Research on Critical Height of Geocell-Reinforced Embankments over Soft Ground

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ABSTRACT: As three dimensional reinforcement, geocell used in embankment cushion shows obvious advantage in increasing overall stability and deformation control, especially on soft ground. But evaluation on the critical height and its overall stability of geocell reinforced embankments is still short of study. By taking both the vertical stress dispersion effects and lateral confinement effects of geocell layer into consideration, a calculation model about critical height of the embankments was proposed by employing the limit equilibrium method and assuming that the foundation failed in circular failure mode, in which geocell and infill soil were regarded as composite material. A numerical model was built using FLAC and calibrated by an embankment model test. The safety factors of the embankments which were calculated by the analytical model and numerical model were compared and they were coincident well with each other.

KEYWORDS: Reinforced embankments; Geocell; Composite material; Stability; Critical height

INTRODUCTION

Embankments, built on soft ground, are always encountered large deformation and are not easy to be constructed. Although several conventional methods can be used to improve the soft soil, replacing and removing the soil and pile supported embankments for example, they cost a lot and require relatively long construction period than reinforcing approaches. As a widely used type of three dimensional reinforcement, geocell shows obvious advantage in construction leveling and deformation control, especially on soft ground.

Several authors have studied geocell reinforced embankments by adopting three methods. In laboratory experiments, Krishnaswamy et al. (2000) conducted a model test and analyzed the effects of modulus of geocell on the ultimate bearing capacity of embankments. Sun et al. (2015) and Gao et al. (2016) evaluated the bearing capacity of multilayer geocells reinforced embankments under strip loading and static and cyclic loading, respectively. However, the embankments were built on stiff foundation and the effects of strength of foundation soil on the overall stability of embankments were not considered. In numerical simulation, Deng et al. (2015) discussed the mechanical behavior of embankments by considering the interaction between infill soil and geocell. The effects of geocell spacing on the distribution of stress and strain in the embankments under cyclic loading were investigated by Leshchinsky and Ling (2013). While in theoretical analysis, the method of researching geosynthetic reinforced foundation was always employed to study the geocell reinforced embankments and several computational models about
the increment of bearing capacity due to geocell were proposed (Zhang et al. 2010; Avesani Neto et al. 2013; Sitharam and Hegde. 2013). The lateral confinement effects of geocell includes direct restriction to the infill soil caused by adjacent cells (former effects) and restriction to the lateral deformation of embankment caused by the shear stress generated between geocell layer and foundation and embankment (latter effects). The former effects were considered in these theoretical models presented before, while the latter effects were ignored.

These results provide certain understanding of the behavior of geocell reinforced embankments, but few authors have proposed a method to calculate its critical height and evaluate its overall stability. This paper proposed an analytical model to calculate its critical height by considering the main reinforcing mechanisms of geocell.

**MODEL FOR CRITICAL HEIGHT OF THE EMBANKMENTS**

For the reinforcing mechanism of geocell, Zhang et al. (2010) attributed it to three main aspects: (1) vertical stress dispersion effects, (2) lateral confinement effects and (3) membrane effects. Nevertheless, the geocell behaves as rigid slab and generates uniform settlement without any significant bending in embankments (Dash et al. 2001a; Yang 2010). Hence, only vertical stress dispersion effects and lateral confinement effects were considered in the analytical studies.

**Vertical stress dispersion effects**

The composite material, consisted with geocell and infill soil, plays a mattress role in embankments. When the upper load acts on it, the load will be dispersed, as shown in Fig.1. For simplicity, the trapezoid distribution form of upper load was converted into rectangle distribution form with the same area and same height. The stress dispersion effects can be described as

\[ P_e = \frac{P_B}{B + 2h \tan \theta_e} \]  

thus

\[ P_e = \frac{P_B}{B + 2h \tan \theta_e} \]  

where \( P_e \) is the upper load and is equal to \( \gamma H \); \( \gamma \) is the unit weight of backfill soil; \( H \) is the height of embankment; \( P_d \) is the load which acts on the foundation after stress dispersion; \( B \) is the width of uniform upper load \( (P_e) \) and is equal to \( L - nH \); \( L \) is the width of embankment; \( n \) is the ratio of slope of embankment; \( h \) is height of geocell and \( \theta_e \) is the angle of stress dispersion.

**Figure 1. Vertical stress dispersion effects. Figure 2. Lateral confinement effects.**

**Lateral confinement effects**

Lateral confinement effects of geocell consist of two aspects which have been
explained in introduction. The infill soil and geocell were treated as composite material in this study due to the close interaction between each other, so the shear stress between geocell and infill soil was not considered. Lateral deformation of embankment and foundation will be generated under the upper load, and so is the geocell. But the deformation of geocell is obviously smaller than the deformation of embankment and foundation because of higher modulus of geocell. Consequently, the shear stress, \(\tau_2\) and \(\tau_3\), will be induced in the interaction surface between embankment and geocell layer and foundation, as shown in Fig. 2.

The failure surface will appear firstly in the soft foundation under the upper load due to its weak strength, and also is difficult to pass through the geocell layer. So the shear stress \(\tau_2\) was only considered in the stability analysis and can be calculated as

\[
\tau_2 = \mu P_d
\]

where \(\mu\) is the friction coefficient between geocell layer and foundation.

**Fig. 3. Stability analysis of geocell-reinforced embankments.**

**Fig. 4. Stress analysis of a unit at point A.**

**Stability analysis**

According to previous researches, the symmetry failure mode appeared in the embankments which was built on soft ground, demonstrated by Zhao et al. (1991) and Liu et al. (2009) using field test and numerical simulation, respectively. Tang et al. (2005) pointed out that the results of theoretical analysis were much closer to the actual ones when the failure surface was assumed to cross the center of embankments. And the failure surface can be assumed to be circular if the embankment was constructed on soft ground according to Hu (1997). Fig. 3 shows the stability analysis of geocell reinforced embankments. The center of failure surface \(O\) was located in extensive line of rectangle’s boundary. The upper load, caused by the self-weight of embankment, was assumed to be symmetry along its width. And the soil of foundation was assumed to be uniform. So one half of the embankments was adopted in this study.

The embankment load can be simplified as uniform ones with the width of \(B/2\) according to the assumption of failure surface described previously. The failure surface was divided into two parts \((BC\) and \(CD)\), representing the location of two different kinds of shear strength \((\tau_{f1}\) and \(\tau_{f2}\)). For the resisting moment provided by the foundation, it can be calculated as

\[
M_{R1} = (\tau_{f1} B + \tau_{f2} C) P
\]

A unit was chosen at point \(A\) to figure out the shear strength, \(\tau_{f1}\) and \(\tau_{f2}\), as shown in Fig. 4. It was assumed that vertical direction and horizontal direction represented major and minor principle stresses, respectively. So the relationship between \(\sigma_1\) and \(\sigma_3\) can be described as
\[ \sigma_i = \sigma_0 \tan^2 \left( \frac{45^\circ - \varphi / 2}{2} \right) - 2c_f \tan \left( \frac{45^\circ - \varphi / 2}{2} \right) \]  \hspace{1cm} (5) 

where \( \varphi \) and \( c_f \) is internal friction angle and cohesion of foundation soil, respectively. The major principle stress, \( \sigma_i \), should be expressed by section due to different upper load. In the \( BC \) part,

\[ \sigma_i = P_i + \gamma f \, y \]  \hspace{1cm} (6)

while in the \( CD \) part,

\[ \sigma_i = \gamma f \, y \]  \hspace{1cm} (7)

where \( \gamma f \) is the unit weight of foundation soil; \( y \) is the vertical coordinate at point A in Fig.3 and can be calculated as

\[ y = R(\cos \alpha - \cos \theta) \]  \hspace{1cm} (8)

where \( \cos \alpha = \sqrt{1 - \sin^2 \alpha} = \sqrt{1 - \left( \frac{B_2 - x}{R} \right)^2} \]  \hspace{1cm} (9)

where \( \theta \) is one half of central angle corresponding to failure surface and is equal to 66.8 based on the ultimate load formula for soft soil foundation (Hu, 1997); \( x \) is the horizontal coordinate at point A in Fig.3; \( R \) is the radius of failure surface and is equal to \( B_1 / (2 \sin \theta) \). According to the Mohr-coulomb criterion, the shear strength of soil can be described as

\[ \tau_f = \sigma \tan \varphi + c_f \]  \hspace{1cm} (10)

By substituting Eqs. (5)−(9) and Eq. (11) into Eq. (10), the shear strength of foundation soil can be calculated. Then, the resisting moment which was generated in \( BC \) part and \( CD \) part can be estimated as

\[ \tau_{f,BC} = \sum_{i=0}^{B_1} \tau_{f,i} \Delta R = \int_0^{B_1} \frac{\tau_{f,i} \Delta R}{\cos \alpha} \]  \hspace{1cm} (12)

\[ \tau_{f,CD} = \sum_{i=0}^{B_2} \tau_{f,i} \Delta R = \int_0^{B_2} \frac{\tau_{f,i} \Delta R}{\cos \alpha} \]  \hspace{1cm} (13)

Another resisting moment, provided by geocell layer, can be calculated as

\[ M_{r2} = \tau_2 \left( \frac{B_2}{2} \right)^2 \cot \theta \]  \hspace{1cm} (14)

The sliding moment was only provided by upper load, because any other external load was not considered in the analysis. And it can be described as

\[ M_s = \frac{1}{8} P_d B^2 \]  \hspace{1cm} (15)

The safety factor of embankment can be calculated as

\[ K = \frac{M_{r1} + M_{r2}}{M_s} \]  \hspace{1cm} (16)

Based on the above equations, the critical height of embankment (\( H_{cri} \)) can be obtained by letting safety factor be equal to 1.

**NUMERICAL SIMULATION**

**General**

The finite difference program FLAC (Fast Lagrangian Analysis of Continua) was utilized in the analysis. The length of geocell layer was kept the same as the width of the bottom of embankments and was equal to 14.5m. The thickness and width of the foundation were 30m and 60m, respectively, as shown in Fig.5. Horizontal fixities were applied to two sides of the model and total fixities were
placed at the bottom of the foundation. To simulate the actual construction process, the embankment was built layer by layer with a thickness of 0.4m until to the critical state. The effects of compaction were simulated by applying a uniform pressure (8kPa) on every layer of backfill, and the pressure was removed after being solved to equilibrium. Computations in the numerical analysis were carried out in large-strain mode to ensure sufficient accuracy in the event of large embankment deformations.

![Figure 5. Dimensions of the embankments.](image)

**Table 1. Physical and mechanical parameters of numerical model.**

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Unit weight (\gamma/(kN\cdot m^{-3}))</th>
<th>Young’s modulus (E/kPa)</th>
<th>Poisson’s ratio (\nu)</th>
<th>Angle of internal friction (\phi/(°))</th>
<th>Cohesive strength (c/kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment</td>
<td>19</td>
<td>5e4</td>
<td>0.30</td>
<td>45</td>
<td>2</td>
</tr>
<tr>
<td>Foundation</td>
<td>17</td>
<td>3e3</td>
<td>0.45</td>
<td>6</td>
<td>10</td>
</tr>
<tr>
<td>Geocell layer</td>
<td>19</td>
<td>/</td>
<td>0.25</td>
<td>45</td>
<td>18</td>
</tr>
</tbody>
</table>

**Backfill soil and foundation**

Sand and soft clay were used to construct the embankment and foundation, respectively. They were modeled as a nonlinear elastic-perfectly plastic material complying with Mohr-coulomb failure criterion and their physical and mechanical parameters were shown in Table 1 (Krishnaswamy et al. 2000). A small value of cohesive strength (2kPa) was introduced in the sand to prevent premature soil yielding in locally low confining pressure zones and to account for possible additional apparent cohesion due to moisture in the backfill soil.

**Geocell layer**

By regarding the geocell and infill soil as composite material, the interfaces between geocell and infill soil will not be needed. But the interface between geocell layer and foundation was modeled as linear spring-slider systems with interface shear strength defined by Mohr-Coulomb failure criterion. The geocell layer can be treated as backfill soil layer with greater cohesion and modulus than the infill soil and the internal friction angle the same as the infill soil according to Madhavi Latha (2000). The increased cohesion strength induced by geocell \((c_r)\) can be described as (Madhavi Latha 2000).

\[
c_r = \Delta \sigma_{xy} \sqrt{K_p} / 2
\]  \hspace{1cm} (17)

where
\[ \Delta \sigma_{3g} = \frac{2M}{D_0} \left( \frac{1 - \sqrt{1 - \varepsilon_u}}{1 - \varepsilon_a} \right) \]  

where \( K_p \) is the coefficient of passive earth pressure; \( \Delta \sigma_{3g} \) is the increased confining pressure due to the confinement of geocell; \( \varepsilon_u \) is the axial strain of geocell at failure; \( M \) is the secant modulus of geocell when the axial strain is \( \varepsilon_a \) and \( D_0 \) is the initial diameter of geocell.

As for the modulus of geocell layer, it can be estimated as (Madhavi Latha 2000)

\[ E_s = 4(\sigma_3^{'})^{0.7}(K_u + 200M^{0.16}) \]  

where

\[ \sigma_3^{'} = K_0 \rho H \]  

Where the dimensionless modulus parameter (\( K_u^{'} \)) corresponds to the modulus number in the hyperbolic model proposed by Duncan and Change (1970); \( M^{'} \) corresponds to the average axial strain of 2.5% in the load-elongation of geocell; \( K_0 \) is the coefficient of at-rest earth pressure. The geocell layer was modeled using continuum zones and a constitutive model with Mohr-Coulomb failure criterion. The Young’s modulus number (\( K_u^{'} \)) and coefficient of at-rest earth pressure (\( K_0 \)) were assumed to be 550 and 0.5, respectively (Madhavi Latha 2007).

MODEL VERIFICATION

The numerical model was verified by a laboratory experiment on geocell reinforced embankment which was conducted by Krishnaswamy et al. (2000). In the experiment, the height of embankment and geocell was 400mm and 100mm, respectively. The thickness and width of foundation were 600mm and 1800mm, respectively. And the ratio of slope (\( n \)) was equal to 1. Clayed sand was adopted for backfill soil and infill soil with unit weight of 19kN/m³. Its cohesion strength was 10kPa and internal friction angle was 30°. Clay with unit weight of 17kN/m³ was adopted for foundation, and its cohesion strength was 10kPa and internal friction angle was 0°. The secant modulus of geocell at 2.5% strain was 160kN/m. In the model test, three dial gages (L1, L2, and L3) were installed along the slope at different heights of embankment to monitor lateral displacements. The results between numerical simulation and model test were compared, and they were coincident well with each other, as shown in Fig.6. This indicated that the numerical model can simulate the behavior of geocell reinforced embankments properly.

![Figure 6. Comparison of lateral displacements.](image1)

![Figure 7. Variation of safety factors.](image2)
The angle of stress dispersion was supposed to be 55° according to Liu et al. (2007) and Dash et al. (2007), and the friction coefficient was assumed to be 0.15 based on Sitharam and Hegde (2013). Fig. 7 shows the results of safety factors of embankments in Fig. 5 which were calculated by theoretical analysis and numerical simulation when the geocell modulus and geocell height were 100kN/m and 0.2m, respectively. Though the difference between these two results was relatively large when the embankment was about to be constructed, they were generally in good agreement with each other. Importantly, the key point in this study is when the embankment reaches to its critical state.

CONCLUSIONS

This study focused on the critical height of geocell reinforced embankments on soft ground by employing the limit equilibrium method and assuming that the foundation failed in circular failure mode. The geocell and infill soil were treated as a composite material and its vertical stress dispersion effects and lateral confinement effects were considered. An analytical model was proposed and its results were compared with ones of a calibrated numerical model. The results between theoretical analysis and numerical calculation were generally good agreement with each other, indicating that this model can be used to make prediction for the critical height of geocell reinforced embankments over soft soils.

REFERENCES


