# Geoencased columns: toward a displacement based design

## Colonnes renforcée par géotextiles: vers une conception basée sur le déplacement

Galli A., Prisco di C.
*Politecnico di Milano*

ABSTRACT: As is largely testified by the scientific literature, in the last decade geoencased columns have become a quite common alternative solution to standard stone columns. This is essentially due to the possibility of employing reinforcements to better the mechanical response of the inclusions without reducing their drainage efficiency. Although GEC are often used to reduce settlements induced by the construction of large embankments on soft soils, up to now a rational displacement based design approach has not yet been introduced. This is thus the final objective of this paper, that, by starting from a critical review of the standards presently available, will illustrate the results of a series of finite difference numerical analyses. The unit cell of an ideal reinforced soil embankment placed on a soft soil stratum will be accounted for and the effect of the main geometrical/mechanical parameters, as well as the response of the system during the construction stages, is discussed.

RÉSUMÉ : Comme il est largement connu dans la littérature de ces vingt dernières années, les colonnes en matériaux granulaires renforcée par géotextiles (GEC) sont devenues une solution très utilisée par rapport aux colonnes ballastées standard. Cela est essentiellement dû à la possibilité d'employer des renforts pour améliorer la réponse mécanique des inclusions sans réduire leur efficacité de drainage. Bien que les GEC soient souvent utilisées pour réduire les tassements induits par la construction de remblais importants sur sols mous, une approche rationnelle de conception basée sur le déplacement n'a, jusqu'à présent, pas encore été mise en place. Cela est donc l'objectif final de cette étude, qui, en partant d'une analyse critique des normes actuellement disponibles, illustrera les résultats d'analyses numériques aux différences finies. Une cellule élémentaire d'un remblai idéal de sol renforcé placé sur un sol mou sera prise en compte et l'effet des principaux paramètres géométriques et mécaniques et la réponse du système au cours des différentes étapes de la construction seront discutés.

KEYWORDS: geoencased granular columns, geotextiles, numerical analyses, displacement based design, earth reinforced structures.

## 1 INTRODUCTION.

As it is well documented in the literature (see e.g. Raithel et al. 2005), since mid-nineties the use of geoencased granular columns (GEC) as foundations of earth structures on soft and very soft soils has been progressively increased. GECs have both mechanical and hydraulic functions: they work not only as reinforcement inclusions, capable of preventing the global collapse of the foundation and reducing differential settlements within the structure, but they work additionally as vertical drains, thus reducing the consolidation time of the soft soil.

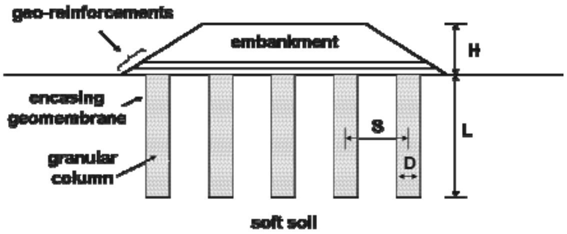


Figure 1. sketch of an earth embankment on GECs.

The GEC foundation system is composed of an array of granular columns of length L and diameter D, placed at a regular spacing S below an embankment of height H (Figure 1). The columns are encased by a geotextile with the double aim of reinforcing the column and filtering to prevent the clogging of the column itself. At the base of the embankment, to redistribute vertical stresses, several layers of geotextile are also inserted during the construction. The effectiveness of this foundation system has been clearly proved both on real scale data (see e.g. Kempfert 2003, where the response of the system is analyzed by varying the spacing among columns and of the stiffness of the encasing geotextile), and by means of numerical and experimental researches (Murugesan and Rajagopal 2006, di Prisco and Galli 2011). The fundamentals of the mechanical behavior of the system is therefore quite well understood. Nevertheless, common design standards are still based on too simplified approaches, unable of capturing the actual mechanical complexity of the system in particular, the interaction between embankment and columns (this point will be tackled in further details in the following section by critically reviewing the most used design standards). Conversely to traditional deep foundation systems (like reinforced concrete piles or jet-grouted columns which can be considered axially rigid with respect to the surrounding soil), GECs are axially deformable inclusions, whose axial deformability is strictly coupled with the stiffness of the surrounding soil. Moreover, since this latter is very often characterized by a very low permeability and high deformability, its mechanical response should be modeled by properly taking into account the hydro-mechanical coupling (for the sake ofbrevity, this aspect will however be disregarded in the following).

In the present paper the attention will be initially focused on a critical review of the most common design standards. Then an engineering displacement based approach will be briefly introduced, and some numerical analyses, with the particular aim of studying the distribution of differential settlement at the top of the embankment, which are generally neglected by the design approaches, will be presented.

## 2 REVIEW OF DESIGN STANDARDS

The two most common design standards available for the design of earth embankment on GECs are the British Standard BS 8006 (1995) and the German Standard EBGEO (Chapter 6.9; 2003). Both of them assume the column to be rigid and, estimate the vertical stress distribution at the base of the embankment to be independent of the mechanical interaction with the foundation (i.e. the GEC and the soft soil). From an engineering point of view, however, the vertical stress redistribution at the base of the embankment is the main parameter governing (i) the design of the reinforcement layers (see Figure 1) and (ii) the evaluation of the differential settlements.

According to BS8006, the vertical stress redistribution does not depend on the mechanical properties of the embankment. In particular, the average vertical stress acting at the column's top (in the following this quantity will be called $\sigma_i$) is determined only as a function of the geometry (H, S, D), as was suggested by the approach proposed by Martson (1913) for buried pipes (see also Jones et al., 1990). The estimation of the average vertical stress $\sigma_e$ acting on the soft soil is instead obtained by means of empirical expressions, depending on the full or partial formation of the arch effect, as a function of the ratio between height H and difference S-D. According to EBGEO, on the contrary, a rather complex analytical procedure, based on the work proposed by Zaeske (2001), is employed to describe the arch effect. This takes into account the geometry (H, S, D) and the friction angle of the granular material constituting the embankment, and imposes the equilibrium of one central slice of a vault shell of the arch that it is supposed to develop within the embankment. No estimation of the vertical stress $\sigma_i$ at the top of the column is provided. For the sake of brevity, the analytical expressions have not been reported here; for further details, see BS8006 and EBGEO (Chapter 6.9).

As far as the evaluation of settlements is concerned, the procedure prescribed by EBGEO follows the work proposed by Ghionna and Jamiolkowski (1981) and consists in subdividing the length L of the column in slices (each one of them is then assimilated to an axisymmetric triaxial soil sample). The following hypotheses are assumed: (i) the granular soil in the column is at critical state (i.e. no changes in volume are possible for the column), (ii) no relative settlement are considered between the column and the soil. These two hypotheses introduce very strong simplifications that can lead to unphysical results. The second one, in particular, makes impossible the superficial differential settlements to be estimated.

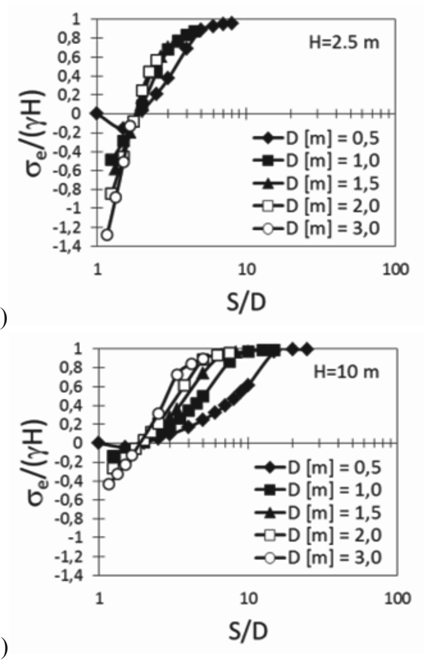
### 2.1 *Parametrical analyses*

In this section parametrical analyses on the values of $\sigma_e$ obtained by employing BS8006, as well as some results concerning the evaluation of the settlement and of the tensile force in the encasing geo-membrane computed according to EBGEO, are presented. In particular, the effect of the embankment height H and of the material friction angle $\phi$' is investigated for increasing values of the relative spacing S/D, and by taking into account several diameters D of the column (the authors are aware of the fact that some values of D and S/D considered are unrealistic, nevertheless they have been chosen in order to test even the asymptotic trend of the design approaches).

#### 2.1.1 *Stress on the soft soil at the base of the embankment*

Figure 2 shows the values of $\sigma_e$ computed according to BS8006, and highlights that unphysical results of $\sigma_e<0$ are obtained for low values of the relative spacing S/D, independently of the embankment height H. This result could in general lead to an overestimation of the arch effect and thus to an unsafe design of the georeinforcement layers at the base of the embankment.

The arch effect tends to vanish for increasing values of S/D, and the value of $\sigma_e$ tends to the weight $\gamma$H of the embankment. The corresponding values of $\sigma_i$ computed according to BS8006 (not reported here for the sake of brevity) are independent of S/D, and only slightly dependent on H.



a)

b)

Figure 2. Evaluation of the stress $\sigma_e$ according to BS8006: (a) H=2.5m and (b) H=10m.

It can be easily demonstrated that $\sigma_e$ and $\sigma_i$ (if computed according to BS8006) do not even satisfy the total equilibrium along the vertical direction with respect to the weight of the embankment, and that the values of the tensile force in the geosynthetic layers at the base of the embankment computed according to BS8006 are not continuous with increasing H (Moraci and Gioffrè 2010).

#### 2.1.2 *Settlements and tensile force in the encasing membrane*

With reference to the values of the mechanical parameters listed in Table 1 (taken from an example of application proposed by EBGEO), in this paragraph a parametrical analyses on the values of the settlement is presented, for increasing values of the embankment weight $\gamma$H and by taking into account several values of stiffness J of the encasing geomembrane. The values of the column length L and of the relative spacing S/D are here considered to be constant and equal to 10 m and 2, respectively (with D=80 cm and S=1.6m).

Table 1. Values of the mechanical properties of the materials considered in the analyses.

| | *Embankment* | *Column* | *Soft soil* |
|---|---|---|---|
| Unit weight (kN/m$^3$) | 20 | 19 | 15 |
| Friction Angle (°) | - | 35 | 15 |
| Cohesion (kPa) | - | - | 10 |
| Young modulus (kPa), at a reference pressure of 100kPa | - | - | 750 |
| Poisson coefficient (-) | - | - | 0.4 |

As it is evident from Figure 3a (where, for the sake of generality, the value of s has been normalized with respect to L), the presence of the encasing geomembrane induces a stiffening effect of the foundation system, thus reducing the expected value of the total settlement (which is considered, according to the adopted hypotheses, to be uniform and coincident with the settlement s at the top of the embankment). The numerical procedure, however, for low values of $\gamma$H (i.e. shallow or light embankments) leads to unrealistic results, for which negative settlements (i.e. uplift) are obtained. This meaningless result is obtained even for a nil value of the stiffness J of the encasing geomembrane. This essentially derives from the assumption concerning the soil within the column which is imposed to be at the critical state along the entire column (similar results have been observed even for other values of L and S/D, but they have not been reported here for the sake of brevity). This is evident when the tensile force $T_g$ in the encasing geomembrane along depth z of the column is considered (Figure 3b, where the case of a shallow embankment is analyzed): at the base of the column the vertical stress is not sufficient to induce an active state of stress, and the only possibility for the column to satisfy the hypothesis of critical state is to reduce its radius, thus inducing a compression (i.e. $T_g<0$) in the encasing geomembrane.

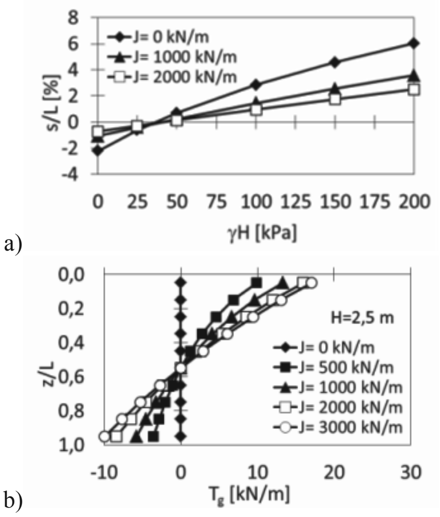


Figure 3. EBGEO: evaluation of (a) settlements and (b) tensile force in the encasing geomembrane.

## 3 A DISPLACEMENT BASED DESIGN APPROACH

In order to overcome the above cited limitations, a consistent and physically based design would require a fully displacement based approach. As was theoretically outlined by Galli and di Prisco (2011), with reference to a single axisymmetric cell (i.e. to a single column together with the surrounding soft soil), the foundation system can be assumed to be composed by two coupled springs, one representing the GEC and the second representing the surrounding soft soil. The two springs work in parallel if and only if the base of the embankment can be considered to be rigid and no differential settlements to arise (Figure 4a). Under this hypothesis, the vertical stress at the base of the embankment is thus uniformly distributed (in Figure 4a γ stands for the unit weight of the granular material constituting the embankment), no differential settlement are observed at the top of the embankment, and no shear stresses develop at GEC-soil interface. The values of vertical stress both in the column and in the soil then can be assumed to depend exclusively on the axial stiffness of the column ($K_{GEC}$) and on the vertical compressibility of the soft soil (represented in Figure 4a by a global stiffness $K_S$).

Real embankments, however, are characterized by a deformable base (Figure 4b), and different values of settlement are expected for the top of the column ($u_c$) and for the soil ($u_s$) at the base of the embankment. Consequently: (i) vertical stresses are redistributed at the base of the embankment between the internal zone of the cell (above the column, characterized by an average stress $\sigma_i$) and the external one (a circular crown above the soil, characterized by an average stress $\sigma_e$) due to the so called arch effect, (ii) shear stresses are activated at GEC-soil interface, and (iii) differential settlements are expected even at the embankment top.

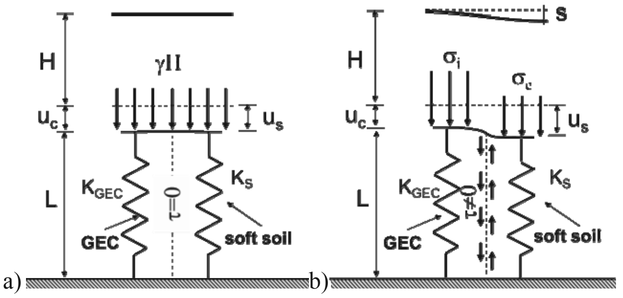


Figure 4. Mechanical response of the foundation system in case of (a) rigid and (b) deformable embankment.

From a modeling point of view, by assuming an engineering approach based on generalized variables, the mechanical behavior of the embankment can be described by means of a generalized constitutive relationship between the average stresses at the base of the embankment and the differential settlements (assumed to be uniform) between the column and the soil:

$$\sigma_i - \sigma_e = f(u_s - u_c)\,, \tag{1}$$

where the values of $\sigma_i$ and $\sigma_e$ must satisfy the equilibrium with respect to the weight of the embankment on the unit cell

$$\sigma_i \cdot D^2 + \sigma_e \cdot \left(S^2 - D^2\right) = \gamma H \cdot S^2\,. \tag{2}$$

The constitutive relationship f can be in general assumed to be described by means of a non-linear curve, whose average stiffness depends (i) on the geometry of the system (S, D, H), (ii) on the mechanical properties of the granular material constituting the embankment and (iii) on the geo-reinforcements at the base of the embankment. Its limit value corresponds instead to the activation of a failure mechanism within the embankment. Depending on the formation of the arch effect, either a "punching" failure mechanism, or a "dome" failure mechanism, with no (or very limited) superficial differential settlements, might develop (Figure 5a-b).

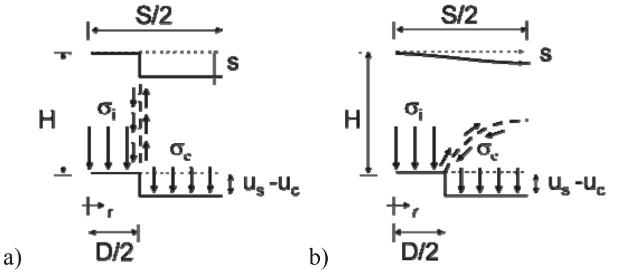


Figure 5. Failure within the embankment: (a) punching mechanism and (b) domed mechanism due to arch effect.

The pattern of superficial differential settlement s=s(r) could then be formally described by a transfer function, ranging from a discontinuous function (in case of punching), to a smooth function (in case of formation of the arch effect).

## 4 NUMERICAL ANALYSES ON SETTLEMENT PROFILE

In order to investigate the settlement distribution s(r) at the top of the embankment for increasing values of H, some preliminary finite difference numerical analyses have been performed by means of the commercial code FLAC. An axisymmetric geometry has been chosen in order to model the cell, and the simplifying hypothesis of rigid column has been assumed; the mechanical behavior of both the materials constituting the embankment and the soft soil has been modeled by assuming an elastic perfectly plastic relationship with a non-associated Mohr-Coulomb failure condition. For the embankment, two different types of material have been considered: a loose sand ($\phi$=20°, with no dilatancy) and a compacted sand ($\phi$=40°, with dilatancy $\psi$=10°). In both cases, for the sake of simplicity the unit weight $\gamma$ is assumed to be equal to 20 kN/m$^3$, the Young modulus equal to 3 MPa and the Poisson coefficient equal to 0.25. No friction has been considered at soil-column interface. For the soft soil, for the sake of simplicity a dry condition has been assumed (i.e. no hydro-mechanical coupling has been modeled). The values of the mechanical parameters are listed in Table 1. Consistently with the parametrical analyses previously discussed, the length L of the column is 10 m and its diameter D is 80 cm. Two ratios S/D have been considered, and the settlement distribution s(r) has been normalized at each stage for the current value of H.

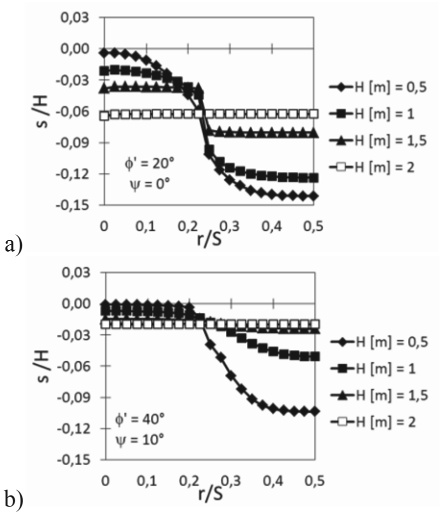


Figure 6. Normalized superficial settlements for S/D=2: (a) loose and (b) dense material.

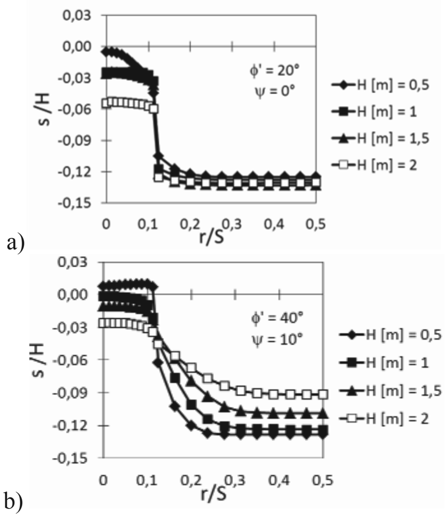


Figure 7. Normalized superficial settlements for S/D=4: (a) loose and (b) dense material.

Figures 6 and 7 describe the evolution of s(r) at the top of the embankment during the construction stages. It appears clearly that the settlement profile ranges from a well localized punching failure mechanism to a smooth distribution of settlements for increasing H (witnessing the progressive mobilization of the arch effect). The influence of relative spacing S/D and of the mechanical properties of the embankment (in terms of both $\phi$' and $\psi$) is opposite: an increase in S/D tends to localize the failure, whilst an increase in $\phi$' and $\psi$ tends to smoothen the settlement profile.

## 5 CONCLUSIONS

The paper critically discussed some results obtained according to the usual Design Standards, and proved that in some cases these approaches lead to unrealistic results. The codes, moreover, disregard the estimation of relative settlements at the top of the embankment, which is actually one of the most important parameters describing the efficiency of the foundation. A consistent, displacement based conceptual framework for describing the behavior of the system has been formulated, and some preliminary numerical analyses have been shown. These latter, in particular, showed on the contrary that the top settlement profile is remarkably affected by both the geometry and the mechanical properties of the embankment.

## 6 ACKNOWLEDGEMENTS

The Authors want to acknowledge TENCATE and ITASCA Italy for financially supporting the research.

## 7 REFERENCES

British Standard BS 8006. 1995. Code of practice for strengthened/reinforced soils and other fills. *British Standards Institution*, London, UK pp.176.

di Prisco C. and Galli A. 2011. Mechanical behaviour of geo-encased sand columns: small scale experimental tests and numerical modeling. *Geomechanics and Geoengineering: An International Journal* 6(4), 251–263

EBGEO. 2003. Empfehlung 6.9 (2003). Bewehrte Erdkörper auf punktoder linienförmigen Traggliedern, Kapitel 6.9 für die Empfehlungen für Bewehrungen aus Geokunststoffen, EBGEO, DGGT (German Geotechnical Society).

Galli A. and di Prisco C. 2011. Un modello concettuale per la progettazione di colonne granulari georinforzate a fondazione di rilevati artificiali (in Italian). *XXIV Convegno Nazionale di Geotecnica*, 231 – 246.

Ghionna V.N. and Jamiolkowski M. 1981. Colonne di ghiaia (in Italian). *X Ciclo di conferenze dedicate ai problemi di meccanica dei terreni e ingegneria delle fondazioni: Metodi di miglioramento dei terreni*. Politecnico di Torino, Atti dell'Istituto di Scienza delle Costruzioni, n.507, pp.1-63.

Jones C.J.F.P., Lawson C.R., Ayres D.J. 1990. Geotextile reinforced piled embankments. *Geotextiles Geomembranes and Related Products*, Den Hoedt (ed.) © 1990 Balkema, Rotterdam, ISBN 90 6191 119 2, pp 155-160.

Kempfert H.G. 2003. Ground improvement methods with special emphasis on column-type techniques. *Int. Workshop on Geotechnics of Soft Soils-Theory and Practice- SCMEP*.

Marston A. and Anderson A.O. 1913. The theory of load on pipes ditches and tests of cement and clay drain tile and sewer pipes. *Bulletin 31. Iowa Engineering Experiment Station*, Iowa State College, Ames, Iowa.

Moraci N. and Gioffrè D. 2010. La progettazione di rilevati su terreni compressibili rinforzati con geosintetici (in Italian). *Rivista Italiana di Geotecnica*, vol. 3/10, 67-100.

Murugesan, S. and Rajagopal, K. 2006. Geosynthetic encased stone columns: Numerical Evaluation. *Geotextiles and Geomembranes*, Vol. 24, 349-358.

Raithel M., Kirchner A., Schade C. and Leusink E. 2005. Foundation of Constructions on Very Soft Soils with Geotextile Encased Columns - State of the Art. *Proceedings ASCE Geo-Frontiers 2005*.

Zaeske D. 2001. Zur Wirkungsweise von unbewehrten und bewehrten mineralischen Tragschichten über pfahlartigen Gründungselementen, Schriftenreihe Geotechnik. Universität Kassel, Heft 10.