the lower and upper tiers are selected, respectively, as $L_1 \ge 0.6H_1$ and $L_2 \ge 0.7H_2$.

For internal stability calculations, additional vertical stresses at depths due to the upper tier are computed based on the criteria given in Figure 2. The location of the potential failure surface required for the pullout capacity calculation is selected based on the offset distance D (Elias and Christopher 1997). 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 the internal stability calculations based on the procedure for a single wall (Elias and Christopher 1997).

3. Problems Considered

3.1 Wall Geometry and Reinforcement Layout

A 10 m high GR-SRW with a two-tier wall arrangement is considered in this study (Figure 3). It is assumed that modular blocks of 520×460 mm in plan and 200 mm in height are used to form the wall facing with no set-back, and that two types of high density polyethylene geogrids, 6T and 10T having an index strength of 60 and 100 kN/m, respectively, are used as primary reinforcements. As a baseline case, the reinforcement layers are assumed to be 5.3 and 3.8 m long, respectively, for the lower and the upper tiers with the vertical spacing of 0.6~0.8 m. Such a reinforcement layout is chosen to satisfy the external and internal stability requirements according to the NCMA design approach as indicated in Figure 3. As in the typical Korean practice, decomposed granite soil



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

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

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

ϕ = internal friction angle

γ = unit weight of select fill

Fig. 2. Calculation model for vertical stress increase due to upper tier (FHWA)

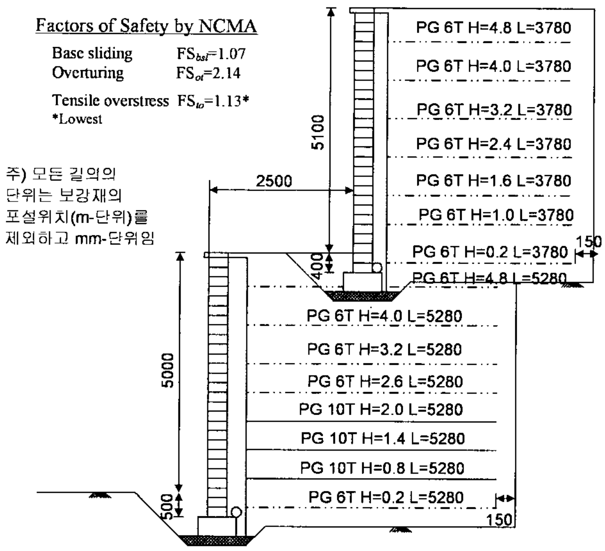


Fig. 3. Geometry of Multitier wall Considered

classified as SP-SM is assumed to be compacted to 95% of standard Proctor to form the reinforced zone. An internal friction angle of $\phi=30°$ is assumed both for the backfill and for the retained soil. The results of the external and internal calculations based on the NCMA design approach are given in Figure 3.

3.2 Conditions Analyzed

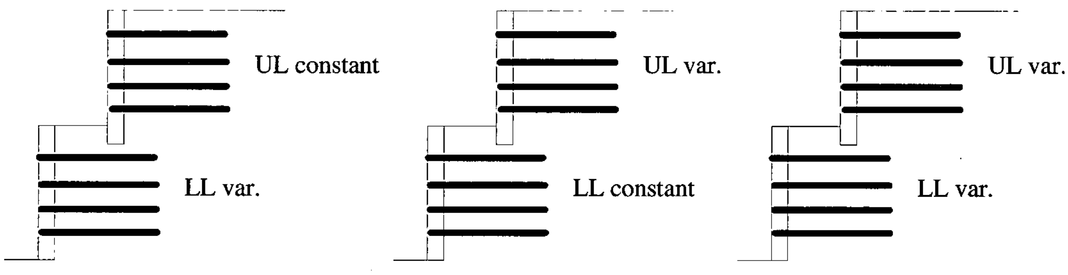
In terms of the foundation condition, both yielding and non-yielding foundation conditions are considered by assigning relevant stress-strain-strength relationships. For the yielding foundation case, a number of different reinforcement layouts are considered in order to get insights into the effect of reinforcement layout on the wall behavior. Also considered is the reinforcement stiffness. The reinforcement layout in Figure 3 was selected as a baseline condition. The reinforcement density in terms of vertical spacing is not considered as a variable in this study since preliminary analyses indicated that the effect of reinforcement spacing was insignificant.

A series of analysis was conducted by varying either the lower or the upper reinforcement length at a time while keeping the other tier's reinforcement length constant (Scheme A and B). For each scheme, a uniform reinforcement length was assumed in each tier. An additional set of cases in which the reinforcement lengths in both tiers were uniformly increased was also analyzed (Scheme C). In addition to the reinforcement length, two levels of reinforcement stiffness were considered, i.e., J=1000 kN/m and 2000 kN/m. Schematic views of the schemes considered are shown in Figure 4.

4. Finite Element Analysis

A commercial finite element code ABAQUS (Hibbitt, Karlsson, and Sorensen 2002) was used for analysis. ABAQUS was used in this study to take advantage of its robustness in numerical solution strategy for soil nonlinearity. In the finite-element modeling, the wall components were carefully modeled including the foundation. The wall facing, the backfill soil, and the foundation were discretized using 8-node plane strain elements (CPE8R) with reduced integration, while the reinforcement was modeled using 3-node truss elements (T3D2).

A refined mesh (Figure 5), consisting of over 5800 nodes and elements, respectively, was adopted to fully



(a) Scheme A (b) Scheme B (c) Scheme C

LL=lower reinforcement legnth UL=upper reinforcement legnth

Fig. 4. Reinforcement distributions considered

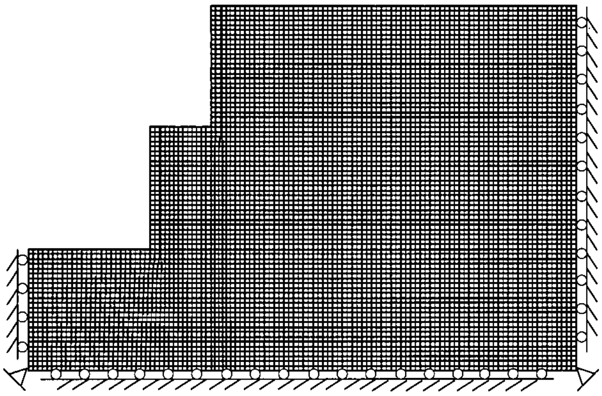


Fig. 5. Finite element mesh

account for the construction procedure and to minimize the effect of mesh dependency on the results of finite-element analyses. The lateral and bottom boundaries were placed at locations with sufficient distance. The interface behavior between the wall facing and the backfill soil was modeled using a layer of interface elements (Desai et al. 1984) with appropriate mechanical properties. No interface was introduced between the soil and the reinforcements assuming no slip between the backfill and the reinforcements. This is justified since pullout tests on many soils show that slip occurs in the soil and not at the interface of the geogrids, unless the confining stress is extremely small.

In the analysis, the backfill and the foundation soils were assumed to follow the modified version of hyperbolic stress-strain and bulk modulus model proposed by Duncan et al. (1980) while the walls facing block and the reinforcement were assumed to behave in a linear elastic manner. In addition, for the interface elements between the walls facing block and the backfill soil, a relatively low shear modulus but with a high bulk modulus was assigned to permit relative movement between the two media to occur. The constitutive laws for the soil and the interface were implemented to ABAQUS with the help of the built-in "User Subroutine" capability. In the hyperbolic model, the stress increments ($\Delta\sigma_d$) are related to the strain increments ($\Delta\epsilon_d$) based on the tangential Young's modulus E_t and/or unloading-reloading modulus E_{ur}, and bulk modulus B which are computed using the Mohr-Coulomb soil strength parameters of c' and ϕ' in conjunction with the hyperbolic model parameters including stiffness modulus number for primary loading K, stiffness modulus number for unloading-reloading K_{ur}, bulk modulus number K_b, stiffness modulus exponent n, bulk modulus exponent m, and failure ratio R_f. Considering the free draining characteristic of typical decomposed granite soils in Korea, a fully drained condition was assumed. Table 1 summarizes the input parameters for various wall components. It should be noted that the hyperbolic parameters for the backfill and the foundation soil were the "best-estimate" parameters based on local experience as well as the database provided by Duncan et al. (1980). It should be noted that the material properties pertaining to the yielding foundation are relevant for a foundation soil with the average Standard Penetration blow counts N value of 15~20.

On account of the discrete nature of the modular block facing, the Young's modulus of the facing block was reduced to 1/10 of that of concrete, giving a wall flexural stiffness of $(EI)_w$=20 MN-m^2/m. The detailed construction sequence was carefully simulated by adding layers of soil and reinforcement at designated steps. The finite-element modeling approach adopted in this study was calibrated against available instrumentation data for a full-scale tiered SRW. Details of the model verification are available elsewhere (Yoo 2003).

Table 1. Material properties used in finite element analyses

Material		c' (kPa)	ϕ'(°)	K	K_{ur}	n	R_f	K_b	m	E_s (MPa)
Backfill		0	30	300	350	0.5	0.8	175	0.2	–
foundation	yielding	20	31	400	450	0.5	0.8	175	0.2	–
	non-yielding	100	40	1000	1500	0.7	0.9	800	0.2	–
facing block		–	–	–	–	–	–	–	–	2000

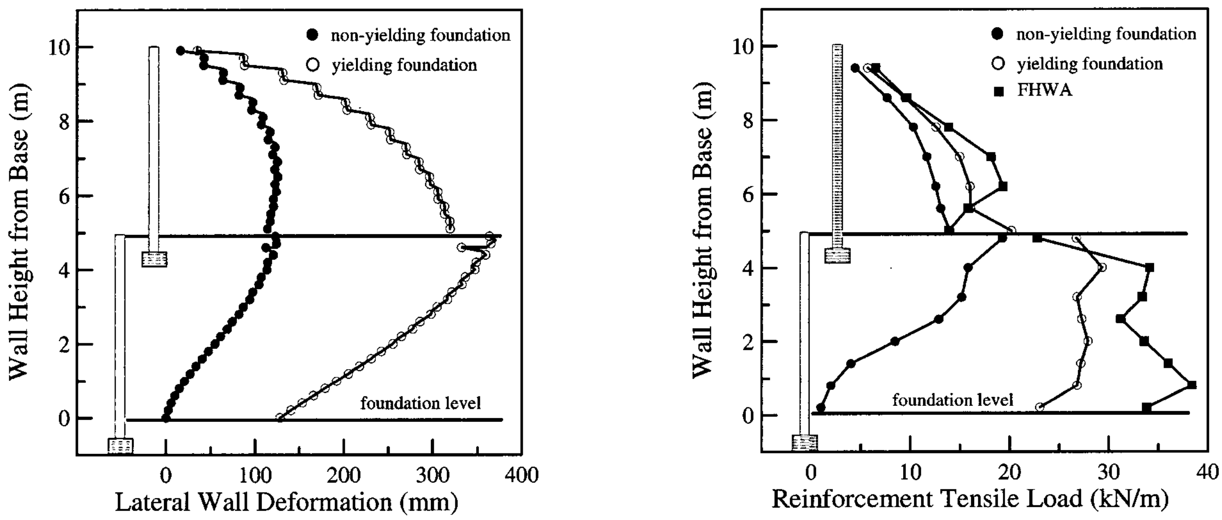
Note: For all materials, unit weight $\gamma=18$ kN/m^3 and poisson's ratio $\upsilon=0.3$

5. Results and Discussion

5.1 Effect of Foundation Yielding

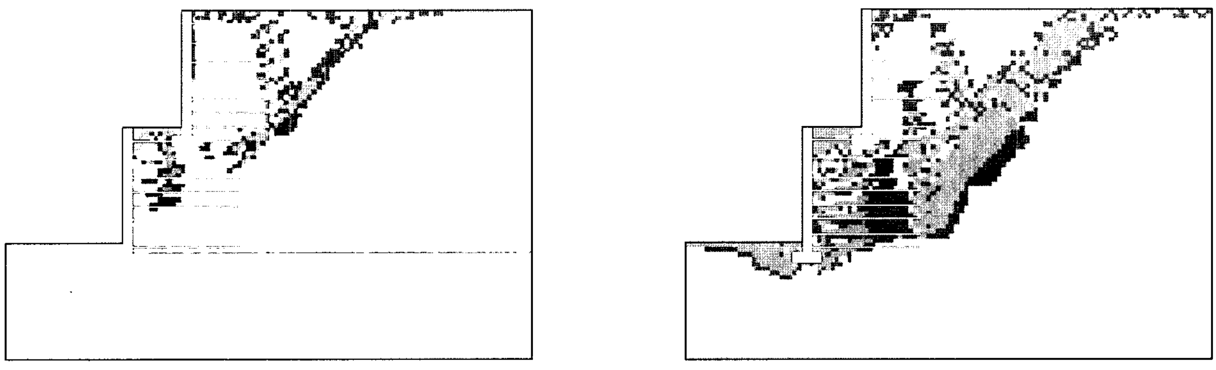
The effect of foundation yielding on the mechanical behavior of the baseline condition was examined in terms of wall deformation, reinforcement load, and stress state for two extreme cases in terms of the foundation rigidity, i.e. yielding and non-yielding foundations.

Figures 6 and 7 illustrate calculated lateral wall deformations, maximum reinforcement loads, and distributions of the shear stress ratio (SR). Note that the shear stress ratio is defined as the ratio of mobilized shear stress to shear strength. Also shown are the calculated reinforcement tensile loads according to the FHWA design approach. As seen in these figures, the effect of foundation yielding is to increase the wall deformation as well as the reinforcement tensile loads in the lower tier. In fact, the foundation yielding caused an over 200% increase in the maximum lateral wall deformation at the top of the lower tier with a significant increase in the associated reinforcement tensile loads in the lower tier as great as 20 kN/m in the lower-most reinforcement layer. The interaction between both tiers also accompanied increases in the wall deformation as well as the reinforcement loads in the upper tier, especially at the base level. This trend is the result of the interaction between the lower and the upper tiers, and is not addressed in the current design assumptions. Although the reinforcement loads from the FE procedure fell below those computed based on the FHWA approach, the significant increases in the reinforcement loads arising from the foundation yielding observed



(a) wall deformation (b) tensile load

Fig. 6. Effect of foundation yielding on wall behavior



(a) non-yielding foundation (b) yielding foundation

Fig. 7. Stress ratio distribution

in the FE results imply that the foundation yielding needs to be correctly accounted for in order to avoid any potential internal stability problems. The distributions of the shear stress ratio (SR) shown in Figure 7, with the zones having a ratio greater than 90% being indicated, suggested that a global shear failure is likely for the yielding foundation case.

The results presented above strongly demonstrate that the performance of multitier walls is significantly influenced by the foundation yielding and warrants a proper consideration of such an effect in the design of both the upper and lower tiers. Also highlighted is the need for carrying out the global slope stability analysis in addition to the external and internal design calculations.

5.2 Effect of Reinforcement Distribution

The variations of the maximum lateral wall deformation ($\delta_{h,\max}$) and the stress ratio distribution are shown in Figure 8 for the different schemes in increasing the reinforcement length. As expected, increasing the reinforcement length, either the lower (LL) or the upper reinforcement length (UL), causes a decrease in the maximum wall deformation in all schemes but with different rates. Of importance trend is that the maximum lateral wall deformation is significantly reduced when keeping LL/H>0.6 in Schemes A and C, suggesting that LL/H=0.6 is a minimum critical length required to avoid excessive lateral wall deformation. Such a trend is not so evident in Scheme B in which UL/H varies, although the rate of decrease with UL/H decreases somewhat when UL/H>0.7.

Figure 9 presents stress ratio distributions for cases having the same average reinforcement length TL/H=0.7 but with different combinations of LL/H and UL/H. Note that TL is an average reinforcement length in both tiers, i.e., TL=(LL+UL)/2. One important trend shown in this figure is that the stress states within the reinforced soil block and the retained soil vary with the reinforcement distribution although TL/H is the same. In fact, the case with a uniform length in both tiers, i.e., LL/H=UL/H=0.7, yielded the most favorable stress state, i.e., least extent of zones of $SR \geq$. Such information cannot be obtained from conventional limit-equilibrium based design/analysis tools, thus illustrating advantages of employing numerical

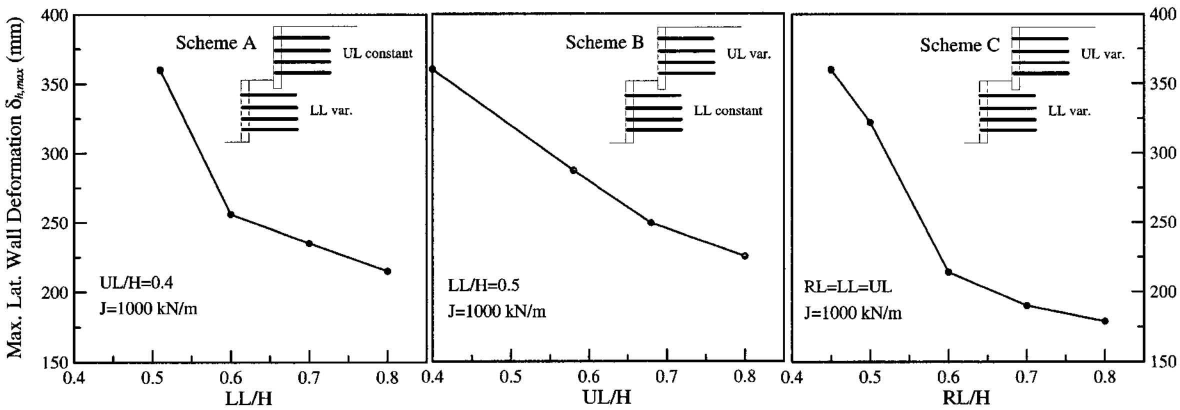


Fig. 8. Variation of $\delta_{h,\max}$ with reinforcement distribution

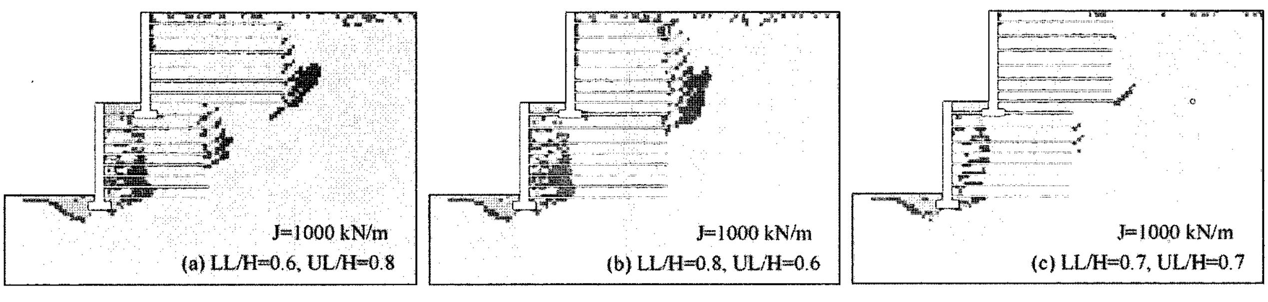


Fig. 9. Stress ratio distributions for cases with different reinforcement distributions

modeling techniques in the design of reinforced soil walls with complex geometry.

5.3 Effect of Reinforcement Length

Figures 10 and 11 illustrate the variations of maximum lateral wall deformation $\delta_{h,\max}$ and maximum reinforcement strain $\epsilon_{\max}$ with LL/H and UL/H for two levels of stiffness, J=1000 and 2000 kN/m. As expected, $\delta_{h,\max}$ and $\epsilon_{\max}$ decrease with increasing either of the reinforcement length to wall height ratios (LL/H, UL/H) but with different rates depending on the reinforcement stiffness. For example, the results for J=2000 kN/m exhibit narrower ranges of variation in $\delta_{h,\max}$ with UL/H than for J=1000 kN/m. Of interest trend shown in Figure 10 is that for a given TL/H, any combination of LL/H and UL/H tends to yield practically the same $\delta_{h,\max}$ at least within the range of the reinforcement lengths considered, as illustrated in Figure 10 (c).

Furthermore, as seen in Figure 11, the maximum tensile strain in the lower tier reinforcement appears to be mainly influenced by the upper tier reinforcement length and remains practically the same regardless of LL/H for a given UL/H, suggesting that the variation in $\delta_{h,\max}$ with LL/H for a given UL/H in fact represents the lateral deformation at the back of reinforcement soil block (external deformation). Such a trend is due in part to the fact that the range of reinforcement lengths considered (LL/H=0.6~0.8) is close to the critical LL/H. Further inspection of Figure 11 reveals that for a given TL/H, the maximum reinforcement strain $\epsilon_{\max}$ can be minimized when adopting longer reinforcement in the upper tier than in the lower tier. This trend is largely due to the fact that the longer upper tier reinforcement reduces the upper tier-induced surcharge load through stress redistribution. In view of reducing reinforcement loads, it is therefore desirable to arrange longer reinforcement in the upper tier than in the lower tier for a given TL/H.

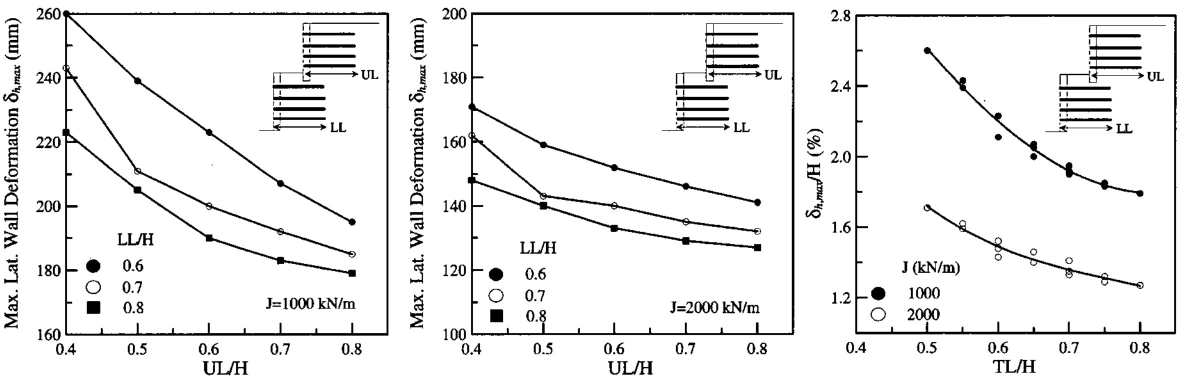


Fig. 10. Variation of $\delta_{h,\max}$ with reinforcement length

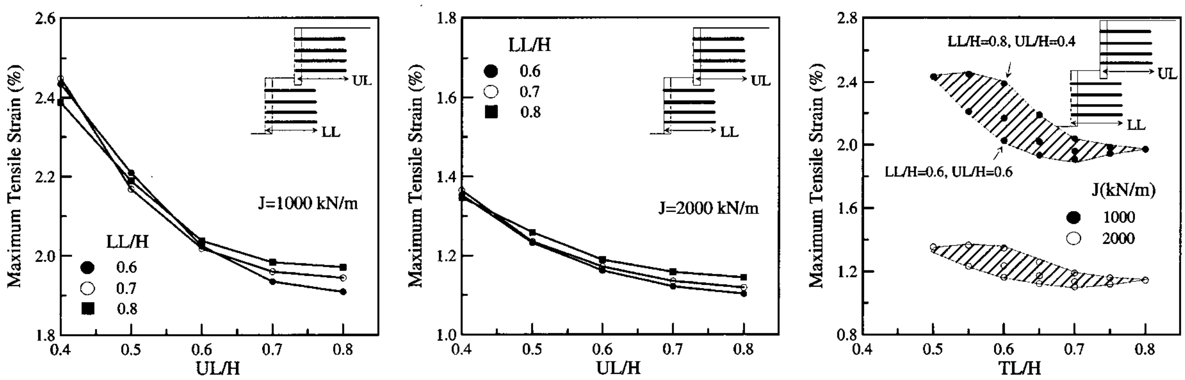


Fig. 11. Variation of $\epsilon_{\max}$ with reinforcement length

6. Conclusions

This paper presents the results of a parametric study on the effect of foundation yielding on the behavior of geosynthetic reinforced two-tier segmental retaining wall. A verified finite-element model was used as a tool for identifying fundamentals pertaining to this subject. A number of cases were analyzed by varying the reinforcement layout as well as the stiffness of the reinforcement.

The results revealed that the foundation yielding may significantly increase the wall deformation and the associated reinforcement loads. Furthermore, for a given average reinforcement length TL/H, any combination of LL/H and UL/H yields practically the same $\delta_{h,\max}$ at least within the range of the reinforcement lengths considered as long as $LL/H \geq 0.6$. The maximum tensile strain in the reinforcement layers in the lower tier is mainly influenced by the upper tier reinforcement length and remains practically the same regardless of the reinforcement length in the lower tier for a given UL/H. Further inspection of the results also indicated that for a given TL/H, it would be a bit more beneficial to employ longer reinforcement in the upper tier in view of reducing reinforcement loads in the lower tier.

Demonstrated in this paper is that the effect of foundation yielding should be explicitly considered in the design of geosynthetic reinforced walls on a yielding foundation, especially for multitiered walls. Also highlighted is the imporatacne of carrying out the global slope stability analysis in addition to the external and internal design calculations for multi-tier walls. Further in-depth studies are warranted to better understand the effect of foundation yielding on multitiered walls with complex geometry.

Acknowledgement

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