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Recent Development of Geosynthetic-Reinforced Column-Supported Embankments

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ABSTRACT: Geosynthetic-reinforced column-supported embankments (GRCSE) have emerged as an effective alternative to conventional geotechnical solutions when constructing on soft soils. This paper provides the state of the art review of recent developments in the use and design of GRCSE. The review includes load transfer mechanisms, failure modes, design considerations, numerical analyses, and applications of GRCSE systems. The review concludes that GRCSE systems are most suitable for situations with a very soft soil underlain by a stiff soil layer or bedrock, when new fill with a certain minimum thickness is needed, rapid construction is necessary, and strict total/differential settlements are required. The common applications of these systems include bridge approach, roadway widening, and railroads or highways across soft soil. Soil arching, tensioned membrane or stiffened platform (or beam) effects, and relative stiffness effects between columns and soil are identified as the load transfer mechanisms above the pile caps or columns. Three common soil arching models are used for estimating the soil arching ratio and the average vertical stress above the geosynthetic reinforcement by assuming vertical slip surfaces, a rigid conical (axisymmetric) or triangular prism, and semi-spheres. A few tensioned membrane theories are available for estimating the strain and the tension developed in the geosynthetic reinforcement. Due to the complexity of the systems, numerical methods are considered as an effective way for analyzing these systems.

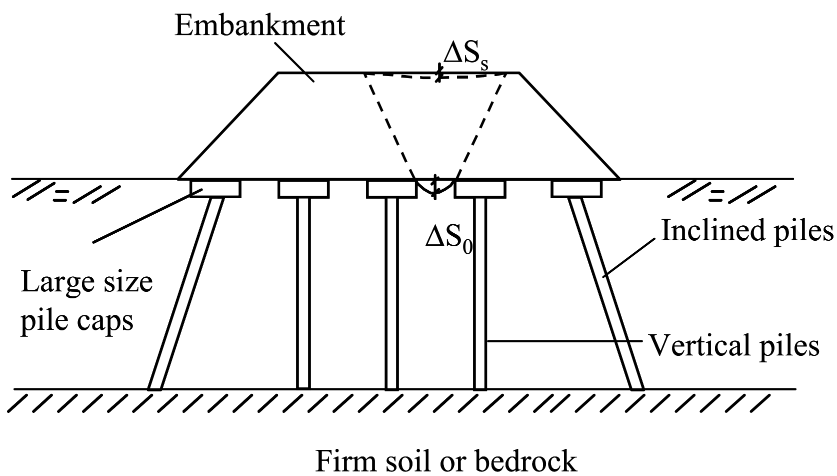
INTRODUCTION

When designing embankments over soft soils, geotechnical engineers must address design concerns related to potential bearing capacity failures, intolerable total and differential settlements, large lateral pressures and movement, and slope instability. A variety of techniques are available to geotechnical engineers to address the above concerns (Magnan, 1994). These techniques include pre-loading or staged construction, using light-weight fill, over-excavation and replacement, geosynthetic reinforcement, soil improvement techniques, and pile-supported embankments. The advantages and disadvantages of these techniques are discussed in the paper by Magnan (1994).

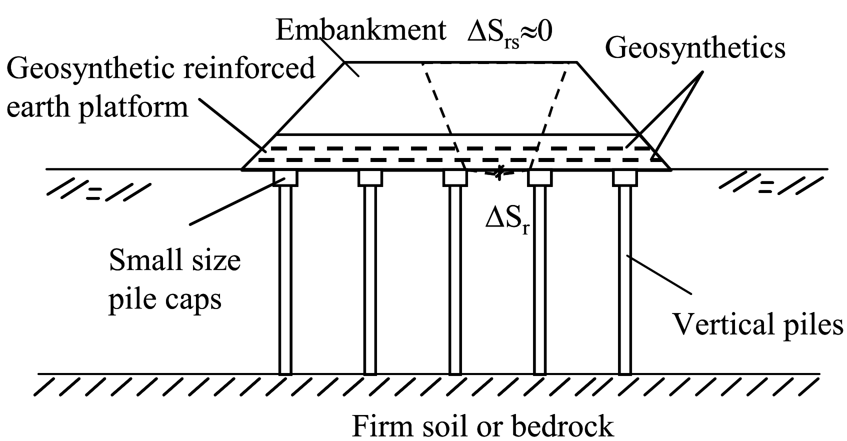
Among these techniques, geosynthetics (such as geogrids and geotextiles, made of polymer materials) as reinforcement have been adopted to reinforce soft foundations, slopes, and embankments. Geosynthetics have a high tensile strength that soils do not have. The function of the geosynthetics for embankments over soft soils is to increase bearing capacity, reduce differential settlement, and prevent slope instability. Many studies have shown that geosynthetics can be used for reducing differential settlements, however, they have limited contributions in reducing total settlements of embankments over soft soils.

In the column-supported embankment system, the columns carry most of the loads from the embankment and the soil is only subjected to small loads. The benefits associated with the use of column-supported embankments are as follows: (1) allows construction of the embankment in a single stage without prolonged waiting time, (2) significantly reduces total and differential settlements, (3) reduces or eliminates global stability concerns.

Column-supported embankment systems have been used with or without geosynthetic reinforcement. A system without geosynthetic reinforcement is referred to herein as the conventional column-supported embankment (CCSE) system while the system with geosynthetic reinforcement is referred to as the geosynthetic-reinforced column-supported embankment (GRCSE) system (Figure 1). For a CCSE, inclined columns are commonly used near side slopes to carry the lateral thrust from the embankment. In addition, columns need to be closely spaced and/or have large column caps in order to transfer surcharge loads through soil arching to the columns and minimize deflection of the soil between column caps and the deflection being reflected to the embankment surface. Shen and Miura (2001) proposed using varying lengths of piles to solve differential settlement of roads on soft soil in Japan. In the GRCSE system, the geosynsthetic reinforcement carries the lateral thrust from the embankment, creates a stiffened fill platform to enhance the load transfer from the soil to the columns, and reduce the differential settlement between pile caps. One single high strength geosynthetic layer may be placed over the column caps or columns acting as a tensioned membrane or multiple layers of geosynethtics with adequate strengths may be placed within granular fill to form a load transfer platform. As a result, the GRCSE system does not require inclined columns, large column caps, and close column spacing. Therefore, the GRCSE system creates a more cost-effective solution.



(a) Conventional column supported embankments



(b) Geosynthetic reinforced column supported embankments

Figure 1. Column-Supported Embankments

In addition to concrete piles and timber piles, vibro-concrete columns (VCC), deep mixed columns, rammed aggregate piers, or stone columns as columnar systems in ground improvement have been used for embankment support as well. The GRCSE systems have been used for a number of applications worldwide, which include: bridge approaching embankments; low height embankments; roadway widening; retaining wall foundation support; storage tanks foundation support; and building foundation support, etc.

Today, there are a several methods available for the design of GRCSE systems that provide very different designs for the same design parameters. There is a need to develop a rational design method for this emerging technology that more accurately predicts GRCSE performance. Several research activities have been going on in the United States in the past few years, which include the FHWA pooled fund project – "Column-Supported Embankments" by Collin and Han, the National Deep Mixing Program project by Han – "Development of Design Charts for Geosynthetic-Reinforced Embankments over Deep Mixed Columns

MECHANISMS OF LOAD TRANSFER

The interactions among column, foundation soil, embankment fill, and geosynthetic reinforcement can be described as follows. Under the influence of fill weight, the embankment fill mass between columns has a tendency to move downward, due to the presence of the soft foundation soil. This movement is restrained by shear resistance from the fill above the columns. The shear resistance reduces the pressure acting on the geosynthetic but increases the load applied onto the columns. This load transfer mechanism in this case was termed the soil arching effect by Terzaghi (1943).

Compared with the unreinforced case, the inclusion of geosynthetic reinforcement is expected to reduce the displacement of the embankment fill between the columns. The reduction of the displacement would reduce the shear stresses in the embankment so that the effect of soil arching in the embankment would be minimized. As a result, the load transferred by soil arching to the columns is reduced. At the same time, however, the load on the columns may be increased by the vertical components of the tension force in the reinforcement. A single geosynthetic layer behaves as a tensioned membrane while a multi-layer system acts as a stiffened platform (or like a beam) due to the interlock of reinforcement with the surrounding soil. Giroud et al. (1990) and British Standard BS 8006 (1995) proposed similar rational for estimating the tension in geosynthetic reinforcement acting as a tensioned membrane. Wang et al. (1996) considered multiple geosynthetic reinforcements in soil providing an addition of apparent cohesion.

Soil arching and tensioned membrane effects also depend on the relative stiffness of the columns to the soil. A rigid column promotes the differential settlement between the columns and the soft soil so that there is more soil arching and tensioned membrane effect. Han and Gabr (2002) have confirmed these phenomena in their numerical analysis.

In summary, the mechanisms of load transfer can be considered as a combination of soil arching, tensioned membrane or stiffened platform effects, and relative stiffness effects between columns and soil. The load transfer contributed by each mechanism depends on a number of factors including number and tensile stiffness of geosynthetic reinforcement layers, properties of embankment fill and foundation soils, and moduli of column materials and soil.

POTENTIAL FAILURE MODES

If not properly designed, the GRCSE system may have the following possible failure mechanisms, failure of foundation soil, failure of columns and caps, failure of geosynethetic reinforcement, and slope instability.

Failure of foundation soil

When CSE are constructed over soft soil, the soft soil between the columns may fail due to low bearing capacity. The inclusion of geosynthetic reinforcement above the columns reduces the load transmitted to the soft soil.

Failure of column and caps

The columns and caps under the embankment may have the following possible failures modes:

- Column caps punching through embankment fill
- Tilt of column caps
- Flexural and/or shear failure of column caps
- Compression failure of column shafts
- End-bearing failure of columns
- Bending failure of columns
- Shear failure of columns

The inclusion of geosynthetic reinforcement would increase the resistance against the punching of column caps, minimize the chance of tilt of column caps and bending failure of columns by reducing lateral thrust from the embankment. However, it may require more flexural and shear capacities of column caps and load capacities of columns since more load is transferred onto the column caps and columns.

Failure of geosynthetic reinforcement

The geosynthetic reinforcement above column caps may fail due to rupture or pullout from the soil especially when the reinforcement is near the edge of the embankment. The reinforcement can also experience excess elongation due to low modulus and/or creep deformation. The tension in a geosynthetic layer when acting as a tension membrane would be reduced as the deflection/elongation of the reinforcement increases (i.e., stress relaxation).

Slope instability

The embankment system may encounter the following possible slope instability situations:

- Lateral spreading due to the thrust from the embankment
- Local slope instability
- Slope instability outside the first row of piles
- Slope instability through piles
- Global slope instability below piles

DESIGN CONSIDERATIONS

Percent coverage or improvement ratio

Based on the performance investigation of conventional pile-supported embankments, Rathmayer (1975) recommended design criteria as shown in Figure 2. The required percent coverage of pile caps, defined as the percentage of the total area of pile caps to that of foundation footprint, depends on the quality of fill materials. For columnar systems, the percent coverage is equivalent to area improvement (or replacement) ratio. The percent coverage of pile caps for thirteen actual GRCSE is plotted in Figure 2 for comparison purposes (Han, 1999). As shown in Figure 2, the area replacement ratio with geosynthetic reinforcement is much lower than that suggested by Rathmayer (1975) for the conventional CSE. The percent coverage of the GRCSE systems is consistently less than 20%. The reduction of percent coverage creates a more economical solution for embankment systems. The percent coverage or area improvement ratio for GRCSE embankment systems mostly ranges from 5% to 30% (Han, 2003).

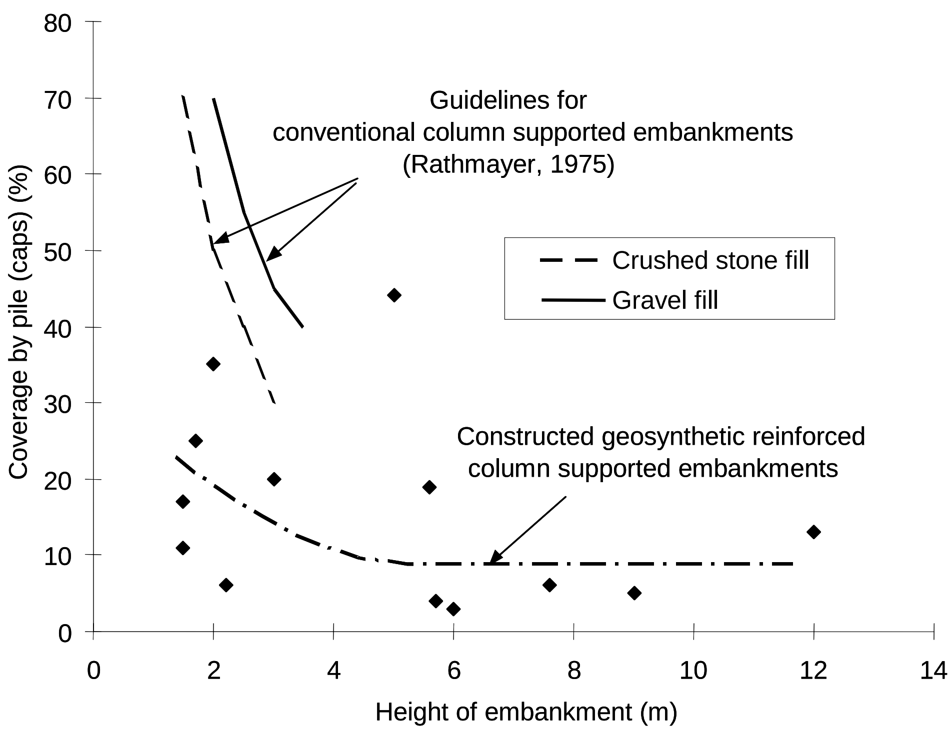


Figure 2. Percent Coverage of Pile Caps for Pile-Supported Embankments (Han, 1999)

Stress above geosynthetic reinforcement

The stresses applied on the soil (in CSE) and geosynthetic reinforcement (in GRCSE) between columns are reduced due to the soil arching effect. In GRCSE, the stress below the geosynthetic reinforcement is further reduced by the geosynthetics membrane effect. In almost all the related publications, the stress applied on the geosynthetic reinforcement is a key variable for computing the tension in geosynthetic reinforcement. Most current design methods ignore the soil resistance below the reinforcement, in other words, a void is assumed below the reinforcement. There are three categories of methods for computing the distributed stress above the geosynthetic reinforcement

1. Soil wedge method

This method has been adopted by a number of researchers, such as Carlsson (1987), Card and Carter (1995), and Svanø et al. (2000). The model is presented schematically in Figure 3. The weight of soil wedge is assumed carried by the geosynthetic reinforcement above the pile caps. Carlsson (1987) assumed $\theta = 15^o$ and Card and Carter (1995) used $\theta = 22.5^o$. Card and Carter (1995) used this θ angle based on the condition that three geogrid layers are spaced at a certain distance and within the triangular area to form a composite load transfer platform with granular fill. Svanø et al. (2000) recommended the θ angle should vary from 15.9^o to 21.8^o and be calibrated.

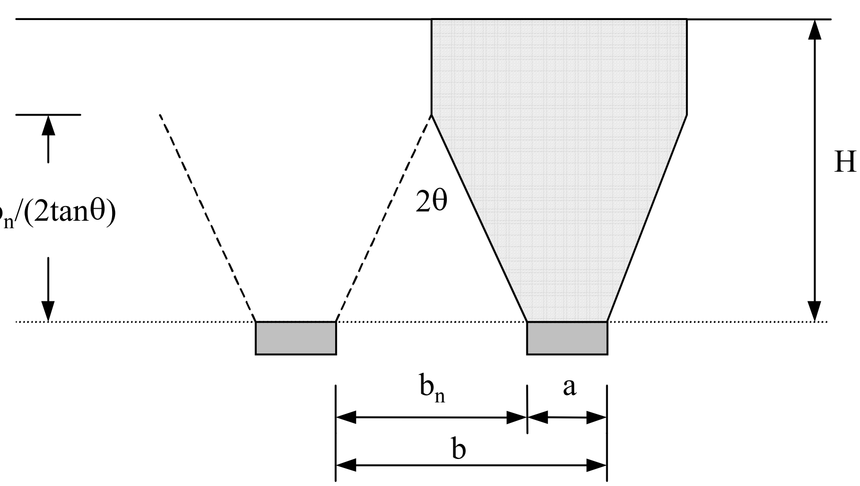


Figure 3. Soil Wedge Model

Carlson (1987) proposed a 2D approach with the height of the embankment above the triangular area so that the weight per unit length can be calculated by

$$w = \frac{(b-a)^2}{4\tan 15^o}\gamma \tag{1}$$

where w = the weight per unit length of reinforcement;
a = the width of square pile caps;
b = the center-to-center spacing of piles;
γ = the unit weight of fill.

Svanø et al. (2000) suggested that the load on the geosynthetic reinforcement is eventually carried by the two strips between column caps. These two strips have a width equal to that of the caps and a length equal to column spacing. They are perpendicular to each other if the columns are installed in a square pattern. This proposed method considers the 3-D effects. The weight per unit cap side length can be calculated by

$$w_s = \frac{\gamma}{2a}\left\{b^2H - \frac{1}{6\tan\theta}\left[(a + H\tan\theta)^3 - a^3\right]\right\} \tag{2}$$

where w_s = the weight per unit pile cap side length;
a = the width of square pile caps;
b = the center-to-center spacing of piles;
γ = the unit weight of fill;
H = the height of the embankment;
θ = the angle depicted in Figure 3.

If the height of the embankment is greater than $(b-a)/(2\tan\theta)$, $H = (b-a)/(2\tan\theta)$ should be substituted in Equation (2).

For multiple geogrid layers in the fill platform, Card and Carter (1995) suggested that each geogrid layer should be designed to carry the weight of the fill above within the soil wedge. Collin (2003) detailed the procedures for designing multiple geogrid layer-reinforced fill platform as a stiffened beam of reinforced soil that distributes the load from the embankment above the load transfer platform (i.e., stiffened beam) to the columns below the platform.
The Collin procedure is based on the following assumptions:

- The thickness (h) of the load transfer platform is equal to or greater than the clear span between columns (b-d), where d is the diameter of columns.
- A minimum of three layers of extensible (geosynthetic) reinforcement is used to create the load transfer platform.
- Minimum distance between layers of reinforcement is 20 cm.
- Select fill is used in the load transfer platform.
- The primary function of the reinforcement is to provide lateral confinement of the select fill to facilitate soil arching within the height (thickness) of the load transfer platform.
- The secondary function of the reinforcement is to support the wedge of soil below the arch.
- The entire vertical load from the embankment above the load transfer platform is transferred to the columns below the platform.
- The initial strain in the reinforcement is limited to 5%.

The vertical load carried by each layer of reinforcement is a function of the column spacing pattern (i.e., square or triangular) and the vertical spacing of the reinforcement. If the subgrade soil is strong enough to support the first lift of fill, the first layer of reinforcement is located 0.15 to 0.25 m above subgrade. Each layer of reinforcement is designed to carry the load from the platform fill that is within the soil wedge below the arch. The fill load attributed to each layer of reinforcement is the material located between that layer of reinforcement and the next layer above.

The uniform vertical load on any layer (n) of reinforcement (w_{Tn}) may be determined from the equation below:

$$w_{Tn} = [(b-d)_n^2 + (b-d)_{n+1}^2]\sin 60° \, h_n \gamma / [(b-d)_n^2 \sin 60°], \text{ triangular spacing} \quad (3)$$

$$w_{Tn} = [(b-d)_n^2 + (b-d)_{n+1}^2]\, h_n \gamma / (b-d)_n^2, \text{ square spacing} \quad (4)$$

where b = the center-to-center spacing of columns;
d = the diameter of columns;
h_n = the height of soil above or between the reinforcement which is carried by the reinforcement;
γ = the unit weight of platform fill.

2. Semi-spherical soil arching model

Hewlett and Randolph (1988) assumed the soil above the pile caps forms a semi-spherical soil arching as shown in Figure 4. The design considers possible failure of soil arching either at the crown of the arch or at the top of the column. The soil arching ratio, defined as the stress above the reinforcement to the overburden stress by the embankment, can be determined as the greater of the values in the following two equations:

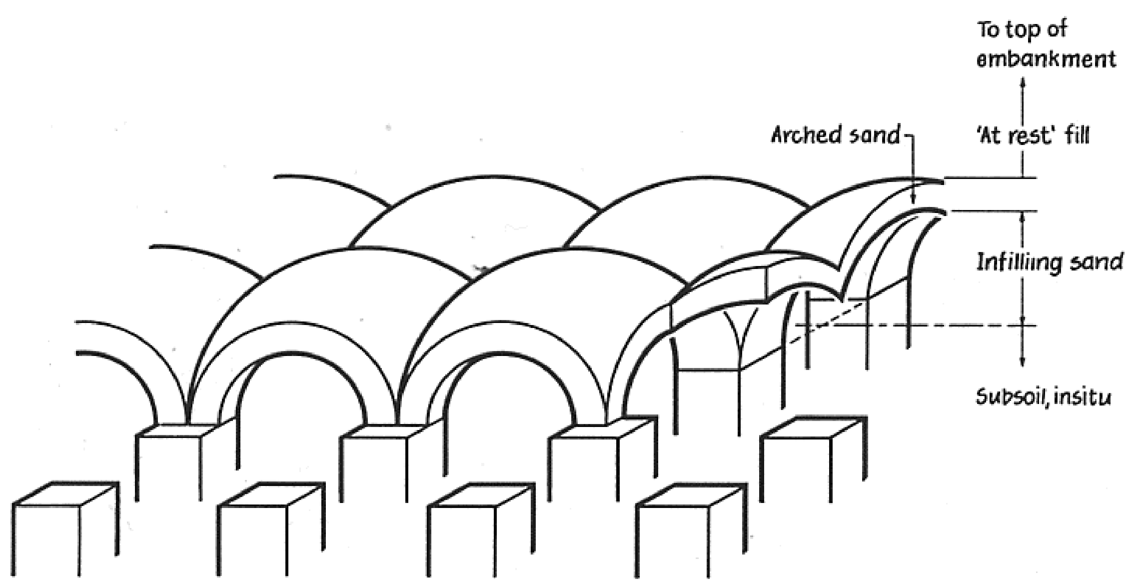


Figure 4. Soil Arching Model Proposed by Hewlett and Randolph (1988)

At the crown,

$$\rho = \left(1 - \frac{a}{b}\right)^{2(K_p - 1)} \left[1 - \frac{b}{\sqrt{2}H}\left(\frac{2K_p - 2}{2K_p - 3}\right)\right] + \frac{b - a}{\sqrt{2}H}\left(\frac{2K_p - 2}{2K_p - 3}\right) \tag{5}$$

At the pile cap,

$$\rho = \frac{1}{\left(\frac{2K_p}{K_p + 1}\right)\left[\left(1 - \frac{a}{b}\right)^{1-K_p} - \left(1 - \frac{a}{b}\right)\left(1 + \frac{a}{b}K_p\right)\right] + \left(1 - \frac{a^2}{b^2}\right)} \tag{6}$$

where ρ = the soil arching ratio;
a = the width of pile cap;
b = the spacing between centers of pile caps;
H = the height of embankment;
K_p = the coefficient of passive earth pressure.

Hewlett and Randolph (1988) suggested that the thickness of well-compacted high grade fill (K_p >3) should not be less than the spacing of piles.

3. Vertical conduit soil arching model (British Standard BS8006)

The British Standard BS8006 considered the columns acting similar to buried rigid pipes in a conduit. The average vertical stress on the top of the column is estimated using Marston's formula for positive projecting subsurface conduits:

$$\sigma_c = p\left(\frac{C_a a}{H}\right)^2 \tag{7}$$

$$C_a = \frac{1.95H}{a} - 0.18 \quad \text{for end-bearing columns}$$

or

$$C_a = \frac{1.5H}{a} - 0.07 \quad \text{for frictional and other columns}$$

where σ_c = the average vertical stress on the top of the column;
p = the overburden stress at the base of the embankment;
C_a = the arching coefficient;
a = the width of square column;
H = the embankment height.

The stress applied on the geosynthetic reinforcement between columns is dependent on the height of the embankment. The distributed load carried by the reinforcement between columns can be determined by

For H > 1.4(b-a):

$$w = \frac{1.4b\gamma(b-a)}{b^2 - a^2}\left[b^2 - a^2 \frac{\sigma_c}{p}\right] \tag{8a}$$

For 0.7(b-a) ≤H≤1.4(b-a):

$$w = \frac{b(\gamma H + q)}{b^2 - a^2}\left[b^2 - a^2 \frac{\sigma_c}{p}\right] \tag{8b}$$

$$w = 0 \text{ if } \frac{b^2}{a^2} \le \frac{\sigma_c}{p} \tag{8c}$$

where w = the distributed load per unit length of reinforcement;
q = the uniform surcharge on the surface of the embankment;
other symbols are defined in Equation (7).

Strain and tension in geosynthetic reinforcement

Geosynthetic reinforcement under applied stresses behaves as a tensioned membrane. A number of methods are available to estimate the strain and tension developed in the geosynthetic reinforcement.

1. Catenary method

The method presented in the reference (John, 1987) included the calculation of the strain and tension developed in the geosynthetic reinforcement:

$$\varepsilon_r = \frac{1}{2}\sqrt{1 + 16\frac{\Delta S_r^2}{b_n^2}} + \frac{b_n}{8\Delta S_r^2}\ln\left(\frac{4\Delta S_r^2}{b_n} + \sqrt{1 + \frac{16\Delta S_r^2}{b_n^2}}\right) - 1 \tag{9}$$

and $$T_r = \frac{1}{2}(\sigma_{sr} - \sigma_s)b_n\sqrt{1 + \frac{b_n^2}{16\Delta S_r^2}} \tag{10}$$

where ε_r = the strain developed in the geosynthetic reinforcement;
ΔS_r = the maximum deflection of the geosynthetic reinforcement;
b_n = the net spacing between the pile caps (b-a);
T_r = the tension developed in the geosynthetic reinforcement;
σ_{sr} = the average vertical stress on the geosynthetic reinforcement;
σ_s = the average vertical stress (soil resistance) below the geosynthetic reinforcement;

The average vertical stress on the geosynthetic reinforcement can be determined based on the methods discussed above. Again, John (1987) assumed $\sigma_s = 0.15\gamma H$ (γ = the unit weight of the embankment fill and H = the height of the embankment). The procedures to determine the strain and tension developed in the geosynthetic reinforcement are as follows: (1) assume a maximum deflection of the geosynthetic reinforcement; (2) use Equation (9) to calculate the strain in the geosynthetic reinforcement; (3) calculate the tension in the geosynthetic reinforcement using Equation (10); (4) use the calculated tension and the tension-strain curve of the geosynthetic reinforcement determined in the lab to calculate the strain; (5) adjust the maximum deflection to repeat the procedures until reaching convergence if the calculated strains in Step (2) and Step (4) do not match.

2. Carlson's method

Carlson (1987) suggested a simple formula to compute the maximum deflection of the geosynthetic reinforcement over a 2-D span as follows:

$$\varepsilon_r = \frac{8}{3}\left(\frac{\Delta S_r}{b-a}\right)^2 \qquad (11)$$

where ε_r = the strain developed in the geosynthetic reinforcement;
ΔS = the maximum deflection of the geosynthetic reinforcement;
b = the center-to-center spacing of piles caps;
a = the width of pile caps.

The tension developed in the geosynthetic reinforcement is calculated by

$$T_r = \frac{\gamma(b-a)^3}{32\Delta S_r \tan 15^o}\sqrt{1+\frac{16\Delta S_r^2}{(b-a)^2}} \qquad (12)$$

where T_r = the tension developed in the geosynthetic reinforcement;
γ = the unit weight of the embankment fill;
ΔS = the the maximum deflection of the geosynthetic reinforcemetn;
b = the center-to-center spacing of piles caps;
a = the width of pile caps.

Rogbeck et al (1998) proposed the following 3-D modification factor to account for 3-D effects:

$$f_{3D} = \frac{a+b}{2a} \tag{13}$$

where f_{3D} = the 3-D modification factor;
a = the width of the pile caps;
b = the center-to-center spacing of piles.

Equation (13) is used to multiply Equation (12) to obtain the tension in the geosynthetic reinforcement accounting for 3-D effects.

3. BS8605 method

The method proposed in the BS8605 standard for estimating the tension in geosynthetic reinforcement between columns is as follows:

$$T_r = \frac{w(b-a)}{2a}\sqrt{1+\frac{1}{6\varepsilon_r}} \tag{14}$$

where T_r = the tension in geosynthetic reinforcement;
w = the distributed load per unit length;
a = the width of column/column cap;
b = the center-to-center spacing of column;
ε_r = the strain in geosynthetic reinforcement.

The BS8605 standard recommends an upper limit of 6% initial tensile strain in geosynthetic reinforcement for a general case and a reduced limit necessary for a low height embankment to prevent differential settlement at the surface of the embankment. In addition, the standard recommends a maximum creep strain of 2% over the design life.

4. SINTEF's method

Svanø et al. (2000) at SINTEF suggested that the elongation over the pile caps should be included in the calculation of the strain in the geosynthetic reinforcement. They proposed the following formula:

$$\varepsilon_r' = \varepsilon_r\left(1+\alpha_T\frac{a}{b-a}\right) \tag{15}$$

where ε_r' = the "corrected" strain in the geosynthetic reinforcement;
ε_r = the strain considering a free span between the net spacing of column caps;
a = the width of column caps;

b = the center-to-center spacing of columns;
α_T = the tension ratio, which is defined as

$$\alpha_T = \frac{T_{rc}}{T_r} \tag{16}$$

where T_{rc} = the average tension in the geosynthetic reinforcement over the column caps;
T_r = the average tension in the geosynthetic reinforcement over the free span.

Svanø et al. (2000) did not provide any guidelines to determine the α ratio. However, the numerical study by Han and Gabr (2002) and Han et al. (2005) showed that the tension in the geosynthetic reinforcement above the column caps or columns is higher than that over the free span. Svanø et al. (2000) proposed the following equation to estimate the tension developed in the geosynthetic reinforcement:

$$T_r = \frac{w}{2}\sqrt{1+\frac{1}{6\varepsilon_r^{'}}} \tag{17}$$

where T_r = the tension developed in the geosynthetic reinforcement;
w = the distributed load above the geosynthetic reinforcement (two strips), which is calculated using Equation (2);
$\varepsilon_r^{'}$ = the corrected strain in the geosynthetic reinforcement, which is calculated using Equation (15);
b = the center-to-center spacing of piles.

5. Giroud et al. method (1990)

The tensioned membrane theory by Giroud et al. (1990) was developed for a geosynthetic layer over a sinkhole. Collin (2003) suggested the use of Giroud et al (1990) method to determine the tensile load in the reinforcement as a function of the amount of strain in the reinforcement. The tension in the reinforcement is determined from the following equation:

$$T_{rpn} = w_{Tn}\,\Omega\,D/2 \tag{18}$$

where: D = the design spanning for tension membrane, $D = (b\text{-}d)_n$ for square column spacing,
$D = (b\text{-}d)_n \tan 30°$ for triangular column spacing;
b = the center-to-center spacing of columns;
d = the diameter of the columns
Ω = the dimensionless factor provided in Table 1.

Table 1. Values of Ω

Ω	Reinforcement Strain (ε)%
2.07	1
1.47	2
1.23	3
1.08	4
0.97	5
0.90	6

APPLICATIONS

The main purpose for the use of the GRCSE system is to transfer the fill loads through piles to a deeper and firm soil layer or rock beneath the soft deposit to reduce embankment settlements and instabilities. This system is most suitable for the situations where

- rapid construction is necessary;
- strict total and/or differential settlement is required.
- soft soil is underlain by a firm soil layer or bedrock;
- and or new fill with certain thickness is needed.

The GRCSE systems have been used for a number of applications worldwide, which include:

- bridge approaching embankments;
- retaining walls foundation support;
- roadway widening;
- storage tanks foundation support;
- low height embankment;
- building foundation support.

Selected case studies are discussed below.

Bridge approach embankments

Reid and Buchanan (1984) reported that this technique was used for preventing differential settlement between an approach embankment constructed over soft soil and a bridge abutment supported by long piles (Figure 5). Piles with varying lengths and spacing were designed for transiting the settlement from near zero at the bridge abutment to a relatively large settlement at the transition from the CSE to the unsupported embankment on the soft soil. The geosynthetic layer was placed to minimize the differential settlement between piles at the pavement surface.

Two similar projects were reported by Broms and Wong (1985) using timber piles and geotextiles to support bridge approach embankments.

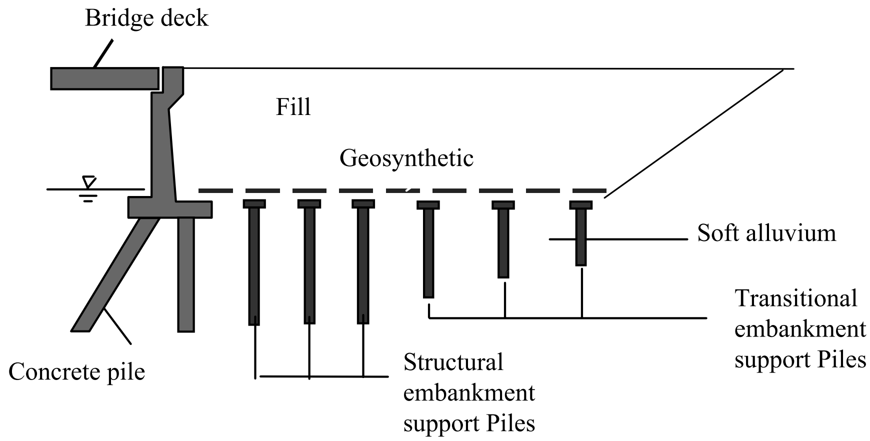


Figure 5. Bridge Approach Support Piling (Reid and Buchanan, 1984)

Retaining walls

An expansion of a highway in the northern area of Sao Paulo, Brazil included the construction of five geogrid-reinforced segmental retaining walls (SRW) with heights ranging from 2.0 to 8.2m. Fine-grained soil with 60 to 70% passing No. 200 sieve (LL<40, PI<20) was used as reinforced fill in these walls. To prevent potential buildup of pore water pressures in the reinforced fill, a drainage system with non-woven geotextile strips in the reinforced fill was installed. A portion of these walls were built on a 9m thick organic silt and clay deposit with SPT blow counts of 0 to 1. Jet-grout columns were selected for improving the very soft soil condition. The original design required 1.2m-diameter jet-grout columns spaced 2.0m on the centers. A geogrid-reinforced fill platform was introduced to enlarge the spacing of columns to 3.0m. These walls were instrumented and their settlements and lateral movements were monitored during the construction up to 90 days after the start of the construction. The typical cross-section of an SRW on a geogrid-reinforced fill platform supported by jet grout columns is illustrated in Figure 6.

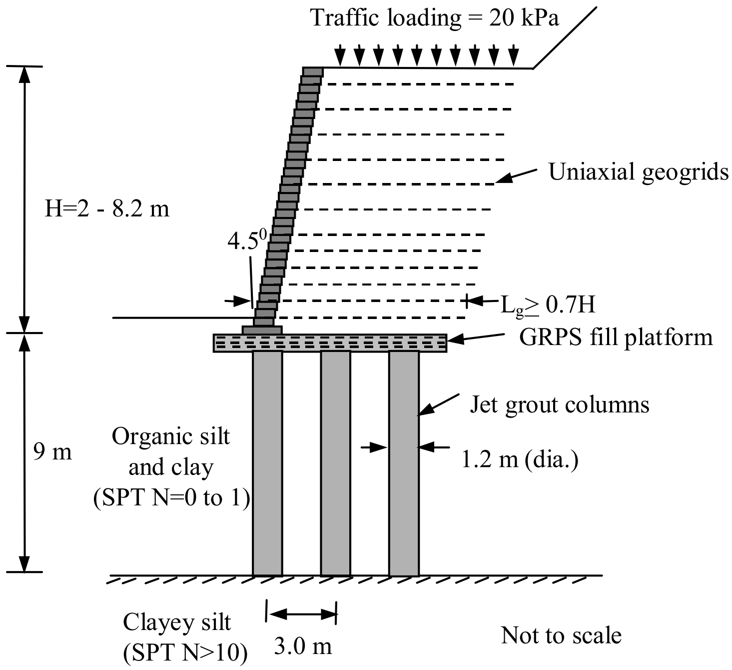


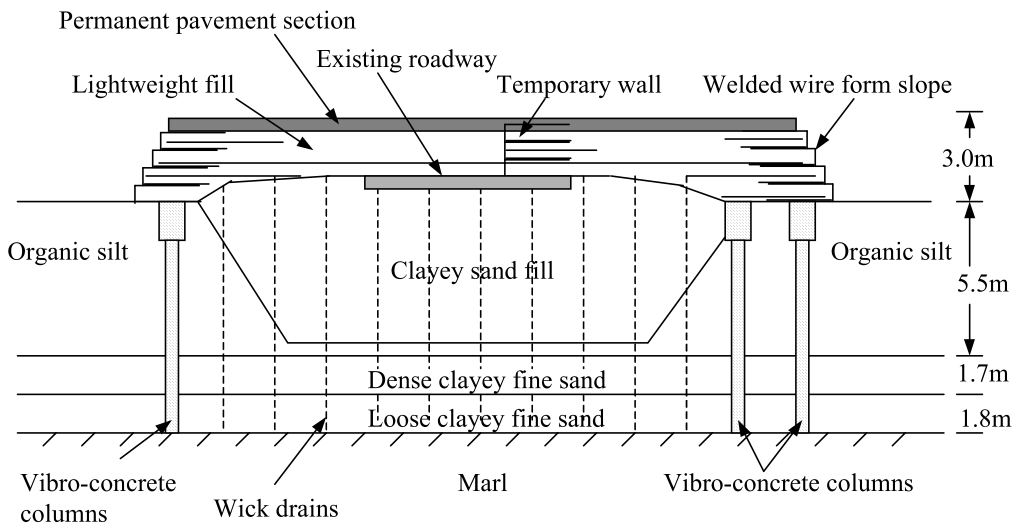
Figure 6. Typical Cross Section of SRWs on the GRPS System (Alzamora et al., 2000)

Roadway widening

Differential settlement is commonly an issue when a new embankment is constructed adjacent to an existing embankment for roadway widening. For most cases, the existing embankment has completed the settlement. The addition of the new embankment would induce not only relatively large settlement itself but also the settlement for the existing embankment. The GRCSE system was used in a project as shown in Figure 7 for preventing differential settlement between a new embankment over soft soil and an existing embankment. This project required not only the widening of the existing roadway but also the raising of the existing roadway elevation to approach a bridge. Vibro-concrete columns with enlarged column heads were used in this project.

Storage Tanks

As shown in Figure 8, the GRPS system was used in conjunction with vibro-concrete columns to minimize total and differential settlements for a storage tank founded over 3.0 to 4.5 m soft organic silt and peat (Shaefer et al., 1997). Three layers of geogrid were placed above the vibro-concrete columns to form the load transfer platform. A similar idea was adopted to construct other storage tanks in Scotland (Thorburn et al., 1984). Instead of geosynthetic-reinforced fill platform, a 150mm thick reinforced concrete membrane was placed over the piles and within the granular fill.



Not to scale

Figure 7. Typical Cross Section of Widening Roadway (Han and Akins, 2002)

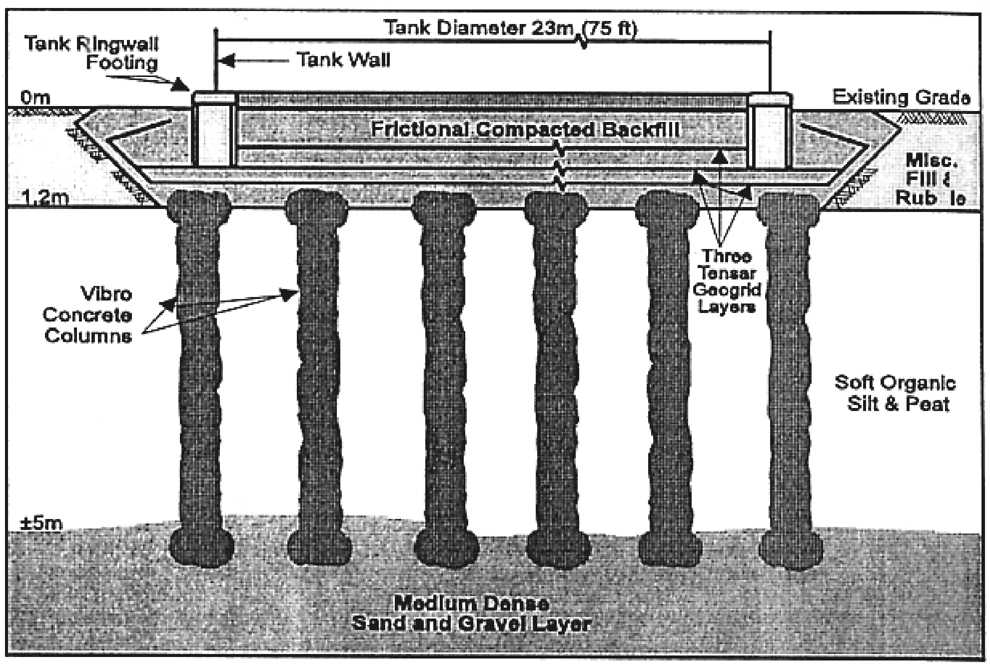


Figure 8. Storage Tank on A Geogrid-Reinforced and VCC-Supported Foundation (Shaefer et al., 1997)

Low height embankment

The major challenge of constructing a low height embankment over piles is the differential settlement between the pile caps to be reflected to the surface. The influence of traffic loading on the settlement also becomes important. A two-lane 10m wide pavement with 2m wide sidewalks on each side was constructed on a soft foundation in Japan. The soft foundation consisted of a 4m thick peat layer with moisture content of 500% and a 4m thick clay layer. Deep mixing (DM) soil-cement columns were installed to improve the soft foundation. As shown in Figure 9, the DM columns were 800mm in diameter and spaced at 2.1m. They had an unconfined compressive strength of 1MPa. A single layer of geogrid was placed on the top of the DM columns. A relatively low (1.5m) embankment (pavement section) was constructed above the geogrid layer. The improvement ratio of 11% was used in this project, which is much less than 50 to 70% coverage of pile caps required in accordance to Rathmayer (1975) for the same height of embankments. The settlements on the top of the columns and the mid-span between the columns at the level of the top of the columns were monitored over a 15-month period. The measured results showed that differential settlements developed between the columns. The maximum differential settlement reached 15mm. In addition, The measured strains in the geogrid layer increased with the differential settlements but less than 0.5%.

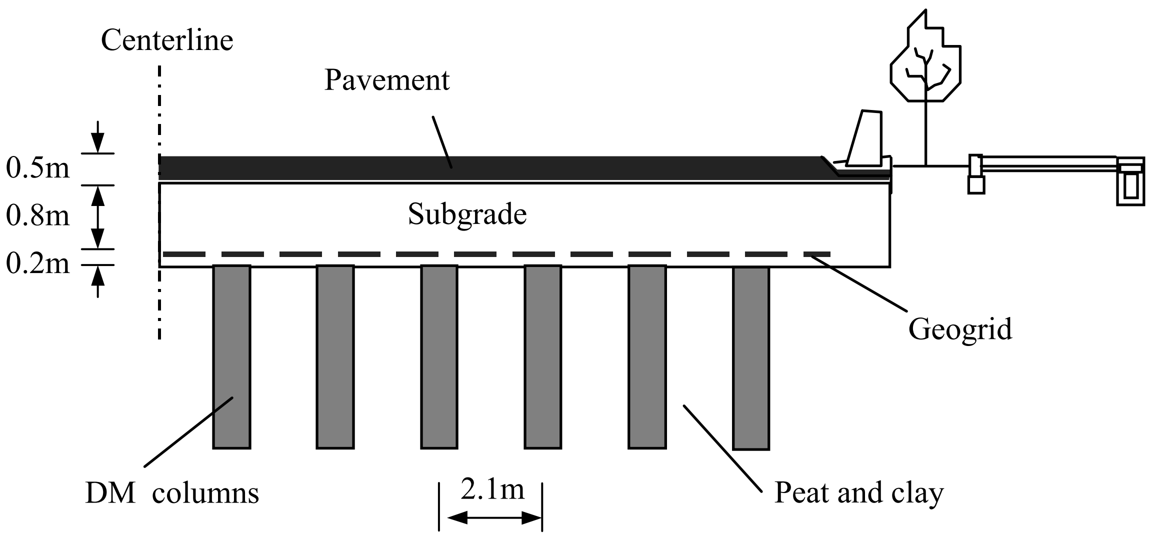


Figure 9. Low Height Embankment over Deep Mixed Columns (Tsukada et al., 1993)

Railroads

Due to the need for upgrading approximately one hundred-year-old railway between Berlin and Magdeburg, Germany, to withstand trains at a speed of 160km/h and higher loads, a geogrid-reinforced and pile-supported railway embankment was constructed over soft organic soil as shown in Figure 10. The details of this project can be found in the literature by Brandl et al. (1997) and Alexiew and Gartung (1999). Numerical analysis was conducted by Huang et al. (2005) to model this geosynthetic-reinforced pile-supported embankment and found the 3-D numerical method is reasonably accurate to estimate the maximum settlement below the geosynthetic layer and the maximum tension in the geosynthetic layers.

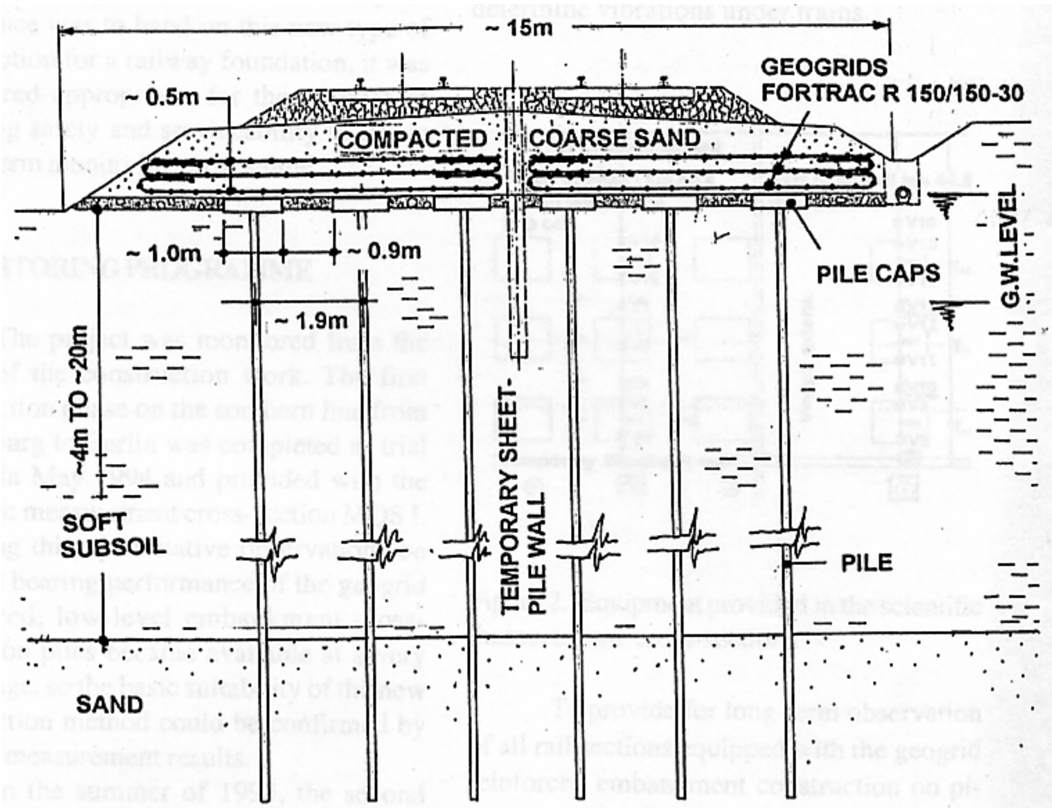


Figure 10. Railway on Geogrid-Reinforced and Pile-Supported Embankment (Alexiew and Gartung, 1999)

Buildings

Han and Akins (2002) reported the use of this GRCSE system for a building constructed on uncontrolled fill. Due to the highly variable uncontrolled existing fill and new fill with non-uniform thickness to be placed, the total and differential settlements are the major concern for this project. The GRCSE system was constructed to provide a stiffened platform to bridge over questionable underlying soils and mitigate differential settlement as shown in Figure 11. This project was designed based on numerical analysis and the predicted settlements were close to the measured. This was one of the earliest applications using geosynthetic-reinforced pile-supported fill platform for buildings.

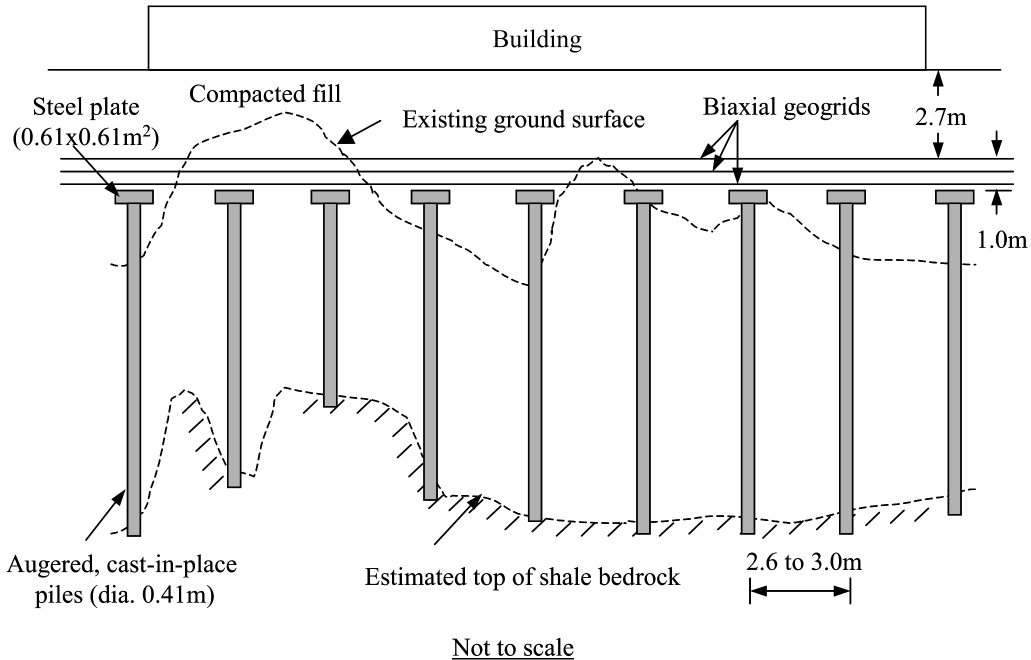


Figure 11. A Building on the GRPS System (Han and Akins, 2002)

CONCLUSIONS

The following conclusions may be drawn:

(1) GRCSE systems are most suitable for situations with a very soft soil underlain by a stiff soil layer or bedrock, new fill with certain thickness needed, rapid construction necessary, and strict total/differential settlement required;
(2) The common applications of these systems include bridge approach, roadway widening, and railroads or highways across soft soil;
(3) The use of geosynthetic reinforcement significantly reduces the required percent coverage or improvement ratio of the columns. The percent coverage or improvement ratio mostly ranges from 5% to 30%;
(4) Possible failure modes of these systems include failure of foundation soil, columns and caps, geosynthetic reinforcement and slope instability. The slope instability includes lateral spreading, local slope stability, general slope stability, and global slope stability;
(5) Soil arching, tensioned membrane or stiffened platform effects, and relative stiffness effects between columns and soil are identified as the load transfer mechanisms above the column caps or columns;
(6) Three common soil arching models are used for estimating the applied stress above the geosynthetic reinforcement;
(7) Tensioned membrane theories with different assumptions of deflected shapes are used for estimating the strain and the tension developed in geosynthetic reinforcement.

REFERENCES

Alexiew, D. and Gartung, E. (1999). "Geogrid reinforced railway embankment on piles – performance monitoring 1994 – 1998." *Proc. of Geossintéticos'99*, Rio de Janeiro, Brazil, 403-411.

Alzamora, D., Wayne, M.H., and Han, J. (2000). "Performance of SRW supported by geogrids and jet grout columns." *Proceedings of ASCE Specialty Conference on Performance Confirmation of Constructed Geotechnical Facilities*, Geotechnical Special Publication No. 94, ASCE, 456-466.

Bell, A.L., Jenner, C., Maddison, J.D., and Vignoles, J. (1994). "Embankment support using geogrids with Vibro Concrete Columns." *Proceedings, 5th International Conference on Geotextiles, Geomembranes and Related Products*, 1, Singapore, 335-338.

Brandl, H., Gartung, E., Verspohl, J., and Alexiew, D. (1997). "Performance of a geogrid-reinforced railway embankment on piles." *Proc. of the 14th ICSMFE*, Hamburg, 3, 1731-1736.

British Standard BS 8006 (1995*). Code of Practice for Strengthened/Reinforced Soils and Other Fills*, British Standard Institution, London, 162p.

Broms, B.B. and Wong, I.H. (1985). "Embankment piles." Third International Geotechnical Seminar – Soil Improvement Methods, Singapore, 27-29 November.

Card, G.B. and Carter, G.R. (1995). "Case history of a piled embankment in London's Docklands." *Engineering Geology of Construction*, Geological Society Engineering Geology Special Publication, No. 10, 79-84.

Carlsson, B. (1987). *Reinforced soil, principles for calculation*. Terratema AB, Linköping. (In Swedish)

Colin, J.G. (2004). "Column Supported Embankment Design Considerations." *Proc. University of Minnesota 52nd Annual geotechnical Engineering Conference*, Minneapolis, Minnesota.

Collin, J.G. (2003). *NHI Ground Improvement Manual – Technical Summary #10: Column Supported Embankments*.

Fluet, J.E., Christopher, B.R., and Slaters, A.R. (1986). "Geosynthetic stress-strain response under embankment loading conditions." *Proc. 3rd Int. Conf. on Geotextiles*, Vienna, Vol. 1, 175-180.

Giroud, J.P., Bonaparte, R., Beech, J.F., and Gross, B.A. (1990). "Design of soil layer-geosynthetic systems overlying voids." *Geotextiles and Geomembrane*, Elsevier, 9(1), 11-50.

Han, J. (1999). "Design and construction of embankments on geosynthetic reinforced platforms supported by piles." *Proceedings of 1999 ASCE/PaDOT Geotechnical Seminar*, Central Pennsylvania Section, ASCE and Pennsylvania Department of Transportation, Hershey, PA, 66-84.

Han, J. and Akins, K. (2002). "Case studies of geogrid-reinforced and pile-supported earth structures on weak foundation soils." *Proc. of International Deep Foundation Congress*, Geotechnical Special Publication No. 116 – *Deep Foundations 2002*, edited by O'Neill and Towsend, ASCE, Orlando, 668-679.

Han, J. and Gabr, M.A. (2002). "A numerical study of load transfer mechanisms in geosynthetic reinforced and pile supported embankments over soft soil." *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, 128(1), 44-53.

Han, J. (2003). *Development of Design Charts for Geosynthetically Reinforced Embankments on Deep Mixed Columns*. Interim Report I – Literature Review, submit to National Deep Mixing Cooperative Research Program, 123p.

Han, J., Huang, J., and Porbaha, A. (2005). "2D numerical modeling of a constructed geosynthetic-reinforced embankment over deep mixed columns." GeoFrontiers Conference, Austin, TX, Jan.

Hewlett, W.J. and Randolph, M.F. (1988). "Analysis of piled embankments." *Ground Engineering*, 21(3), 12-18.

John, N.W.M. (1987). *Geotextiles*. Blackie, Glasgow and London.

Magnan, J. (1994). "Methods to reduce the settlement of embankments on soil clay: a review." *Foundations and Embankments Deformations*, ASCE, Geotechnical Special Publication No. 40, 77-90.

Marston, A. and Anderson, A.O. (1913). "The theory of loads on pipes in ditches and tests of cement and clay drain tile and sewer pipe." Bulletin No. 31, Iowa Engineering Experiment Station, Ames, Iowa.

Pham, H.T.V., Suleiman, M.T., and White, D.J. (2004). "Numerical analysis of geosynthetic-rammed aggregate pier supported embankment." Geo-Trans Conference, Los Angles, CA, July.

Rathmayer, H. (1975). "Piled embankment supported by single pile caps." *Proceedings, Istanbul Conference on Soil Mechanics and Foundation Engineering*, 283-290.

Reid, W.M. and Buchanan, N.W. (1984). "Bridge approach support piling." *Piling and Ground Treatment*, Thomas Telford Ltd, London, 267-274.

Rogbeck, Y., Gustavsson, S., Södergren, and Lindquist, D. (1998). "Reinforced piled embankments in Sweden-design aspects." *Proc. 1998 Sixth International Conference on Geosynthetics*, 755-762.

Schaefer et al. (1997). *Ground Improvement, Ground Reinforcement, Ground Treatment Developments 1987-1997*. Geotechnical Special Publication No. 69, V.R. Schaefer et al. (ed.), 616p.

Shen, S. L., and Miura, N. (2001). "A technique for reducing settlement difference of road on soft clay." *Computer Methods and Advances in Geomechanics*, *Proc. 10 IACMAG*, Vol.2, Edited by C.S. Desai et al., A.A. Balkema, 1391-1394.

Stewart, M.E., Navin, M.P., and Filz, G.M. (2004). "Analysis of a column supported test embankment at the I-95/Route 1 interchange." Geo-Trans 2004 Conference, Los Angles, CA, July.

Svanø, G., Ilstad, T., Eiksund, G., and Want, A. (2000). "Alternative calculation principle for design of piled embankments with base reinforcement." *Proc. 4th International Conference on Ground Improvement Geosystems*, Helsinki, June.

Terzaghi, K. (1943). *Theoretical Soil Mechanics*, John Wiley & Sons, New York, 66-75.

Thorburn, S., Laird, C.L., and Randolph, M.F. (1984). "Storage tanks founded on soft soils reinforced with driven piles." *Piling and Ground Treatment*, Thomas Telford Ltd, London, 157-164.

Tsukada, Y., Isoda, T., and Yamanouchi, T. (1993). "Geogrid Subgrade Reinforcement and deep foundation Improvement: Yono City, Japan." *Proceedings, Geosynthetics*

Case Histories, International Society for Soil Mechanics and Foundation Engineering, Committee TC9, 158-159.

Wang, M.C., Feng, Y.X., Jao, M. (1996). "Stability of geosynthetic-reinforced soil above a cavity." *Geotextiles and Geomembranes*, Elsevier, 14, 95-109.

Zanzinger, H. and Gartung, E. (2002). "Performance of a geogrid reinforced railway embankment on piles."'"Geosynthetics -–7th ICG, Delmas, Gourc, and Girard (eds.), Swets & Zeitlinger, Lisse, 381-386.