EVALUATION OF VARIOUS DESIGN METHODS FOR PREDICTING REINFORCEMENT LOADS WITHIN GEOSYNTHETIC-REINFORCED SOIL STRUCTURES

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ABSTRACT

Proper estimate of reinforcement loads is a key to evaluate internal stabilities of Geosynthetic-Reinforced Soil (GRS) structures. Prediction methods for reinforcement loads within GRS structures in current research and practice can be categorized into two approaches: force equilibrium approach (i.e., earth pressure method and limit equilibrium method) and deformation-based approach (i.e., K-stiffness method and finite element method). Until today, the accuracy of these methods has not been extensively examined and evaluated yet. This paper first introduced each method and discussed their advantages and disadvantages. Afterward, the reinforcement loads measured from a 3.6m high full-scale GRS structure with careful construction and instrumentation were used to examine the prediction of reinforcement loads by the aforementioned methods. Comparison results indicated the force equilibrium approach including earth pressure method and limit equilibrium method overly predicted the reinforcement loads. The finite element method agreed well with the measured data at working stress conditions but numerical illness (i.e., convergence problem) may occur earlier than the actual failure of structure at large loading conditions. The K-stiffness method showed an obvious underestimate under surcharging conditions. Reasons of discrepancy between predicted reinforcement loads and measured data were discussed. The results obtained from this study provide insightful information for the design of GRS structures.

Key Words: Geosynthetic-reinforced soil structure, Reinforcement load, Force equilibrium, Deformation

1 INTRODUCTION

Mechanical Stabilized Earth (MSE) retaining structures are now widely used in various projects including residences, highways, bridge abutments, and slope stabilization for the purposes as increasing Right of Way (ROW), resisting earth pressures, and providing load bearing on top of MSE structures, and allowing for changes of elevation in highway projects. A number of factors have propelled the acceptance of MSE retaining structures. These include qualities like aesthetics, reliability, and low cost. Moreover, good construction techniques, impressive seismic performances, and a striking ability to withstand large deformations without structural distress account for MSE structures desirability. The study presented in this paper will specifically focus on MSE retaining structures with extensible reinforcements, commonly called Geosynthetic-Reinforced Soil (GRS) retaining structures.

The three primary agencies identified in the most recent MSE structure design specifications in North America are the American Association of State Highway and Transportation Officials (AASHTO 2002), Federal Highway Administration (FHWA) (Elias et al. 2001), and National Concrete Masonry Association (NCMA 2010). In these design guidelines, the design of MSE retaining structures is the result of a synergistic approach. Figure 1 shows the wall system is analyzed for internal, external, global and seismic stability as well as deformability. MSE structures must meet certain factors of safety, $FS$, against all failure models.

In analyzing the internal stability of GRS structures, it requires to predict the maximum reinforcement tensile load, $T_{max}$, in each reinforcement layer. Having knowledge of the forces in reinforcements enables one to select reinforcements having adequate long-term strength (against breakage), to calculate the length required to resist pullout within the stable soil zone (against pullout), and to calculate the required connection strength at facing (against connection failure). As a result, the evaluation of $T_{max}$ is a key for the internal stability analyses of MSE structures. Prediction methods for reinforcement loads within GRS structures in current research and practice can be categorized into two approaches: force equilibrium approach (i.e., earth pressure method and limit equilibrium method) and deformation-based approach (i.e., K-stiffness method and finite element method). However, until today, the accuracy of these methods has not been extensively examined evaluated yet.

2 METHODS TO PREDICT $T_{MAX}$
2.1 Earth Pressure Method

Earth pressure method has been adopted in many current design guidelines (AASHTO 2002, Elias et al. 200, NCMA 2010) to predict reinforcement loads of MSE walls. The design rationale assumes the tensile forces developed in reinforcements are in local equilibrium with the lateral earth pressure generated in MSE walls. FHWA design guidelines recommend using Eq. (1) to predict $T_{\text{max}}$ of each reinforcement layer.

$$T_{\text{max}} = \left( \frac{k_r}{K_a} \right) K_a (\gamma z + q) S_v$$

where $T_{\text{max}}$ is the maximum reinforcement load of each reinforcement layer; $k_r/K_a$ is the normalized lateral earth pressure coefficient; $K_a$ is the theoretical Rankine or Coulomb active earth pressure coefficient; $\gamma$ is the backfill unit weight; $z$ is the depth below the top of the backfill; $q$ is the surcharge; $S_v$ is the tributary area (equivalent to the reinforcement vertical spacing when analyses are carried out per unit length of wall). The $k_r/K_a$ varies with the type of reinforcements; for flexible MSE walls or GRS walls, the $k_r/K_a$ has a value of 1.0 and remains constant throughout the depth of wall. This implies that for flexible MSE walls or GRS walls the horizontal movement occurring during construction is sufficient for the soil reaching active stress state and generating active earth pressure. The final computed reinforcement tensile loads increases linearly from the topmost layer of reinforcement to the bottommost layer of reinforcement (proportional to the overburden pressure).

Christopher et al. (2005) discussed the limitations of earth pressure method as follows: 1. It is theoretically-based and thus limited to relatively simple geometric structures and difficult to extrapolate to complex geometries, such as narrow walls and multi-tiered walls; 2. Limited to uniform granular soil and difficult to extrapolate to non-ideal reinforced fill soils; 3. Drainage should be adequate because pore water pressure or seepage forces in the reinforced fill are not considered; 4. Cannot evaluate global stability; 5. Downdrag at connections is not evaluated; 6. Unable to evaluate wall deformation. In addition, Allen et al. (2003) and Bathurst et al. (2008, 2005) investigated quantitatively the accuracy of reinforcement loads predicted by the earth pressure theory using careful interpretation of a database of 30 well-monitored full-scale walls. By the comparison between the reinforcement loads (interpreted from measured strains) in various instrumented GRS walls and the reinforcement loads predicted using the earth pressure theory, they concluded that loads predicted using earth pressure theory were excessively conservative. The predicted loads for GRS walls were on average three times greater than estimated values for full-scale instrumented walls. Furthermore, the distribution of reinforcement loads in the instrumented walls was seen to be generally trapezoidal in shape rather than linear with depth as assumed in the earth pressure theory for walls with uniform reinforcement spacing. Last, Yang et al. (2012) found from a series of finite element simulations that the mobilization of soil stress was non-uniform along the failure surface. This finding contradicts the basic assumption in the earth pressure method that the soil shear strength along the failure surface mobilizes equally and reaches peak shear strength simultaneously. Overall, the earth pressure method produces safe structures, but conservative, in terms of reinforcement strength for MSE structures on firm foundations and reinforced fills with no positive pore water pressures.

2.2 Limit Equilibrium Method

Limit equilibrium method has been used to analyze slope stability for many years by assuming the soil at failure obeys the perfectly plastic Mohr-Coulomb criterion and searching for a critical
failure surface that contains a minimum factor of safety. Limit equilibrium method can be applied to design complex geometric structures and, in general, to non-homogeneous soils. It can include the effects of pore water pressure on the system stability. Limit equilibrium method can also evaluate the global stability as well as local stability at any location or interface of interest. Limit equilibrium analyses of the reinforced soil structures have also been successfully reported (Zornberg et al. 1998). The stabilizing forces contributed by the reinforcement loads are incorporated into the equilibrium equation (balance of force or moment) at “limit” state (right between stable and unstable states).

Christopher et al. (2005) discussed the limitations of design based on limit equilibrium method as follows: 1. Currently FHWA limits the use of limit equilibrium method to reinforced soil slopes (facing inclinations less than 70°); however, this limitation is arbitrary and there is no theoretical reason why it could not be extended to MSE walls (facing inclination larger than 70°); 2. Needs modifications so that connection load and effect of facing element can be assessed within the limit equilibrium analysis; 3. Downdrag at connections is not evaluated; 4. Unable to consider wall deformation. In addition, the problem of GRS structures in limit equilibrium analysis is statically in determinate. In particular, determination of $T_{max}$ developed at each reinforcement layer requires assumptions. It needs further verification that the distribution of reinforcement tensile loads with depth is a valid assumption. A triangular distribution of $T_{max}$ (proportional to the overburden pressure) has been assumed in the design of reinforced soil structures (Schmertmann et al. 1987, Leschinsky and Boedeker 1989, Jewell 1991). FHWA design guidelines for reinforced soil slopes also recommend a linear distribution of $T_{max}$ with by dividing $T_{max}$ into 2 or 3 zones for the case of structures higher than 6m (Elias et al. 2001). However, as addressed previously, measured data shows nearly uniform mobilization of $T_{max}$ with depth for GRS walls under working stress conditions (Allen et al. 2003, Bathurst et al. 2008, 2005). For GRS slopes, the conventional triangular distribution of $T_{max}$ with depth is also not supported by a centrifuge investigation that evaluated the behavior of reinforced soil slopes under working stress conditions (Zornberg and Arriaga 2003) and failure conditions (Zornberg et al. 1998). In their studies, analysis of reinforcement strains results shows that the location of the maximum reinforcement strain among all reinforcement layers does not occur near the toe of the structure. It was located, rather, at approximately midway up the reinforced slopes, at a point along the critical failure surface directly below the crest of the slope.

2.3 K-Stiffness Method

Allen et al. (2003) and Bathurst et al. (2008, 2005) proposed a new working stress method for estimate of reinforcement loads in GRS walls, known as K-stiffness Method. In the development of the K-stiffness method, a database of 30 wall case studies was used to establish an empirical expression for predict $T_{max}$ at each reinforcement layer. The K-stiffness method has altered the conventional equation, Eq. (1), for computing $T_{max}$ by adding many influence factors, calculated as:

$$T_{max} = \frac{1}{2} K_o (\gamma H + q)S_t D_{tmax} \Phi$$

$$\Phi = \Phi_g \Phi_{local} \Phi_{fs} \Phi_{fb} \Phi_c$$

where $T_{max}$ is the maximum reinforcement load; $K_o$ is the at-rest earth pressure coefficient; $\gamma$ is the backfill unit weight; $H$ is the wall height, $q$ is the surcharge; $S_t$ is the tributary area or the reinforcement vertical spacing; $D_{tmax}$ is the load distribution factor; $\Phi$ is the influence factor that is the product of factors that account for the effects of global and local reinforcement stiffness $\Phi_g$ and $\Phi_{local}$, facing stiffness $\Phi_{fb}$, face batter $\Phi_{fb}$, and backfill cohesion $\Phi_c$.

The limitations of design based on the K-stiffness method include: 1. The applicability of K-stiffness method is limited to walls with the range of parameters matching the database of case histories used to calibrate the K-stiffness method; 2. Compared to the earth pressure theory, K-stiffness method involves many design variables and long design procedure; 3. Connection loads is not included into the K-stiffness method; 4. This method is limited to relatively simple geometric structures; 5. Pore water pressure in the reinforced fill are not considered. It is implicitly assumed that soils are well compacted and that good drainage practice is exercised to keep water from entering the reinforced soil zone. In addition, the application of K-stiffness method is limited to GRS walls under working stress conditions. Allen et al. (2003) demonstrated a good prediction of $T_{max}$ for GRS walls under working stress conditions (developed soil strain ≤ 3%). However, for GRS structures under large soil strain conditions (developed soil strain > 3%), the K-stiffness method consistently under-predicts $T_{max}$.

2.4 Finite Element Method

Finite element method has been widely applied to model the behavior of GRS structures (e.g., Hatami and Bathurst 2005 and 2006, Karparapu and Bathurst 1995, Ling et al. 2000, Lopes et al. 1994). Analysis based on finite element method considers full continuum mechanics, e.g., the constitutive relationships of all materials involved. It satisfies boundary conditions, considers local conditions like the interface between soil and reinforcement, and can be applied to any loading condition and sequence (i.e., traffic loading, seismic loading, and step loading to
simulate construction sequence). Unlike the earth pressure theory and limit equilibrium method, it’s potential to produce displacements and compatibility between dissimilar materials is analytically assured. It can represent a problem in a most realistic fashion, and its prediction of performance can be quite accurate. It provides rich information (i.e., stress, strain, force, and displacement) and can be obtained at any location of interest (i.e., nodal and Gaussian Point).

Christopher et al. (2005) discussed the limitations of design based on finite element method as follows: 1. Typically it requires a computational effort by a trained analyst; 2. It requires comprehensive characterization of strength and compressibility for all soils, reinforcements and facings to produce relevant results; 3. Requires careful modeling to replicate the effects of soil–reinforcement-facing interactions; 4. Predictions can be non-conservative, requiring careful evaluation of the reliability of input values and appropriate safety and/or resistance factors. In addition, although it has provided good predictions of the behavior of GRS structures under working stress conditions, finite element has not been reported to successfully predict under failure or large deformation conditions. Numerical difficulties often occur under failure or large deformation conditions. This is a crucial problem for the evaluation of the structure behavior, specifically for comparatively flexible structures such as GRS structures. Some specific development and implementation into finite element program are in need for modeling GRS structures under large deformation conditions. For example, a soil constitutive model is required to model the soil post-peak behavior. Also, special care is needed for numerical accuracy and stability for simulation under large deformation conditions.

3 FULL-SCALE GRS WALL TEST

The aforementioned methods to predict $T_{max}$ are examined by comparing with the $T_{max}$ measured from a full-scale and carefully instrumented GRS wall conducted by Bathurst et al. (2006) in the Royal Military College (RMC). The GRS wall is 3.6m high and constructed with reinforcement 6 layers at a spacing of $S_v=0.6m$. Different from a typical wrapped-face GRS wall that each facing wrap was extended back into the reinforced soil zone, each facing wrap in this test wall was attached to the reinforcement layer above using a metal bar clamp to form the wall face with a facing slope of $\omega=8^\circ$. Figure 2 illustrates the cross-section of the GRS test wall.

The backfill, named RMC sand, is a clean, uniform graded, beach sand classified as poor sand (SP) according to USCS. The backfill soil has $D_{50}=0.34mm$, coefficient of curvature $C_c=2.25$, coefficient of uniformity $C_u=1.09$, unit weight $\gamma=16.7kN/m^3$ and soil peak friction angle $\phi_p=35^\circ$ by triaxial compression tests and $\phi_p=42^\circ$ by plane strain tests. The reinforcement is a polypropylene geogrid with a total length of 2.52m measured from the front wall face. The ultimate tensile strength was of $T_{ult}=13kN/m$ obtained from the wide width strip tensile test (ASTMD4595). Because the reinforcement strain rate (10%/min) in the wide width tensile test is much larger than the strain rate possibly developed in the test wall, a series of constant-load creep tests were carried out by Bathurst et al. (2006) to determine the isochronous load-strain responses of reinforcement at 1000hr which is similar to the duration of the wall test.

After completion of wall construction, uniform surcharges were applied on the top of the wall with load increment of 10kPa until final loading of 80kPa was reached. This wall was intensively instrumented to measure the performance of the wall at the end of construction and during staged uniform surcharging;
for instance, the strain gauges and extensometers attached to reinforcements were used to measure the reinforcement strains along each reinforcement layer. The measured maximum reinforcement strain at each reinforcement layer then multiplied by the reinforcement secant stiffness \(T_{\text{max}} = J(\varepsilon)\varepsilon\) determined from the isochronous load-strain responses at the same strain level to estimate the reinforcement loads in this study.

4 CALCULATION DETAIL

4.1 Earth Pressure Method

For the test wall without backslope, the lateral earth pressure coefficient \(K_a\) in Eq. (1) can be calculated according to Rankine and Coulomb theories, as shown in Eq. (4) and Eq. (5), respectively.

\[
K_a = \tan^2(45 - \frac{\phi}{2})
\]  
(4)

\[
K_a = \frac{\cos^2(\omega + \phi)}{\cos^2(\omega)\cos(\delta - \omega)} \left[ 1 + \frac{\sin(\phi + \delta)\sin\phi}{\cos(\delta - \omega)\cos\omega} \right]^{-2}
\]  
(5)

where \(\phi\) is the backfill friction angle; \(\omega\) is the facing batter; \(\delta\) is the soil-facing interface friction angle. Different from Rankine theory, Coulomb theory can account for the effect of wall facing batter and soil-face interaction on \(K_a\), resulting in the calculated \(K_a\) is less than the \(K_a\) from Rankine theory. The peak plane strain friction angle of \(\phi_p=42^\circ\) was inputted into Eqs. (4) and (5) to characterize the backfill shear strength in the test wall conditions. For Coulomb theory, \(\delta=\phi\) was used, assuming the facing column creates a soil-to-soil interface for the wrapped-face wall. The normalized lateral earth pressure coefficient of \(k/K_a=1\) is applied for the GRS test wall. Input values for other parameters in Eq. (1) correspond to the physical wall test.

4.2 Limit Equilibrium Method

Limit equilibrium analyses were performed using the Modified Bishop method with circular surfaces as coded in the commercial slope stability analysis software, STEDwin. Figure 3 shows the limit equilibrium modeling of the GRS test wall. The geometry of the wall model follows the dimensions of physical wall test. The peak plane strain friction angle of \(\phi_p=42^\circ\) was used. The limit equilibrium analysis assumed that the reinforcement loads had a uniform distribution with depth and considered the contribution of geogrid overlap layers to system stability. Unlike the recommended use of allowable tensile strength in the conventional analysis, the limit equilibrium analyses in this study did not consider reduction factors due to installation damage, creep or degradation (i.e., all reduction factors were 1.0). A series of uniform loadings were applied on the top of limit equilibrium model to simulate the surcharges. The mobilized reinforcement loads \(T_{\text{max}}\) at different surcharges were determined by varying the values of \(T_{\text{max}}\) until FS=1 was reached at each surcharge level.

4.3 K-Stiffness Method

As suggested by the K-stiffness method, the peak plane strain friction angle of \(\phi_p=42^\circ\) was used in the calculation. The reinforcement stiffness at 2% strain \(J_{2\%}\) determined from the project-specific isochronous load-strain response was applied to calculate the influence factor for the effects of global and local reinforcement stiffness (i.e., \(\Phi_g\) and \(\Phi_{\text{local}}\), respectively). The facing stiffness of \(\Phi_{\text{fs}}=1\) was applied according to the suggestion in the K-stiffness method. This implies the wrapped-around face has no influence on \(T_{\text{max}}\). Specifically, the wrapped-around face cannot reduce the reinforcement loads by resisting a part of lateral earth pressure. Because there is no cohesion in the backfill, the effect of cohesion is not considered in the calculation (i.e., \(\Phi_c=1\)). Input values for other parameters in Eqs. (2) and (3) corresponds to the physical wall test.

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**Fig. 3. Limit equilibrium model and results of the GRS test wall**
4.4 Finite Element Method

The finite element program, PLAXIS version 8.2 (PLAXIS, 2005), was used to develop a numerical model for the GRS test wall and to simulate their plane-strain response at the end of construction and during staged uniform surcharging. Figure 4 shows the finite element model of the GRS test wall.

The backfill, RMC sand, was modeled as a stress-dependent, hyperbolic elasto-plastic material using the Hardening Soil model. Table 1 lists the material properties of RMC sand. Figures 5 show the calibration results of stress-strain-volumetric response of RMC sand. A small cohesion value, $c=1$ kPa and $2$ kPa were introduced in the soil model at construction and during staged uniform surcharging, respectively, to improve numerical stability. In addition, because each facing wrap was fixed using a metal bar clamp in the test wall, a cohesion of $c=10$ kPa was applied to the soil elements in the wrapped-around face to simulate this effect. The reinforcements were modeled as elasto-plastic bar elements with an axial stiffness $E_A$, maximum axial tensile strength, $N_p$ and no compressive strength. Table 1 lists the reinforcement properties determined from the isochronous load-strain response at 1000hr. Figure 6 show the calibration results of geogrid load-strain response. Note that in order to model the nonlinear load-strain response, the reinforcement stiffness were inputted as $E_A= 100$kN/m and $70$kN/m for construction and during staged uniform surcharging, respectively. These input values of reinforcement stiffness correspond to the average mobilized reinforcement strain of 2% and 7% at construction and during staged uniform surcharging, respectively.

Stage construction was included in the simulation by conducting layer-by-layer construction in PLAXIS. The uniform surcharges were applied on the top of the finite element model with load increment of 10kPa until target loading of 80kPa was reached. Updated mesh was activated to account for large deformations, especially important at significant loading conditions. Notably, the calculated FE failure was earlier than the actual failure of soil, a clear internal failure surface, observed in the test wall at $q=90$kPa. The FE simulation terminated at the next 10kPa loading increment after completing 40kPa due to numerical difficulties occurred in the computation. Inspection of soil elements at the termination of simulation revealed that most of soil elements along the failure surface reached their peak shear strength. That may cause the numerical instability in the simulation and result in the termination of simulation. Last, the accuracy of the numerical model was verified by quantitatively comparing the reinforcement strains along each layer and the comparison results showed the prediction and measurement were in satisfactory agreement.

5 RESULTS AND DISCUSSION

5.1 Results

The accuracy of each method is examined by comparing the predicted $T_{\text{max}}$ with the measured $T_{\text{max}}$ from the test wall. Figure 7 shows the comparison of $T_{\text{max}}$ at the end of construction ($q=0$kPa). The “measurement” in Fig. 7, indicates the “measured” reinforcement load calculated by multiplying the measured strain by the isochronous stiffness value at the same strain level for each reinforcement layer.

![Fig. 4. Finite element model of the GRS test wall](image)
Table 1. Material properties for RMC sand and geogrid

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<th>Value</th>
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<tr>
<td>$\gamma$ (unit weight) (kN/m$^3$)</td>
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</tr>
<tr>
<td>$\phi$ (peak friction angle) (degree)</td>
<td>42</td>
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</table>
| $c$ (cohesion) (kPa) | 1 for construction  
2 for surcharging  
10 for wrapped-around face |
| $\psi$ (dilation angle) (degree) | 11 |
| $E_{50}^{ref}$ (secant stiffness) (kPa) | 6.2$\times$10$^4$ |
| $E_{50}^{\phi}$ (tangent stiffness for primary oedometer loading) (kPa) | 6$\times$10$^4$ |
| $E_{ur}^{ref}$ (unloading/reloading stiffness) (kPa) | 1.8$\times$10$^5$ |
| $m$ (modulus exponent) | 0.5 |
| $R_f$ (failure ratio) | 0.8 |
| **Reinforcement** | |
| $N_p$ (maximum tensile strength) (kN/m) | 7.7 |
| $E_A$ (axial stiffness) (kN/m) | 100 for construction  
70 for surcharging |

Note: $E_{ur}^{ref}$ was assumed to be $3E_{50}^{ref}$ as the default setting in PLAXIS.

Fig. 5. Measured and predicted stress-strain-volumetric response of RMC sand: (a) stress-strain response from plane strain tests; (b) axial strain-volumetric strain from triaxial tests. Note that no volumetric strain response was taken in plane strain tests.

Fig. 6. Measured and predicted load-strain response of geogrid.
The range bars in Figure 7 represent 10% of uncertainties on measured $T_{max}$ to account for the estimate error of the strain measurements and isochronous stiffness values. Comparison results indicate the earth pressure methods using both Rankine and Coulomb theories overly predict the reinforcement loads. The earth pressure method using Coulomb theory is considered superior to the one using Rankine theory because Coulomb theory can account for the effect of wall facing batter and soil-face interaction on $K_a$. Although the $T_{max}$ predicted by the limit equilibrium method is in a good agreement with the maximum value of the measured $T_{max}$, the uniform distribution of reinforcement load with depth assumed in the limit equilibrium method does not match the distribution of measured data. The K-stiffness method seems to slightly underestimate the measured $T_{max}$. The finite element predictions agree fairly well with the measured $T_{max}$.

Figure 8 shows reinforcement loads at different surcharge levels at reinforcement layer 3, where the maximum of the measured $T_{max}$ occurred within the test wall. To predict the value of maximum $T_{max}$ accurately is very important because the value of maximum $T_{max}$ is conventionally used to determine the ultimate reinforcement tensile strength in the design of GRS wall internal stability against reinforcement breakage. Overall, every method is able to predict the increase of $T_{max}$ with increasing surcharges. However, except for a good prediction of $T_{max}$ at $q=0$ kPa by the limit equilibrium method, the force equilibrium approach, including the earth pressure methods using Rankine and Coulomb theories and the limit equilibrium method, overly predicts the reinforcement loads at different surcharge levels. The magnitude of the discrepancy between predicted and measured results increases as the surcharge increases.

As for the deformation-based approach, the K-stiffness method slightly underestimates the measured $T_{max}$ at $q=0$ kPa but shows an obvious underestimate under surcharging conditions. This observation is consistent with one of the limitations of the K-stiffness method discussed by Allen et al. (2003). The prediction by the finite element method agrees well with the measured $T_{max}$. As mentioned earlier, the FE simulations terminated at the next 10 kPa loading increment after completing 40 kPa for the test wall due to numerical difficulties occurred in the computation. Therefore, the FE results are only presented until $q=40$ kPa in Figure 8. Table 2 presents the ratio of predicted $T_{max}$ to measured $T_{max}$ at various surcharge levels to quantitatively assess the degree of accuracy of these methods. In general, the earth pressure method using Rankine theory has the most significant overestimate of the $T_{max}$ value with an average ratio of 2.39. In contrast, the K-stiffness method underestimates the $T_{max}$ value most significantly with an average ratio of 0.61.

Figure 9 shows summation of reinforcement loads $T_{max}$ from all reinforcement layers at various surcharge levels. Because Bathurst et al. (2006) only reported the strain magnitude and distribution in the six layers of reinforcements at the end of construction and $q=80$ kPa, only the measured $\Sigma T_{max}$ at $q=0$ kPa and 80 kPa are plotted in Fig. 9. Similar to previous observations in Fig. 8, the force equilibrium approach overly predict the mobilized reinforcement loads. The finite element method agrees well with the measured data at working stress conditions but numerical illness occurs at large loading conditions (i.e., $q \leq 40$ kPa). The K-stiffness method shows a good agreement at working stress conditions (the end of construction) but underestimates the measured $T_{max}$ in particular at large loading conditions.
Methods Surcharge level, \( q \) (kPa) Average

<table>
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Fig. 8. Comparison of reinforcement load \( T_{max} \) at layer 3 at different surcharge levels

Fig. 9. Comparison of summation of reinforcement loads \( \Sigma T_{max} \) from all reinforcement layers at different surcharge levels
5.2 Discussion on the Discrepancy

Holtz (2010) in the 46th Karl Terzaghi lecture discussed the discrepancy between predicted and measured $T_{\text{max}}$ may generally come from: 1. Selection of soil shear strength properties to input into the prediction methods (i.e., use $\phi_{\text{max}}$, $\phi_{\text{plane strain}}$ or $\phi_{\text{residual}}$); 2. Error and uncertainty from field instrumentation and measurement; 3. Existence of apparent cohesion in the field unsaturated conditions; 4. Influence of facing stiffness. The discrepancy caused by the apparent cohesion was also discussed by Leshchinsky (2009 and 2010). He declared a trace of apparent cohesion from capillary suction or soil matrix potential in the field unsaturated conditions may dramatically increase system stability and result in reducing the requirement of the mobilization of $T_{\max}$ within GRS structures. Bathurst et al. (2006) compared the influence of facing stiffness on the measured reinforcement strain and commented that the wall facing is a structural element that acts to reduce the magnitude of deformations and reinforcement strains of GRS structures.

For this study, the peak plane strain friction angle was used to characterize the backfill shear strength in the test wall conditions because the plane strain condition was convinced to be most representative to the real test conditions. Error and uncertainty from measurement and data interpretation were also considered using the range bar to represent the uncertainties on the measured $T_{\max}$. The effect of apparent cohesion due to suction was not considered in the prediction of $T_{\max}$ in this study. This effect may be important for the field wall as discussed by Leshchinsky (2009 and 2010); however, it is believed that this effect has little influence on the measured $T_{\max}$ for the test wall discussed in this study. That is because the backfill used in the test wall was a uniform sand with relatively coarse sand particle (i.e., $D_{50}=0.34$mm) and less than 1% of fine soil. The backfill was compacted at a little moisture content of 3% to 5%. The apparent cohesion under this backfill condition is likely little (i.e., $<2$ kPa).

In author’s opinion, the facing stiffness as mentioned by Holtz (2010) and Bathurst et al. (2006) is a major source of conservatism in the force equilibrium approach for the case in this study. However, the influence of facing stiffness is typically not accounted for in the current design procedures which are established based on the force equilibrium approach. To demonstrate the statement above, another limit equilibrium analysis was conducted by inputting an additional cohesion of $c=10$ kPa to the soil elements in the wrapped-around face to simulate the effect of facing stiffness. This way of modeling facing stiffness is similar to the finite element modeling as discussed in Section 4.4. The results of limit equilibrium analysis considering the effect of facing stiffness are shown in Fig. 9. The limit equilibrium results demonstrate modeling of facing stiffness in the limit equilibrium analysis can improve the prediction of $T_{\max}$. Some discrepancies between prediction and measurement still can be observed in Fig. 9. That is because the influence of facing stiffness on the reduction of reinforcement loads cannot be easily and quantitatively implemented in the force equilibrium method. More specifically, the effect of facing stiffness should also develop with increasing loadings rather than a constant value assumed in this study. Further studies are needed to more accurately simulate the effect of facing stiffness in the force equilibrium approach.

6 CONCLUSIONS

In this paper, the accuracy of various design methods to predict the reinforcement loads $T_{\max}$ was evaluated by comparing with the measured data from a full-scale and carefully instrumented GRS structure. Specific important conclusions and discussion points are summarized as follows.

- Comparison results indicate the force equilibrium approach including the earth pressure method and limit equilibrium method overly predict the reinforcement loads. Among all methods, the earth pressure method using Rankine theory has most significant overestimate of the $T_{\max}$ value with an average ratio of 2.39.
- The finite element method agrees well with the measured data at working stress conditions but numerical illness may occur earlier than the actual failure of structure at large loading conditions.
- The K-stiffness method shows a good agreement at working stress conditions (the end of construction) but an underestimate in particular under large loading conditions. Among all methods, the K-stiffness method underestimates the $T_{\max}$ value most significantly with an average ratio of 0.61.
- Ignorance of the effect of facing stiffness is a major source of conservatism in the force-equilibrium approach for the wall case in this study. That is because the facing stiffness can reduce system deformation and, consequently, decrease reinforcement loads. This study demonstrated that modeling of facing stiffness in the limit equilibrium analysis can improve the prediction of $T_{\max}$. However, the influence of facing stiffness on the reduction of reinforcement loads cannot be easily and quantitatively implemented in the force equilibrium method.

ACKNOWLEDGEMENTS

The financial support for this study was provided by the National Science Council of the Republic of China, Taiwan under Grant No. NSC99-2218-E-001-006 and partially by the National Taiwan University of Science and Technology under the new faculty research funding. The author also sincerely
acknowledges the recommendation from the Taiwan Geotechnical Society for author’s participation in the 7th Asia Young Geotechnical Engineers Conference.

REFERENCES


