Comparison of FHWA and BS8006 design methods for geosynthetic reinforcement over piles

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ABSTRACT: The construction of embankments or retaining walls over soft foundation soils is a common challenge for geotechnical engineers. Problems associated with it include poor bearing capacity, excessive differential settlements and lateral sliding instability. This led to the development of several ground improvement techniques. However, when settlements need to be controlled, rigid intrusions are commonly installed to provide support and settlement control by transferring the load to a stiffer foundation. To effectively transfer the load from the soil structure to the piles and develop soil arching between rigid intrusions, piles spacing and cap size need to properly designed. To optimize the design and reduce the number of piles needed, geosynthetic reinforcement can be used to transfer the vertical load from the soil structure to the piles while maintaining the settlement control. In literature several design methods are available to calculate the vertical stresses and the strain mobilized in the geosynthetic reinforcement. However, two major design approaches are currently used in the USA and Europe: the beam method (FHWA-NH1-16-028, 2017) and the catenary method (BS 8006:2010). The first one considers multiple layers of low strength geosynthetic reinforcement equally spaced within a select granular structural fill that act as a single rigid beam. The second approach considers high strength geosynthetic that acts as a catenary single layer at the interface between the columns and the soil structure. This paper presents an overview and comparisons between these two design approaches, design examples, and cost analysis.

Keywords: Geosynthetic reinforcement, basal reinforcement, column supported embankments, design methods

1 INTRODUCTION

The presence of poor ground conditions in a construction site is historically the most frequent cause of long construction durations. Exist several alternatives to overcome this challenge, including relocating the project, removing and replacing the unsuitable soil, design the structure accordingly to the soil conditions or modifying the existing soil through ground improvement techniques. However, the current economic approach and political realities of highway projects make long duration ground improvements too costly and unacceptable in many instances. In addition, many new highway projects are widening of existing roads or they cross special soil deposits where traditional methods are not technically viable. (FHWA-RC-BAL-04-0015).

Modern ground modification and geoconstruction technologies are still the economically preferred solution in many cases to meet the demanding project requirements (Schaefer et al., 2017). Ground modification can be defined as the alteration of site foundation conditions or project earth structures to provide better performance under deign and/or operational loading conditions (USACE 1999). Common primary functions include: increase shear strength and bearing resistance, increase density and drainage, decrease permeability, control deformations, provide lateral stability, accelerate consolidation, decrease imposed loads, provide lateral stability, increase resistance to liquefaction and transfer embankment loads to more competent layers (Schaefer et al., 2017). Typically used ground modification are categorized by their primary function as listed above and include among others: vertical drains, vibro-compaction, dynamic compaction, grouting, chemical and mechanical stabilization, aggregate columns and deep mixing methods.

Each ground improvement technique has inherent limitations on their applicability as related to as the type of soil, depth or treatment and performed function. Furthermore, project constrains such as schedule and time, right of way, budget, cost and utilities among others can highly affect the selection of one type of ground improvement technique over another.

2 COLUMN SUPPORTED SOIL STRUCTURES

When time constraint is a critical aspect of the project, and primary and secondary settlements need to be controlled, rigid intrusions (i.e. piles, columns) are commonly installed to provide support and settlement control by transferring the load through the soft compressible soil layer to a stiffer foundation. Figure 1 shows examples of structures on soft soil supported by rigid intrusions. Many different types of columns can be used, including stone columns, vibro-concrete columns, deep-mixing method columns, driven piles and auger piles (Collin, 2004 and 2005). A load transfer platform (LTP) or a bridging layer is commonly installed immediately above the columns to help transfer the load from the soil structure to the columns. The use of geosynthetic reinforcement is not dependent on any one column type.

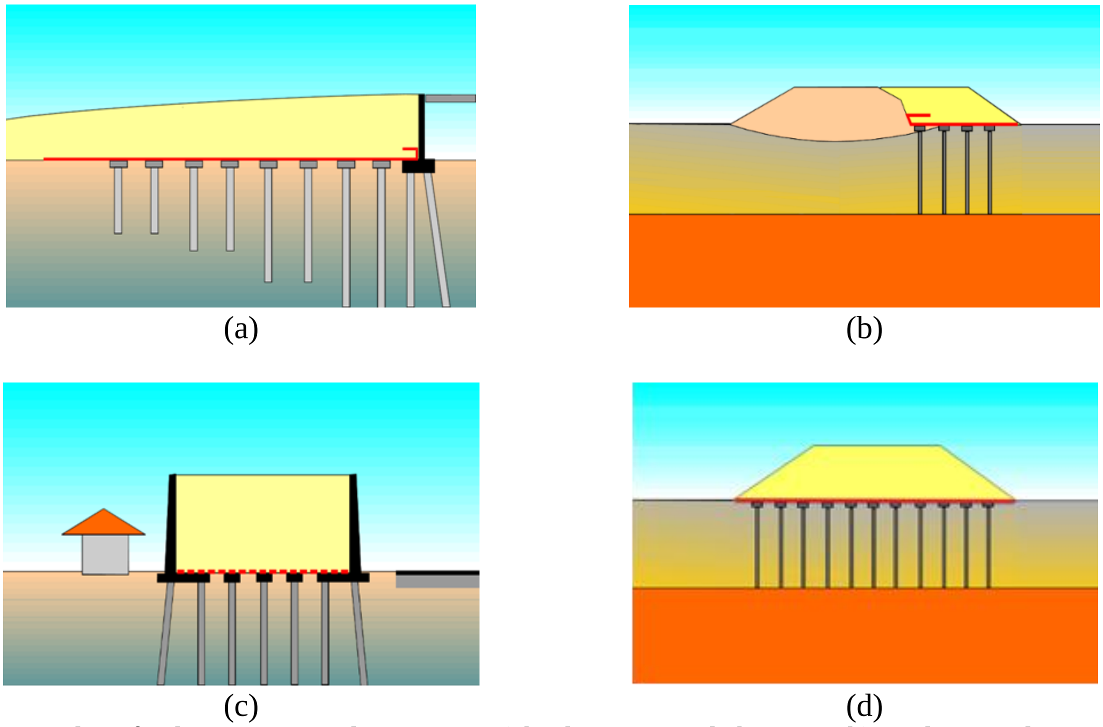


Figure 1. Examples of column supported structures: a) bridge ramp and abutment, b) roadway widening, c) storage tank and d) embankment stabilization

Probably the most critical aspect when designing column supported structures is that the load must be transferred effectively to the columns to prevent punching of the columns into the soil structure; this would lead to severe differential settlements in the surface of the structure. The mechanisms through which soil transfer the load to the columns is called soil arching. It is commonly defined as the ability of a granular material to transfer loads from the yielding part of the soil to an adjacent rigid zone through a system of shear stresses that oppose the relative movement between stationary and moving masses. This transfer mechanism within column supported soil structures was first termed and described by Terzaghi (1943). Through this mechanism, when columns are placed close enough together, soil arching occurs between columns, developing shear stresses resisting the movement of the soft soil and thereby reducing the stresses at the base of the moving soft soil and transferring them to the rigid stationary column. The load is therefore transferred from the soil to the columns which in turn transfer the loads to the deeper firm bearing stratum.

The spacing between piles is crucial to develop an efficient soil arching. To optimize the design of piles it is the common practice to include one or a series of geosynthetic reinforcement layers to create a LTP or a bridging layer directly on top of the columns. The goal of the geosynthetic reinforcement is to span across pile caps carrying all or a portion of the load and transferring it to the columns. By doing so, spacing between piles can be significantly increased and piles caps size can be decreased. In addition, the reinforcement can be used to counteract the horizontal thrust of the embankment fill, eliminating the need of raking piles along the extremities of the foundation.

The design of column supported soil structures usually need to consider both limit state and serviceability state failure criteria. Most of these design steps are followed using engineering practices covered either by the FHWA-NHI-16-028 or the BS8006. However, the design of geosynthetic reinforced pile supported embankments involves the calculation of the vertical stresses on the geosynthetic reinforcement. From which, strain and tension developed in the geosynthetic can be calculated (Sloan, 2003). Several models have been developed to calculate the vertical stresses and several others to calculate the strain and required strength in the geosynthetics. However, two fundamental design models are currently used in the US and Europe: the beam method and the catenary method. The beam method (FHWA-NHI-16-028) considers multiple layers (3 or more) of low strength geosynthetic reinforcement equally spaced within a select granular structural fill (i.e. load transfer platform) that act as a single rigid beam. The catenary theory (BS8006) considers one single high strength geosynthetic that acts as a catenary single layer at the interface between the columns and the soil structure. This paper herein summarizes and compares both design approaches.

3 THE FHWA LTP DESIGN METHOD (FHWA-NHI-16-028)

The newly published FHWA-NHI-16-028 describes in detail the design process of column supported embankments with and without LTP. The beam method that has been predominantly used in the United States is being replaced by a more analytically sophisticated design method based on load and displacement compatibility, named the load displacement compatibility (LDC) method. However, the beam design method, as illustrated in Figure 2, is still commonly used and recommended for preliminary design to determine the LTP thickness and the geosynthetic requirements. The LDC method is then used to perform the settlement analysis.

The beam design method assumes a stiffened beam of select fill with a minimum of three layers of geosynthetic reinforced with 200 mm minimum vertical spacing. LTP thickness should be equal or greater than one-half of the clear span between columns.

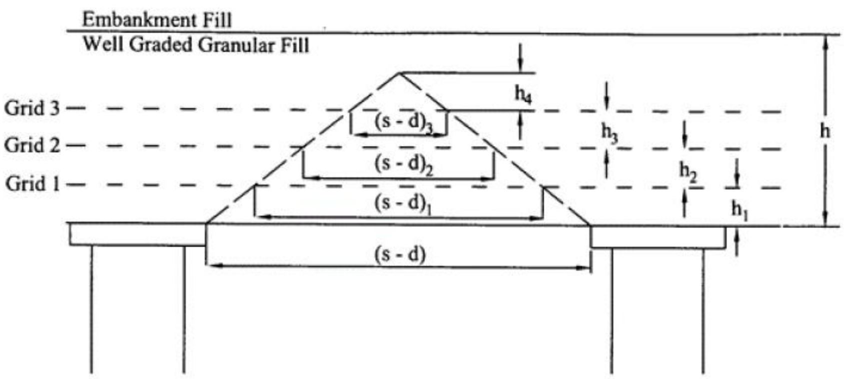


Figure 2. The Beam Method

Each geosynthetic layer provides lateral confinement on the select fill, facilitates soil arching and supports the weight of the soil in the section of the pyramid between itself and the next geosynthetic layer. For a square array of columns, the uniform vertical load on any layer of reinforcement is calculated as follows:

$$W_{Tn} = [(s-d)_n^2 + (s-d)_{n+1}^2]\left(\frac{H_n\gamma}{(s-d)_n^2}\right) \tag{1}$$

Where γ is the soil unit weight. This load is then used to calculate the tension in the geosynthetic layers using tensioned membrane theory:

$$T_{rpn} = W_{Tn}\Omega\left(\frac{D}{2}\right) \tag{2}$$

Where D = design spanning for tension membrane and Ω = dimensionless factor function of the reinforcement strain. Collin, 2007 modified the beam method by introducing a fourth geosynthetic reinforcement layer directly above the pile caps that acts as a catenary reinforcement.

The LDC method was first developed by Filz and Smith (2006, 2007) for rigorously analyzing the net vertical load acting on the pile caps and the geosynthetic reinforcement in the LTP. The LDC method models the actual load transfer mechanisms by analyzing the net vertical load that act on the geosynthetic reinforcement. The method assumes an axisymmetric approximation of a unit cell with vertical load equilibrium and displacement compatibility at the level of the geosynthetic. The vertical equilibrium of the axisymmetric system is satisfied when:

$$\gamma H + q = a_s\sigma_{col,geotop} + (1 - a_s)\sigma_{soil,geotop} = a_s\sigma_{col,geobot} + (1 + a_s)\sigma_{soil,geobot} \tag{3}$$

Where, γ = unit weight of the embankment soil, H = height of the embankment, q = surcharge pressure, α_s = area replacement ratio, $\sigma_{col,\ geotop}$ = vertical stress acting down on the top of the geosynthetic in the area underlain by the column, $\sigma_{soil,\ geotop}$ = vertical stress acting down on the top of the geosynthetic in the area underlain by the soil foundation, $\sigma_{col,\ geobot}$ = vertical stress acting up on the top of the geosynthetic in the area underlain by the column, $\sigma_{soil,\ geobot}$ = vertical stress acting up on the top of the geosynthetic in the area underlain by the soil foundation.

The method uses the Adapted Terzaghi Method (Russel and Pierpoint, 1997) to determine the limiting distribution of stresses acting up on the base of the embankment and a parabolic deformation pattern of the geosynthetic between columns to calculate tensile stresses in the geosynthetic reinforcement. Like the Collin method though, this model is calibrated for biaxial and triaxial geogrids, commonly installed in a series of parallel layers. For a rectangular array of columns, the tension in a biaxial geogrid can be estimated solving the following stress-strain compatibility equation:

$$6T^3 - 6T\left(\frac{\sigma_{net}A_{soil}}{p}\right)^2 - J\left(\frac{\sigma_{net}A_{soil}}{p}\right)^2 = 0 \tag{4}$$

Where, T = tension in the geogrid, $\sigma_{net} = \sigma_{soil,\ geotop} - \sigma_{soil,\ geobot}$ = net vertical stress acting on the geogrid, A_{net} = area of geogrid in a unit cell underlain by soil, p = pile cap perimeter, J = sum of the stiffness of the geogrid layers.

The equation can be modified for radially isotropic geogrids (i.e. triaxial geogrids) and for triangular arrays of columns. As input parameters, the model uses geogrid stiffness, J, and long-term tensile strength as geogrid properties. Isochronous curves are used to determine the long-term required tensile strength at a specific design strain value. Creep, installation damage and durability reduction factors are then applied to the short-term tensile strength to determine the available long-term design tensile strength.

The reinforcement layers should extend across the width of the embankment beyond the crest towards embankment toe to achieve an adequate bond with the adjacent soil to prevent lateral sliding (spreading). The minimum reinforcement length is determined using the following equation:

$$L_e = \frac{P_{lat}}{0.5\gamma H c_{iemb} \tan \Phi_{emb}} \tag{5}$$

Where P_{lat} = required tensile force to prevent lateral sliding, c_{iemb} = coefficient of interaction for sliding between the reinforcement and the embankment fill and Φ_{emb} = friction angle of embankment fill.

4 THE CATENARY THEORY LTP DESIGN METHOD (BS8006:2010)

The BS8006 design method is based on the use of a single layer of geosynthetic reinforcement that acts as a catenary system and deforms in a parabolic shape. However, the system allows the use of multiple layers of reinforcement. Because of the very high stresses that one single reinforcement need to carry, the method requires the use of high strengths geosynthetic, commonly one order of magnitude higher than using the beam method. The standard also allows the use of multiple reinforcement layers; however, it states that for large settlements (> H/25), the stronger geogrid or the lowest layer attracts a much higher amount of mobilized resistance force. Since the distribution of forces is not yet fully understood the standard recommends here possible to design the lower level reinforcement to the maximum limit state tensile force.

The BS8006:2010 provides two methods for determining the distributed load on the reinforcement: the Marston's formula and the Hewlett and Randolph method.

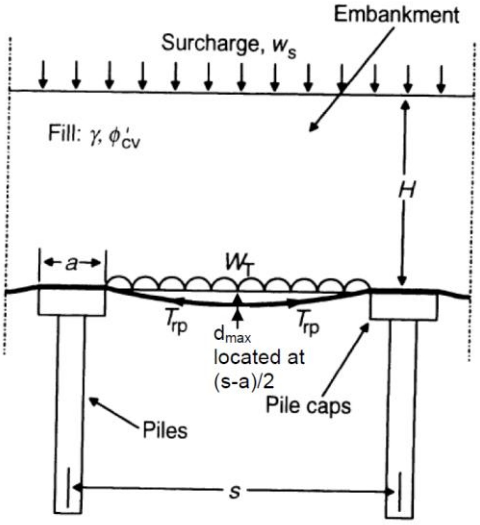


Figure 3. Definition of variables for BS8006:2010 design method

The Marston's formula estimates the ratio of the vertical stress exerted on top of the pile caps to the average vertical stress at the base of the embankment as:

$$\frac{p_c'}{\sigma_v'} = \left[\frac{C_c}{H}\right]^2 \tag{6}$$

Where p_c' = vertical stress on the pile caps, σ_v' = factored average vertical stress at the base of the embankment, C_c = arching coefficient (function of pile arrangement), H = embankment height. The distributed load carried by the reinforcement between adjacent pile caps can be determined from:

$$\text{For } H > 1.4(s - a) \qquad W_t = \frac{1.4 s f_{fs} \gamma (s-a)}{s^2 - a^2}\left(s^2 - a^2 \frac{p_c'}{\sigma_v'}\right) \tag{7}$$

$$\text{For } 0.7(s-a) \le H \le 1.4(s-a) \qquad W_t = \frac{s(f_{fs}\gamma H + f_q w_s)}{s^2 - a^2}\left(s^2 - a^2 \frac{p_c'}{\sigma_v'}\right) \tag{8}$$

The BS8006:2010 allows for an alternative theoretical solution to determine the vertical load acting across the geosynthetic reinforcement based on the work of Hewlett and Randolph, 1988. The method calculates the arching efficiency as the proportion of the embankment weight carried by the piles, hence the proportion of the embankment weight carried by the geosynthetic reinforcement is (1-E). The system can fail at either the crown of the arch, E_{crown}, or at the pile cap, E_{pile}. For equations on how to determine arching efficiency, refer to BS8006:2010.

The maximum distributed load carried by the reinforcement between adjacent pile caps is determined using the minimum between E_{crown} and E_{pile}.

$$W_t = \frac{s\left(f_{fs}\gamma H + f_q w_s\right)}{s^2 - a^2}(1 - E_{min})s^2 \tag{9}$$

The tensile load generated in the reinforcement resulting from the distributed load W_t, can then be estimated using the following equation.

$$T_{rp} = \frac{W_t(s-a)}{2a}\sqrt{1+\frac{1}{6\varepsilon}} \tag{10}$$

The above equation has two unknowns, T_{rp} and the strain in the reinforcement ε. It is solved for T_{rp} by considering the maximum allowable strain in the reinforcement and by an understanding of the stress-strain characteristics of the reinforcement. The tensile load in the reinforcement develops as the geosynthetic deforms under the weight of the embankment. This normally occurs during embankment construction but in situations where the reinforcement cannot deform during construction the reinforcement will not carry the applied loads until the foundation settles.

It is essential to ensure enough bond with the adjacent soil at the extremities of the piled area to guarantee that the maximum limit state tensile load can be generated. The reinforcement bond beyond the out row of piles can be determined using the following equation:

$$L_b \geq \frac{f_n f_p\left(T_{rp} - T_{ds}\right)}{\gamma H\left(\frac{a_1' \tan \Phi_{cv1}'}{f_{ms}} + \frac{a_2' \tan \Phi_{cv2}'}{f_{ms}}\right)} \tag{11}$$

Where f_n and f_p = partial factor of safety, T_{rp} = factored tensile load in the reinforcement, T_{ds} = factored tensile load needed to resist outward thrust, a_1' and a_2' = interaction coefficients between embankment fill and reinforcement for the upper and lower side of the reinforcement respectively, Φ'_{cv1} and Φ'_{cv2} = constant volume friction angle of embankment fill and soil below the reinforcement. If adequate bond length cannot be achieved because of embankment geometry limitations, the reinforcement can be wrapped around a thrust block positioned along the outer row of piles into the embankment (i.e. gabions, concrete blocks) and returned into the embankment.

The maximum allowable strain in the reinforcement ε_{max} should be limited to ensure differential settlements do not occur at the surface of the embankment. This can be a problem with shallow embankments where the soil arch cannot develop fully within the embankment fill. The initial tensile strain in the reinforcement is needed to generate a tensile load. BS 8006:2010 imposes a practical upper limit of 6% strain to ensure all embankment loads are transferred to the piles. With shallow embankments this upper strain limit may have to be reduced to prevent differential movements at the surface of the embankment.

The maximum mid-span deflection of the reinforcement is recommended to be limited to 300mm. However, the maximum deflection generally occurs diagonally to the mid-span between adjacent pile caps and is assumed to be twice the maximum deflection calculated orthogonally the two pile caps.

The long-term strain (due to creep) of the reinforcement should be kept to a minimum to ensure that long-term localized deformations do not occur at the surface of the embankment. BS 8006:2010 restricts the maximum creep strain over the design life of the reinforcement to 2%.

5 DESIGN EXAMPLE COMPARISON

To compare these two design methods, a design of a new column supported embankment over soft soil was completed following both design methods. The design of the load transfer platform using the FHWA design method was carried out using the GeogridBridge2.0 spreadsheet developed by Filz and Smith (2006, 2007). The spreadsheet implements the simultaneous nonlinear equations described in Chapter 2 using the Adapted Terzaghi Method to determine the load on the geosynthetic reinforcement, the Parabolic Method to determine the tension in the reinforcement and load-displacement compatibility principles to incorporate the support provided by the weak subgrade.

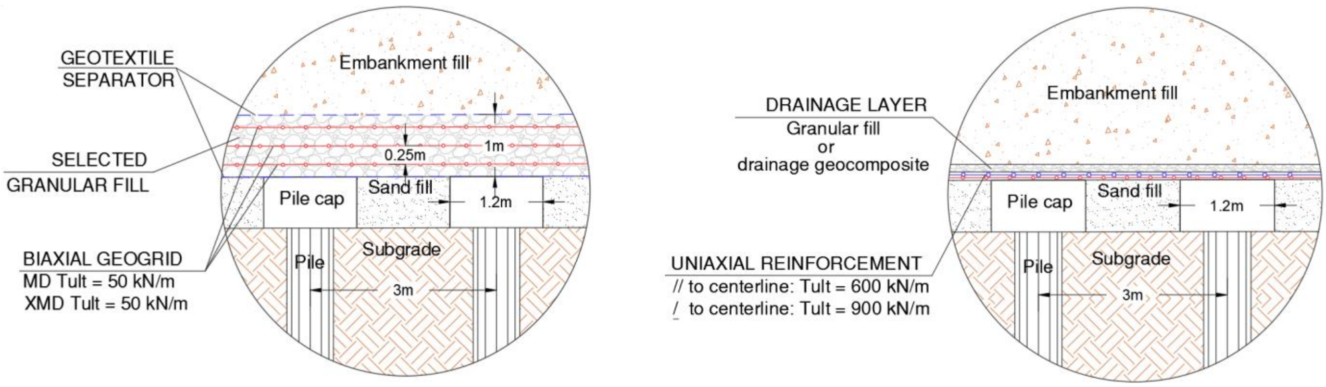
The design of the geosynthetic reinforcement following the BS8006:2010 was carried out using Maccaferri design software MacBARS. Given an embankment geometry, soil properties, piles type and layout, the software calculates the distributed load on the geosynthetic reinforcement using either the Marston's formula or the Hewlett and Randolph method. The software also checks the pile and embankment geometry, strain compatibility in the longitudinal and transversal direction and creep strain over the design life. The output of the software is the factored required ultimate tensile strength of the geosynthetic reinforcement both longitudinal and transversal to the embankment centerline.

Several parameters of the subgrade stratigraphy and soil parameters play a key role in the design of column supported embankments (i.e. layers thickness, unit weight, column-soil interface friction angle, compression rate, and coefficient of consolidation). All these parameters govern the design of the piles layout, pile cap size and post-construction differential settlement. To fairly compare these two design methods, the same piles layout and pile cap size were used during this exercise. A similar level of geosynthetic reinforcement strain in the geogrids was also maintained. The design parameters are summarized in the Table 1.

Table 1. Design parameters

Embankment	End Bearing Piles	Bridging layer
• Height (H): 4m • Unit weigh (γ): 18 kN/m^3 • Friction angle (Φ): 32° • Young's Modulus (E): 14,400 kPa • Poisson's ratio (υ): 0.33 • Traffic: 14 kN/m^2	• Center to center spacing (s): 3m • Pile diameter: 0.6m • Pile cap (a): square, 1.2m	• Thickness: 1m • Unit weigh: 20 kN/m^3

The results of the analyses are summarized in Figure 3.



(a) (b)

Figure 3. Analyses results for (a) FHWA LDC and (b) BS8006:2010 design methods.

When designing load transfer platforms, the cost of the structure is something that always needs to be taken into consideration. The three components that affect the cost of a LTP are the geosynthetic reinforcement, the selected granular fill and labor to install the materials. A cost comparison between the two solutions is summarized in Table 2. The comparison conservatively assumes same labor cost for both solutions even though the installation of two layer of geosynthetics with no selected granular fill between them would be exempted be more cost effective. A drainage geocomposite layer on top of the uniaxial geogrids is included in the BS8006:2010 cost analysis. Table 2 shows that the main cost difference between the two alternatives is the cost of the selected granular backfill that highly affect the total cost and covers the difference in cost between geogrids. Therefore, the availability of granular selected fill highly influences the final cost of the LTP.

Table 2. Cost comparison

Components	Typical unit cost	Average unit cost	FHWA LDC	BS8006:2010
Selected granular fill	7 to 20 \$/ton	\$13.5/ton	\$29/m^2	/
Biaxial geogrid	2 to 4 \$/m^2	\$3/m^2	\$9/m^2	/
Uniaxial geogrid	8 to 13 \$/m^2	10.5 \$/m^2	/	\$21/m^2
Drainage Geocomposite	4 to 6 \$/m^2	\$5/m^2	/	\$5/m^2
Geotextile separator	1 to 2 \$/m^2	1.5 \$/m^2	\$3/m^2	/
Labor	\$45/m^2	\$45/m^2	\$45/m^2	\$45/m^2
TOT			\$84/m^2	\$71/m^2

6 CONCLUSIONS

The design approach of the BS8006:2010 determines the requirement of two orthogonal layers of uniaxial or biaxial geosynthetics reinforcement directly on top of the pile caps. One longitudinal parallel to the centerline of the embankment to resist the redistributed vertical load and one transversal to resist both vertical load and lateral thrust. On the other hand, the FHWA LTP design method is calibrated to use biaxial or triaxial geogrids embedded in a thick layer of selected granular material to create a stiff beam that redistributes the loads. Biaxial and triaxial geogrids currently present in the market offer a limited range of tensile strengths compared to uniaxial geogrids that can vary to T_{ult} over 1500 kN/m. Both design procedures use the parabolic method to determine the tension and strain in the geosynthetic but use different methods to calculate stresses on the geosynthetic; the Adapted Terzaghi method is used by the FHWA LTP while the Hewlett and Randolph method is used by the BS8006:2010. the However, both methods have shown consistent results when compared. From a cost perspective, the main factor affecting the cost effectiveness of one method over the other is the availability of a relatively close source of selected granular material.

REFERENCES

BS8006-1:2010. Code of Practice for Strengthened/Reinforced Soils and Other Fills

Collin, J.G. (2004) “Column Supported Embankment Design Considerations”, Proceedings 52nd Annual Geotechnical Conference, University of Minnesota, Minneapolis, MN. February 27, 2004.

Collin, J.G., Watson, C.H., and Han, J. (2005), “Column Supported Embankment Solves Time Constraint for New Road Construction”, ASCE Geo-Frontiers Conference, Austin TX, January 2005.

Collin, J.G. (2007). “U.S. state-of-practice for the design of geosynthetic reinforced load transfer platforms in column supported embankments.” GeoDenver 2007.

FHWA-RC-BAL-04-0015. Piles Supported Embankments for Accelerated Construction

Filz, G.M. and Smith, M.M. 2006. Design of Bridging Layers in Geosynthetic-Reinforced, Column-Supported Embankments, Virginia Transportation Research Council Report No. VTRC 06-CR12.

Filz, G. M. and Smith, M. E. 2007. “Net vertical loads on geosynthetic reinforcement in column-supported embankments.” GeoDenver 2007, GSP-172: Soil Improvement.

Hewlett, WJ, Randolph, MA. 1988. Analysis of Piled Embankments. Ground Engineering. April pp.12-18.

Holtz, R.D., Christopher B.R., Berg R.R. 2008. Geosynthetic Design & Construction Guidelines Reference Manual. Federal Highway Administration (FHWA), U.S. Department of Transportation, Publication No. FHWA-NHI-07-092, National Highway Institute, Washington D.C., USA

Russell, D. and Pierpoint, N. (1997). An Assessment of Design Methods for Piled Embankments. Ground Engineering, 30(11): pp. 39-44.

Schaefer, V. R., Berg, R. R., Collin, J. G., Christopher, B. R., DiMaggio, J.A., Filz, G. M., Bruce, D. A., Ayala, D. 2017. Ground Modification Methods Reference Manual – Volume I. Federal Highway Ad-ministration (FHWA), U.S. Department of Transportation, Publication No. FHWA-NHI-16-027, National Highway Institute, Washington D.C., USA

Schaefer, V. R., Berg, R. R., Collin, J. G., Christopher, B. R., DiMaggio, J.A., Filz, G. M., Bruce, D. A., Ayala, D. 2017. Ground Modification Methods Reference Manual – Volume II. Federal Highway Administration (FHWA), U.S. Department of Transportation, Publication No. FHWA-NHI-16-028, National Highway Institute, Washington D.C., USA

Sloan, J.A. 2011. Column-Supported Embankments: Full-Scale Tests and Design Recommendations. Doctor of Philosophy Dissertation, Virginia Polytechnic Institute and State University, Blacksburg, VA

Sloan, J.A., Filz, G.M., Collin, J.G. and Kumar, P. (2014). Column-Supported Embankments: Field Tests and Design Recommendations, 2nd Edition. Virginia Tech Center for Geotechnical Practice and Research, CGPR #77, February.

Terzaghi, K. (1943). Theoretical Soil Mechanics.

USACE. (1999). Guidelines on Ground Improvement for Structures and Facilities. Technical Letter No. 1110-1-185, Department of the Army. U.S. Army Corps of Engineers, Washington, D.C., 42p.

van Eekelen, S.J.M, Bezuijen, A., Oung, O. (2003). Arching in piled embankments; experiments and design calculations. Foundations: Innovations, observations, design and practice, Thomas Telford, London, 2003, 885-894.