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A simplified model for geosynthetic-reinforced pile-supported embankment with cohesive fills

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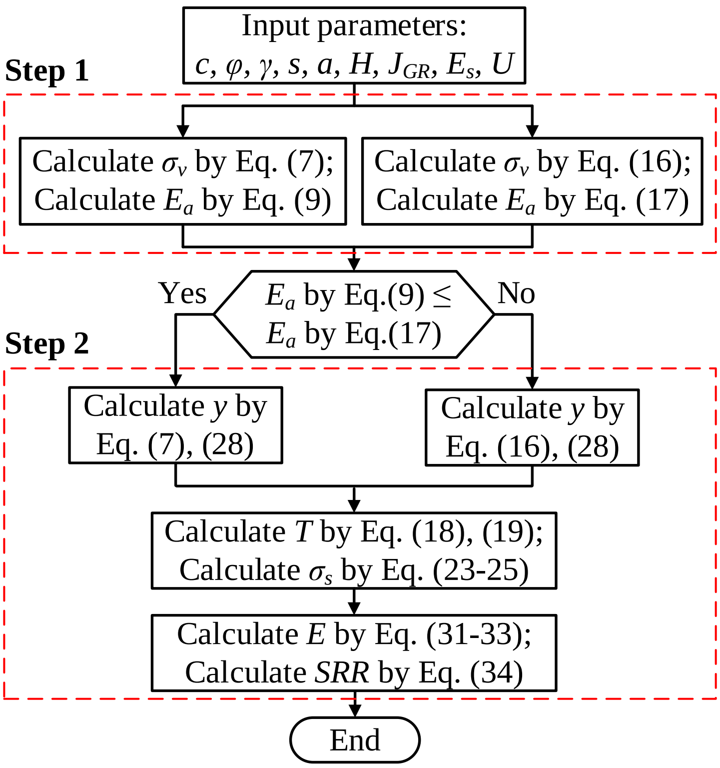

Fig. S1. Design flowchart for the GRPS embankment using the proposed model (following the tow-step approach).

Section S1 Full-scale field test in Shanghai (Liu et al., 2007)

The monitoring results of a GRPS highway embankment located in a northern suburb of Shanghai, China, were presented by Liu et al. (2007). The improvement area ratio (IAR), representing the percentage coverage of the pile caps over the total foundation area (Chen et al., 2021), is relatively low and equal to 8.7%. Beneath the embankment, there is a 2.3 m thick deposit of silty clay overlaid by a 1.5 m thick coarse-grained fill. This deposit overly a 10.2 m thick of soft silty clay. Below the soft silty clay layer, there is a 2 m thick layer of medium silty clay, followed by a layer of sandy silt. Cast-in-place annulus concrete piles support the embankment, arranged in a square pattern. The piles, each 16 m in length, are supported on the relatively stiffer sandy silt layer. The pile cap is circular in shape, with a diameter of 1.01 m (equivalent side width of 0.9 m can be converted for square caps). The embankment fill material primarily consisted of pulverized fuel ashes. The embankment is 5.6 m in height, 120 m in length, and has a crown width of 35 m. The side slope is 1:1.5 (height : width). The field monitoring program was concluded in 180 days after the embankment construction start (or 125 days after the embankment completion). A summary of the input parameters for the analytical models is listed in Table S1.

Table S1. Input parameters for analytical models from full-scale field test of Liu et al. (2007).

Embankment fills	h = 5.6 m, γ = 18.5 kN/m^3, c = 10 kPa, φ = 30°, q = 0 kPa
Pile (arranged in square pattern)	d = 1.01 m, a = 0.9 m, s = 3 m (circular pile cap)
Geosynthetic reinforcement	J_{GR} = 1125 kN/m
Soft subsoil	k_s = 550 kPa/m, U = 0.7

Note: h is embankment height; γ, c, φ are the average unit weight, internal friction angle and cohesion of embankment fills, respectively; q is surcharge load; d is diameter of circular pile caps; a is width of square pile caps ($a = d\sqrt{\pi}/2$); s is center-to-center pile spacing; J_{GR} is geosynthetic tensile stiffness; k_s is subsoil reaction modulus by Eq. (24); U is consolidation degree of subsoil.

Section S2 Full-scale field test in Zhejiang (Liu et al., 2015)

Liu et al. (2015) reported a case study on a GRPS embankment for a highway construction project over soft ground in Ningbo, China. Below the embankment, four type of soil layers were identified: a 1.5 m thick of silty clay, a 6.9 m thick of mud clay, a 19.1 m thick of soft clay and a 2.4 m thick of silty clay from top to bottom. The bearing capacity of the foundation soil before ground treatment was assessed to be equal to 83 kPa through cone penetration testing (CPT). Grouted gravel columns were installed for the piled system. The columns are 28 m long with tips extending to the stiff silty clay layer. The columns are positioned in a square grid with a center-to-center spacing of 2.4 m. On the top of each column, a square reinforced concrete column cap is cast, measuring 1.0 × 1.0 × 0.3 m (length × width × height). Cohesive soil mixed with about 40% fly ash was used as the fill material for the embankment. The embankment has a height of 4.6 m and a crown width of 42.5 m. The side slope is 1:1.5 (height : width). The IAR is calculated as 17.2%. The embankment construction was completed within 150 days, while the monitoring lasted 360 days after the embankment construction started. The measured data includes the soil pressures, the pore-water pressures and the differential settlements. The consolidation degree is determined based on the measured settlements and consolidation time. Details of input parameters for the analytical models can be found in Table S2.

Table S2. Input parameters for analytical models from full-scale field test of Liu et al. (2015).

Embankment fills	h = 4.6 m, γ = 19 kN/m^3, c = 11 kPa, φ = 30°, q = 0 kPa
Pile (arranged in square pattern)	a = 1 m, s = 2.4 m (square pile cap)
Geosynthetic reinforcement	J_{GR} = 2250 kN/m
Soft subsoil	k_s = 500 kPa/m, U = 0.8

Note: The notations are consistent with those in Table S1.

Section S3 Full-scale field test in Zhejiang (Chen at al., 2021)

Chen at al. (2021) described a series of field tests for a GRPS embankment on a soft marine deposit, located in the coast of the East China Sea. The performance of the GRPS embankment was investigated under three testing procedures: construction, surcharge loading and consolidation. The soil stresses, cumulative settlements and excess pore pressures were monitored using pre-embedded soil pressure sensors, settlement plates and pore pressure sensors, respectively. The field monitoring program was concluded 99 days after the consolidation procedure starts. The subsoil beneath the embankment primarily consisted of lean clay (CL). Pre-stressed high-strength concrete tube piles were utilized for the piled system. All piles were arranged in a square pattern with a center-to-center distance of 3 m. A cast-on-site concrete pile cap measuring 1.8 × 1.8 × 0.5m (length × width × height) was constructed on the top of each pile. The bottom of the testing embankment is 31.8m long and 31.8m wide. The embankment fills mainly consisted of poorly graded gravel with clay, with a height of 3m and a side slope gradient of 1:2 (height : width). The IAR is calculated as 36%. In this project, the consolidation procedure was followed after an unloading procedure, in which the 50 kPa surcharge released back to 0 kPa in 1 day. The input parameters employed for the analytical models are detailed in Table S3.

Table S3. Input parameters for analytical models from full-scale field test of Chen et al. (2021).

Embankment fills	h = 3 m, γ = 22 kN/m^3, c = 5 kPa, φ = 39°, q = 0 kPa
Pile (arranged in square pattern)	a = 1.8 m, s = 3 m (square pile cap)
Geosynthetic reinforcement	J_{GR} = 3280 kN/m
Soft subsoil	k_s = 459 kPa/m, U = 0.7

Note: The notations are consistent with those in Table S1.

Section S4 Full-scale model test by Wang et al. (2019)

Wang et al. (2019) constructed a full-scale model of a GRPS railway track-bed, utilizing a concrete slab and a water bag to simulate the pile caps and the surrounding subsoil, respectively. This model was constructed within a steel-constituted chamber measuring 15 m in length, 5.5 m in width, and 4 m in height, adhering to the Chinese standard TB10621-2009 (Ministry of Railways, 2009). The behaviour of the GRPS railway track-bed was investigated under three testing procedures: model construction, static loading and subsoil settlement (discharging of water bags). The pile cap was square in shape with a width of 1 m. The pile caps were set up in a square layout, with a center-to-center distance of 1.8 m. The embankment was constituted by well-graded gravels in the surface layer and clayey gravels in the bottom layer, with a thickness of 0.4 m and 2.3 m, respectively. The side slope is 1:1.5 (height : width). There was a 7.42 kPa surcharge load on the embankment surface due to the self-weight of the superstructure (8 pairs of rails and fasteners, the track slab and the concrete base). The test was concluded when the soil stresses on the pile cap top were stabilized. The input parameters for the analytical models are outlined in Table S4.

Table S4. Input parameters for analytical models from full-scale model test of Wang et al. (2019).

Embankment fills	h = 2.7 m, γ = 22.7 kN/m^3, c = 16.3 kPa, φ = 43.6°, q = 7.42 kPa
Pile (arranged in square pattern)	a = 1 m, s = 1.8 m (square pile cap)
Geosynthetic reinforcement	J_{GR} = 2459.5 kN/m
Soft subsoil	k_s = 520 kPa/m, U = 1.0

Note: The notations are consistent with those in Table S1.

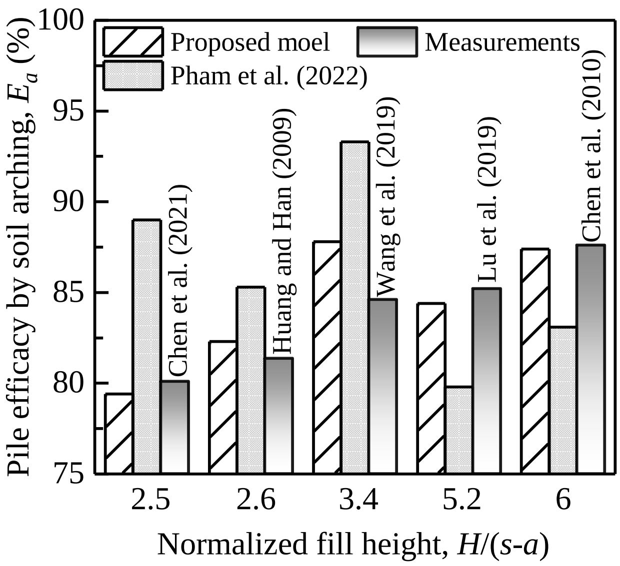

Fig. S2. Comparison of predicted pile efficacy by soil arching (E_a) from the proposed model and the model of Pham et al. (2022) with measurements under relatively high fill conditions.

Parametric study

Table S5. Basic parameters used in parametric study.

Parameters	Values
Normalized fill height, $H/(s\text{-}a)$	1, 2, 3, 4, 5, 6, 7
Improvement area ratio, IAR (%)	5, 10, 15, 20, 25, 30, 35, 40
Internal friction angle of fills, φ (°)	5, 10, 15, 20, 25, 30, 35, 40, 45, 50
Cohesion of fills, c (kPa)	0, 10, 20
Geosynthetic tensile stiffness, J_{GR} (kN/m)	200, 500, 800, 1500, 3000, 5000
Subsoil reaction modulus, k_s (kPa/m)	100, 250, 500, 1000, 1500, 2000, 3000

Section S5

Nordic design guideline (NGG, 2004)

The wedge-shaped model for the soil arching calculation, introduced by Carlsson (1987), is adopted in the Nordic guideline (NGG, 2004). According to which, the distributed load acting across the geosynthetic reinforcement in two dimensions W_{2Dd} is determined as:

$$W_{2Dd} = \frac{(s-a)^2}{4\tan 15^\circ}\gamma = 0.93(s-a)^2\gamma \tag{S1}$$

And the distributed load in three dimensions is estimated by:

$$W_{3Dd} = \frac{1+s/a}{2}W_{2Dd} \tag{S2}$$

The vertical stress distributed on geosynthetic by soil arching effect σ_v, is calculated as:

$$\sigma_v = W_{3Dd}/s \tag{S3}$$

Then, the calculation of the tension in geosynthetic T resulting from the vertical load in three dimensions is performed using:

$$T = \frac{W_{3Dd}}{2}\sqrt{1+\frac{1}{6\varepsilon}} \tag{S4}$$

Combining with the equilibrium condition ($T = J_{GR}\varepsilon$), the solution of ε can be determined by:

$$J_{GR}{}^2\varepsilon^3 - (\frac{W_{3Dd}}{2})^2\varepsilon - \frac{(W_{3Dd})^2}{24} = 0 \tag{S5}$$

Finally, the maximum deflection y of the geosynthetic is approximated by:

$$y = (s-a)\sqrt{\frac{3\varepsilon}{8}} \tag{S6}$$

British standard BS8006 (BS8006-1, 2010)

Due to the simplicity, the British standard BS8006 has been one of the most widely employed method for GRPS embankment design. The updated version of BS8006-1 (2010) introduces two different methods to calculate the distributed loads acting on the reinforcement, considering the soil arching effect in embankment. The one method uses the model proposed by Marston and Anderson (1913), which originally derived according to the equal settlement surface above the underground pipelines. The Marston's model is therefore following the plane strain condition. To be an alternative solution, the soil arching model proposed by Hewlett and Randolph (1988) is used in another method. It relies on the observed mechanisms from model tests, considering a series of hemispherical domes. In this study, the model of Hewlett and Randolph (1988) is adopted for the BS8006-1 (2010).

Accordingly, the soil arching failure is presumed to occur at either the arch crown or the pile cap. Considering the equilibrium equation of the crown at ultimate state, the pile efficacy E_{a1} is determined by:

$$E_{a1} = 1 - [1 - (a/s)^2](X_1 - X_1 X_2 + X_3) \tag{S7}$$

where X_1, X_2 and X_3 are calculation coefficients given by:

$$X_1 = [1 - (a/s)]^{2(K_p - 1)};\ X_2 = \frac{s}{\sqrt{2}H}\left(\frac{2K_p - 2}{2K_p - 3}\right);\ X_3 = \frac{s - a}{\sqrt{2}H}\left(\frac{2K_p - 2}{2K_p - 3}\right) \tag{S8}$$

For the pile cap (the foot of soil arching) at ultimate state, the pile efficacy E_{a2} is calculated by:

$$E_{a2} = \frac{\beta}{1 + \beta} \tag{S9}$$

where the β is a parameter determined as:

$$\beta = \frac{2K_p}{(K_p + 1)(1 + a/s)}\left[\left(1 - \frac{a}{s}\right)^{-K_p} - \left(1 + K_p \frac{a}{s}\right)\right] \tag{S10}$$

The final efficacy E_a contributed by soil arching should be the lower one between E_{a1} and E_{a2}. Then, the maximum distributed load W_T carried by the geosynthetic reinforcement between adjacent pile caps is defined as:

$$W_T = \frac{s^3(\gamma H + q)}{s^2 - a^2}(1 - E_a) \tag{S11}$$

The vertical load acting on geosynthetic P_s^a, is therefore determined as:

$$P_s^a = W_T \bullet (s - a) = \frac{s^3(\gamma H + q)}{s + a}(1 - E_a) \tag{S12}$$

The tension in geosynthetic T is then calculated by:

$$T = \frac{P_s^a}{2a}\sqrt{1 + \frac{1}{6\varepsilon}} \tag{S13}$$

where ε is the average geosynthetic strain. A design strain of 5% is recommended by the BS8006-1 (2010). However, the 5% strain value may not be representative of all cases in reality. Therefore, the equilibrium condition ($T = J_{GR}\varepsilon$) is substituted for the estimation of ε for the comparing. Accordingly, the ε can be determined by:

$$J_{GR}{}^2\varepsilon^3 - \left(\frac{\sigma_v}{2a}\right)^2\varepsilon - \frac{1}{24}\left(\frac{\sigma_v}{a}\right)^2 = 0 \tag{S14}$$

Finally, the maximum deflection y is approximated as:

$$y = (s - a)\sqrt{\frac{3\varepsilon}{8}} \tag{S15}$$

All the input parameters for the calculation using BS8006-1 have the same meaning as that in the model proposed by this study. It should be noted that the supporting from subsoil is not taken into account in the BS8006-1. That is the entire load from the overlying embankment is assumed to be distributed to the piles by the soil arching effect and the tensioned membrane effects. The total efficacy E (sum of E_a and E_m) is therefore equal to 1.

German standard EBGEO (DGGT, 2011)

The nonconcentric hemispherical model proposed by Zaeske (2001) is utilized in the German Standard EBGEO (DGGT, 2011). In this analytical model, the pile efficacy E_a by the soil arching effect is defined as:

$$E_a = \frac{[(\gamma H + q - \sigma_v)s^2 / a^2 + \sigma_v]a^2}{(\gamma H + q)s^2} \tag{S16}$$

where σ_v is the vertical stress distributed on the geosynthetic reinforcement between pile caps by soil arching, determined as:

$$\sigma_v = \lambda_1^{\alpha}(\gamma + \frac{q}{H})\{H(\lambda_1 + h_e^{\,2}\lambda_2)^{-\alpha} + h_e[(\lambda_1 + \frac{h_e^{\,2}\lambda_2}{4})^{-\alpha} - (\lambda_1 + h_e^{\,2}\lambda_2)^{-\alpha}]\} \tag{S17}$$

In this equation, h_e is the height of soil arching, recommended by $h_e = s_d / 2$ for $H \geq s_d / 2$ and $h_e = H$ for $H < s_d / 2$, respectively; s_d is the diagonal center spacing between neighbouring pile caps; λ_1, λ_2 and α are parameters, determined as:

$$\lambda_1 = \frac{1}{8}(s_d - a)^2 \,;\; \lambda_2 = \frac{s_d^{\,2} + 2as_d - a^2}{2s_d^{\,2}} \,;\; \alpha = \frac{a(K_p - 1)}{\lambda_2 s_d} \tag{S18}$$

The tensile force in the geosynthetic reinforcement is determined using the membrane theory. The average geosynthetic strain ε can be estimated using the design charts provided in the EBGEO [graph 9.16 in EBGEO (2011)], in which two input parameters F_1 and F_2 are required and can be calculated respectively as:

$$F_1 = \frac{\sigma_v(s^2 - a^2)}{2J_{GR}a} \,;\; F_2 = \frac{k_s(s-a)^2}{J_{GR}} \tag{S19}$$

The value of the tensile force in the geosynthetic reinforcement T is then calculated by:

$$T = J_{GR}\varepsilon \tag{S20}$$

Finally, the maximum deflection of geosynthetic can be estimated as follows:

$$y = (s-a)\sqrt{\frac{3\varepsilon}{8}} \tag{S21}$$

Dutch Design Guideline (CUR226, 2016)

The concentric arches model developed by van Eekelen et al. (2013) is adopted in the Dutch Design Guideline (CUR226, 2016) to estimate the soil arching effect in GRPS embankment. In the CUR226 (2016), the residual load part supported by the geosynthetic reinforcement and subsoil P_s^a (kN) is determined as:

$$P_s^a = F_{GRsquare} + F_{GRstrips} \tag{S22}$$

Therefore, the load part directly transferred to the pile cap by the soil arching effect P_c^a (kN) is represented as:

$$P_c^a = (\gamma H + q)s^2 - F_{GRsquare} - F_{GRstrips} \tag{S23}$$

where $F_{GRsquare}$ is the total vertical load exerted by the 3D hemisphere on the geosynthetic reinforcement square area (kN). $F_{GRstrips}$ is the total vertical load exerted by the 2D arches on the geosynthetic reinforcement strips (kN). Due to space limitations, the derivation and equations of $F_{GRsquare}$ and $F_{GRstrips}$ can be referred to CUR226 (2016) or van Eekelen et al. (2013).

In CUR226 (2016), a uniform load distribution on the geosynthetic reinforcement is used when the subsoil support is considered. For the geosynthetic reinforcement design, the geosynthetic tension $T(x)$ is calculated as follows:

$$T(x) = T_H \sqrt{1 + y'(x)^2} \tag{S24}$$

where $y(x)$ represents the geosynthetic deflection curve and $y'(x)$ represents the derivative of deflection. T_H represents the horizontal component of the tensile force, which can be determined using:

$$\int_0^{(s-a)/2} \sqrt{1 + y'(x)^2}\, dx - (s-a)/2 = \frac{1}{J_{GR}} \int_0^{(s-a)/2} T(x) dx \tag{S25}$$

Note that the maximum tension in the geosynthetic reinforcement T occurs at the pile cap edge ($x = \frac{s-a}{2}$). And the maximum deflection of geosynthetic reinforcement y is estimated by:

$$y = (s-a)\sqrt{\frac{3T}{8J_{GR}}} \tag{S26}$$