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ABSTRACT

Soils with a large percentage of fines (silt and clay) are considered of marginal quality for the purposes of their use as backfill in reinforced soil structures because they exhibit poor drainage capacity. Accordingly, such soils are not allowed in the US, at least for reinforced retaining walls and steep slopes constructed by public transportation agencies. However, in spite of the significant caution against the use of such soils in the US, reinforced soil structures in Brazil have often been built using soils with a large percentage of fines. Indeed, the reported performance of these structures, many of them with field instrumentation, has shown a very good long-term performance. The good performance can be attributed to the significantly different characteristics of fine-grained soils in the US and Brazil. Specifically, most of the fine-grained soils used as backfill material in Brazil are residual soils, and often lateritic soils, which have shown excellent performance in engineered embankments. Accordingly, existing guidelines for reinforced soil construction should be refined as the sole use of grain size distributions to define the adequacy of backfill soils may be oversimplified. This paper presents an overview of Brazilian case histories involving the construction of reinforced walls and steep slopes using poorly draining soils, and documents the basis for their design, aspects of their construction, and their long-term performance. Some of the structures built using poorly draining soils are now over 20 years old and show no signs of distress.

INTRODUCTION

Soil reinforcement is now a common design alternative in Latin America for the construction of retaining walls and steep slopes. This is because of reduced costs of the reinforced alternatives as well as because of their excellent long-term behavior when compared to that of conventional retaining structures. The behavior of reinforced soil structures is a function of the properties of both the reinforcing elements and of the geotechnical characteristics of the backfill soil.

High shear strength and adequate (free) drainage capacity are the typical requirements expected from the soil selected as backfill for reinforced soil structures. The need of good drainage is because backfill materials must be capable to quickly dissipate any pore water pressures that may developed both during construction and throughout the lifetime of the structure. Granular soils fully attend these two design requirements regarding strength and drainage. A review by Zornberg and Leshchinsky (*1*) of current design guidelines worldwide indicates that most countries that explicitly establish criteria based on grain size distribution end up indicating very stringent requirements regarding the maximum allowable percentage of fine-grained material.

However, in Brazil and other countries of tropical climate, granular materials are typically not readily available in the vicinity of typical construction sites. Indeed, poorly draining soils (i.e. silts and clays) cover large areas of the Brazilian territory. Consequently, the use of granular material may become prohibitive because of the significant hauling costs. Unlike the typical fine-grained soils in countries with temperate climate such as the US, most of the fine-grained soil deposits in Brazil are of residual, and often lateritic, origin. Consequently, in spite of their comparatively lower drainage capacity than free-draining granular soils, they present high shear strength and low compressibility, which makes them an excellent backfill material when compared to more conventional clay soils.

Many reinforced soil structures designed and built in Brazil were constructed using poorly draining backfill soils. Some of these structures have been even instrumented. The overall long-term performance of these structures has been reported to be excellent. Specifically, the measured and observed vertical and horizontal displacements of the wall face have been negligible, and the overall systems have not shown signs of distress. A review and documentation of these systems is timely as there is a current reevaluation not only in Brazil, but also in the US, regarding the need of stringent grain size distribution requirements for backfill material.

The objective of this paper is to evaluate the performance of some of the Brazilian reinforced soil structures. The analysis of these case histories involving reinforced soil walls and steep slopes built with marginal soils, can provide significant insight regarding the adequacy of current guidelines for backfill material in the US and the potential need of such guidelines in Brazil. The paper also discusses the potential use of permeable inclusions, which may be incorporated within the fill mass to facilitate the dissipation of the excess pore water pressures caused by infiltration.

TECHNICAL GUIDELINES FOR REINFORCED SOIL STRUCTURES

Guidelines are provided by the Federal Highway Administration (FHWA) (*2*) indicate that backfill soils for reinforced soil walls should must be free from organic and deleterious materials and should conform to the stringent grain size limits indicated in Table 1. The AASHTO Manual (*3*) specifies free drainage backfill that should exclude any type of expansible soils. The manual indicates that silts and clays should not be used in permanent structures.

An additional international reference regarding reinforced soil construction are the British Standards (*4*), which provide design criteria for permanent reinforced soil backfills. Cohesive soils are not allowed by the British Standards for structures of Categories 2 and 3, as described in Table 2.

TABLE 1 Grain Size Limits for the Soils Used in Reinforced Soil Walls According to FHWA

U. S. Sieve size	Percent passing
102 mm	100
0.425 mm (n° 40)	0 – 60
0.075 mm (n° 200)	0 – 15

TABLE 2 Categories of Reinforced Soil Structures According to BS 8006

Category	Examples of structures
1 (Low)	Retaining walls smaller than 1.5m.
2 (Medium)	Retaining walls where the failure could result in a moderated damage
3 (High)	Bridge abutments, retaining walls that support directly main roads, railroads and dams.

The aforementioned guidelines apply for the design and construction of public projects in the US and the UK. Private projects in these countries typically have fewer restrictions and often specify soils with a larger percentage of fines.

Transportation agencies in Brazil have not issued guidelines regarding the selection of backfill soils for geosynthetic reinforced structures. This has often created controversial situations because Brazilian engineers either follow international recommendations, such as those issued by FHWA, AASHTO or British Standards that pose stringent backfill requirements, or follow local experience, which recognize the good mechanical properties of residual soils. Accordingly, a reevaluation is made in this paper of structures that have been constructed in Brazil using criteria that do not meet current US standards.

POORLY DRAINING SOILS IN BRAZIL

Most of the Brazilian territory is covered by fine-grained soils (silts and clays). It is estimated that approximately 60% of the total area of the country is constituted by fine-grained soils, with a large percentage of tem being of residual origin. That is, these soils are the product of in-situ weathering of the original rock, which is typical of regions with tropical climate. When compared with soils from temperate climate of similar same grain size distribution, the fine-grained soils from tropical regions show comparatively better mechanical properties. Among the various attempts to establish an appropriate classification for tropical soils, Nogami & Villibor (*5*) developed the MCT classification to attend the geotechnical peculiarities of tropical compacted soils. This classification takes into account, in addition to the grain size distribution, aspects such as workability of the soil and its mineralogical and structural characteristics. Two distinct soil types are defined in this classification that account for the soil genesis: lateritic soils and saprolitic soils.

Lateritic soils constitute the most superficial layer of tropical, well-drained regions. They are characterized by red and yellow colored soils, reaching horizons with a thickness of over 2.0 m. The particles of these soils are mechanically and chemically resistant to the various agents (sand and gravel fractions) and the clay fraction is constituted essentially by kaolinite. Clay particles show low potential for volumetric changes. Soil particles are covered and agglutinated by hydroxides and oxides of iron and aluminum forming aggregates with dimensions ranging from several microns to a few centimeters. The strength of these materials under dry dried condition is very high, mainly due to the action of available cements (*6*).

The saprolitic soils often constitute the underlying layers of the lateritic soils, with horizons that reach a thickness of dozens of meters. Its mineralogical constitution shows a significant number of minerals. These soils show a high percentage of silt-size particles that behave differently than typical silts from temperate climate regions. This is because these saprolitic soil particles are essentially inert, as they contain kaolinite micro-crystals and mica that shows some plasticity even without the presence of clay-size particles (*7*).

In spite of showing a comparatively large percentage of fines, the typical shear strength parameters of tropical soils are comparatively high. For illustration purposes, Table 3 presents shear strength parameters of soils used in dam construction in the Southern areas of Brazil (*8*). The magnitude of the cohesion and friction angle exhibited by soils listed in Table 3 are typical of many compacted lateritic soils found throughout Brazilian territory.

TABLE 3 Shear Strength and Pore Pressure Parameters of Residual Soils Used in the Construction of Dams in Southern Areas of Brazil

Soil (parent rock)	ø (°)	c´(kPa)	$\bar{B}_{opt}$ (%)
Sandy soils (sandstone)	30 - 35	0 - 20	5 - 20 (5 - 10 usual)
Silts/clays (granite/gneiss)	26 - 32 (30 - 32 usual)	0 - 40	6 - 10 (silt) 20 - 45 (clay)
Silts/clays (filite/siltstone/claystone)	23 - 29	0 - 25	5 - 10 (silt) 8 - 25 (clay)
Clays (Basalts)	24 - 31	10 - 70	16 - 35

PORE WATER PRESSURES IN REINFORCED SOIL BACKFILLS

Significant problems are associated with the use of marginal soils in reinforced soil construction. The use of comparatively wet soils leads, for example, to construction problems associated with compaction difficulties during placement. However, the most serious concerns are related to stability problems associated with the potential development of pore water pressures or loss of strength due to wetting within the reinforced fill mass. Christopher et al. (9) identified three adverse conditions of pore water pressure generation and/or loss of strength due to wetting are of concern when reinforcing poorly draining backfills. These conditions, illustrated in Figure 1, are as follows:

- *Condition (a): Generation of pore water pressures within the reinforced fill.* When fine grained, poorly draining soils are used in reinforced soil construction (particularly if placed wet of optimum moisture), excess pore water pressure can develop during compaction, subsequent loading, and surcharging. The designer must then account for these pore water pressures for the evaluations of stability and consolidation-induced settlements.
- *Condition (b): Wetting front advancing into the reinforced fill.* This is the case for fills placed comparatively dry (i.e. no pore water pressure generation is expected during construction). However, loss of soil shear strength may occur due to wetting of the backfill soils as a consequence of post-construction infiltration. This loss of strength due to wetting could be expected, even if no positive pore water pressures are generated and no seepage flow configuration is established within the fill.
- *Condition (c): Seepage configuration established within the reinforced fill.* Seepage flow may occur within the reinforced soil mass, for example, in the case of sliver fills constructed on existing embankment side slopes and cut slopes in which infiltration occurs from the adjacent ground. Significant seepage forces may occur either during rainy or spring thaw seasons. Water level fluctuations and rapid draw down conditions can also induce seepage forces in structures subjected to flooding or constructed adjacent to or within bodies of water. Seepage forces may also occur during ground wetting, inducing an additional destabilizing effect to the loss in shear strength described by Condition (b).

The pore water pressures generated under the Condition (a) have been evaluated for the case of tropical soils using the parameter $\overline{\mathbf{B}}$,which relates the pore water pressures and the vertical total stress. Cruz (*8*) summarizes the values of $\overline{\mathbf{B}}_{opt}$ ($\overline{\mathbf{B}}$ at optimum moisture content) from various soils from southeastern areas of Brazil (Table 3). As can be observed by inspection of these values, with exception of clays, average values of $\overline{\mathbf{B}}$ at optimum moisture content are comparatively small. For most soils, the ranges of $\overline{\mathbf{B}}$ with moisture content can be significant, especially when the soils are compacted with a moisture content dry of optimum moisture (according to Standard Proctor). That is, for soils compacted dry of optimum, small variations in the moisture content will result in significant variations of $\overline{\mathbf{B}}$. With few exceptions (basalt plastic clays) values of $\overline{\mathbf{B}}$ are very small (generally less than 0.1) if the backfill soils are compacted dry of optimum. In some situation this value can even be negative. Consequently, pore pressures generated during construction using tropical soils are very small and usually can be neglected.

Regarding the condition involving infiltration of moisture, it should be noted that there is only small amount of reliable data in Brazil involving water infiltration on natural or compacted slopes in Brazil. Piezometer data available from the few instrumented reinforced backfills built accross the country show very small pore pressure variations throughout the year, some of these readings have been even negative. Available data of water infiltration on natural slopes in Brazil, obtained from finite element simulations, show that the depth and volume of water that infiltrates the soil profile depends on the magnitude of the hydraulic conductivity, and on the rain intensity and duration (*10*). In clay soils displaying hydraulic conductivity above 10^{-07} m/s, water infiltration concentrates at the topmost 2m of the slope even for an intense rain with duration of 50 hours.

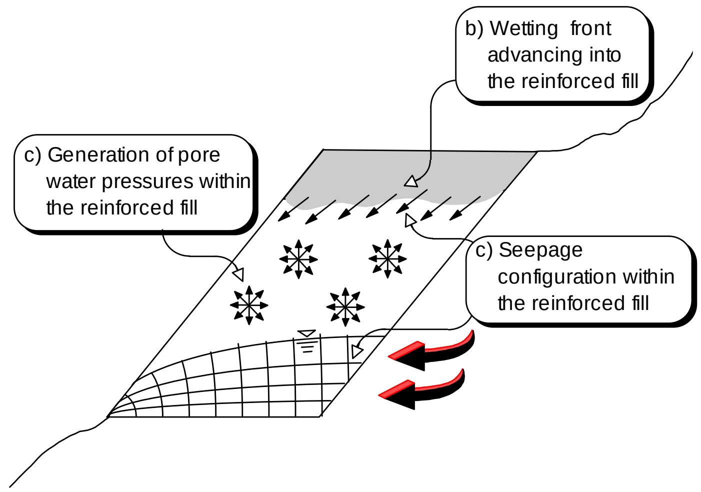

FIGURE 1 Different conditions for the generation of pore water pressure in poor draining backfills (*9*)

DESIGN OF REINFORCED STRUCTURES WITH POOR LYDRAINING SOIL

Design of structures built with poor draining soils requires analysis at the end of construction (short-term analysis), after the total dissipation of pore water pressure (long term analysis) and at intermeadiary periods (*11*). The soil parameters to feed short term analysis can be obtained either from triaxial UU tests (assuming undrained condition) or in terms of effective stresses (drained tests or undrained triaxial tests with the measurement of pore pressures). The analysis in term of effective stresses requires a reliable estimate of the distribution of the field pore water pressures inside the reinforced soil mass.

The long term analysis is always performed in drained conditions. Laboratory tests have either to simulate the total dissipation or to be performed at slow deformation rate to allow the measurement of the pore water pressure in order to get the effective stresses acting in the soil sample. The stability analysis at intermeadiary periods are generaly performed in effective stress term thus requiring the use of efective shear strength prameters and a reliable estimate of filed pore pressures. Table 4 (*11*) resumes the form of analysis and the soil parameters used to represent site conditions.

TABLE 4 Choose of Stability Analysis of Reinforced Backfills (Total and Effective Stress Methods)

Situation	Preferred method
1. End of construction, saturated soil. Short construction time.	$ø = 0$; $c = S_u$;
2. Stability at intermediary periods	$c´$ e $ø´$ with estimates of pore water pressures
3. End of construction, partly saturated soil.	c_u e $ø_u$ from UU tests or $c´$ and $ø´$ plus reliable estimates of pore water pressures.
4. Long term stability	$c´$ and $ø´$; Analysis with pore water pressure given by the equilibrium ground water conditions

For the specific case of reinforced soil structures, Christopher et al. (*5*) proposed that the analysis should account for the three adverse conditions of potential pore water pressure generation. The general design philosophy proposed herein is to consider a two-phase evaluation:

- *Analysis (i)* in each adverse condition is performed ignoring the drainage contribution provided by the reinforcements. This is a total stress analysis which considers that stability is mostly provided by the reinforcements with minimum contribution of the soil shear strength. Due to the conservative nature of this assumption, a relatively low design factor of safety is suggested.
- *Analysis (ii)* in each adverse condition is performed accounting fully for the drainage contribution provided by the reinforcements (i.e. zero pore water pressure is considered within the reinforced fill for analysis purposes). Considering that no pore water pressures are assumed to develop, this is an effective stress analysis. Design factors of safety used in conventional engineering practice are considered in this case.

These analyses are summarized in Table 5. They involve the following:

- Total stress analysis ignoring reinforcement lateral drainage. This analysis neglects the dissipation of pore water pressures through the permeable inclusions to provide a conservative estimate of the stability of the structure at the end of construction. Considering the short-term condition and the conservative assumptions in this analysis, a factor of safety of 1.1 is recommended. This analysis determines minimum reinforcement requirements that will preclude collapse during construction of the structure. That is, it provides reinforcement requirements for a short-term situation in which stability is provided mostly by the tensile forces in the reinforcements with only a minor contribution by the undrained shear strength of the backfill. The undrained soil shear strength of the backfill for this analysis should be based on unconsolidated undrained (UU) triaxial tests. The specimens should be prepared at representative field densities and moisture placement conditions, and tested at these placement conditions under project-specific confining pressures.
- Effective stress analysis accounting for full lateral drainage by the reinforcement. Full drainage of the reinforced fill is assumed for the long-term conditions. This analysis provides a realistic evaluation of the long-term stability of the structure, because dissipation of pore water pressures generated during construction should have occurred through the permeable inclusions. This analysis determines the minimum reinforcement requirements that will provide adequate stability under long-term conditions following dissipation of pore water pressures generated during construction of the structure. It is emphasized that the transmissivity of the reinforcements should be selected so that generation of pore water pressures is prevented at the soil-reinforcement interface. Typically, the soil shear strength should be based on isotropically consolidated undrained (CIU) triaxial tests performed on saturated samples with pore pressure measurements or on consolidated drained (CD) triaxial tests. The long term design factor of safety typically required for reinforcement of granular fills (e.g. 1.3 to 1.5) should be used in this analysis.

Table 5 Summary of Analyses for Reinforced Soil Structures with Poorly Draining Backfills (*5*)

Condition	***Characteristics***	***Analysis i: Ignoring lateral drainage***	***Analysis ii: Accounting for full drainage***
a) Generation of pore water pressures within reinforced fill	Type of analysis:	Total Stress	Effective Stress
	Case:	Generation of pore pressures due to short-term loads	Long-term drained condition due to lateral drainage
	Design Criteria:	FS = 1.1	FS = 1.3 to 1.5 (*)
	Reinforcement Transmissivity:	Ignored in analysis	Conveys fully the flow from consolidation process
	Soil shear strength:	ϕ and c from UU tests. Specimen condition: as placed	ϕ' and c' from CIU or CD tests. Specimen condition: saturated
b) Wetting front advancing into reinforced fill	Type of analysis:	Total Stress	Total Stress
	Case:	Loss of shear strength due to soaking	Unsaturated condition maintained due to permeable reinforcements
	Design Criteria:	FS = 1.1	FS = 1.3 to 1.5 (*)
	Reinforcement Transmissivity:	Ignored in analysis	Prevents advancement of wetting as defined by testing
	Soil shear strength:	ϕ and c from CIU tests. Specimen condition: saturated	ϕ and c from CIU or CD tests. Specimen condition: highest anticipated moisture
c) Seepage flow configuration established within reinforced fill	Type of analysis:	Total Stress	Effective Stress
	Case:	Development of seepage forces within fill	Saturation of fill, without development of seepage forces due to permeable reinforcements
	Design Criteria:	FS = 1.1	FS = 1.3 to 1.5 (*)
	Reinforcement Transmissivity:	Ignored in analysis	Conveys fully the seepage flowing into the backfill
	Soil shear strength:	ϕ and c from CIU tests. Specimen condition: saturated	ϕ' and c' from CIU or CD tests. Specimen condition: saturated

(*) Design criteria for Analysis (ii) should be selected based on design guidelines for reinforced soil structures with granular backfill.

REINFORCED SOIL STRUCTURES INVOLVING THE USE OF POORLY DRAINING SOILS IN BRAZIL

Some examples of retaining walls and steep slopes built using poorly draining soils in Brazil are presented in this section. So far, all these structures are reported to behave very well although not all of them were fully instrumented to collect internal displacements, total stresses and pore water pressure. The cases where an instrumentation monitoring program has been implemented, results obtained during and after construction show that most movement took place only during the construction phase. The results also show that pore pressures developed during construction are typically very small. Post-construction horizontal end vertical movements have also been reported to be very small and tend to stabilize a short period of time after construction. The main projects involving reinforced steep slopes and reinforced walls are detailed next.

Reinforced Steep Slopes

The first reinforced soil structure ever built in Brazil was a reinforced steep slope with a facing inclination of 1H:2V. This structure was constructed in 1984 along highway SP-123, which links Taubaté to Campos de Jordão, in São Paulo state. Figure 2 shows a view during construction and a cross section of the system. The 10 m high reinforced structure is the central part of the reconstruction project involving the repair of a failed road slope with a height of approximately 30 m. The reinforced zone involved 500 m² of front face and was built using sandy silt material as backfill material. The grain size distribution of the soil would not meet current FHWA guidelines. The vertical spacing between reinforcing elements was 0.60 m. Two different geotextiles were used in this project: a woven geotextile manufactured of slit polypropylene (PP) tape and a nonwoven, needlepunched geotextile made of continuous Polyester (PET) filaments (*12*). For comparison purposes, the two geotextile reinforcements that were selected for this project showed a similar unconfined ultimate tensile of 35 kN/m.

The project was instrumented using piezometers, inclinometers, earth pressure cells and tell-tails. This allowed gathering important variables on the response of both portions (woven and nonwoven) of the reinforced soil structure. The instrumentation data showed that most of the vertical and horizontal movements occurred during the construction phase. Also, the collected data indicates that the horizontal movements of the slope face collected on the portion of the reinforced slope that used woven geotextiles were larger than those collected in the zone reinforced with nonwoven geotextiles (Figure 30. Specifically, the maximum horizontal displacement obtained after construction corresponds to ratios displacement/height (δ_h/H) of 1.2% and 0.6% for the woven and nonwoven zones, respectively.

Pore pressure data collected for this project indicated only very small values, with the pore water pressures often having negative values. The excellent behavior of the nonwoven geotextile was attributed to the effect of soil confinement on the mechanical properties of the geotextile reinforcements. Instrumentation data was subsequently reviewed by Ehrlich et al. (*13*), who confirmed the excellent geotechnical long-term response of the reinforced structure after 10 years of service life.

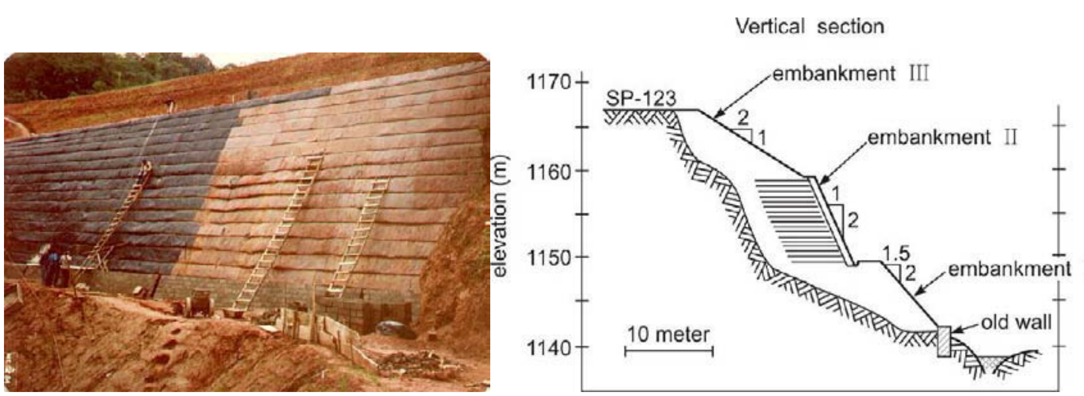

FIGURE 2 Pioneering reinforced soil structure built in Brazil: (a) front view; (b) cross section.

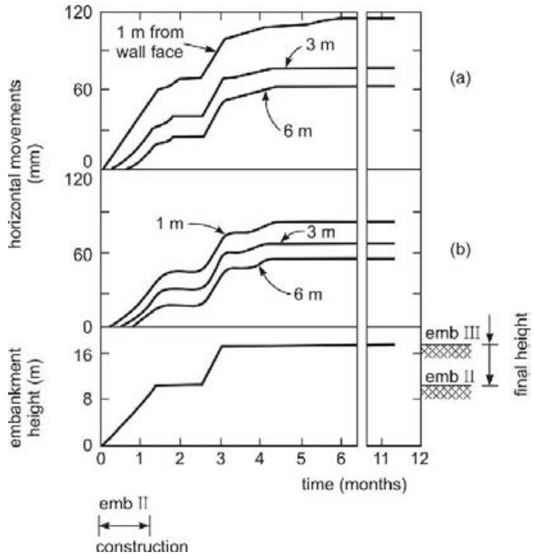

FIGURE 3 Horizontal movements of the woven and nonwoven reinforced zones at three different locations (1, 3 and 6 m from the wall face): (a) woven; (b) nonwoven.

A second reinforced steep slope project is a structure built as part as part of a slope protection in the city Presidente Epitáceo, state of Sao Paulo. The structure was constructed using silty clay as backfill material. The 7.0 m high steep slope was reinforced using nonwoven needlepunched geotextiles. The reinforcements were 4.8 m long and had a non-uniform vertical spacing, ranging from 0.3 m at the base to 0.7 m at the top of the slope (Figure 4). The facing was protected using shotcrete. The backfill soil had a shear strength characterized by an effective cohesion of 10 kPa and an effective friction angle of 29^o (*14*). The project was instrumented using tell-tails placed to monitor the internal displacements within the reinforcement layers. Specifically, these instruments were placed at two horizontal levels (elevations 251.1 m and 253.5 m) and at two vertical sections (D and F). Also, settlement plates were used to monitor the internal vertical displacements.

The results of horizontal displacement obtained at elevation 251.1 m at the end of construction reached a maximum value of 12 mm at section F, while the horizontal displacements at elevation 253.5 m showed a maximum value was of 25 mm. The ratios δ_h/H for these two cases corresponds to 4.8 and 3.9%, respectively.

Figure 5 shows the internal horizontal displacements of sections D and F (elevation 251.1 m) measured during and after construction. As shown in the figure, most of the movements were very small and occurred only during construction. The horizontal displacements reached a maximum value of approximately 10 mm and took place observed within the reinforced fill, at a distance of 1 m from the front face. Post construction movements were very small, with a maximum horizontal displacement of only 2 mm obtained after 40 days.

A third reinforced steep slope project built in Brazil using poorly draining soils is the reinforce slope built along Highway BR 381, which links the city of São Paulo to Belo Horizonte. Figure 6 shows a cross-section of this reinforced steep slope (*15*). This structure is 18 m high and 270 m long. The decision to build a geosynthetic-reinforced soil structure was based not only on cost, but also on the limited space behind the slope face, which was insufficient for a conventional soil slope.

This reinforced steep slope was subdivided into three distinct sections, each 6.0 m high. The facing inclination is 1H:2V, and the structure includes 3 m wide berms. A 10 m high unreinforced slope with a facing inclination of 2H:3V was built on the top of the reinforced soil structure. Overall, the total height of the composite structure was 28 m (Figure 6). The facing of the reinforced slope was built using soil cement bags. A drainage layer was built at the back of the reinforced soil compacted zone along the natural retained soil.

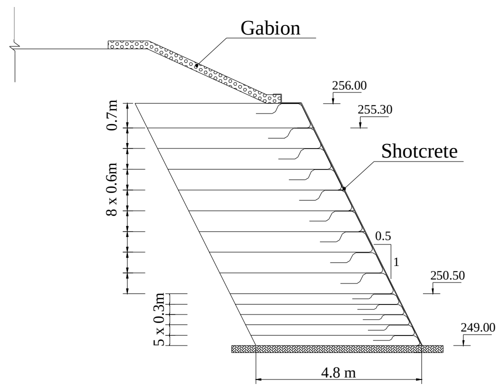

FIGURE 4 Cross section of a reinforced soil that was structure constructed in Presidente Epitáceo, Sao Paulo, Brazil.

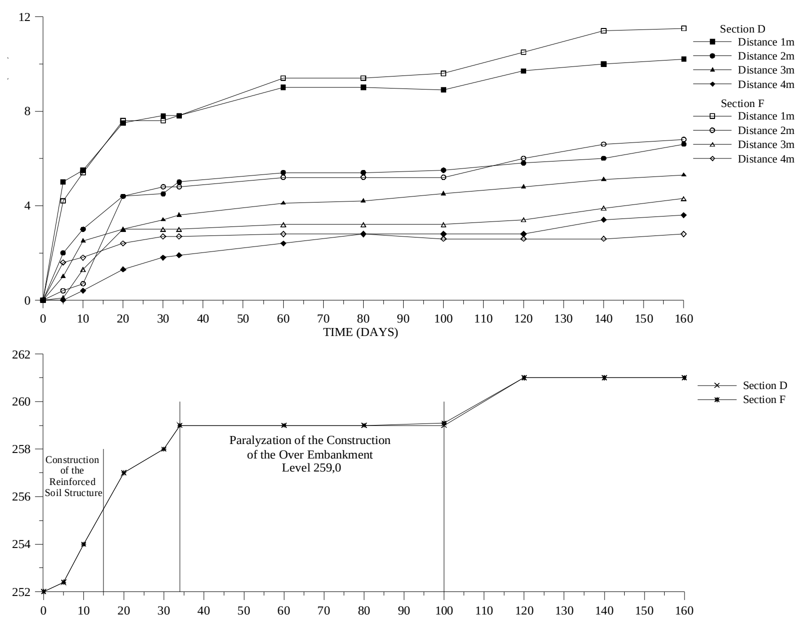

FIGURE 5 Horizontal movements collected during and after construction in two adjacent sections (elevation 251.1m).

The soil used as backfill material involved two types of materials. Crushed stone was placed at the bottom portion of the slope while tailings ore was used at the middle and top portions of the reinforced slope. The tailing ore involved a silty sand with a reported friction angle of 38.9^o and a cohesion of 19.6 kPa. Two types of inclusions were used as reinforcement: (1) a nonwoven needle punched geotextile with a mass per unit area of 600 g/m^2 and an ultimate unconfined tensile strength of 40 kN/m, and (2) a woven slit film PP geotextile with a mass per unit area of $445g/m^2$ and an unconfined tensile strength of 75 kN. The bottom slope was built using 8 geotextile inclusions of PP woven geotextiles while middle and top slopes were built using 16 layers of nonwoven needle punched geotextile.

Horizontal displacements were estimated by monitoring the internal strains of the inclusions at 1 and 5% deformations. At 1% deformation, maximum horizontal displacements values reached 162 and 177 mm at heights of 0.014 H and 0.015 H, respectively. At 5% deformation, the maximum horizontal displacement values reached 253 and 278 mm at 0.021 H and 0.023 H, respectively. Even though the instrumentation data was limited, visual inspection of the structure shows that the slope is performing as designed.

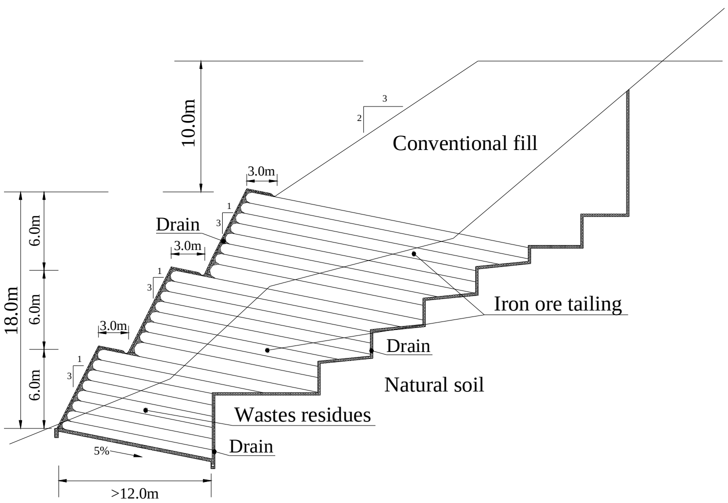

FIGURE 6 Cross section of reinforced slope at BR 381 (10).

Reinforced Walls

A number of reinforced soil walls were constructed as part of a restoration program of hillsides in the historic city of Petrópolis (state of Rio de Janeiro). In order to minimize transportation costs, the structure was built using poorly draining soils obtained from excavations conducted as part of the project. Nonwoven geotextiles were used as reinforcement inclusions (*13*, *16*). Table 5 presents the grain size distribution for the soils used as the reinforced backfills. Triaxial tests were conducted using compacted unsaturated soil sample (Soil 1), which defined a shear strength characterized by a cohesion of 50kPa and friction angle of 33^{o}.

A back-analysis for one of these structures was performed (*16)*. The global stability analysis (using the tie-back wedge method) demonstrated that the unsaturated compacted soil mass could remain stable without the reinforcements, with a factor of safety equal to 3.9. Such high factor of safety results from accounting for the unsaturated conditions of the soil, where negative pore water pressures were registered. However, the unsaturated condition of the soil cannot be guaranteed and, consequently, the design was performed using the saturated condition which resulted in a need for reinforcements with a vertical spacing of 0.30 m and a reinforcement length of 4 m. A drainage blanket was constructed behind the reinforced zone in order to minimize the potential generation of pore pressures.

Figure 7 shows a typical cross section for these walls, which were designed with a battered face of inclination 1H:8V. As expected, horizontal movements took place during construction and, towards the end of construction the facing of the structures became typically vertical.

TABLE 5 Grain Size Distribution of the Soils Used for the Retaining Wall Constructions in Petrópolis/RJ/Brazil (8)

Local	Grain size distribution		
	% < 0.002mm	% < 0.02mm	% < 2mm
1	36	54	100
2	31	55	100

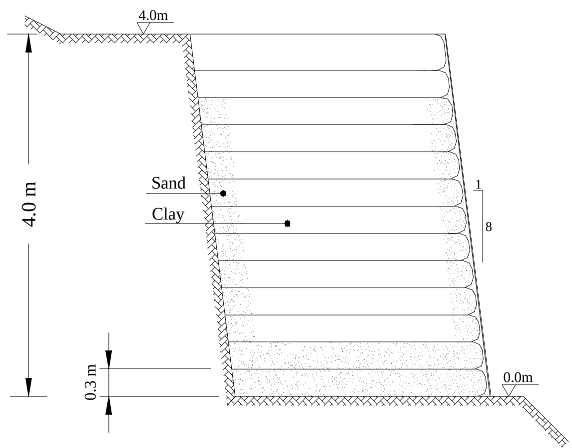

FIGURE 7 Cross section of a reinforced soil structure constructed in Petrópolis, Rio de Janeiro, Brazil (*13*).

The structures were well instrumented and showed very small values for post-construction horizontal and vertical movements. Also, the recorded pore water pressures were very small. The horizontal displacements both walls, for example, were less than 10 mm. The settlement plates did essentially not record any vertical movement. During the entire period over which pore water pressure readings were taken, the transducers did not record any positive value, even during the rainy period (*13*).

An additional reinforced soil wall built using poorly draining soils in Brazil is a reinforced soil wall built in the state of Minas Gerais along highway MG 030 (Figure 8) (*15*). The wall was 9.20 m high and used residual itabirite soil from local excavations as backfill material. This soil has 71.2% of fines (soil particles passing sieve #200). The shear strength of the soil was characterized by an effective friction angle of 48.4^o and an effective cohesion of 24.3 kPa. A woven geotextile with mass per unit area of $250 g/m^2$ and ultimate tensile strength of 42 kN/m was used as reinforcement. The wall facing was built included soil cement and shotcrete. A 200 mm thick drainage blanket was constructed at the base and behind the compacted reinforced fill.

The displacements for this structure were predicted using the approach proposed by Jewell and Milligan (*17*). The estimated maximum horizontal displacement ranged from 57 to 60 mm (at elevations of 0.62 H and 0.65 H, respectively) and from 58 to 69 mm (at elevations of 0.63 H and 0.75 H, respectively). This corresponded to reinforcement deformations obtained at tensile strains of 1 and 5%, respectively.

Poorly draining soils were also used during construction of a segmental retaining wall (*18*). This project was conducted as part of the reconstruction of a failed road slopes in the state of Rio de Janeiro State. The wall was 5.5 m high and was reinforced using PET geogrids with an ultimate tensile strength of 35 kN/m. The reinforcement layout involved a vertical spacing of 0.4 m and a reinforcement length of 3.5 m. The wall built using a battered facing with inclination of 1H:4V, as shown in Figure 9. The backfill soil consisted of a clayey silt obtained from excavations conducted as part of the project. The backfill soil had approximately 50% of material passing sieve #200, and its shear strength was characterized by a cohesion of 5kPa and a friction angle of 26º.

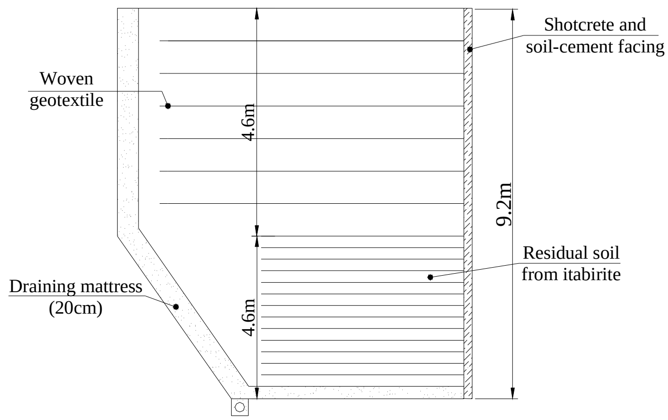

FIGURE 8 Cross section of reinforced wall at MG 030.

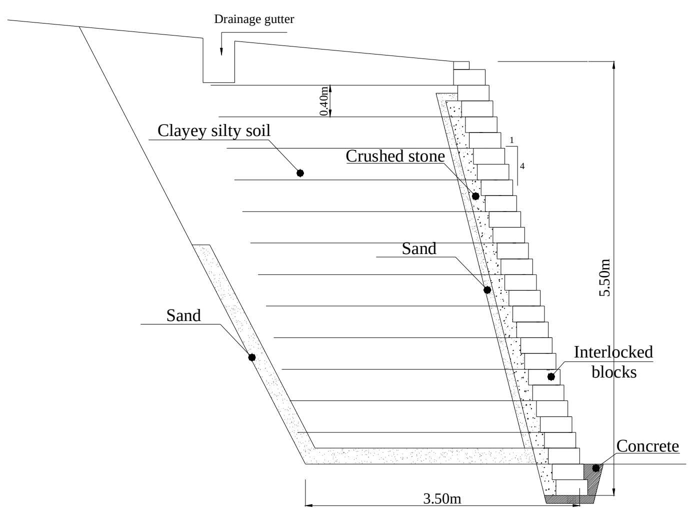

FIGURE 9 Cross section of a reinforced embankment at north of Rio de Janeiro State (*18*).

FINAL REMARKS

Soils with a large percentage of fines (silt and clay) are considered of marginal quality for the purposes of their use as backfill in reinforced soil structures because they exhibit poor drainage capacity. Accordingly, such soils are not allowed in the US, at least for reinforced retaining walls and steep slopes constructed by public transportation agencies. However, in spite of the significant caution against the use of such soils in the US, reinforced soil structures in Brazil have often been built using soils with a large percentage of fines. Indeed, the reported performance of these structures, many of them with field instrumentation, has shown a very good long-term performance. The good performance can be attributed to the significantly different characteristics of fine-grained soils in the US and Brazil. Specifically, most of the fine-grained soils used as backfill material in Brazil are residual soils, and often lateritic soils, which have shown excellent performance in engineered embankments. Accordingly, existing guidelines for reinforced soil construction should be refined as the sole use of grain size distributions to define the adequacy of backfill soils may be oversimplified.

A summary is provided in this paper of well-documented Brazilian case histories that involved the construction of reinforced walls and steep slopes using poorly draining soils. These structures, some of them over 20 years old, show no signs of distress as judged by the following:

- The magnitude of horizontal displacements that were recorded during construction was a function of the care taken during soil placement and compaction in the vicinity of the facing of the structure. However, no differences were observed in relation to the response of structures constructed with granular soils. Specifically, when care is taken during construction, the ratio δ_h/H ranges between 1 to 5%;
- Post-construction movements were observed to be significantly smaller than those taking place during construction. They stabilized within short periods of time after construction. The post-construction movements were typically characterized by ratios δ_h/H well below 1%;
- Measured values of pore pressure for the cases documented as part of this study were negligible, and typically negative;

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