STABILITY ANALYSES FOR GEOSYNTHETIC-REINFORCED STEEP-FACED SLOPES SUBJECTED TO TOE SCOURING

Ching-Chuan Huang, Bo-Shin Huang, and Yue-Wing Chen

ABSTRACT

Knowledge on the influence of a scoured slope toe to the stability of earth structures is key to their long-term maintenance and disaster mitigation. As a follow-up of an experimental study on geosynthetic-reinforced steep-faced slopes subjected to toe scouring, limit-equilibrium-based stability analyses were performed to investigate their effectiveness in evaluating the stability of the slope. Results of analyses showed that the safety factors for pull-out, tie-break, facing connection, base sliding, and overturning failures were insensitive to the boundary changes induced by the toe scouring. The toe-scouring-induced instability of the slope was reflected by the reduction of safety factors against bearing capacity and circular sliding. Among the methods used, the circular failure analysis demonstrated the highest versatility and potentiality in detecting the toe-scouring-induced instability of reinforced steep slopes.

Key words: Geosynthetic-reinforced slope, limit equilibrium analysis, toe scouring, internal stability, external stability, circular failure analyses.

1. INTRODUCTION

Results of post-earthquake and post-tsunami site reconnaissance suggested that flawed or scoured slope toe conditions may account for the catastrophic failure of soil structures such as slopes, retaining walls, and sea walls (Huang and Chen 2004, 2005, Yamaguchi et al. 2012; Kuwano et al. 2014). Results of the site reconnaissance after a devastating tsunami in the 2011 Great East Japan Earthquake in Japan indicated that catastrophic failures of coastal dykes might have been initiated by overflow-induced scouring at the toe of the downstream slope (Yamaguchi et al. 2012 and Kuwano et al. 2014). As a result of these post-tsunami studies, a steep-faced geosynthetic-reinforced soil dyke was proposed by Yamaguchi et al. (2012). In contrast to the above-mentioned natural forces, human activities such as excavations also play a role in destabilizing natural or manmade slopes, causing landslides (Guadagno et al. 2005). Huang (2015) proposed a technique of reinforced earth slab using two layers of reinforcement to mitigate possible excessive settlement of a footing at the crest of the slope. El-Eman and Bathurst (2004, 2007) showed the importance of toe-restraining conditions to the seismic displacement and lateral earth pressure distributions of steep-faced geosynthetically-reinforced soil retaining walls based on the results of model tests. Tatsuoka et al. (2007) and Huang et al. (2008) performed experimental and analytical studies on using soil bags with integrated reinforcement strips as the facing of reinforced slopes to enhance the stability of earth dams against over-flow-induced scouring and instability. Huang and Chen (2012a, b) investigated the stability and deformation of geosynthetic-reinforced vertically faced soil retaining walls subjected to simulated toe scouring. Results of limit equilibrium analyses performed by Huang and Chen (2012a, b) showed that to avoid catastrophic failures of the wall subjected to toe scouring, the required safety factors against internal failures should be moderately increased. In general, experimental and/or analytical works exploring the stability and deformation characteristics of reinforced soil slopes (or retaining wall) subjected to a progressive loss of the support at the slope toe are limited (e.g., Miyata et al. 2015, Huang and Chen 2012a, b). Investigations on the failure mechanism of reinforced slopes subjected to toe scouring are performed based on the results of a series of model tests using limit-equilibrium-based stability analyses to gain new knowledge on the behavior of steep-faced reinforced slopes with a scoured toe.

2. MODEL TEST FACILITY AND TEST RESULTS

The model test system consists of a steel frame of 2.5 m-long, 1.0 m-wide, and 0.15 m-thick for containing a 433 mm-high idealized two-dimensional slopes. Stainless steel rods with a uniform diameter of 1.96 mm and a length of 150 mm were stacked in a rhombic pattern having a unit weight of $\gamma = 68.5 \text{ kN/m}^3$ which is approximately 4 times that of a typical soil ($\gamma = 17 \text{ kN/m}^3$), simulating a stress level of a 4-g condition (gravitational acceleration), i.e., a similar stress level in a 1.7 m-high (0.433 m $\times$ 4) slope is achieved in the model slopes. Other details of the model slope were reported by Huang (2015). The facing of steep reinforced slope was jointed with 10 load cells which are capable of measuring normal and shear forces simultaneously with a negligibly small coupling effect which has been reported in-detail by Huang and Chen (2012a). Displacement sensors were installed at various locations to measure the displacement of facing and the settlement at the crest of the slope, as shown by Disp1 ~ Disp8 in Fig. 1.
The internal friction angle ($\phi$) of the steel rod assembly has curved Mohr-Coulomb failure envelope, expressed as $c = 0$ kPa, and $\phi$ as a function of effective confining stress ($\sigma$), based on the result of a medium-scale direct shear tests using a 105 mm-long, 100 mm-high (upper + lower boxes), and 150 mm-wide shear box:

$$\phi = 40.9^\circ - 30.2^\circ \times \log(\sigma / \sigma_{0})$$

where

- $\sigma_{0}$: reference confining stress (= 18.25 kPa)
- $\sigma$: confining stress in the range of 18.25 ~ 50 kPa

In the following stability analyses, three values of internal friction angle were determined based on three different values of overburden pressures ($\sigma$) corresponding to three depths, namely, depths of 216 mm (= 433 mm / 2), 366 mm (= 733 mm / 2), and 583 mm (= 433 mm + 300 mm / 2). Values of $\sigma$ (including self-weight and surcharge, $q = 10$ kPa) at the above-mentioned depths are 25, 35, and 50 kPa, respectively, which in-turn yield $\phi = 37^\circ$, $32^\circ$, and $27^\circ$, for lateral pressure, circular failure, and bearing capacity analyses, respectively. Note that in the following, $\phi = 38^\circ$ rather than $37^\circ$ is used for lateral pressure (or internal stability) analyses, in order to conform with that used in a similar study performed by Huang and Chen (2012a).

A heat-bonded nonwoven geotextile with an ultimate tensile strength ($T_f$) of 4.8 kN/m at a breakage strain of 35% is used as reinforcement in the tests of reinforced slopes. The connection between the reinforcement sheets and the facing blocks is referred to as a "high-strength connection (HC)" based on the pull-out tests reported by Huang and Chen (2012a). Large-strain type strain gages (YFLA-20, Tokyossoki Co., Japan) with a strain limit of 10% ~ 15% are attached to the surface of reinforcement sheets to measure the reinforcement strains during the tests. The measured reinforcement strains were then converted to the reinforcement stress based on the results of strain gage calibration which has been reported in-detail by Huang (2015). It is conceivable that an infinite number of the patterns of scouring could be applied to a structure because the process of scouring is dominated by the dynamics of water currents and the geometries of the structure. As a preliminary study, limited configurations of the scoured patterns were used. Figs. 2(a) and 2(b) schematically show shallow and deep scouring procedures, respectively. Both testing procedures comprise removing 20 mm-thick of the test medium at each step, followed by steepening the slope of the foundation at a rate of 2°/step. Along the planned lines of scouring as shown in Figs. 2(a) and 2(b), steel rods are removed piece-by-piece manually in a top-down manner to simulate a progressive loss of soil particles at slope toe. Removing steel rods in such a manner may not reproduce a scouring process in reality. However, the present study focused on the failure mechanism of the slope with a scoured toe, not on the mechanism of scouring itself. In shallow scouring tests, a maximum cutting of foundation, $D_f$ is 80 mm which is about 20% of $H_f$ (= 433 mm; $H_f$: The full height of the reinforced slope). In deep scouring tests, a maximum value of $D_f = 300$ mm which is about 70% of $H_f$. 

![Fig. 1 Geometry of model test facility](image1)

![Fig. 2 Schematic scouring procedures](image2)
3. OBSERVED FAILURE MECHANISMS

Four model tests are performed and the test results are analyzed here. The test conditions, including \( L_t / H_t \) (\( L_t \): Length of reinforcement), ratios between the total reinforcement area and the facing area, and the angle of scoured slope at ultimate failure states (\( \theta \)) are summarized in Table 1. These tests are selected in the present study because of the reinforcement configurations (\( L_t / H_t = 0.7 \) and 1.0) are of practical significance. Figure 3(a) shows a side-view of an intact slope (\( L_t / H_t = 0.7 \)) before testing. A side-face of the 2-D backfill is marked with white horizontal lines to facilitate the observation of failure mechanisms. Lines are also marked to facilitate the scouring test procedure. Figure 3(b) show the slope at a scoured slope angle of \( \theta = \theta_f = 40^\circ \). It can be seen that local shear planes, in the sequence of ‘1’ to ‘4’ appeared in the lower part of the reinforced zone, as a pre-warning of ultimate collapse. Figure 3(c) shows the slope at the verge of total collapse at \( \theta = \theta_f = 57^\circ \). A compound sliding through the unreinforced and the reinforced zone can be observed. The following two typical failure patterns were observed: (1) a bearing capacity failure of facing associated with pull-out of bottom layers of reinforcement (as shown in Fig. 4(a)), and/or a bearing capacity failure of facing associated with intensive shear bands behind the facing (as shown in Fig. 4(b)) occurred exclusively for the slope subjected to a shallow scouring; (2) a compound failure consisting of a bearing capacity failure in sloped foundation and/or a global sliding scouring (as shown in Figs. 4(c) and 4(d)).

4. OBSERVED SLOPE DISPLACEMENTS

Figure 5 shows the vertical displacements measured at the top of facing during the process of surcharging and horizontal scouring expressed using a ratio between the equivalent of the wall (\( H_e \)) and the total wall height (\( H_t \)) defined as:

\[
H_e = H + \frac{q}{\gamma} + D_s
\]  

where

- \( H \): unsupported height of the wall at-completion of the slope (\( = 333 \text{ mm} \))
- \( q \): intensity of surcharge (\( = 0 \sim 10 \text{ kPa} \))
- \( D_s \): depth of horizontal scouring (\( 0 \sim 100 \text{ mm} \))

It can be seen in Fig. 5 that the slopes subjected to shallow scouring have higher values of \( \theta \) than those for the deep scouring, suggesting a significant influence of the depth of scouring (\( D_s \)) on the behavior of reinforced slopes. In addition, the slopes subjected to shallow scouring have two-step displacement curves as a result of possible stress re-distributions before the ultimate collapse.

5. MEASURED REINFORCEMENT FORCES

Figures 6(a) compare the effect of scouring patterns on the distributions of reinforcement forces measured at \( \theta = 40^\circ \) for the slopes using \( L_t = 330 \text{ mm} = 0.7H_t \). At \( \theta = 40^\circ \), similar reinforcement force intensities and distributions were obtained regardless of the types of scouring. However, distinctly large

<table>
<thead>
<tr>
<th>Test</th>
<th>Scouring type</th>
<th>Reinforcement length (( L_t / H_t ))</th>
<th>Reinforcement area / facing area</th>
<th>Failure state</th>
</tr>
</thead>
<tbody>
<tr>
<td>TE30</td>
<td>Deep scouring</td>
<td>300 mm (0.7)</td>
<td>3.0</td>
<td>( \theta_f (1) = 40^\circ )</td>
</tr>
<tr>
<td>TE30(2)</td>
<td>Shallow scouring</td>
<td>300 mm (0.7)</td>
<td>3.0</td>
<td>( \theta_f (1) = 57^\circ )</td>
</tr>
<tr>
<td>TE43</td>
<td>Deep scouring</td>
<td>430 mm (1.0)</td>
<td>4.3</td>
<td>( \theta_f (1) = 40^\circ )</td>
</tr>
<tr>
<td>TE43(2)</td>
<td>Shallow scouring</td>
<td>430 mm (1.0)</td>
<td>4.3</td>
<td>( \theta_f (1) = 69^\circ )</td>
</tr>
</tbody>
</table>

\( \theta_f \) denotes the angle of the scoured foundation slope at which a total failure is observed.
reinforcement forces were measured at \( \theta = \theta_f = 57^\circ \) for the case of shallow scouring suggesting that the reinforcement forces mobilized more effectively in shallow scouring case than did the deep scouring. Figure 6(b) shows the distributions of measured reinforcement forces for the slopes using \( L_t = 1.0 \) \( H_t \). The effect of scouring patterns on the distributions of reinforcement forces at \( \theta = 40^\circ \) for tests TE43 and TE43(2) is rather small. It is also noted that at the moment of ultimate collapse, significantly higher reinforcement forces can be found in the case of shallow scouring (at \( \theta = \theta_f = 69^\circ \)) than those in deep scouring (at \( \theta = \theta_f = 40^\circ \)). The significantly increased reinforcement forces for lower layers of reinforcement observed at \( \theta_f = 57^\circ \) in Fig. 6(a) and at \( \theta_f = 69^\circ \) in Fig. 6(b) support the failure mechanism observations in Figs. 4(a) and 4(b), i.e., additional tensile forces were mobilized as a result of the pull-out of reinforcement in response to the bearing capacity failure of facing. This observation also agrees well with the two-step settlement of facings prior to the ultimate collapse of the slopes subjected to shallow scouring as discussed for Fig. 5.

6. INTERNAL STABILITY ANALYSES

Figure 7 schematically shows the forces considered in internal stability analyses, i.e., the tie-break and pull-out failures on the potential failure plane and the pull-out (or connection) failure at facing. The safety factor against tie-break failure \( (FS_t) \) is defined as:

\[
FS_t = \frac{T_f}{K_a \cdot \sigma_v \cdot S_v}
\]

where

- \( T_f \): ultimate tensile strength of reinforcement
  \( (= 4.8 \text{ kN/m}) \)
- \( K_a \): Coulomb’s active earth pressure coefficient
  \( (= 0.217 \text{ for } \varphi = 38^\circ) \)
- \( \sigma_v \): effective overburden pressure at the level of reinforcing sheets

Fig. 5 Settlemetn measured at the crest of facing during surcharging and scouring
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Fig. 6 Comparisons of measured reinforcement force distributions between deep and shallow scouring

(a) \( L_t = 0.7 H_t \)

(b) \( L_t = 1.0 H_t \)

Distance from wall face / Wall height

Slope angle \( \alpha = 60^\circ \), \( L_t = 0.7 H_t \)

- TE30(2) (Shallow scouring) at \( \theta_f = 40^\circ \)
- TE30(2) (Shallow scouring) at \( \theta_f = 57^\circ \)
- TE30 (Deep scouring) at \( \theta_f = 40^\circ \)

Tensile force (kN/m)

Sv: tributary height for a certain layer of reinforcement (\( = 0.087 \) m)

The safety factor against pull-out from the stationary zone of backfill (\( F_{S_p} \)) is defined as:

\[
F_{S_p} = \frac{2 \cdot \sigma_v \cdot \tan \mu \cdot L_t}{K_a \cdot \sigma_v \cdot S_v}
\]  

(4)

where,

\( \mu \) : soil-reinforcement interface friction angle

\( L_t \) : length of embedded reinforcement against pull-out (as shown in Fig. 7)

The safety factor against facing pull-out (or connection failure), \( F_{S_f} \) is defined as:

\[
F_{S_f} = \frac{2 \cdot L_f \cdot (c_f + \sigma_v \cdot \tan \varphi_f)}{K_a \cdot \sigma_v \cdot S_v}
\]  

(5)

\( L_f \) : reinforcement length embedded in facing (= width of facing= 0.05 m)

\( c_f \) : cohesion intercept (\( = 0.7 \) kN/m) obtained in reinforcement-facing interface pull-out tests reported by Huang and Chen (2012a)

\( \varphi_f \) : friction angle (\( = 60^\circ \)) obtained in reinforcement-facing interface pull-out tests reported by Huang and Chen (2012a)

Values of \( F_{S_p} \), \( F_{S_p} \), and \( F_{S_f} \), calculated for various types of scouring and values of \( L_t \) are shown in Figs. 12(a) ~ 12(d) which will be discussed in-detail later.

7. BASE SLIDING AND OVERTURNING FAILURE ANALYSES

The safety factor against base sliding (\( F_{S_b} \)) is defined as:

\[
F_{S_b} = \frac{P_a \cdot \cos \varphi_w + Q_0 \cdot \tan \varphi_b}{Q_b}
\]  

(6)

\( Q_b = W_1 + W_2 - P_{a1} \sin \varphi_w - P_{a2} \sin \varphi_w + q \cdot L_t + P_a \cdot \sin \varphi_w
\]  

(7)

where

\( P_a \) : active lateral thrust induced by the uniform surcharge (\( = K_a \times q \times H \))
$P_{a2}$: active lateral thrust induced by the self-weight of backfill ($= 0.5 \times K_a \times \gamma \times H^2$)

$P_p$: passive lateral thrust induced by the self-weight of backfill in front of the wall ($= 0.5 \times K_p \times \gamma \times D^2$; $D$: Height of passive zone in front of the facing; $K_p = 9.64$ for $\phi = 38^\circ$)

$W_1$: weight of facing

$W_2$: weight of reinforced backfill

$q$: uniform surcharge at the crest of the slope

$L_t$: total reinforcement length

$\phi_w$: interface friction angle could be applied to a structure between reinforced and unreinforced zones ($= \phi/2 = 19^\circ$)

$\phi_b$: friction angle at the base of reinforced zone ($= 0.8\phi = 31^\circ$)

The factor safety of overturning around the rotation center (the toe of facing ‘o’ shown in Fig. 9) is defined as:

$$FS_o = \frac{M_r}{M_o}$$

(9)

$$M_r = P_p \cos \phi_w L_p + W_1 L_1 + W_2 L_2 + q L_q L_7$$

(10)

$$M_o = P_{a2} \cos \phi_w L_3 + P_{a1} \cos \phi_w L_6 + P_{a1} \sin \phi_w L_6 + P_{a2} \sin \phi_w L_4$$

(11)

where, $L_1$, $L_2$, ..., $L_6$, $L_{7}$ and $L_o$ are arms of rotation defined in Fig. 9. The calculated values of $FS_o$ and $FS_b$ for the cases of deep and shallow scouring are summarized in Figs. 12(a) ~ 12(d), respectively, which will be discussed later. Note that to analyze base sliding and overturning failures as schematically shown in Figs. 8 and 9, two types of imaginary boundary lines were used to separate reinforced and unreinforced zones, namely, Type-1: A vertical line passing through point o’ (see Figs. 8 or 9), and Type-2: A line parallel to the facing and passing through point o’.

Based on the result of a comparative study using two types of imaginary boundaries, Type-2 was not used in the present study because it yielded: (1) unreasonably large backward eccentricity of resultant reaction force at the base of reinforced zone; (2) higher values of $FS_o$ and $FS_b$ than those obtained using Type-1. This is attributable to: (1) a part of $W_2$ located to the left of point o’ (see Figs. 8 and 9) transmitting the self-weight to the base (o-o’) when using Type-1 boundary; (2) smaller values of $P_{a1}$ and $P_{a2}$ than those using Type-2 imaginary boundary. Regarding (1), the authors believe that the transmission of that part of self-weight to the base of reinforced zone (o-o’) hasn’t been verified. Therefore, further discussions on Type-2 is beyond the scope of the present study.

8. BEARING CAPACITY FAILURE ANALYSES

The safety factor ($FS_b$) against bearing capacity failure of a rigid footing placed on a horizontal or sloped ground is defined as (see Fig. 10):

$$FS_b = \frac{q_u}{q_{max}}$$

(12)

where $q_u$ is the ultimate bearing capacity of a surface footing calculated using the following equation:

$$q_u = \frac{1}{2} \gamma \cdot B' \cdot N_f \cdot F_{\theta} \cdot F_{\eta}$$

(13)

$B'$: effective footing width ($= B - 2e$; $B(= L_o)$: full width of footing; $e$: eccentricity of loading at the base of reinforced slope)

$N_f$: bearing capacity factor for the self-weight of soil (= 10 and 15 suggested by Meyerhof (1963) and Vesic (1973), respectively, for $\phi = 27^\circ$)

$F_{\theta}$: correction factor for ground inclinations (see Eq. 15 by Hansen 1970; Vesic, 1973, 1975; Huang and Kang 2008)

$F_{\eta}$: correction factor for load inclinations represented by the angle of load inclination, $\eta$ (see Eq. 15 by Meyerhof 1963; Huang and Kang 2008)

$$F_{\theta} = \left[ 1 - (1.062 - 0.014 \cdot \phi) \cdot \tan \phi \right]^{0.12}$$

(14)

$$F_{\eta} = \left[ 1 - \left( \frac{\eta}{\phi} \right)^{0.10 \cdot 1.21} \right]$$

(15)
The value of $q_{\text{max}}$ is determined using one of the following two equations:

1. Based on Meyerhof’s effective width of foundation:

$$q_{\text{max}} = \frac{Q}{B}$$

(17)

where

$Q$: total vertical load at the base of reinforced slope

2. Based on the elastic stress distribution at the base of reinforced slope:

$$q_{\text{max}} = \frac{Q}{B} \left( 1 + \frac{e}{6 \cdot L} \right)$$

(18)

$$e = \frac{M}{Q_{\nu}} - \frac{L}{2}$$

(19)

Values of $FS_b$ during surcharging and simulated toe scouring are summarized in Figs. 12(a), 12(b), 12(c), and 12(d) for tests TE30, TE30(2), TE43, TE43(2), respectively. It can be seen that curves for $FS_b$ showed a slight increase of $FS_b$ during surcharging and a drastic decrease for the 2nd stage (from the beginning of sloped scouring to the ultimate collapse). The 1st segment reflects the increase of $q_s$ due to the application of surcharge ($q$) at the crest of the slope. The decreased $FS_b$ in the 2nd segment reflects a combined effect of increased slope angles ($\theta$), increased load inclination angles ($\eta$), as well as the decrease of $B^\prime$. It is noted, however, that current knowledge regarding the ultimate bearing capacity as summarized in Eqs. (13) ~ (15) cannot address the effect of the height of the scoured slope ($D_f$) as shown in Figs. 2(a) and 2(b) to the ultimate bearing capacity of footing. As a result, no difference in the curves of $FS_b$ between deep scouring (Figs. 12(a), 12(c)) and shallow scouring (Figs. 12(b), 12(d)).

9. CIRCULAR FAILURE ANALYSES

As a commonly acknowledged definition of safety factor ($FS_c$) in slope stability analyses:

$$FS_c = \frac{\tau_f}{\tau}$$

(20)

$\tau_f$: ultimate strength at the base of slice $i$ according to the Mohr-Coulomb failure criterion

$\tau$: shear stress at the base of slice $i$

Based on the force equilibrium in the vertical direction, the moment equilibrium and the Mohr-Coulomb failure criterion, Bishop (1955) proposed the following equation of $FS_c$ for a circular failure surface (see Fig. 11):

$$FS_c = \left[ \sum C_i + (W_i - U_i \cdot \cos \alpha_i) \cdot \tan \phi \cdot \sec \alpha_i \right]$$

$$\sum \left( W_i \cdot \sin \alpha_i \right) - \sum (T_i \cdot \cos \alpha_i)$$

(21)

$C_i$, $B_i$: cohesion intercept of soil ($c = 0$ in the present study)

$W_i$: self-weight of slice $i$

$T_i$: Reinforcement force acting at the base of slice $i$ (= 0.45 kN/m based on the averaged value of maximum tensile forces as shown in Figs. 6(a) and 6(b) and the result of preliminary study reported by Huang 2016)

$\alpha_i$: inclination angle of slice base $i$

$u_i$: pore water pressure acting at the base of slice $i$ ($u_i = 0$ in the present study)

$l_i$, $B_i$: the length of the base and the width, respectively, for slice $i$. 

$$C_i = c \cdot l_i = c \cdot B_i \cdot \sec \alpha_i$$

(22)

$$U_i = u_i \cdot l_i = u_i \cdot B_i \cdot \sec \alpha_i$$

(23)
10. RESULTS OF STABILITY ANALYSES

Figures 12(a) ~ 12(d) summarize the results of various stability analyses, in terms of various $FS$ vs. $H_e$ and $FS$ vs. $\theta$ relationships. It can be seen that the internal stability analyses (pull-out, tie-break, and facing connection failures) and the external stability analyses (base sliding and overturning) responded insensitively to the change of $H_e$ and $\theta$. Figures 12(a) ~ 12(d) also show that circular ($FS_c$) and bearing capacity ($FS_b$) failures dominate the ultimate state of reinforced slopes, regardless of the type of scouring. For bearing capacity analyses, upper and lower bound lines of $FS_b$ are presented. The upper bound of $FS_b$ was derived using Vesic’s value of $N_\gamma$ ($\gamma = 15$) in conjunction with the effective footing width $B'$ (Eq. 17); the lower bound was derived using Meyerhof’s value of $N_\gamma$ ($\gamma = 10$) in conjunction with elastic stress distributions (Eqs. 18 and 19). Figure 12(a) shows the result for the slope with $L_t/H_t = 0.7$ and deep-scouring. It can be seen that $FS_b$ dropped drastically and ultimately controlled the failure of the slope subjected to deep scouring. Figure 12(a) also shows that the intersection between lines of $FS_c$ and $FS = 1.0$ fall in the range of $\theta = 20 \sim 30^\circ$ which slightly underestimated the observed value of $\theta_f = 40^\circ$. A similar prediction for the failure condition ($\theta = 30^\circ$) is also derived based on circular failure analyses ($FS_c$). Figure 12(b) shows that for shallow scouring, the circular failure analysis ($FS_c$) outperformed the bearing capacity analysis ($FS_b$), and rendered a predicted value of $\theta_f \approx 60^\circ$ which is comparable with the measured value of $\theta_f = 57^\circ$. Figure 12(c) shows that the circular failure analysis predicted a failure angle of $\theta_f = 37^\circ$ which is close to the measured value of $\theta_f = 40^\circ$. Figure 12(c) also shows that the upper bound line of $FS_b$ also predicted $\theta_f = 37^\circ$ suggesting that bearing capacity and circular analyses are equally useful for the stability calculations in the case of deep-scouring. However, this is not true for the case of shallow scouring as shown in Fig. 12(d) which indicated that only the circular failure analysis can provide a predicted value of $\theta_f$ which is comparable with the measured $\theta_f = 69^\circ$.

![Variations of safety factors during surcharging and toe scouring](image-url)
The delayed total collapse in the case of shallow scouring (at \( \theta_t = 57^\circ \) and 69° in Figs. 12(b) and 12(d), respectively) than those for deep-scouring (at \( \theta_t = 40^\circ \) in Figs. 12(a) and 12(c)) is due to: (1) a local bearing capacity failure (under the base of facing) occurred in the case of shallow scouring, contradicting to the global (or compound) failure in the case of deep-scouring; (2) a secondary support provided by the pull-out resistance of reinforcement as shown in Figs. 4(a) and 4(b) which is also illustrated by the significantly increased tensile forces at lower layers of reinforcement shown in Figs. 6(a) and 6(b). It is also noted that the safety factor \( FS \) obtained using circular failure analysis is the only indicator capable of detecting different boundary conditions induced by shallow and deep scouring. This is because:

1. Equations of internal stability (Eqs. 3 – 5) and external stability of base sliding and overturning (Eqs. 6 – 11) are irrelevant to the scoured profile of the foundation.

2. Equations of bearing capacity (Eqs. 12 – 19) take into account the influences of foundation slope angle (\( \theta_t \)), load inclination (\( \eta_t \)), and load eccentricity (\( e \)). However, the influence of the height of scoured foundation, \( D_f \) (see Figs. 2(a) and 2(b)) on the value of \( q_u \) is not taken into account.

Although the circular failure analysis is proved useful based on the above-mentioned investigation, this limit-equilibrium method requires input reinforcement force as an essential input parameter. In the present study, an averaged value of the measured maximum tensile forces (Figs. 6(a) and 6(b)) was used as the input reinforcement force \( T_r = 0.45 \text{kN/m} \). This drawback may be overcome via using a force-equilibrium-based finite displacement method (FFDM) proposed by Huang (2014) in which the reinforcement force is a part of the output rather than an input.

11. CONCLUSIONS

Limit equilibrium analyses were performed to evaluate the stability of reinforced slopes at various stages of scoured toe conditions. The process of toe scouring was simulated by removing the backfill horizontally at the passive zone, followed by steepening the foundation ground in front of the slope toe. Results of analyses suggested that bearing capacity and circular failures are dominant failure mechanisms for the slope reinforced with uniform reinforcement lengths of \( L_r = 0.7 \) and 1.0 \( H_r \) (height of reinforced slopes). It was found that results of current internal stability (pull-out, tie-break, and facing connections) and external stability (base sliding and overturning) analyses responded insensitively to the boundary changes induced by the toe scouring. The circular failure analysis using a slice method is useful to evaluate the stability status changes of reinforced slopes induced by deep and shallow foundation scouring, provided that adequate input values for the reinforcement force are used. Although the bearing capacity analysis was incapable of detecting the difference between shallow and deep scouring, it was potentially useful for predicting bearing capacity failures for the slopes subjected to deep-scouring.

REFERENCES


