Numerical analisys of instrumented granular columns

Gregório L. Silva Araújo, Ennio Marques Palmeira & Renato Pinto da Cunha
Department of Civil and Environmental Engineering, University of Brasilia, Brasilia/DF, Brazil

ABSTRACT
Construction of embankments on soft soils can present stability problems as well as high post-construction settlements. Similar problems can occur also for embankments on collapsible soils. One recent construction technique to reduce or avoid these problems is the use of geotextile encased granular columns. This paper presents results of numerical analyses on the behaviour of conventional and encased sand columns in a collapsible soil using the Finite Element Method. Results of field load tests were used for comparisons with numerical predictions. The results obtained showed good agreement between measurements and predictions for the initial stages of the load settlement curve, but deviations regarding the strains in the columns. These deviations were in part caused by the column construction conditions.

RÉSUMÉ

Durante y después de la construcción de terraplenes sobre suelos blandos o sobre suelos proclives al colapso, pueden presentarse problemas de estabilidad y de asentamientos. Últimamente, es común el uso de columnas de material granular confinado como metodología de solución de este problema. Este artículo pretende mostrar un análisis numérico que mediante métodos de elementos finitos analiza el comportamiento de las columnas de arena, convencionales y confinadas, en suelos proclives al colapso. Por último y con el objetivo de validar el análisis numérico, se hicieron pruebas de carga en campo. Los resultados mostraron una buena concordancia de los datos predichos con los tomados de las curvas de carga vs asentamiento, construidas a partir de los ensayos de campo; y algunas diferencias entre los valores de las tensiones internas, medidas y modeladas, en las columnas de arena.

1 INTRODUCTION

The construction of an embankment on soft soil can be a complex task. This kind of construction can present some drawbacks such as stability problems and/or excessive deformations. The use of geosynthetic reinforcement has increased markedly to guarantee embankment stability. Some solutions that can be used to reduce soil settlements in such problems are partial or total soft soil removal, acceleration of soft soil consolidation using vertical drains and the use of piles. A more recent technique is the used of encased granular columns to reduce embankment settlements.

Due to the low lateral confinement provided by soft soils, the radial deformations of granular columns can be excessive yielding to vertical displacements of the top of the column, and reducing the column load capacity (Chummar, 2000). Hence, column encasement can avoid excessive bulging of the column.

Some constructions using this technique can be found in the literature. For example, Raithel et al. (2002) describe the construction of a dyke to protect the extension of the facilities of the Airbus 380 factory, in Hamburg, Germany. In this case, sand columns were encased by a strong and stiff woven geotextile. The performance of the column supported dyke was excellent. Other recent examples are the first South American embankment construction using encased granular columns in Sao Jose dos Campos City, Sao Paulo State, in Brazil (De Mello *et al.*, 2008) and the construction of the runway for the Atlantic Steel Company, in the Rio de Janeiro State, in Brazil (Alexiev & Moorman, 2009).

Stability problems and excessive vertical displacements can also occur in embankments on collapsible soils. This kind of soil is common in several parts of the city of Brasilia, the capital of Brazil and presents a structure breakdown phenomenon caused by loading and/or increase of moisture content. However, there are very few studies of applications of encased granular columns in this type of soil (Araujo *et al.*, 2009).

This paper presents a numerical study on encased granular columns in an unsaturated collapsible foundation soil.

2 LOADING TESTS ON GRANULAR COLUMNS

As part of a research programme on the behaviour of geotextile encased sand columns, loading tests were carried out on such columns in the Foundation Experimental Site of the Graduate Programme of Geotechnics of the University of Brasília. The columns tested were 0.4 m in diameter and 8.0 m long. A woven geotextile, made of polyester was used as encasement. The tensile stiffness and tensile strength of the geotextile used were equal to 2000 kN/m and 200 kN/m, respectively. The methodology employed in the loading tests on the granular columns followed the procedure established by the Brazilian standard NBR 12131 for pile loading tests. Displacements of the top of the column were measured by displacement transducers and the applied loads were measured by a load cell. Besides, six strain gauges were positioned along the column length, aiming at assessing the column deformation during the tests. At the depths of 1.0 m and 4.5 m, horizontal strain gauges were also installed. The strain gauges consisted of electric strain gauges with the ends anchored in epoxy disks. Figure 1 shows the locations of the strain gauges in the column.

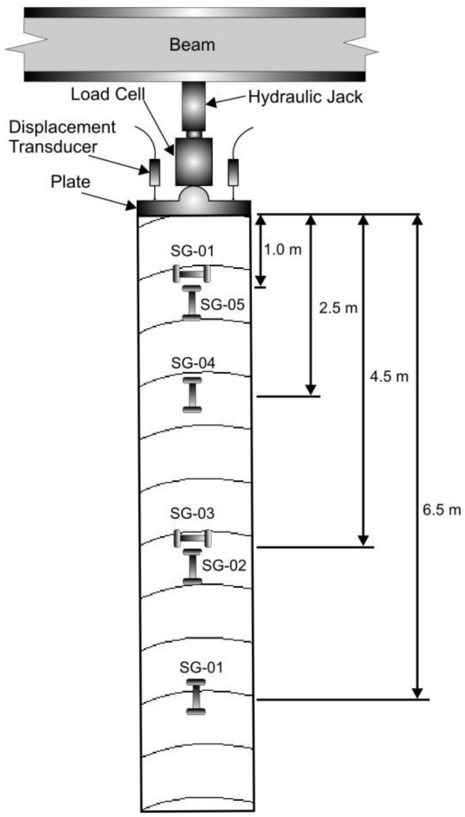

Figure 1. Strain-gauge location.

The methods by Brinch-Hansen (1970), Chin (1970), Decourt (1996) and Van Der Veen (1953) were used to obtain the column load capacity from the results of the field loading tests. The load capacity of the columns increased between 66% and 81% due to geotextile encasement, depending on the method considered (Araujo 2009, Araujo *et al.* 2010).

3 NUMERICAL ANALYSIS

The finite element code used in the back-analyses of the field tests was PLAXIS 7.2 (Brinkgreve & Vermeer, 1998). Axi-symmetric conditions were assumed because each column tested was in isolation. Initially, the influence of the size of the finite element mesh was investigated to minimize boundary effects. Displacements at the lateral boundaries were allowed only along the vertical direction, whereas no displacement was allowed along the bottom boundary. The mesh was refined in the region of the column-soil interface to increase the accuracy of the predictions (Araujo, 2009). Prescribed vertical displacements of the column top were imposed. Figure 2 shows the finite element mesh employed. The *hardening-soil* model present in Plaxis was used for the sand of the column. The *soft-soil* model was used to simulate the collapsible foundation soil.

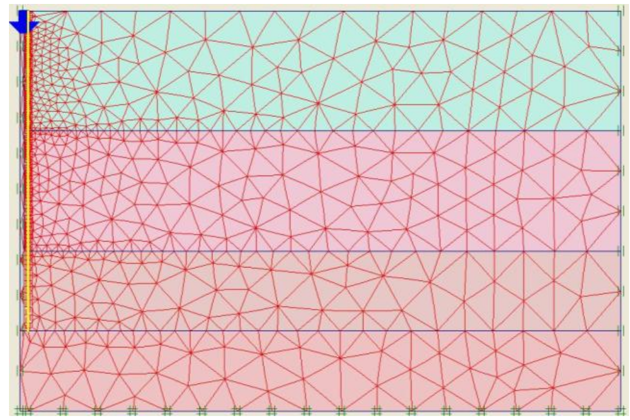

Figure 2. Geometry and finite element mesh of the problem.

For the simulation of the encasement, a geotextile element was used. This type of element is available in the Plaxis code. Interface elements were used along the soil-geotextile interfaces.

The geotechnical parameters used for the unsaturated collapsible clay were based on 15 years of researches in the experimental site, where different testing methods were investigated. Based on these previous studies the collapsible foundation soil was divided in three layers. The first and second layers are 3 m thick and the last one is 2 m thick. Underneath the latter layer a rigid layer was assumed, also based on the field test results in the region.

Tables 1 to 3 present the parameters used in the numerical analysis. Some of them were obtained based on field and laboratory tests (Guimarães, 2002 and Mota, 2003). In this paper, the Young modulus and the oedometric modulus of the soil were varied until the best comparisons between predictions and measurements were reached.

Table 1. Geotechnical properties of the porous soil.

Layer	γ_s (kN/m^3)	γ_d (kN/m^3)	γ (kN/m^3)	e_0
01	26,58	10,70	13,88	1,58
02	26,22	11,80	13,88	1,31
03	26,39	13,34	18,29	1,03

Notes: γ_s: unit weight of soil particles; γ_d: dry soil unit weight; γ: soil unit weight; e_0: initial void ratio.

Table 2. Deformability and stress history parameters of the porous soil used in the numerical analysis.

Layer	C_c	C_s	λ^*	κ^*	k_0	OCR
01	0,54	0,022	0,090	0,0058	0,36	1,8
02	0,46	0,018	0,087	0,0054	0,45	1,1

03	0,28	0,016	0,061	0,0054	0,54	0,9

Notes: C_c:compression index; C_s: swelling index; λ^*: modified compression Index; κ^*: modified swelling index; k_0: at rest earth pressure coefficient; OCR: over consolidation ratio.

Table 3. Strength parameters of the porous soil in the numerical analysis.

Layer	c`(kPa)	ϕ`(^{0})
01	30	26
02	19	30
03	37	26

Notes: c': cohesion; ϕ': friction angle.

Table 4 presents the values of the sand parameters used in the numerical analysis and Table 5 presents the parameters used in the analysis of the encased column for best fit between predictions and measurements.

Table 4. Sand parameters of conventional sand column in the numerical analysis.

c' (kPa)	ϕ' (^{0})	ψ (^{0})	E_{50}^{Ref} (kPa)	E_{oed} (kPa)	γ (kN/m^3)	R_{inter}
0	42	15	28.000	32.000	16	1

Notes: c': cohesion; ϕ': friction angle; ψ: angle of dilation; E_{50}^{Ref}: Young modulus at 50%; E_{oed}: oedometric modulus; γ: soil unit weight; R_{inter}: interface parameter.

Table 5. Encased sand column parameters of the numerical analysis.

c' (kPa)	ϕ' (^{0})	ψ (^{0})	E_{50}^{Ref} (kPa)	E_{oed} (kPa)	γ (kN/m^3)	R_{inter}
0	42	15	32.000	36.000	16	0,8

Notes: c': cohesion; ϕ': friction angle; ψ: angle of dilation; E_{50}^{Ref}: Young modulus at 50%; E_{oed}: oedometric modulus; γ: Soil unit weight; R_{inter}: Interface parameter.

4 RESULTS OBTAINED

Figure 3 shows predicted and observed load settlement curves for tests on conventional sand columns (without geotextile encasement). The results show that there is a good agreement between predicted and measured values up to a load of 40 kN. After 40 kN, deviation between results can be noted, which can be in part due to the increase of the earth pressure coefficient because of radial column deformation (as suggested by Domingues *et al.* (2007). However, the influence of some simplifications adopted in the modeling technique employed cannot be ruled out.

Figure 4 presents predicted and measured strains in the location of some strain gauges in the test on the conventional sand column. Some deviations can be noted, although for the complexity of the conditions in the field the comparisons might be considered rather satisfactory. Field and numerical results showed that the strains in the column are larger close to its top.

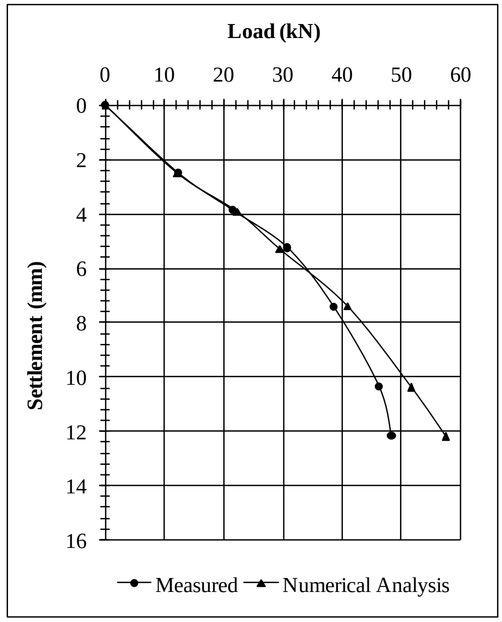

Figure 3. Comparison between measurements and predictions for the test on the conventional sand column.

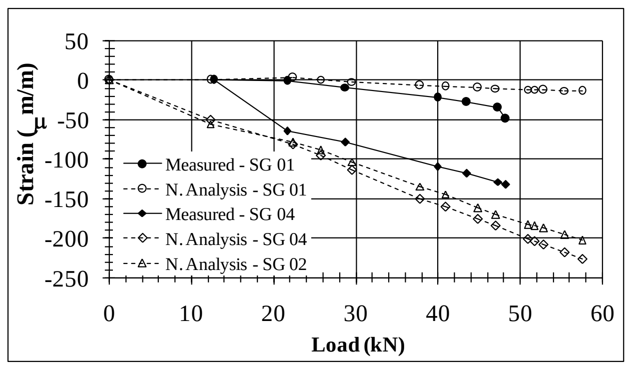

Figure 4. Vertical strains measured by strain gauges 01, 02 and 04 of the conventional column.

Figure 5 shows the results of radial deformations in the test on the conventional column. It was observed that the predicted and the measured strains in the central portion of the column (strain-gauge 3) compared well. However, a great deviation between results can be observed for strain-gauge 6. This was due to mal functioning of this strain gauge. The lateral deformation at the top of the column is greater than that at its central part due to a lower confinement of the column close to the groung surface.

Figure 6 illustrates comparisons between predicted and observed load-settlement curves for the geotextile encased column. For loads below 62 kN there is a good agreement between the two curves. However, deviation between these curves occur beyond that value. This in part can be attributed to the fact that the encased column was not in direct contact with the surrounding soil along its length.

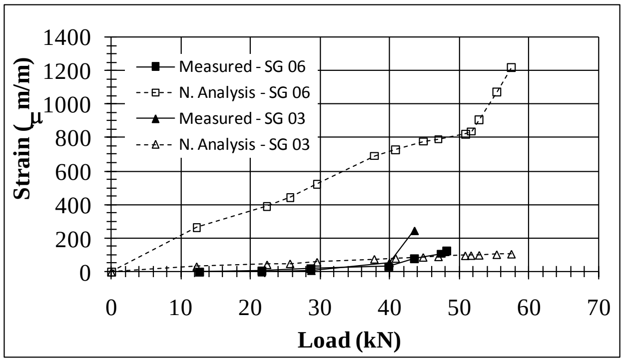

Figure 5. Radial strains for the conventional column.

The diameter of the hole executed in the ground to construct the column was slightly greater than the outer diameter of the column. In addition, the non uniformity of the hole diameter along the depth cannot be ignored as well. Figure 7 shows the execution of the hole in the ground for one of the columns tested and Figure 8 shows an example of a gap between the column and the hole wall. The investigation of these aspects were beyond the objectives of this study.

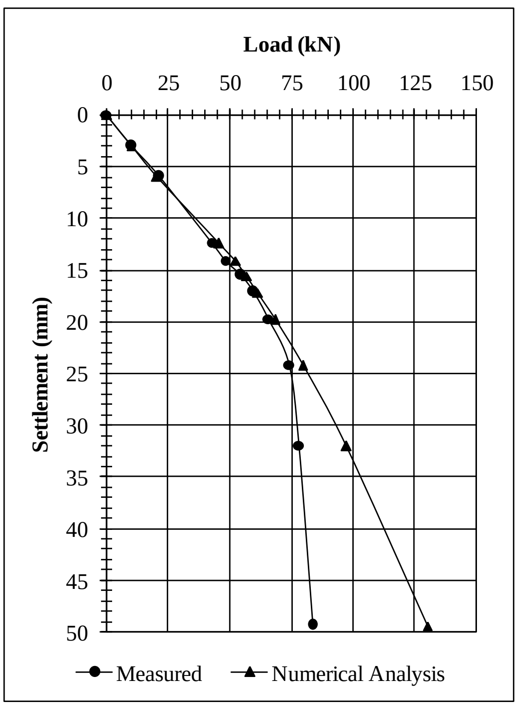

Figure 6. Comparison between predictions and measurements for the test on the encased sand column.

Figure 9 shows predicted and measured vertical strains of the encased column for strain gauges 1, 2 and 4. Again, large deviations can be observed, which shows the limitations of predicting column strains using routine numerical analyses.

The same applies for the comparisons between radial strains measured by strain-gauge 6 in the test on the encased column, as shown in Figure 10.


Figure 7. Column borehole execution.


Figure 8. Gap between encased column and borehole due to the column execution process.

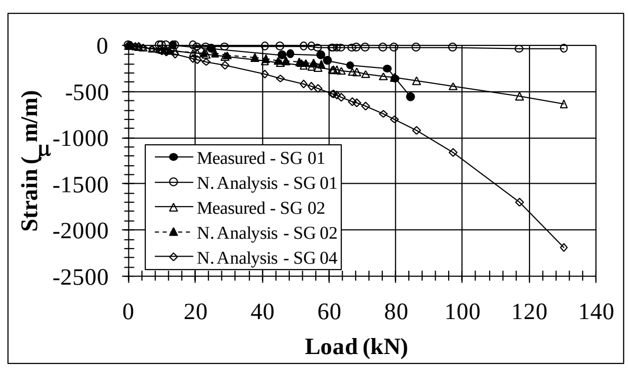

Figure 9. Vertical strains of strain gauges 01, 02 and 04 of the encased column.

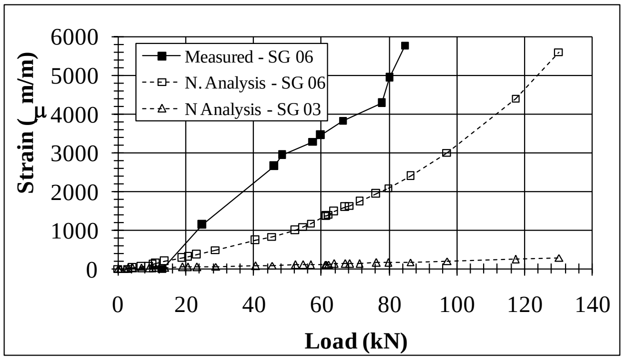

Figure 10. Lateral strains for the encased column.

5 CONCLUSIONS

This paper presented comparisons between predictions and measurements regarding tests on two granular columns in an unsaturated tropical collapsible soil. One of the columns was encased by a woven geotextile and the other was constructed in the conventional way. The main conclusions of this study are summarized below.

- Geotextile casing increases the load capacity of the granular column. This was observed in both the numerical study and in the field loading tests performed.
- With proper adjustment of some soil parameters, a reasonably good comparison between numerical predictions and measurements for load-settlement curves could be achieved in the early stages of the tests. However, deviations between predictions and measurements were significant at the later stages of the tests, particularly for the geotextile encased column.
- Large deviations between predicted and measured strains in the columns were observed.
- To some extent, the deviations between predictions and measurements for the encased column may be associated with the lack of full contact between the column and the borehole surfaces. Gaps present certainly influenced the encased column behavior and that was not modeled in the present study. Further investigation is under way trying to simulate the influence of these gaps.

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