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Membrane effect of geosynthetic reinforcement subjected to localized sinkholes

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Abstract: Membrane effect often occurs in geosynthetic-reinforced structures, where subsoil may have voids or sinkholes. An analytical model is proposed to estimate membrane effect of geosynthetic reinforcement subjected to localized sinkholes. The upper interface friction in subsided area and vertical deformation of supporting soil in anchorage area are considered simultaneously. The maximum geosynthetic strain and the maximum surface settlement, served as key design points, can be determined. Based on the proposed method verified using a full-scale experiment, a parametric study is conducted. The results show that ignoring upper interface friction results in significant undervaluation of maximum geosynthetic strain, and ignoring vertical deformation of supporting soil leads to obvious undervaluation of maximum surface settlement. A practical design framework is also proposed and it is an applicable tool for preliminary design of geosynthetic-reinforced structures especially for cases with soft ground.

Key words: Geosynthetic; Membrane effect; Arching effect; Maximum strain; Maximum surface settlement.

Introduction

Geosynthetic-reinforced structures are extensively used in geotechnical engineering. In such an structure where subsoil may have voids or sinkholes, membrane effect often occurs along with soil arching (e.g., Giroud 1981; Kinney and Connor 1987; Giroud et al. 1990; Wang et al. 1996; Han and Gabr 2002; Rogbeck et al. 2003; Briançon and Villard 2008; Abusharar et al. 2009; Lu and Miao 2015; Feng and Lu 2015; Huckert et al. 2016; King et al. 2017). During a subsiding process, geosynthetic interacts with overlying soil: subsided soil deflects geosynthetic and is also prevented from sinking by the deflected geosynthetic at the same time. Such interactions result in load transmission from subsided zone to surrounding less-deformed zone (i.e., arching effect) and load-deformation coordination of deflected geosynthetic (i.e., membrane effect). The former is evaluated to obtain the load acting on geosynthetic over cavity (e.g., Villard et al. 2016; Feng, et al. 2017a, b); the latter directly decides the maximum tensile strain of geosynthetic and greatly influences the surface settlement (e.g., Briançon and Villard 2008; Villard et al. 2016), both of which are key design points. This study will focus on the force and deformation characteristics in membrane effect, which are important for rational design of geosynthetic-reinforced structures.

To evaluate the anti-settling performance of geosynthetic over cavity, ultimate state and some simplification assumptions are adopted. First of all, it is generally assumed that the geosynthetic is fixed at the edges of cavity and its strain is uniform. Based on this, the geosynthetic is assumed to deflect into regular shapes: circular arc (e.g., Giroud 1981; Giroud et al. 1990; Kuo et al. 2005), parabolic curve (Giroud 1995) and combined parabolic-circular curve (Shukla and Sivakugan 2009). Geosynthetic strain calculated by all these methods only depends on the ratio between deflection

and cavity width, and is not affected by overburden load and even mechanical properties of geosynthetic. Moreover, there does not exist a point with maximum strain in deflected geosynthetic. Obviously, these fly in the face of the real scenario (e.g., Villard and Briançon 2008; Huckert et al. 2016).

Differently, Espinoza (1994) deduced non-uniform strain of deflected geosynthetic with irregular shape based on its force equilibrium conditions. Then, Gourc and Villard (2000) obtained the maximum deflection and maximum tensile force for serviceability design of geosynthetic-reinforced structures subjected to localized sinkholes. But at this point, the slippage between soil and geosynthetic has not been taken into account. Additionally, Kinney and Connor (1987) firstly divided the deflected geosynthetic into two parts (i.e., subsided and anchorage areas) to consider the deformation coordination. Later, Villard and Briançon (2008) analyzed the relative slippage between soil and geosynthetic in anchorage area based on Coulomb friction law. Besides, Feng and Lu (2015) expanded its applicability to geosynthetic reinforcement over two adjacent voids. However, all the above methods ignored the vertical deformation of supporting soil in anchorage area, which may significantly influence the evaluation of anchoring effect of the geosynthetic in this area and then membrane effect (e.g., Yu and Bathurst 2017), especially for soft ground. Furthermore, the relative slippage between overlying soil and geosynthetic in subsided area has rarely been considered by the existing analytical methods, which changes the external load on deflected geosynthetic. Therefore, it is necessary to improve the evaluation model of membrane effect.

This study presents an effective framework for estimating the membrane effect in geosynthetic-reinforced structures under local subsidence. An analytical model is developed first. The model is then verified using a full-scale experiment. Afterward,

the influences of load distribution types, upper interface friction angle, modulus of subgrade reaction and subsidence width on maximum geosynthetic strain and maximum surface settlement are studied using the proposed model. Finally, a practical design method for geosynthetic-reinforced structures is proposed.

Formulation and solution

Fig. 1 depicts a geosynthetic-reinforced structure consisting of embankment fill, geosynthetic and supporting soil. Void or sinkhole in the soil substratum results in the occurrence of soil arching in the overlying soil and membrane effect of the deflected geosynthetic (Fig. 2). During the subsidence, the downward moving soil in the subsided area transfers load to the surrounding areas with less deformation, leading to redistribution of stress acting on the geosynthetic. Meanwhile, combination of the embankment fill and the supporting soil deflects the geosynthetic, as shown in Fig. 2. The arching effect, with representation of load transfer mechanism, can be accurately estimated by the improved Terzaghi formulation reported by Villard et al. (2016). The membrane effect, symbolizing force and deformation characteristics of the deflected geosynthetic, can be described by the proposed model in the later part.

Model characteristics

The deflected geosynthetic can be divided into two parts (i.e., subsided area and anchorage area in Fig. 2) and then to be analyzed separately. In the process of subsidence, the geosynthetic in the subsided area is stretched with no interaction with the supporting soil and there is no disengagement between the overlying soil and the geosynthetic (Fig. 2), and the upper interface friction is considered in this area. The geosynthetics on both sides of the subsided area play an anchoring role and the

vertical deformation of the supporting soil should not be neglected. To develop the analytical model, some more assumptions are made as follows:

(1) The behavior of the geosynthetic serving as a reinforcement is assumed to be linearly elastic (i.e., $T = J\varepsilon$ where T is the tension in the geosynthetic, J is the stiffness for a unit width of the geosynthetic, and ε is the strain) and the strain of deflected geosynthetic is always less than its allowable tensile strain, ε_a .

(2) The interface friction between soil and geosynthetic agrees with the Coulomb friction law (Fig. 3). If the relative displacement between soil and geosynthetic, u , is larger than the critical relative displacement when the interface friction is fully mobilized (e.g., Gilbert and Byrne 1996), u_0 , the frictional stress can be defined as $\tau = \tau_{\max} = \sigma_n \tan \phi$ where $\tau_{\max}$ is the maximum frictional stress, σ_n is the normal stress, and ϕ is the interface friction angle. Otherwise, τ is expressed as

$$\tau = \tau_{\max} \frac{u}{u_0} = \sigma_n \tan \phi \frac{u}{u_0} \quad (1)$$

(3) A vertical stress-displacement relationship is assumed for the supporting soil:

$$p - q = k_s w \quad (2)$$

where q and p are respectively the stresses on the upper and lower interfaces between soil and geosynthetic, and are assumed to be distributed along the horizontal direction; k_s is the modulus of subgrade reaction; w is the vertical displacement. The overlying and supporting soils of the geosynthetic in the anchorage area are assumed to have no horizontal displacement. Thus, the tensile displacement of geosynthetic can be used as the relative displacement between soil and geosynthetic.

Membrane effect can be deciphered by coupling the forces and deformations of the geosynthetic in subsided and anchorage areas based on the point A in Fig. 2. Thus,

evaluation of membrane effect can be achieved by four steps: (i) determining the vertical stress acting on the geosynthetic; (ii) analyzing the force and deformation of geosynthetic in the subsided area; (iii) analyzing the force and deformation of geosynthetic in the anchorage area; (vi) calculating the maximum surface settlement caused by the void or sinkhole.

Determination of load applied to the geosynthetic

The load applied to the geosynthetic can be determined by Terzaghi's formulation and is generally assumed to be uniform (Giroud et al. 1990; Villard et al. 2000; Briançon and Villard 2008; Lu and Miao 2015; Feng et al. 2017a, b). There are two types of void: a long void and a circular void (Giroud et al. 1990). For a long void with width B_L , the vertical stress acting on the geosynthetic in the subsided area, q_{1L} , can be expressed as

$$q_{1L} = \frac{B_L \gamma_s}{2K \tan \phi_s} \left[1 - e^{-K \tan \phi_s (2H / B_L)} \right] + q_0 e^{-K \tan \phi_s (2H / B_L)} \quad (3)$$

where K is the lateral pressure coefficient and defined as the ratio between the horizontal and vertical stresses; γ_s , ϕ_s , and H are the unit weight, internal friction angle and height of the overlying soil, respectively; q_0 is the surcharge applied on the ground surface. For a circular void with diameter B_C , q_{1C} can be expressed as

$$q_{1C} = \frac{B_C \gamma_s}{4K \tan \phi_s} \left[1 - e^{-K \tan \phi_s (4H / B_C)} \right] + q_0 e^{-K \tan \phi_s (4H / B_C)} \quad (4)$$

Based on discrete-element modeling and full-scale experiment, Villard et al. (2016) proposed an appropriate value for K (i.e., 1.2 for sand and 1.5 for coarse granular materials) in Eqs. (3) and (4), and pointed out the vertical load (q_{1L} or q_{1C}) acting on the geosynthetic is not uniform. From the cavity opening modes (i.e., progressive process of cavity diameter opening and gradual settlement process), there

are two load types (Fig. 4) for a circular void: approximately conical load and inverted parabolic load. For the two types, the total vertical loads carried by the geosynthetic in the subsided area are the same and determined by Eq. (4) and void size. Thus, the non-uniformly load along the horizontal direction, $q_{1C}(x)$, can be obtained as

$$\left\{ \begin{array}{l} \text{Approximately conical load:} \\ q_{1C1} = \frac{3rq_{1C}}{3+r}, q_{1C2} = \frac{3q_{1C}}{3+r} \\ q_{1C}(x) = q_{1C1} + q_{1C2} - \frac{2q_{1C1}x}{B_C} \end{array} \right. \quad \text{or} \quad \left\{ \begin{array}{l} \text{Inverted parabolic load:} \\ q_{1C1} = \frac{2rq_{1C}}{2+r}, q_{1C2} = \frac{2q_{1C}}{2+r} \\ q_{1C}(x) = q_{1C2} + \frac{4q_{1C1}x^2}{B_C^2} \end{array} \right. \quad (5)$$

where q_{1C1} is the maximum value of non-uniform part of the load, and q_{1C2} is the value of uniform part (see Fig. 4); r is the ratio q_{1C1}/q_{1C2} ; x is the distance to the middle of cavity.

Response of geosynthetic in subsided area

A geosynthetic reinforcement is symmetrically embedded in the supporting soil with a void (Fig. 2). In order to analyze the response of geosynthetic in subsided area to local subsidence, a half part of the geosynthetic and a differential element with length of ds are analyzed (e.g., Espinoza 1994; Patra and Shahu 2012, 2015), as shown in Fig. 5. Thus, considering force equilibrium in horizontal and vertical directions, the following equations are obtained:

$$dT_{Lh} = \tau_{1u} ds \cos \beta_L \quad (6a)$$

$$dT_{Lv} = q_1 dx + \tau_{1u} ds \sin \beta_L \quad (6b)$$

where T_{Lh} and T_{Lv} are the horizontal and vertical components of tension (T_L), respectively; q_1 and τ_{1u} are the vertical and frictional stresses acting on the upper interface, respectively; β_L is the angle between tangential direction of ds and the

horizontal direction; x is the horizontal distance from the differential element to the origin of x - y cartesian coordinate system (Fig. 5). Then, dividing Eqs. (6a) and (6b) by dx gives

$$\tau_{lu} = \frac{dT_{Lh}}{dx} \quad (7a)$$

$$q_1 = \frac{dT_{Lv}}{dx} - \tau_{lu} \tan \beta_L \quad (7b)$$

And based on geometry of the deflected geosynthetic, the following relationship can be established:

$$\begin{cases} T_{Lh} = T_L(x) \cos \beta_L(x), & T_{Lv} = T_L(x) \sin \beta_L(x) \\ y'(x) = \tan \beta_L(x) = \frac{dy}{dx} \end{cases} \quad (8)$$

which gives

$$T_{Lv} = T_{Lh} y'(x) \quad (9)$$

Taking the first-order derivative of Eq. (9) and placing the resulting equation into Eqs. (7a) and (7b), yield

$$q_1 = T_{Lh} y''(x) \quad (10)$$

Invoking the assumption (1) and the strain-displacement relationship of deflected geosynthetic, the following equations can be given:

$$\begin{cases} T_{Lh}(x) = J \varepsilon(x) \cos \beta_L = J \frac{\varepsilon(x)}{\sqrt{1 + y'^2(x)}} \\ u'(x) = \varepsilon(x) \sqrt{1 + y'^2(x)} \end{cases} \quad (11)$$

which gives

$$T_{Lh}(x) = J \frac{u(x)}{1 + y'^2(x)} \quad (12)$$

where u is the tensile displacement of a s -length geosynthetic from the origin.

Substituting Eq. (12) in Eq. (10), yields

$$y''(x) = \frac{q_1}{J} \frac{1 + y'^2(x)}{u(x)} \quad (13)$$

Additionally, by taking the first-order derivative of Eq. (12) and substituting the resulting equation and Eq. (13) into Eq. (7a), yields

$$u''(x) = 2 \frac{q_1}{J} y'(x) + \tau_{1u} \frac{1 + y'^2(x)}{J} \quad (14)$$

Invoking the assumption (2), τ_{1u} can be formulated as

$$\begin{cases} \tau_{1u} = q_1 \cos^2 \beta_L \tan \phi_u \frac{u(x)}{u_0} & \text{if } u < u_0 \\ \tau_{1u} = q_1 \cos^2 \beta_L \tan \phi_u & \text{if } u \geq u_0 \end{cases} \quad (15)$$

where ϕ_u is the upper interface friction angle between soil and geosynthetic. Thus, by substituting Eq. (15) into Eq. (14) and combining the resulting expression and Eq. (13), the governing equations representing the force and deformation of deflected geosynthetic in subsided area can be obtained:

$$\begin{cases} u''(x) = 2 \frac{q_1}{J} y'(x) + \frac{q_1 \tan \phi_u}{J} \frac{u(x)}{u_0} & \text{if } u < u_0 \quad \textcircled{1} \\ u''(x) = 2 \frac{q_1}{J} y'(x) + \frac{q_1 \tan \phi_u}{J} & \text{if } u \geq u_0 \quad \textcircled{2} \\ y''(x) = \frac{q_1}{J} \frac{1 + y'^2(x)}{u(x)} & \quad \textcircled{3} \end{cases} \quad (16)$$

Response of geosynthetic in anchorage area

Similar to the subsided area, the force equilibrium of a differential element, dS , from the geosynthetic in anchorage area (Fig. 6) also yields

$$\tau_{2u} + \tau_{2l} = \frac{dT_{Rh}}{dX} \quad (17a)$$

$$p_2 - q_2 = \frac{dT_{Rv}}{dX} - (\tau_{2u} + \tau_{2l}) \tan \beta_R \quad (17b)$$

where T_{Rh} and T_{Rv} are the horizontal and vertical components of tension (T_R) for the

geosynthetic in anchorage area, respectively; q_2 and p_2 are the vertical stresses acting on the upper and lower interfaces between soil and geosynthetic, respectively; τ_{2u} and τ_{2l} are the frictional stresses acting on the upper and lower interfaces, respectively; β_R is the angle between tangential direction of dS and the horizontal direction; X is the horizontal distance from the differential element to the origin of X - W coordinate system (Fig. 6). For the anchorage area, similar equations to Eqs. (10), (12) and (15) can be reformulated:

$$p_2 - q_2 = T_{Rh} W''(X) \quad (18)$$

$$T_{Rh}(X) = J \frac{U(X)}{1 + W'^2(X)} \quad (19)$$

$$\begin{cases} \tau_{2u} + \tau_{2l} = (q_2 \tan \phi_u + p_2 \tan \phi_l) \cos^2 \beta_R \frac{U(X)}{u_0} & \text{if } U < u_0 \\ \tau_{2u} + \tau_{2l} = (q_2 \tan \phi_u + p_2 \tan \phi_l) \cos^2 \beta_R & \text{if } U \geq u_0 \end{cases} \quad (20)$$

where U is the tensile displacement of a S -length geosynthetic from the origin of X - W coordinate system; ϕ_l is the lower interface friction angle. Then, invoking the assumption (3) for Eq. (18) and substituting Eq. (19) into the resulting equation yield

$$W''(X) = \frac{k_s}{J} \frac{W(X) [1 + W'^2(W)]}{U'(X)} \quad (21)$$

Similarly, by taking the first-order derivative of Eq. (19), substituting the resulting expression and Eq. (20) into Eq. (17a) and then combining these equations and Eq. (21), the governing equations representing the force and deformation of deflected geosynthetic in anchorage area can also be given:

$$\begin{cases}
U''(X) = 2 \frac{k_s}{J} W(X) W'(X) + \frac{q_2(\tan \phi_u + \tan \phi_l)}{J} \frac{U(X)}{u_0} + \frac{k_s W(X) \tan \phi_l}{J} \frac{U(X)}{u_0} & \text{if } U < u_0 \quad (1) \\
U''(X) = 2 \frac{k_s}{J} W(X) W'(X) + \frac{q_2(\tan \phi_u + \tan \phi_l)}{J} + \frac{k_s W(X) \tan \phi_l}{J} & \text{if } U \geq u_0 \quad (2) \\
W''(X) = \frac{k_s}{J} \frac{W(X) [1 + W'^2(X)]}{U'(x)} & (3)
\end{cases}
\quad (22)$$

Next, considering the overall equilibrium at point A in Fig. 2 as the connecting point between the subsided area (Fig. 5) and the anchorage area (Fig. 6), the coupling relationship can be obtained as follows:

$$\begin{cases}
T_{AL} = T_{AR} \\
\beta_{AL} = \beta_{AR}
\end{cases} \quad (23)$$

where T_{AL} and T_{AR} are the tensile forces of the deflected geosynthetic at point A at the right edge of subsided area (Fig. 5) and the left edge of anchorage area (Fig. 6), respectively; β_{AL} and β_{AR} are the intersection angles on both sides of point A. Moreover, the compatibility of deformation occurs in the deflected geosynthetic and can be expressed as

$$(s_L + s_R) - \left(\frac{B}{2} + L\right) = u_{AL} + U_{AR} \quad (24)$$

where L is the initial length of geosynthetic in anchorage area; s_L and s_R are the lengths of deflected geosynthetic in subsided and anchorage areas, respectively; u_{AL} and U_{AR} are the displacements on both sides of point A, respectively. Additionally, the boundary conditions are as follows: (1) for the subsided area, $u(x=0)=0$, $y(x=0)=0$ and $y'(x=0)=0$; (2) for the anchorage area, $U(X=0)=0$, or $U'(X=0)=10^{-12}$ and $W'(X=0)=0$. Thus, by combining the governing equations (16) and (22), coupling relationship, deformation compatibility and boundary conditions, the membrane effect symbolizing force and deformation characteristics of the deflected geosynthetic over cavity can be evaluated effectively.

Prediction of maximum surface settlement

The design of a geosynthetic-reinforced structure depends on the stiffness requirement of geosynthetic and uneven surface settlement criterion. The maximum tensile strain ($\varepsilon_{\max}$) of deflected geosynthetic, which can be calculated by the proposed method in this study, must be less than the allowable tensile strain (ε_a). The ratio (A_e) between $\varepsilon_{\max}$ and ε_a is called the anchorage coefficient (i.e., $A_e = \varepsilon_{\max}/\varepsilon_a$), which reflects the anchoring effect of geosynthetic in anchorage area as well as the membrane effect. For the same geosynthetic reinforcement, geosynthetic strain is proportional to its tensile force, and the larger $\varepsilon_{\max}$ at point A, the greater anchoring effect. On the other hand, the membrane effect of geosynthetic depends on its stretch degree after subsidence, also symbolized by A_e . When $A_e = 1$, the membrane effect has been fully developed. In contrast, when the subsoil has no voids or sinkholes, the membrane effect and anchoring effect are not developed (i.e., $A_e = 0$).

In addition, there exists a limitation value of surface settlement (s_a), corresponding to one subsidence width (B_L or B_C), defined by the RAFAEL method (Villard et al. 2000, 2016; Briançon and Villard 2008) for meeting the surface settlement requirement. The RAFAEL method considers a cylindrical collapse over the cavity, a parabolic shape of the deflected geosynthetic and the ground surface, and soil expansion with a global expansion factor. For a long void with width B_L , the maximum surface settlement, $s_{\max}$, can be determined as

$$s_{\max} = f_{\max} - 3H \frac{(C_e - 1)}{2} \quad (25)$$

where C_e is the expansion coefficient and defined as the ratio between the soil volume (V_s) in subsided area after subsidence and the initial soil volume (V_0) before subsidence ($C_e = V_s/V_0$); $f_{\max}$ is the maximum vertical displacement of deflected

geosynthetic. For a circular void with diameter B_C , $s_{\max}$ can also be determined as

$$s_{\max} = f_{\max} - 2H(C_e - 1) \quad (26)$$

Solution

The membrane effect is jointly decided by the stresses and deformations of geosynthetic in the subsided and anchorage areas. For geosynthetic in the subsided area, there may exist two kinds of deformation response based on the assumption (2): (a) $u_{AL} < u_0$, $[0, B/2]$ governed by Eq. (16 ①+③); (b) $u_{AL} \geq u_0$, $[0, x_C]$ governed by Eq. (16 ①+③) and $[x_C, B/2]$ governed by Eq. (16 ②+③). And likewise, for the geosynthetic in the anchorage area, there may exist three kinds of responses: (a) $U(X=0) \geq u_0$, $[0, L]$ governed by Eq. (22 ②+③); (b) $U(X=0) < u_0$ and $U_{AR} < u_0$, $[0, L]$ governed by Eq. (22 ①+③); (c) $U(X=0) < u_0$ and $U_{AR} \geq u_0$, $[0, X_C]$ governed by Eq. (22 ①+③) and $[X_C, L]$ governed by Eq. (22 ②+③). x_C and X_C are the horizontal distances from the point with u_0 to the origin for the subsided and anchorage areas, respectively.

An iterative method is adopted for the whole solution, as shown in Fig. 7. To begin with, an arbitrary initial value of $u'(x=0)$ is assumed. The response of the deflected geosynthetic in the subsided area is solved by combining the governing equations Eq. (16) and the boundary conditions (1). Next, arbitrary initial values are assigned to $U(X=0)$, or $U'(X=0)$ and $W(X=0)$. At this point, the response of the deflected geosynthetic in the anchorage area is also solved by combining the governing equations Eq. (22) and the boundary conditions (2). Then, by updating these unknown boundary values and executing iterative calculation for the anchorage area, the coupling condition Eq. (23) is satisfied. Additionally, updating the value of $u'(x=0)$ makes the elongation of geosynthetic to meet the compatibility deformation

condition Eq. (24). Thus, the membrane effect of geosynthetic for local subsidence can be evaluated through the solution procedures.

Model verification

This section compares the results of the analytical calculations by this study and the full-scale experimental tests conducted by Villard and Briançon (2008). Table 1 shows the geometry and material properties of soil and geosynthetic. For ease of comparison, the same simplifications from Villard and Briançon (2008) are made here: the arching effect in the overlying soil after subsidence is ignored, and the load over the geosynthetic is assumed to be uniformly distributed (i.e., $q_1 = q_2 = \gamma_s H = 8.5 \text{ kPa}$). The modulus of subgrade reaction, k_s , which is close to the elastic modulus (E_s) as a result of $\nu = 0.3$ (Daloglu and Vallabhan 2000), is assumed to be 30 MPa/m. The strain and vertical displacement of deflected geosynthetic are compared, as shown in Fig. 8.

In Fig. 8, the values of strain and vertical displacement of deflected geosynthetic in the anchorage area, calculated by the present method, are close to those of the full-scale experimental tests. It is noteworthy that the vertical deformation of supporting soil after subsidence is well described by the present method, as shown in Fig. 8b. In the subsided area, the results of the present method also match those of the experimental tests reasonably well. Obviously, from Fig. 8a, the maximum strain of deflected geosynthetic occurs at point A. Thus, the method in this study can be used to estimate the membrane effect of deflected geosynthetic over cavity.

Parametric study

Effect of load type on geosynthetic

In this part, three types of distributed load are considered (Fig. 9a): uniform load – UL, conical load – CL and inverted parabolic load – IPL. The load applied to the geosynthetic can be determined: q_1 in Eq. (16) = q_{1C} in Eq. (4) or $q_{1C}(x)$ in Eq. (5), and $q_2 = \gamma_s H$. To be consistent with Villard et al. (2016), the experimental test conducted by Huckert et al. (2016) is only adopted for this analysis. The needed parameters are shown in Table 2. And based on the numerical simulations reported by Villard et al. (2016), $K = 1.3$ (Eq. 4) and $r = q_{1C1}/q_{1C2} = 4$ (Eq. 5) were obtained when $B_C/H = 2.2$.

Fig. 9 compares the strains (ε) and vertical displacements (y) of deflected geosynthetic between experimental results (Huckert et al. 2016) and analytical calculations (i.e., Villard et al. 2016 and this study). As shown in Fig. 9, ranking the strains and vertical displacements of deflected geosynthetic under the three load types, it is clear that $CL > UL > IPL$, and the difference is mainly embodied in the subsided area. With the same total load, load UL is uniformly distributed along the geosynthetic, load CL is mainly distributed near the middle of subsided area while load IPL is mainly distributed near point A according to Eq. (5). The different load distributions along the geosynthetic cause the significantly different ε and $|y|$. Thus, it is crucial to accurately determine load distribution type when considering membrane effect. Additionally, compared with load IPL, the values of ε and $|y|$ under load CL are closer to the experimental results of Huckert et al. (2016), since the latter is generated by a progressive process of cavity diameter opening similar to the full-scale experiment.

From Fig. 9a, the effect of load type on the variation of geosynthetic strain in the subsided area is more significant for the present method than the analytical method

proposed by Villard et al. (2016). From the middle of subsided area to its edge, the changing curves (UL, CL, and IPL) calculated by the former are steeper than those by the latter. The main reason is that the upper interface friction in the subsided area is considered by the present method but ignored by the method proposed by Villard et al. (2016). From Fig. 2, the upper interface friction promotes the stretch of geosynthetic relative to point A and the anchoring effect (A_e or $\varepsilon_{\max}$).

In order to further investigate the variations of strain and vertical displacement of geosynthetic in Fig. 9, the variations of relative displacement (u or U) between soil and geosynthetic along the interface under different load types are shown in Fig. 10. From the marginal edges of subsided and anchorage areas to the junction point A (see Fig. 2), u and U both gradually increase. And based on the compatibility condition (Eq. 24), the summation of relative displacement is equal to the difference between the length of deflected geosynthetic and the initial length prior to deflection. When $u > u_0$ or $U > u_0$, the interface slippage occurs and thus there exists some critical sliding points (x_C in subsided area and X_C in anchorage area) in Fig.10. Same as y or ε in Fig. 9, $CL > UL > IPL$ for u and U . Thus, the higher load concentration level at the middle of subsided area leads to the larger relative displacement (u and U), stronger membrane effect ($\varepsilon_{\max}$ and A_e), greater geosynthetic deflection ($y_{\max}$) and maximum surface settlement ($s_{\max}$).

However, the ratio r in Eq. (5) determining the non-uniformly distributed load, which changes with B_C/H , still cannot be obtained by means of an analytical method. Thus, load UL applied to the geosynthetic in the subsided area is adopted for analysis in the following parts. And based on the model verification, the experimental test conducted by Villard and Briançon (2008) is also adopted.

Effects of width-height ratio and anchorage length

Fig. 11 shows the variations of maximum geosynthetic strain and maximum surface settlement with width-height ratio and anchorage length. All the required parameters are shown in Table 1 except L and H . The expansion coefficient of overlying soil, C_e , in Eq. (25) is 1.1, varying between the recommended range (1.03 ~ 1.15) reported by Villard et al. (2016). The arching effect is considered: q_1 in Eq. (16) = q_{1L} in Eq. (3) with $K = 1.3$, and $q_2 = \gamma_s H$. As shown in Fig. 11, with increasing B/H , $\varepsilon_{\max}$ decreases and $s_{\max}$ increases. Under constant B , larger B/H means smaller H , leading to smaller overlying load (q_1 and q_2), and then smaller $\varepsilon_{\max}$ and $f_{\max}$. And from Eq. (25), $s_{\max}$ increases with $f_{\max}$ but decreases with H . Thus, the increase of $s_{\max}$ is mainly caused by a prevailing effect of H instead of $f_{\max}$. Additionally, from Fig. 11b, $s_{\max}$ approaches zero when $B/H < 1.25$ (i.e., $H > 1.6$ m). Thus, for the cases with greater H , the cavity below the geosynthetic does not affect the ground surface.

Then, $\varepsilon_{\max}$ and $s_{\max}$ for different L are also analyzed. The anchoring effect (A_e) determined by $\varepsilon_{\max}$ is influenced by the length of geosynthetic in the anchorage area (L). When L is close to zero, there is almost no anchoring effect ($A_e \rightarrow 0$). The exertion of anchoring effect (A_e) relies on lengthening L . When $B/H \geq 2.5$ (i.e., $H \leq 0.8$ m), $\varepsilon_{\max}$ for the case of $L = 1.5$ m is obviously smaller than that of $L = 2.0$ or 2.5 m (Fig. 11a). When $B/H < 2.5$ (i.e., $H > 0.8$ m), $\varepsilon_{\max}$ with different L are almost the same, which reveals that the anchoring effect from the geosynthetic in the anchorage area has been fully developed even for $L = 1.5$ m. When subsidence happens, the anchoring effect of geosynthetic near point A first develops by means of its overlying and underlying soils. The higher overlying soil in the anchorage area, the greater anchoring effect. When $H > 0.8$ m, the anchoring effect has been fully developed and lengthening L has no favoring effect. Similar to $\varepsilon_{\max}$, when $L \geq 2.5$ m for $B/H = 1.0 \sim$

4.0, the change in L has no effect on $s_{\max}$ (Fig. 11b). Thus, the fixed length of geosynthetic ($L = 2.5$ m) can be used for analyses in the following parts.

Fig. 11 also compares the results of three analytical calculations: Villard and Briançon (2008), the modified method with consideration of upper interface friction in the subsided area and the present method. From Fig. 11a, $\varepsilon_{\max}$ is undervalued by the first method but slightly overvalued by the second method. And in Fig. 11b, $s_{\max}$ is undervalued by both the first and the second methods. Thus, the present method is very necessary for rational design of geosynthetic-reinforced structures.

Effects of upper interface friction angle and modulus of subgrade reaction

Figs. 12 and 13 respectively show the effects of upper interface friction angle and modulus of subgrade reaction on the strain and vertical displacement of deflected geosynthetic. All the required parameters are shown in Table 1 except k_s and ϕ_u . Under the same load, the upper interface friction in the subsided area is reflected by ϕ_u . From Fig. 12, with increasing ϕ_u , $\varepsilon_{\max}$ increases obviously and $f_{\max}$ decreases slightly. Thus the change in ϕ_u has greater influence on $\varepsilon_{\max}$ but less influence on $f_{\max}$. Observing Fig. 2, it is obvious that the increase of ϕ_u enhances the interface friction on both sides of point A and then promotes the anchoring effect, which enlarges $\varepsilon_{\max}$. The increase of interface friction in the anchorage area hinders the deflection of geosynthetic, but that in the subsided area is on the contrary. Furthermore, the former plays a dominant role, resulting in the slight decrease of $f_{\max}$.

According to the assumption (3) or Eq. (2), the modulus of subgrade reaction, k_s , governs the vertical deformation of supporting soil. As shown in Fig. 13, with increasing k_s , $\varepsilon_{\max}$ increases slightly and $f_{\max}$ decreases obviously. And the increasing ($\varepsilon_{\max}$) and decreasing ($f_{\max}$) rates moderate with the increase of k_s . The smaller vertical

deformation of supporting soil caused by larger k_s promotes the anchoring effect of geosynthetic in the anchorage area. However, this hinders the deflection of geosynthetic in the anchorage area, hence leading to smaller $f_{\max}$ with increasing k_s . Thus, ϕ_u and k_s should be synthetically considered for satisfying the stiffness requirement of geosynthetic and maximum surface settlement criterion.

Effect of subsidence width

Fig. 14 shows the effect of subsidence width on the strain and vertical displacement of deflected geosynthetic over cavity. Due to the assumption that the geosynthetic in the subsided area has no supporting soil, B is regarded as the unique parameter representing the local subsidence condition. Same as before, all the required parameters are shown in Table 1 except B . As shown in Fig. 14, at any point of geosynthetic, ε and $|y|$ increase significantly with increasing B . For the cases of $B = 1.5, 2.0$ and 2.5 m, the obtained $\varepsilon_{\max}$ and $f_{\max}$ are 1.37%, 1.83%, 2.24% and 15.77, 23.06, 31.02 cm, respectively. The increase of subsidence width not only enlarges the vertical load acting on the geosynthetic in subsided area but also extends the contact length between soil and geosynthetic, which promotes the membrane effect. Fig. 15 shows the variations of relative displacement (u or U) between soil and geosynthetic along the interface for different B . From Fig. 15, with increasing B , the critical sliding points (x_C in subsided area and X_C in anchorage area) move to the opposite direction, away from the junction point A. Different from H , the increase of B only leads to the increase of the overlying load acting on the geosynthetic in the subsided area. The wider subsided area leads to the greater anchoring effect. In conclusion, the change in B has significant influence on $\varepsilon_{\max}$ and $f_{\max}$ or $s_{\max}$, and accurate determination of subsidence width is crucial to the design of geosynthetic-reinforced structures.

Practical design

This section proposes a practical design framework for geosynthetic-reinforced structures under local subsidence including the design charts, design method and an example application. It is noteworthy that the design framework is only applicable for the sheet-shape geosynthetic reinforcements (e.g., geotextiles), but not for the hole-shaped geosynthetic reinforcements (e.g., geogrids). When using the latter, the resistance resulting from the interaction between the soil strike-through within the apertures and the transverse ribs will play a significant role in the reinforcing effect, which is not considered by the proposed method.

First of all, the design charts (i.e., Fig. 16) are proposed based on the present method. The design charts give the maximum geosynthetic strain and maximum surface settlement for a given geometry. During a design process, the maximum geosynthetic strain must be less than the corresponding allowable tensile strain (ε_a), and the maximum surface settlement must meet the maximum surface settlement criterion (less than s_a). Only in this way can we obtain the optimal design.

Then, the design method for geosynthetic reinforcement subjected to localized sinkholes is introduced on the basis of design charts. The design consists of determining the stiffness of geosynthetic over cavity for a width-height ratio and a given maximum surface settlement criterion. For a given stiffness of geosynthetic, ε_a is also known. In general, the anchorage length of geosynthetic is also seen as a key design point and can be determined reasonably by the same way as Fig. 11. To control the computation, a fixed length of geosynthetic is adopted in this section, e.g. $L = 2.5$ m the same as that in the previous analysis. Now, we know all the parameters from Table 1 except the stiffness of geosynthetic. On condition that the interface parameters between soil and geosynthetic are known, we can select a geosynthetic reinforcement

based on a determined J_S (short-term stiffness for geosynthetic). The long-term performance of geosynthetic is considered by adopting the long-term stiffness for geosynthetic (J_L), which can be determined as $J_L = J_S / (R_C \times R_{ID} \times R_{CBD})$ by taking into account the reduction factors for creep R_C , installation damage R_{ID} and chemical and biological degradation R_{CBD} . The specific design can be divided into three steps: (i) find a value of J_L for a given B/H according to Fig. 16b to comply with the maximum surface settlement criterion (i.e., s_a); (ii) determine $\varepsilon_{\max}$ of deflected geosynthetic by placing the pair of values (i.e., a given B/H and determined J_L in the first step) into Fig. 16a and check if it is less than ε_a ; (iii) repeating the first two steps and find a J_L until $\varepsilon_{\max} < \varepsilon_a$.

Finally, an example application is done based on the proposed design process. The value of s_a is given as 10 cm. For the case of $B/H = 2.0$, we can determine $J_L = 1340 \text{ kN/m}^3$ in Fig. 16b based on the prerequisite of $s_{\max} = s_a = 10 \text{ cm}$. The value of ε_a is 2.5% for $J_L = 1340 \text{ kN/m}^3$. Then, we can obtain $\varepsilon_{\max} = 2.46\%$ by using Fig. 16a. We find $\varepsilon_{\max} < \varepsilon_a$ and thus the selected geosynthetic can be served as an applicable reinforcement for the case.

Conclusions

In this study, an analytical model is proposed to estimate membrane effect of geosynthetic reinforcement subjected to localized sinkholes. In subsided area, the upper interface friction is considered and the relative slippage between overlying soil and deflected geosynthetic is also analyzed based on Coulomb friction law. In anchorage area, the vertical deformation of supporting soil is considered. The strain of geosynthetic and ground surface settlement under different load types, different anchorage lengths or width-height ratios, different upper interface friction angles or

moduli of subgrade reaction, and different subsidence widths are investigated. The major conclusions are as follows:

(1) Overlying vertical load on geosynthetic and its distribution type with consideration of arching effect significantly influence membrane effect of deflected geosynthetic. In cases when the geosynthetic in anchorage area is well anchored, the larger vertical load applied near the middle of subsided area leads to greater membrane effect.

(2) Anchoring effect first develops near junction point A (see Fig. 2) and then spreads toward the marginal edge of anchorage area during a subsiding process. Increasing the anchorage length can promote the anchoring ability before the anchoring effect is fully developed.

(3) Ignoring upper interface friction results in undervaluation of maximum geosynthetic strain but overvaluation of maximum surface settlement. Anchoring effect is highly influenced by the vertical deformation of supporting soil. Without considering this, the anchoring effect is overestimated, resulting in undervaluation of maximum surface settlement.

(4) Design charts for geosynthetic reinforcement under local subsidence are given based on the present method. A practical design method is proposed and illustrated by an example case. The design framework is an applicable tool for preliminary design of geosynthetic-reinforced structures.

List of symbols

A_e	anchorage coefficient
B	subsidence width (m)
B_C	diameter of a circular void (m)

B_L	width of a long void (m)
C_e	expansion coefficient of overlying soil
E_s	Young's modulus (MPa)
$f_{\max}$	maximum vertical displacement of deflected geosynthetic (m)
H	height of overlying soil (m)
J	tensile stiffness of geosynthetic (kN/m)
k_s	modulus of subgrade reaction (MPa/m)
K	lateral pressure coefficient of overlying soil
L	anchorage length (m)
p_2, q_2	vertical stresses on lower and upper interfaces between soil and geosynthetic for anchorage area (kPa)
q_0	surcharge on ground surface (kPa)
q_1	vertical stress on upper interface for subsided area (kPa)
q_{1C}, q_{1L}	vertical stresses acting on deflected geosynthetic over a circular void and long void (kPa)
q_{1C1}, q_{1C2}	maximum values of non-uniform and uniform parts of q_{1C} (kPa)
r	ratio q_{1C1}/q_{1C2}
s	length of deflected geosynthetic (m)
s_L, s_R	lengths of deflected geosynthetic for subsided and anchorage areas (m)
$s_{\max}$	maximum surface settlement (m)
T_L	tension in geosynthetic for subsided area (kN)
T_{Lh}, T_{Lv}	horizontal and vertical components of T_L (kN)
T_{Rh}, T_{Rv}	horizontal and vertical components of tension in geosynthetic for anchorage area (T_R) (kN)
T_{AL}, T_{AR}	left and right tensile forces of point A (kN)

u, U	relative displacements between soil and geosynthetic for subsided and anchorage areas (m)
u_0	critical relative displacement (m)
u_{AL}, U_{AR}	left and right displacements of point A (m)
x, X	horizontal distances to origin in x - y and X - W coordinate systems (m)
x_C, X_C	horizontal distances from point with u_0 to origin for subsided and anchorage areas (m)
y, W	vertical distances to origin in x - y and X - W coordinate systems (m)
β_L, β_R	angles between tangential direction of ds and horizontal direction for subsided and anchorage areas ($^\circ$)
β_{AL}, β_{AR}	left and right intersection angles of point A ($^\circ$)
ϕ, ϕ_u	lower and upper interface friction angles ($^\circ$)
τ_{1u}	frictional stress on upper interface for subsided area (kPa)
τ_{2l}, τ_{2u}	frictional stresses on lower and upper interfaces for anchorage area (kPa)
$\tau_{\max}$	maximum frictional stress (kPa)

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References

- Abusharar, S.W., Zheng, J.J., Chen, B.G., and Yin, J.H. 2009. A simplified method for analysis of a piled embankment reinforced with geosynthetics. *Geotextiles and Geomembranes*, 27(1): 39-52. doi:10.1016/j.geotexmem.2008.05.002.
- Briançon, L., and Villard, P. 2008. Design of geosynthetic-reinforced platforms spanning localized sinkholes. *Geotextiles and Geomembranes*, 26(5): 416-428. doi:10.1016/j.geotexmem.2007.12.005.
- Daloglu, A.T., and Vallabhan, C.G. 2000. Values of k for Slab on Winkler Foundation. *Journal of Geotechnical and Geoenvironmental Engineering*, 126(5): 463-471. doi:10.1061/(ASCE)1090-0241(2000)126:5(463).
- Espinoza, R.D. 1994. Soil-geotextile interaction: evaluation of membrane support. *Geotextiles and Geomembranes*, 13(5): 281-293. doi:0.1016/0266-1144(94)90024-8.
- Feng, S.J., Ai, S.G., and Chen, H.X. 2017a. Estimation of arching effect in geosynthetic-reinforced structures. *Computers and Geotechnics*, 87: 188-197. doi:10.1016/j.compgeo.2017.02.014.
- Feng, S.J., Ai, S.G., Chen, H.X., and Xie, H.J. 2017b. An analytical method for predicting load acting on geosynthetic overlying voids. *Geotextiles and Geomembranes*, 45(6): 570-579. doi:10.1016/j.geotexmem.2017.07.007.
- Feng, S.J., and Lu, S.F. 2015. Deformation analysis of a geosynthetic material subjected to two adjacent voids. *Geotextiles and Geomembranes*, 43(4): 317-331. doi:10.1016/j.geotexmem.2015.04.007.
- Gilbert, R.B., and Byrne, R.J. 1996. Strain-softening behavior of waste containment system interfaces. *Geosynthetics International*, 3(2): 181-203. doi:10.1680/gein.3.0059.

- Giroud, J.P. 1981. Designing with geotextiles. *Matériaux Et Construction*, 14(4): 257-272.
- Giroud, J.P. 1995. Determination of geosynthetic strain due to deflection. *Geosynthetics International*, 2(3): 635-641. doi:10.1680/gein.2.0028.
- Giroud, J.P., Bonaparte, R., Beech, J.F., and Gross, B.A. 1990. Design of soil layer-geosynthetic systems overlying voids. *Geotextiles and Geomembranes*, 9(1): 11-50. doi:0.1016/0266-1144(90)90004-V.
- Gourc, J.P., and Villard, P. 2000. Reinforcement by membrane effect: Application to embankments on soil liable to subsidence. *In Proceedings of the 2nd Asian Geosynthetics Conference, ASIA 2000, Kuala Lumpur, Malaysia, 29-31 May 2000, Vol. 1, pp. 55-72.*
- Han, J., and Gabr, M.A. 2002. Numerical analysis of geosynthetic-reinforced and pile-supported earth platforms over soft soil. *Journal of Geotechnical and Geoenvironmental Engineering*, 128(1): 44-53. doi:10.1061/(ASCE)1090-0241(2002)128:1(44).
- Huckert, A., Briançon, L., Villard, P., and Garcin, P. 2016. Load transfer mechanisms in geotextile-reinforced embankments overlying voids: experimental and analytical approaches. *Geotextiles and Geomembranes*, 44(3): 442-456. doi:10.1016/j.geotexmem.2015.06.005.
- King, D.J., Bouazza, A., Gniel, J.R., Rowe, R.K., and Bui, H.H. 2017. Load-transfer platform behaviour in embankments supported on semi-rigid columns: implications of the ground reaction curve. *Canadian Geotechnical Journal*, 54(8): 1158-1175. doi:10.1139/cgj-2016-0406.
- Kinney, T.C., and Connor, B. 1987. Geosynthetics supporting embankments over voids. *Journal of cold regions engineering*, 1(4): 158-170.

doi:0.1061/(ASCE)0887-381X(1987)1:4(158).

- Kuo, S.S., Desai, K., and Rivera, L. 2005. Design method for municipal solid waste landfill liner system subjected to sinkhole cavity under landfill site. *Practice Periodical of Hazardous, Toxic, and Radioactive Waste Management*, 9(4): 281-291.
- Lu, W., and Miao, L. 2015. A simplified 2-D evaluation method of the arching effect for geosynthetic-reinforced and pile-supported embankments. *Computers and Geotechnics*, 65(65): 97-103. doi:10.1016/j.compgeo.2014.11.014.
- Patra, S., and Shahu, J.T. 2012. Pasternak model for oblique pullout of inextensible reinforcement. *Journal of Geotechnical and Geoenvironmental Engineering*, 138(12): 1503-1513. doi:10.1061/(ASCE)GT.1943-5606.0000720.
- Patra, S., and Shahu, J.T. 2015. Behaviour of extensible reinforcement resting on non-linear Pasternak subgrade subjected to oblique pull. *Géotechnique*, 65(9): 770-779. doi:10.1680/geot.14.P.233.
- Rogbeck, Y., Alén, C., Franzén, G., Kjeld, A., Odén, K., Rathmayer, H., Watn, A., and Øiseth, E. 2003. Nordic guidelines for reinforced soils and fills. Nordic Geosynthetic Group of the Nordic Geotechnical Societies, Nordic Industrial Fund.
- Shukla, S., and Sivakugan, N. 2009. A general expression for geosynthetic strain due to deflection. *Geosynthetics International*, 16(5): 402-407. doi:10.1680/gein.2009.16.5.402.
- Terzaghi, K. 1943. *Theoretical Soil Mechanics*, John Wiley & Sons, New York, NY.
- Villard, P.V., and Briançon, L.B. 2008. Design of geosynthetic reinforcements for platforms subjected to localized sinkholes. *Canadian Geotechnical Journal*, 45(2): 196-209. doi:10.1139/T07-083.

- Villard, P., Gourc, J.P., and Giraud, H. 2000. A geosynthetic reinforcement solution to prevent the formation of localized sinkholes. *Canadian Geotechnical Journal*, 37(5): 987-999. doi:10.1139/t00-002.
- Villard, P., Huckert, A., and Briançon, L. 2016. Load transfer mechanisms in geotextile-reinforced embankments overlying voids: Numerical approach and design. *Geotextiles and Geomembranes*, 44(3): 381-395. doi:10.1016/j.geotexmem.2016.01.007.
- Wang, M.C., Feng, Y.X., and Jao, M. 1996. Stability of geosynthetic-reinforced soil above a cavity. *Geotextiles and Geomembranes*, 14(2): 95-109. doi:10.1016/0266-1144(96)84939-9.
- Yu, Y., and Bathurst, R.J. 2017. Influence of selection of soil and interface properties on numerical results of two soil–geosynthetic interaction problems. *International Journal of Geomechanics*, 17(6). doi:10.1061/(ASCE)GM.1943-5622.0000847, 04016136.

List of Table Captions

Table 1. Geometry and material properties of the soil and geosynthetic.

Table 2. Parameters taken from experimental studies conducted by Huckert et al. (2016).

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Table 1. Geometry and material properties of the soil and geosynthetic.

Property	Value
Geometry	
Cavity width, B_L (m)	2
Height of overlying soil, H (m)	0.5
Anchorage length, L (m)	4
Overlying soil	
Unit weight, γ_s (kN/m ³)	17
Internal friction angle, ϕ_s (°)	30
Cohesion, c_s (kPa)	0
Geosynthetic	
Tensile stiffness, J (kN/m)	1100
Critical relative displacement between soil and geosynthetic, u_0 (m)	0.005
Upper interface friction angle between soil and geosynthetic, ϕ_u (°)	30
Lower interface friction angle between soil and geosynthetic, ϕ_l (°)	25
Supporting soil	
Young's modulus, E_s (MPa)	30
Poisson's ratio, ν	0.3
Surcharge applied on the ground surface, q_0 (kPa)	0

Table 2. Parameters taken from experimental studies conducted by Huckert et al. (2016).

Parameter	Value
Geometry	
Cavity diameter, B_C (m)	2.2
Height of overlying soil, H (m)	1
Anchorage length, L (m)	3
Overlying soil	
Unit weight, γ_s (kN/m ³)	15.65
Internal friction angle, ϕ_s (°)	36
Cohesion, c_s (kPa)	0
Geosynthetic	
Tensile stiffness, J (kN/m)	2988
Critical relative displacement between soil and geosynthetic, u_0 (m)	0.005
Upper interface friction angle between soil and geosynthetic, ϕ_u (°)	23
Lower interface friction angle between soil and geosynthetic, ϕ_l (°)	40
Supporting soil	
Young's modulus, E_s (MPa)	19
Poisson's ratio, ν	0.3
Surcharge applied on the ground surface, q_0 (kPa)	0

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- Fig. 16.** Design chart of stiffness of geosynthetic based on its maximum strain and maximum surface settlement.

Fig. 1. Schematic diagram of geosynthetic-reinforced structure.

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