



Evaluation of Penetration Resistance of Soils Reinforced with Geosynthetics Using CBR Tests

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Abstract

The application of geosynthetics in roads, paved or unpaved, can extend the service life of pavements, reduce base course thickness and delay rutting development, as well as reinforce weak subgrade layers, the base–subgrade interface, or the base layer. Regarding the application of geosynthetics in roads, this paper uses laboratory CBR tests to investigate how geosynthetics can improve the penetration resistance of two site-won soils (coarse and fine), analysing the influence of several parameters. The test conditions influenced unreinforced and reinforced specimens similarly. The CBR test captured the beneficial effect of the inclusion of one reinforcement layer for both soils. However, when the coarse soil was reinforced with two or more layers of sheet reinforcements, the limitations of the CBR test had a significant impact on the test results. The lateral walls of the mould and the reinforcement layout may lead to misleading results in the CBR tests. These conclusions were qualitatively corroborated by comparisons with the stress–strain behaviour from triaxial tests. Considering its popularity in the design of roads, and to address limitations of the CBR test, a modified setup to carry out CBR tests of reinforced soil solutions is proposed. This setup aims to ensure that the reinforcement mechanisms mobilised during the CBR test are realistic.

Keywords Reinforced soil · Geosynthetics · CBR · Granular soil · Fine soil · Modified CBR setup

Introduction

Forest roads, which are essential infrastructures that enable access to the forest, are unpaved. Often, such roads are formed using a superficial layer of unsealed gravel or aggregate, local, site-won soils or a mix of these two types of material. The application of geosynthetics in unpaved roads can extend the service life of pavements, reduce base course thickness for a given service life, and delay rutting

development [1]. In addition, geosynthetic reinforcement can be used in roadways to reinforce weak subgrade layers or it can be placed at the base–subgrade interface or within the base layer [2, 3].

When used in roads, geosynthetic reinforcement can mobilise three different mechanisms [4–6]: lateral restraint, bearing capacity increase, membrane tension support. The lateral restraint is mobilised by friction and/or interlocking (depending on the geosynthetic) between the soil and the geosynthetic; and it occurs when the geosynthetic reinforcement takes part of the shear stresses due to loading, reducing the shear stress transmitted to the soil below it, which otherwise would be transmitted directly to that lower soil layer. The geosynthetic reinforcement pushes the potential failure surfaces away from the foundation layer, to develop along surfaces with higher shear strength, increasing the bearing capacity of the soil. Under loading, a geosynthetic reinforcement may become concave and withstand tensile loads; in this case, the reinforcement will take part of the normal stresses transmitted to the soil above it, reducing the normal stress transmitted to the soil layer below the geosynthetic [7]. This membrane effect is mobilised only when the

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soil below the geosynthetic undergoes plastic deformations and rutting occurs; it is particularly important for reinforcement with high tensile modules [4].

Lower quality site-won soils can be used in paved or unpaved roadways by improving their mechanical performance, for example using reinforcements. This paper explores the response of two local site-won soils reinforced with geosynthetics using California bearing ratio, CBR, tests. The CBR of compacted soils (laboratory and/or field tests) is generally used for road design, often as input in empirical or semi-empirical methods. In the CBR test, the resistance of a soil to penetration by a punch is quantified and compared to that of standard material, for similar strain values [8]. Although the CBR test is standardised in many countries, there is little agreement on how representative the test is when estimating deformability and strength of the soil [9]. Even though the CBR test conditions (high strain level and low strain rates) do not replicate the loading on subgrades below pavements (low strain levels and higher strain rates), often subgrade design is based on this test [10]. While some authors agree that the CBR test measures the strength of subgrade layers (without providing information on their shear strength), other authors consider that a deformability parameter can be derived from the force–penetration curve [9]. Nevertheless, although the CBR is not a fundamental material property, it is a relatively easy and cost-effective test and is often used to estimate soil properties, such as the resilient modulus and the shear strength of unbound granular materials [11]. Due to sample disturbance and poor laboratory testing conditions, results from CBR laboratory tests are not always accurate [12]. In fact, CBR values (most of which are obtained from laboratory tests) are highly dependent on the representativeness of the samples tested. The boundary conditions in the laboratory CBR tests do not represent fully the boundary conditions that occur in the field. In addition, the testing conditions can significantly influence the CBR. For example, CBR values of soaked samples compacted to the optimum moisture content and density conditions can be significantly lower than those of similar unsoaked samples. Nevertheless, the CBR value is used frequently as a direct input to empirical pavement design methods and in correlations with resilient modulus and other engineering properties. For example, the bearing capacity of materials for subbase in road and airport pavements [13] can be estimated using CBR [14].

Some authors tried to modify the CBR test to overcome some of its limitations. For example, Haghghi et al. [15] used a staged repeated load CBR to investigate the stress–strain behaviour of marginal pavement materials and showed that the test method was able to determine the resilient modulus of the materials tested.

Mendoza and Caicedo [8, 16] used 3D finite-element analysis to illustrate how CBR test results are correlated with

the resilient modulus (key parameter in current mechanistic pavement design methods) and with other variables (particle size and shape, crushing, elastic behaviour of the soil). Mendoza and Caicedo [8] concluded that the CBR is mostly influenced by the Young's modulus of the soil, followed by the yield stress and the compression index; they observed that high values of the compression index of the soil lead to low CBR and that low values of the yield stress for compressibility influence the CBR (with reduced importance for increasing yield stress).

Raja et al. [17] evaluated the ability of the several intelligent models such as artificial neural network (ANN), least median of squares regression, and Gaussian processes regression, to estimate the CBR of reinforced soil. Due to the overall excellent performance of ANN, the model was converted into a trackable functional relationship to estimate the CBR of reinforced soil.

Despite their limitations, CBR tests can be used to demonstrate the qualitative benefit of geosynthetic reinforcement solutions, compared to unreinforced soil tested under the same conditions [18] and a number of authors have used this approach (e.g. [19–21]). Data from CBR tests showed that using a larger number of layers to reinforce soils can both increase and decrease their bearing capacity, depending on the type of soil considered and on the position of the reinforcement layers [22, 23]. Increasing the depth of the reinforcement layer within the CBR specimen decreases its bearing ratio [24]. The improvement in strength due to the geosynthetics depends on the type of soil and is larger for soils with a lower bearing capacity [25]. The type of geosynthetic used to reinforce the soil influences the change in bearing capacity [23–27]; however, in the literature no clear trend has been identified. The influence of the soil water content on its bearing capacity is similar for both unreinforced and reinforced samples; however, the permeability of the reinforcement layer may influence the response of the reinforced soil. Most studies on the influence of the soil water content of reinforced soil compare different test conditions (soaked *versus* unsoaked), such as Naeini and Ziaie Moayed [23] and Nguyen and Yang [28]; the influence in CBR is similar to that observed for unreinforced soil samples. Adams et al. [29] reported the influence of parameters such as plasticity index and gradation of soils on the bearing ratio of reinforced soil: the CBR decreased with increased soil plasticity and with decreased proportion of coarse particles; often two layers of geogrid resulted in a marginally improved performance compared to one layer of geogrid (if placed at adequate depth). Thus, as stated above, the CBR test procedure was developed for homogenous materials (soils and aggregates); however, many authors have used it to assess the response of composite materials, such as layered soils and reinforced soil [17, 21, 22, 25–32].

Moayed et al. [32] used CBR tests to analyse two-layered soils (granular soil as base layer and cohesive soil as subgrade layer) unreinforced and reinforced with geotextile and with geogrid at the interface between the two soils. They concluded that the water content of the subgrade layer influenced the response of the two-layered soil, namely the efficiency of the reinforcement; the type of reinforcement (geogrid or geotextile) was found to affect the behaviour of the layered soil.

This paper focuses on the assessment of the penetration resistance of two local site-won soils and how that response was affected by geosynthetic inclusions. Three different reinforcement solutions were studied, representing typical scenarios in reinforced soil. The influence of a number of parameters on the penetration resistance was assessed and the main limitations of the CBR test in reproducing a realistic response of the reinforced solutions are discussed. Some possible mechanisms are put forward, to explain the changes in response of the reinforced specimens relatively to that of the unreinforced soil. The test results were used to estimate the Young's modulus of the soils and of the soil–geosynthetic composite material, using relations from the literature. To address some of the limitations of CBR tests and to support the application of geosynthetics in soil reinforcement, the results were compared with the stress–strain response of the similar composite materials obtained using triaxial tests (tests performed by [33] and [34]). Finally, changes to the

test setup are suggested, to address some of the limitations of the CBR test identified for reinforced soil specimens.

The specific objectives of this research are to:

- Measure the penetration resistance of two soils (coarse and fine) unreinforced and reinforced with three different geosynthetic solutions using CBR tests.
- Assess the influence of the reinforcement (reinforcement solutions, number of reinforcement layers), the compaction energy used to prepare the specimens, the initial water content, and the test conditions on the penetration resistance.
- Compare qualitatively responses for the penetration resistance, from CBR tests, and for the stress–strain, from triaxial tests.
- Identify and address limitations of CBR tests to analyse reinforced soil solutions.
- Quantify the Young's modulus of the soil and soil–geosynthetic composite material using relations from the literature.

Materials and Methods

The test program (Table 1) encompassed studying the penetration resistance of two site-won soils reinforced with geosynthetics using CBR laboratory tests. The influence of several factors on the penetration response was analysed,

Table 1 Test program and number of specimens tested

| Soil | Reinforcement | | Initial water content w (%) | Number of blows per layer | Test | |
|--------|------------------|-----------|----------------------------------|------------------------------|------------------|---------------------|
| | Number of layers | Type | | | Conditions | Number of specimens |
| Soil 1 | 0 | – | 9.5; 11.5; 13.5 | 25 | Soaked | 5 |
| | 0 | – | 11.5 | 55 | Soaked; unsoaked | 2 |
| | 1 | GCR | 9.5; 11.5; 13.5 | 25 | Soaked | 3 |
| | 1 | GGR | 9.5; 11.5; 13.5 | 25 | Soaked | 3 |
| | 1 | GGR + GTX | 9.5; 11.5; 13.5 | 25 | Soaked | 3 |
| | 1; 2; 4 | GCR | 11.5 | 55 | Soaked; unsoaked | 6 |
| | 1; 2; 4 | GGR | 11.5 | 55 | Soaked; unsoaked | 6 |
| | 1; 2; 4 | GGR + GTX | 11.5 | 55 | Soaked; unsoaked | 6 |
| Soil 2 | 0 | – | 11.9; 13.9; 15; 17; 19 | 25 | Soaked; unsoaked | 8 |
| | 1 | GCR | 11.9; 13.9; 15; 17; 19 | 25 | Soaked | 5 |
| | 1 | GGR | 11.9; 13.9; 15; 17; 19 | 25 | Soaked | 5 |
| | 1 | GGR + GTX | 11.9; 13.9; 15; 17; 19 | 25 | Soaked | 5 |
| | 1 | GCR | 11.9 | 25 | Unsoaked | 1 |
| | 1 | GGR | 11.9 | 25 | Unsoaked | 1 |
| | 1 | GGR + GTX | 11.9 | 25 | Unsoaked | 1 |
| | 2; 4 | GCR | 11.9 | 25 | Soaked; unsoaked | 4 |
| | 2; 4 | GGR | 11.9 | 25 | Soaked; unsoaked | 4 |
| | 2; 4 | GGR + GTX | 11.9 | 25 | Soaked; unsoaked | 4 |

using the response of the unreinforced soils as a reference. The following sections describe in detail the test procedure, the site-won soils (soil 1 and 2) and the geosynthetics (GCR, GGR and GTX) studied.

Test Procedure

The CBR tests were performed according to the procedure described in LNEC [35], similar to ASTM D1883–07 [13]. Their main differences refer to the test velocity and the number of blows applied during compaction. Before the tests, the soil was dried; then it was prepared to the desired water content and allowed to rest under controlled

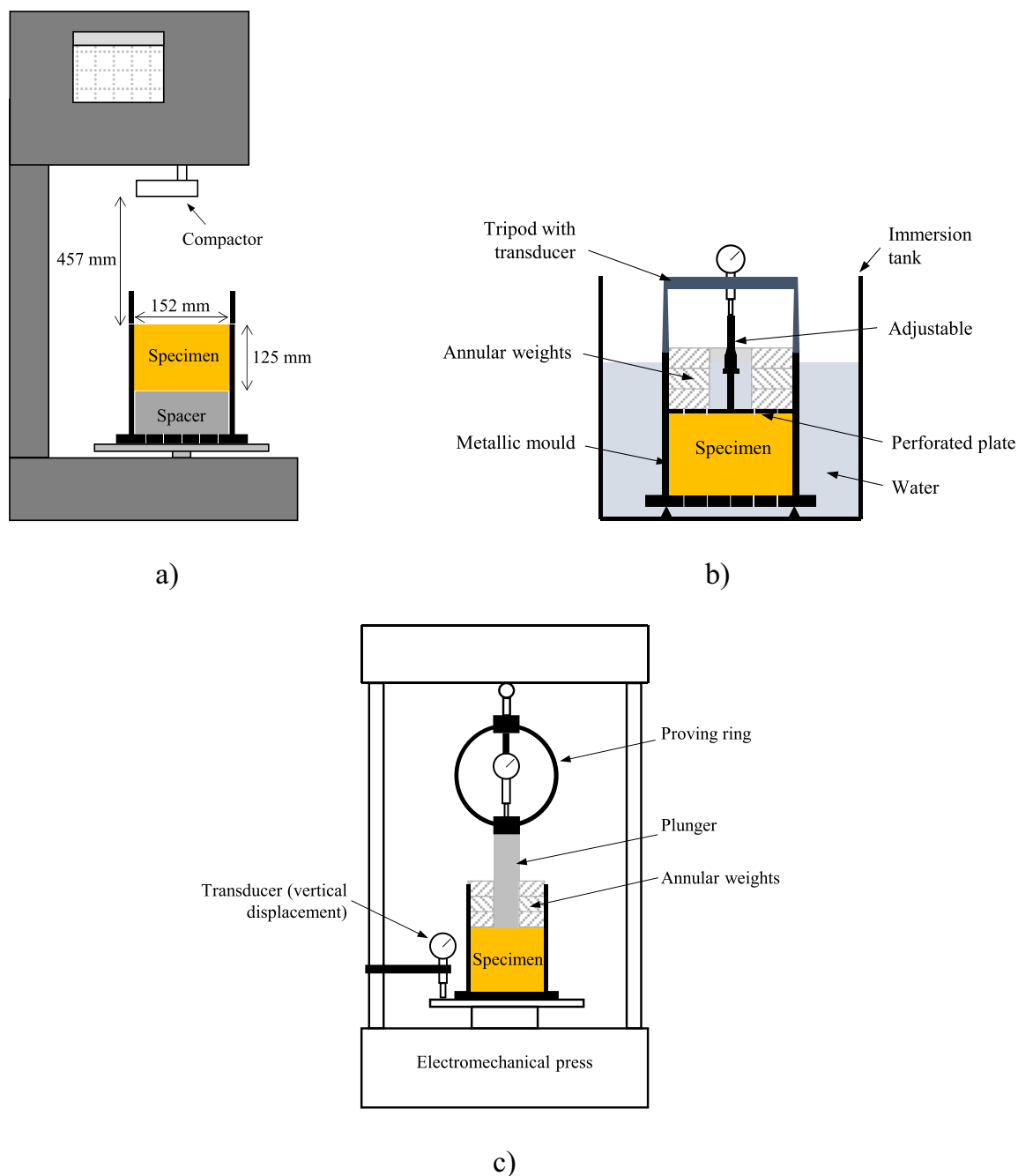


Fig. 1 Schematic representation of the equipment used for the CBR tests: **a** automatic compaction; **b** immersion tank; **c** electromechanical press for the CBR test

conditions (temperature 20 °C, relative humidity 65%), for 0.5 h (soil 1) and 24 h (soil 2), following the recommendations in ASTM D1557 [36] (0–3 h for SP-SM soils, > 16 h for ML soils). The cylindrical specimens (diameter 152 mm, height 125 mm) were prepared using a metallic mould. Each specimen was formed by 5 layers of soil (each 25 mm high) compacted with 25 or 55 blows using a rammer (44.5 N; drop height 457 mm) (Fig. 1a). When required, the specimens were soaked for 96 h in an immersion tank filled with water and under a surcharge (Fig. 1b); the swell observed was negligible.

The reinforced specimens were prepared in a similar way: a layer of soil was placed and compacted; this was repeated until the position of the reinforcement was reached; then, a reinforcement layer was placed horizontally on top of the compacted soil; next, the following soil layer was placed and compacted. This process was repeated until the final height of the specimen was reached, and all reinforcement layers were placed at the

desired heights. The reinforcement layers were discs of geosynthetic included at the desired positions forming three different layouts (Fig. 2). The discs of reinforcement were not attached to the mould and the diameter of the discs was identical to the internal diameter of the mould. Before placing the reinforcement, the soil was levelled to ensure that the reinforcement was free of wrinkles and to minimise damage during compaction.

During the CBR test, three annular plates (24.52 N each) were placed at the upper surface of the specimen. All CBR tests were performed at an imposed axial displacement rate of 1 mm/min and with a piston of 50 mm diameter (Fig. 1c). Both soaked and unsoaked tests were carried out.

Soils

This study analysed two different local soils, to assess their potential applications (Fig. 3): soil 1, a residual soil from granite; soil 2, a fine soil (from a salt pan in the Aveiro

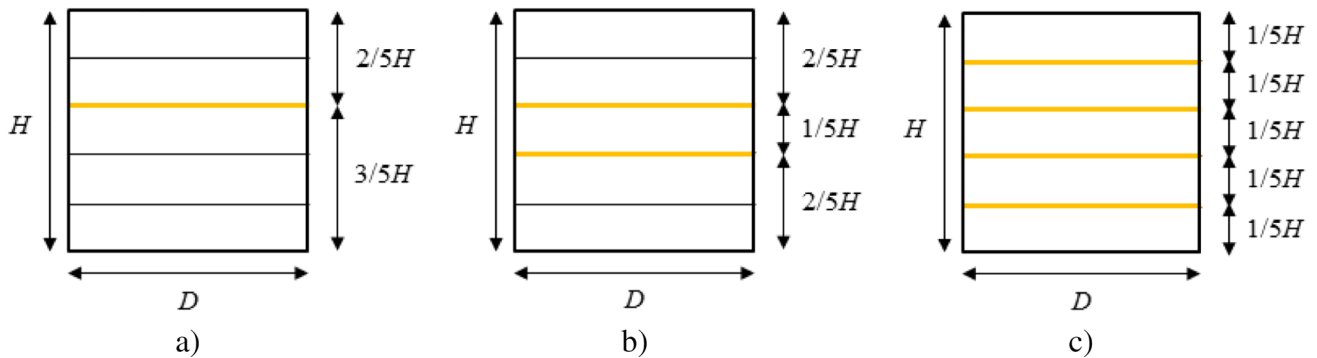


Fig. 2 CBR tests with different number of reinforcement layers, placed at designated heights within the mould (relative to the top of the specimen): **a** 1 layer (position $2/5H$); **b** 2 layers (positions $2/5H$ and $3/5H$); **c** 4 layers (positions $1/5H$, $2/5H$, $3/5H$ and $4/5H$)

Fig. 3 Particle size distribution of soils 1 and 2

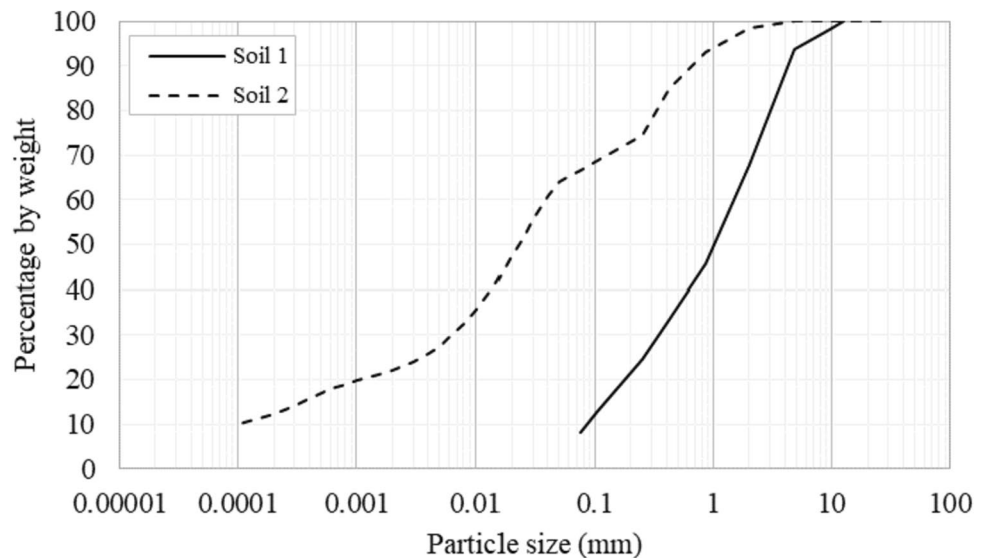


Table 2 Properties of soils 1 and 2 and of the fine fraction of soil 1 (soil 1_{fine})

| Property | Unit | Soil 1 | Soil 2 | Property | Unit | Soil 1 _{fine} | Soil 2 |
|--------------------|----------------------|--------|--------|----------|------|------------------------|--------|
| % < 0.074 mm | (%) | 8 | 65.7 | w_L | (%) | 28.9 | 35 |
| D_{10} | (mm) | 0.084 | 0.0001 | w_P | (%) | 18.0 | 25 |
| D_{50} | (mm) | 1.002 | 0.023 | I_P | (%) | 10.9 | 10.4 |
| D_{max} | (mm) | 12.7 | 4.76 | I_C | (%) | 1.8 | 1.3 |
| C_U | (–) | 16.7 | 380 | A_t | (–) | – | 0.47 |
| C_C | (–) | 0.9 | 10.8 | | | | |
| G_s | (–) | 2.55 | 2.64 | | | | |
| e_{min} | (–) | 0.48 | – | | | | |
| e_{max} | (–) | 1.00 | – | | | | |
| $\rho_{dmax}^{\#}$ | (g/cm ³) | 1.925 | 1.845 | | | | |
| $w_{opt}^{\#}$ | (%) | 11.5 | 13.9 | | | | |

[#]Modified Proctor (ASTM D1557 [36])

lagoon, Portugal). According to the Unified Soil Classification System, USCS (ASTM D2487–11 [37]), and AASHTO classification system (AASHTO M 145–91-UL [38]), soil 1 is SW-SM, well-graded sand with silt, or A-2, respectively, and soil 2 is ML, sandy silt, or A-4, respectively. Table 2 includes some relevant soil properties: percentage of fine particles (<0.074 mm); 10%, 50% and maximum grain sizes (D_{10} , D_{50} , D_{max} , respectively); uniformity coefficient (C_U); coefficient of curvature (C_C); relative density of the particles (G_s); minimum (e_{min}) and maximum (e_{max}) void ratio; soil compaction characteristics (ASTM D1557-12 [36], modified Proctor tests), maximum dry density (ρ_{dmax}) and optimum water content (w_{opt}). Additional information is provided for the fine fraction of soil 1 (soil 1_{fine}) and for soil 2: consistency limits (w_L and w_P), plasticity index (I_P), consistency index (I_C) and activity (A_t). The shear strength of the soils was estimated from triaxial tests and the effective cohesion intercept c' and the angle of friction, ϕ' , are, respectively: $c'=0$ kPa and is $\phi'=37.7^\circ$, for soil 1; and $c'=5$ kPa and is $\phi'=32.3^\circ$, for soil 2.

Geosynthetics

Three geosynthetics were used in the tests (Fig. 4): geogrid (GGR), consisting of high tenacity polyester (PET) yarns covered with black polymeric coating woven into a grid structure (openings 25 mm × 25 mm); reinforcement geocomposite (GCR), uniaxial geocomposite composed of high modulus PET fibres attached to a continuous filament non-woven geotextile backing; geotextile (GTX), consisting of continuous thermo-bonded polypropylene (PP) filaments. Table 3 summarises some physical and tensile properties of the geosynthetics. Their short-term tensile properties (EN ISO 10319 [39]) (mean values and standard deviation for 5 specimens and for both the machine direction, MD, and the cross-machine direction, CMD) include the tensile strength (T_{max}) and the corresponding strain (ϵ_f), and the force for 2% strain ($T_{2\%}$). The physical properties included in Table 3 are: thickness under a stress of 2 kPa, d_{2kPa} , (EN ISO 9863–1 [40]); mass per unit area, μ (EN ISO 9864 [41]). The mass per unit area of GGR was estimated using specimens with 1 m² area.

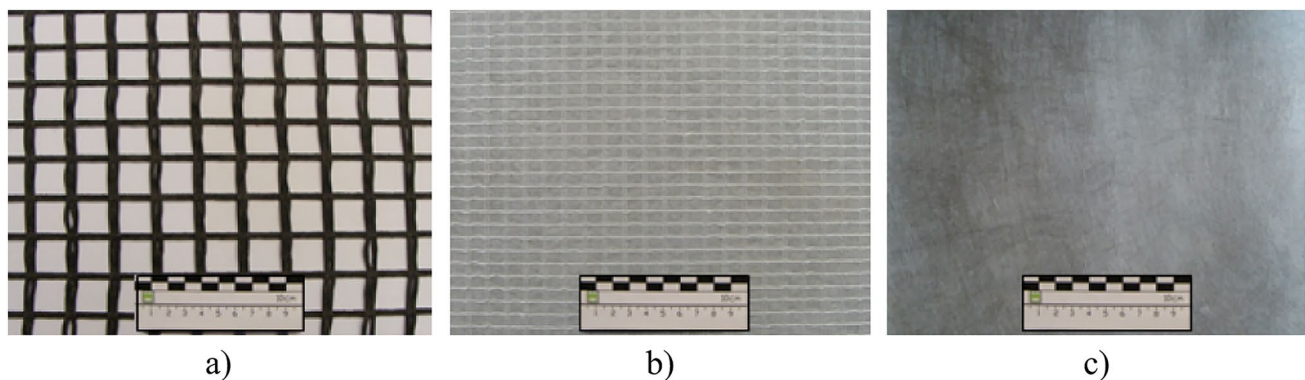


Fig. 4 Geosynthetics studied: **a** geogrid GGR; **b** geocomposite GCR; **c** geotextile GTX

Table 3 Tensile properties of the geosynthetics studied, tensile strength (T_{max}) and corresponding strain (ϵ_f), tensile load for 2% strain ($T_{2\%}$), and physical properties, thickness (d_{2kPa}) and mass per unit area (μ)

| Property | Unit | Direction | Value | Geosynthetics | | |
|---------------------|--------------|---------------------|-------|---------------|------|------|
| | | | | GCR | GGR | GTX |
| Tensile properties | T_{max} | MD | Mean | 54.6 | 43.9 | 12.6 |
| | | | SD | 1.5 | 0.9 | 0.9 |
| | | | Mean | 15.6 | 50.8 | 12.4 |
| | | CMD | SD | 0.3 | 1.5 | 0.5 |
| | | | Mean | 10.6 | 7.9 | 45.5 |
| | | | SD | 0.4 | 0.4 | 2.5 |
| | ϵ_f | MD | Mean | 10.6 | 7.9 | 45.5 |
| | | | SD | 0.4 | 0.4 | 2.5 |
| | | | Mean | 79.7 | 8.7 | 34.6 |
| | | CMD | SD | 3.2 | 0.3 | 2.9 |
| | | | Mean | 12.6 | 8.4 | 3.89 |
| | | | SD | 1.6 | 0.4 | 0.2 |
| $T_{2\%}$ | MD | Mean | 12.6 | 8.4 | 3.89 | |
| | | SD | 1.6 | 0.4 | 0.2 | |
| | | Mean | 1.2 | 9.9 | 3.7 | |
| | CMD | SD | 0.3 | 0.6 | 0.7 | |
| | | Mean | 2.14 | 1.72 | 0.49 | |
| | | Mean | 325 | 252 | 164 | |
| Physical properties | d_{2kPa} | (mm) | – | | | |
| | μ | (g/m ²) | | | | |

MD machine direction, CMD cross-machine direction, SD standard deviation

In the CBR tests, three different reinforcement solutions were used: 1) GCR; 2) GGR; 3) GGR + GTX, association of geogrid GGR and geotextile GTX. These represent different reinforcement scenarios: GCR, sheet reinforcement stronger in MD; GGR, planar reinforcement with openings, allowing soil particles to move between layers adjacent to the reinforcement, with similar strength in MD and CMD; GGR and GTX, sheet reinforcement (the GTX prevents the soil particles from moving between layers), while taking advantage of the stiffness and strength of GGR. The tensile strength of the composite GGR + GTX was not measured using tensile tests, but it was estimated to be similar in both MD and CMD, as both geosynthetics (GGR and GTX) exhibited a biaxial response, characterised by a similar tensile strength along MD and CMD. For this reinforcement solution, GGR + GTX, a disc of GGR was placed directly on top of a disc of GTX, with the MD directions of the geosynthetics aligned; thus, GGR overlaid GTX with no bonding between them.

The three different reinforcement solutions included correspond to typical situations in soil reinforcement. Two different scenarios for the tensile strength on the MD and the CMD of the geosynthetics were studied (one solution with orthogonal anisotropy of tensile strength, and two other solutions with similar strength along the MD and the CMD). In addition, two different scenarios for the contact area between the reinforcement and the adjacent soil were analysed (two solutions with sheet reinforcement and one with grid reinforcement).

Results and Discussion

Interpretation of Results

Usually, the CBR test is interpreted using a CBR index or value (CBR_x , Eq. 1) calculated as the ratio of the force, F_x , measured during the test for a value of penetration x (typically, $x=2.5$ mm or 5.0 mm) to the corresponding force for a standard material, F_{xS} ($F_{2.5S}=13.29$ kN; $F_{5.0S}=19.94$ kN, recommended in [35]). For soils, normally $CBR_{2.5}$ is reported; whenever the ratio at 5.0 mm penetration is greater than $CBR_{2.5}$, the test needs to be rerun and in case of a similar trend, $CBR_{5.0}$ should be reported. Herein, CBR values at penetrations of 2.5 mm ($CBR_{2.5}$) are included; $CBR_{5.0}$ were also calculated, and some values are also reported.

$$CBR_x = \frac{F_x}{F_{xS}} \times 100 \quad (1)$$

This approach is empirical, but its results are often used in civil engineering and may help to appreciate the influence of the reinforcements (and the different parameters analysed) on the response of the soils studied.

Nevertheless, to overcome this empiricism, other quantities were defined. The influence of the reinforcement was quantified by defining an improvement ratio, IR (Eq. 2), where $F_{x,u}$ and $F_{x,r}$ are the force values measured for a penetration of x mm for the unreinforced and reinforced samples, respectively, tested under similar conditions:

$$IR = \frac{F_{x,r}}{F_{x,u}} \quad (2)$$

The improvement ratio for the maximum penetration force is usually designated as bearing capacity ratio (BCR). Herein, the maximum force always corresponds to a penetration $x = 12.5$ mm (maximum penetration depth in the tests). In this paper, the test results are also analysed using the full force–penetration curves. Thus, analysing the improvement ratio (Eq. 2) and the full force–penetration curves, as well as the density of the different specimens, overcomes the empiricism of the typical interpretation of CBR tests (where the force F_x is compared to a corresponding force for a standard material, F_{xS}).

Before the tests, the water content of each specimen was assessed, to check if the desired water content (w) had been achieved. The real water content of the specimens was measured (before soaking, when applicable), w_{real} , and was reported.

The dimensions of the specimens (diameter 152 mm, height 125 mm) and of the piston (diameter 50 mm) used in the CBR test are relatively small, limiting the range of materials for which the test results are appropriate (maximum particle size up to 20 mm). Nevertheless, that was not an issue for the soils reported herein.

Repeatability of Results

To assess the repeatability of results, the responses of two unreinforced soil specimens (Unr) tested under the same conditions were compared. The force–penetration curves

obtained for the repeated tests were almost identical (Fig. 5). The repeatability of the CBR values was found adequate, with only small differences, higher for soil 1 (S1) than for soil 2 (S2): for S1 Unr $w = 11.5\%$ CBR_{2.5} varied between 22.1% and 21.4% and CBR_{5.0} varied between 25.1% and 24.7%; for S2 Unr $w = 11.9\%$ CBR_{2.5} varied between 9.5% and 9.7% and CBR_{5.0} varied between 10.0% and 10.1%. Most likely, those differences are due to small variations in the real water content, w_{real} , of the specimens (measured before the soaking period): S1 Unr $w = 11.5\%$ w_{real} was 11.4% and 11.7%, respectively; for S2 Unr $w = 11.9\%$, w_{real} was 11.9% for both specimens. The type of soil tested had no significant influence on the repeatability of results.

The repeatability of results was satisfactory. Thus, to avoid increasing in several dozens the number of tests to carry out, afterwards only one specimen was tested per condition. However, the tests with results outside of the expected were repeated to assess their repeatability. The results with significant discrepancies were analysed and the results considered anomalous were discarded. The approach adopted is often reported in the literature. For example, Mehrpazhouh et al. [19], Singh et al. [20] and Lakshmi et al. [21] presented test results of one specimen per test condition only (for both, unreinforced and reinforced soils).

Influence of the Reinforcement

The influence of the reinforcement depends on several parameters, such as the type of geosynthetic, number and

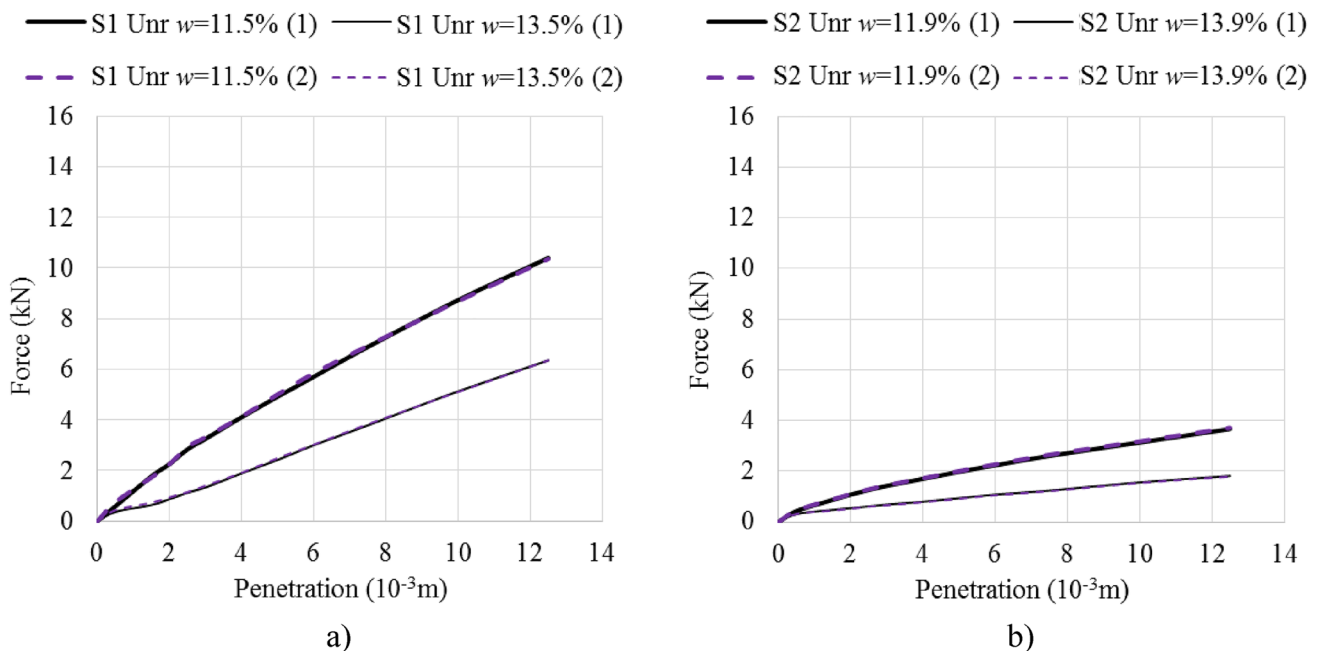


Fig. 5 Repeatability of CBR results (unreinforced specimens, Unr): **a** soil 1 (soaked specimens, $w = 11.5\%$ or 13.5% , 25 blows); **b** soil 2 (soaked specimens, $w = 11.9\%$ or 13.9% , 25 blows)

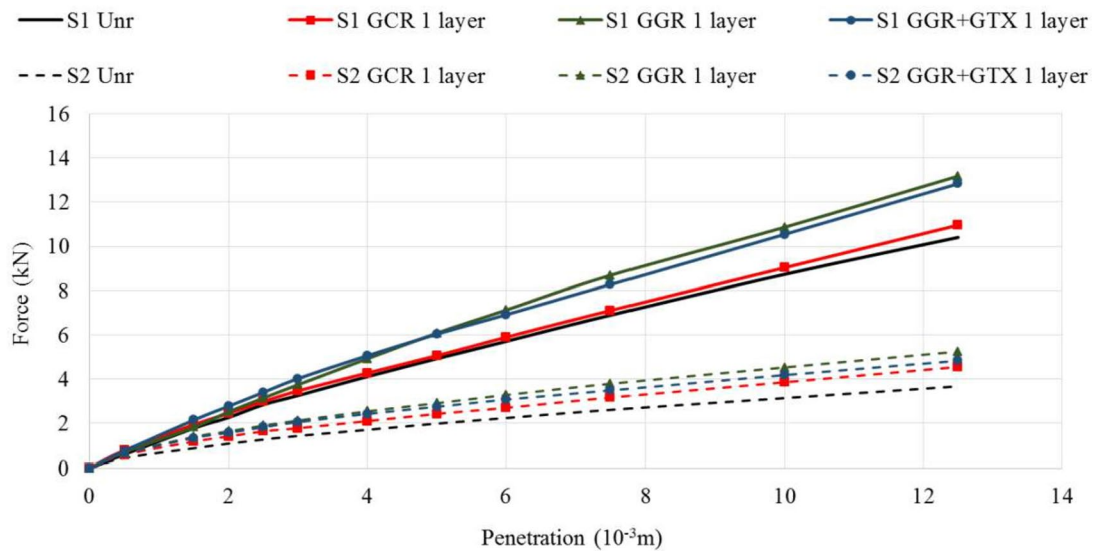


Fig. 6 Force–penetration response of unreinforced specimens (Unr) and specimens reinforced with one layer of geosynthetic (GCR, GGR, GGR + GTX): soil 1 (soaked, $w = 11.5\%$, 25 blows); soil 2 (soaked, $w = 11.9\%$, 25 blows)

position of the reinforcement layers, type of soil. Herein, these factors are analysed separately.

Unreinforced Versus Reinforced Soil

To discuss the influence of the reinforcement on the response of the soil, the results for the unreinforced and reinforced specimens with one layer of reinforcement were compared (Fig. 6). For both soils 1 and 2, specimens with one layer of reinforcement achieved a higher load at the same penetration, relative to the corresponding unreinforced specimen. For example, using a layer of GGR led to an increase of $CBR_{2.5}$ between 10.6% (soil 1) and 49.2% (soil 2), the corresponding increase of maximum penetration force, F_{max} , ranged between 26.7% (soil 1) and 42.8% (soil 2).

After the tests, all specimens, unreinforced and reinforced, exhibited similar permanent deformations immediately below the penetration piston (Figs. 7 and 8), with no

upward movement of the soil surrounding the piston. The internal changes of the specimens were varied: the unreinforced specimens became denser below the penetration piston; the reinforced specimens exhibited an additional feature, as the reinforcement followed the deformation of the soil, becoming concave and sustaining tensile loads (Fig. 8). Figure 8 illustrates qualitatively the deformations occurred in the interior of the test specimens: the densification of the soil and the deformations of the reinforcement layers were observed visually during the disassembly of the specimens, after the tests. Each soil layer was carefully removed, and the reinforcement was observed (the deformations were not measured).

Due to the small size of the CBR specimen, an analogy between the responses observed and the reinforcement mechanisms discussed before (lateral restraint, bearing capacity increase and membrane tension support) can be difficult. Nevertheless, the deformation of the

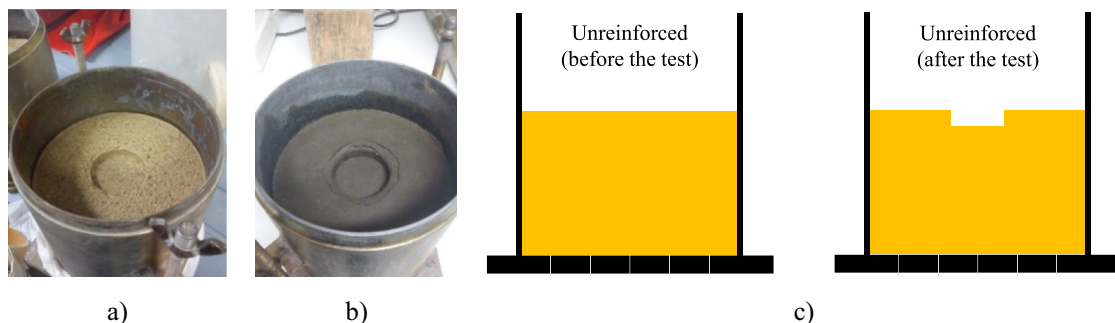


Fig. 7 Specimen after the test **a** soil 1 and **b** soil 2; **c** schematic representation of unreinforced specimen cross-section before and after the test

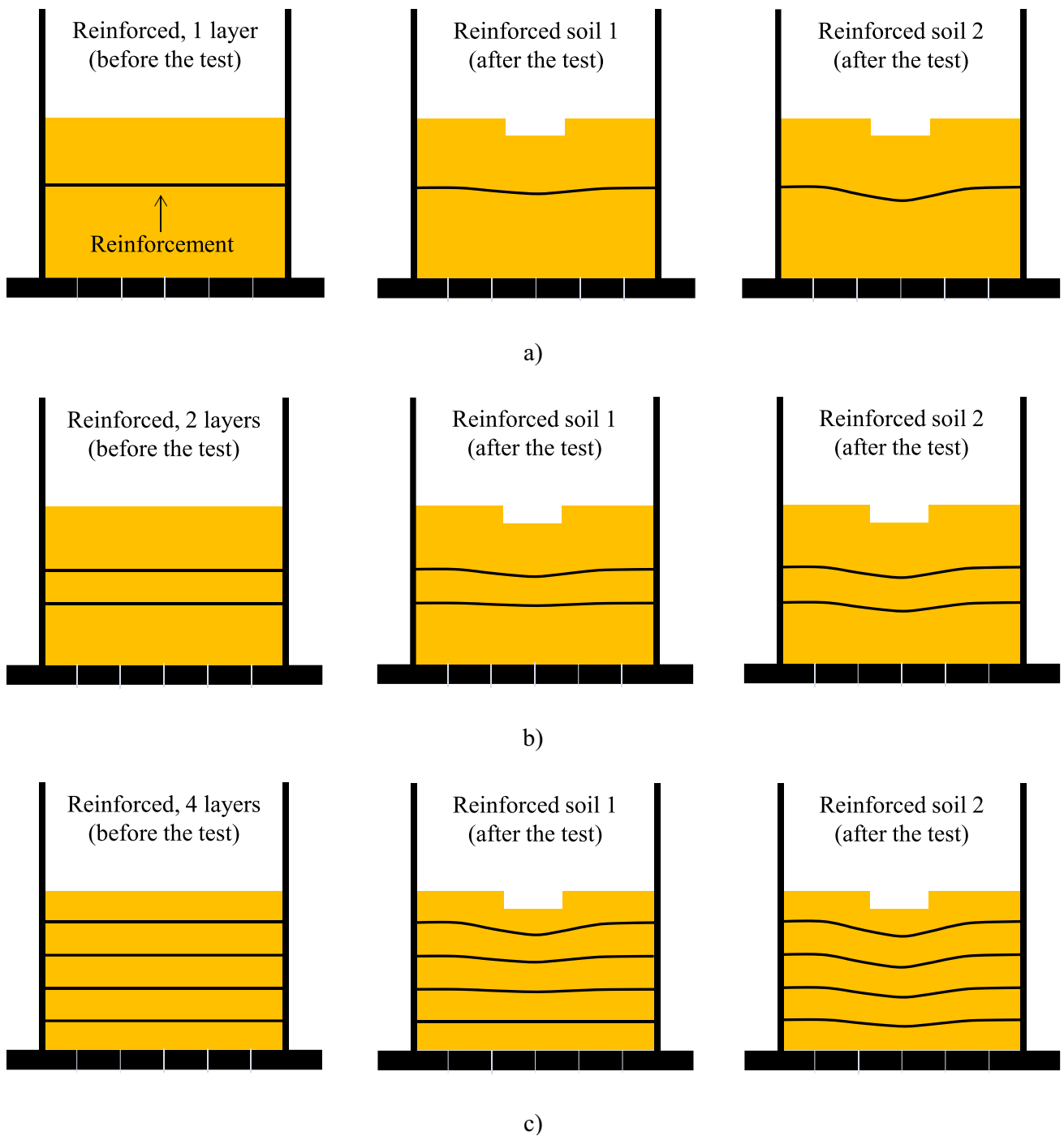


Fig. 8 Schematic representation of the cross-section of the reinforced soil specimens before and after the test illustrating qualitatively the differences observed for soils 1 and 2: **a** one layer; **b** two layers; **c** four layers

reinforcement and the associated plastic deformations observed in the soil underlying the reinforcement, particularly for soil 2, point to a membrane tension support reinforcement mechanism (described by Giroud and Noray, [7]). The importance of this mechanism depends on the ability to mobilise sufficient shear strength outside the area influenced directly by the piston load.

The higher deformability of soil 2 (compared to soil 1) may also contribute to the mobilisation of this mechanism. Simultaneously, the presence of the layer of reinforcement is likely to have forced the potential failure surface to follow an alternative deeper path (compared to that of the unreinforced soil). This allowed for an increase of the bearing capacity of the soil, which was particularly

important for the finer soil (soil 2). The third reinforcement mechanism (lateral confinement) could not be identified, as the movement of the soil particles could not be observed. The test results and the cross-section of the specimens at the end of the tests show that the improved response of the reinforced samples tested may be due to both membrane tension support and bearing capacity increase. The relative contribution of these two reinforcement mechanisms to the responses observed cannot be quantified. Nevertheless, due to the small length of reinforcement available for the mobilisation of shear strength, it is likely that the membrane tension support mechanism is less important than the bearing capacity increase described above.

The CBR test can be compared to a load test on circular foundation, while considering several limitations (e.g. relative size of the piston base and the soil particles, and influence of the mould). Similarly, the penetration response of specimens reinforced with geosynthetics can be compared to the stress–settlement response of circular footings on soils reinforced with geosynthetics. There are different proposals for the optimum reinforcement layout for a circular footing (diameter d) on soil reinforced with one circular layer of geosynthetic (diameter D_r). According to Lavasan and Ghazavi [42], the maximum depth of the layer of geosynthetic reinforcement is $u = 0.3d - 0.50d$ and the diameter of the reinforcement should be $D_r = 4d$. Chakraborty and Kumar [43] carried out upper bound axisymmetric finite-element analysis to evaluate the bearing capacity of circular footing on soils reinforced with geogrids. Their analyses have some limitations, as they refer to ultimate limit state only and are based on the shear strength parameters of the soil and soil–reinforcement interface, ignoring the stiffness and the Poisson's ratio of both soil and reinforcement. Still, their work may help to explain the responses observed in the CBR tests reported herein. Chakraborty and Kumar [43] concluded that, to avoid boundary effects, the area of analysis should be extended laterally of $3d - 7d$ beyond the edge of the footing and $1.8d - 3.8d$ below the footing; they also found that the optimum layout for one layer of reinforcement was $u = 0.20d - 0.40d$ (for this layout, $D_r > 1.5d$ had no significant influence on the bearing capacity), and the optimum value for D_r varied from $1.85d$ to $3.55d$, depending on the type of soil and on the shear strength parameters considered. The depth of the reinforcement layer should be small enough to prevent failure of the soil above the reinforcement, while the diameter of the reinforcement should be large enough to allow its full mobilisation within potential failure surfaces.

In the CBR tests carried out, the mass of soil extended laterally of $1d$ beyond the edge of the footing and $5d$ below the footing and the reinforcement layout for specimens with one layer of reinforcement was $u = d$ and $D_r = 3.04d$. Thus, the lateral walls of the mould acted as a rigid boundary

and restrained lateral movements of the soil, while the reinforcement layer was too deep relative to its optimum value. These two effects are contradictory, and the test results show that, for the conditions considered herein, the contribution of the lateral walls of the mould was higher than the detrimental effect of the depth of the reinforcement, thus resulting in an improved penetration resistance of the specimens reinforced with one layer of geosynthetic relative to the unreinforced soil.

Number of Reinforcement Layers

The response of similar specimens unreinforced and reinforced with different numbers of layers was analysed for tests carried out with soil 1 (soaked, $w = 11.5\%$, 55 blows) and soil 2 (soaked, $w = 11.9\%$, 25 blows). Increasing the number of reinforcement layers did not always result in increased CBR and F_{max} , relative to the unreinforced soil. The influence of the number of reinforcement layers varied with both the type of geosynthetic and soil (Figs. 9 and 10).

A larger number of reinforcement layers improved the response of soil 2, for all types of reinforcement (Figs. 9b, d, f and 10b, d). Compared to the corresponding unreinforced specimens (soil 2 soaked), the $CBR_{2.5}$ increased 29% to 49% (1 reinforcement layer), 42% and 75% (2 reinforcement layers) and 56% and 94% (4 reinforcement layers), depending on the reinforcement solution analysed. For soil 2, the influence of the number of reinforcement layers was greater when using GGR; the least effective reinforcement was GCR. The influence of the type of geosynthetic is discussed in the following section.

When analysing the results obtained for soil 1 (coarser), increasing the number of reinforcement layers did not always lead to a better response of the specimens (Figs. 9a, c, e and 10a, c). While one layer of reinforcement was beneficial, using two reinforcement layers improved the response of the specimens relative to that of the unreinforced soil (except for GCR), but not relative to that of specimens with one layer of reinforcement. For specimens reinforced with one and two layers of reinforcement, the upper layer was placed at the same depth (at $2H/5 = d$ from the top, Fig. 2). When four reinforcement layers of GCR or GGR + GTX were included in the specimens of soil 1, the response was significantly worse than that of the unreinforced specimens (for example, $CBR_{2.5}$ reduced 45% with GCR and 19% with GGR + GTX). The specimens reinforced with four layers of GGR exhibited a response between specimens reinforced with one and two layers of GGR, depending on the penetration considered (Fig. 9c).

Naeini and Mirzakanlari [22] and Vinod and Minu [24] found similar trends (reduced penetration resistance with larger number of reinforcement layers) but did not attempt an explanation. Rashidian et al. [44] tested coarser soils

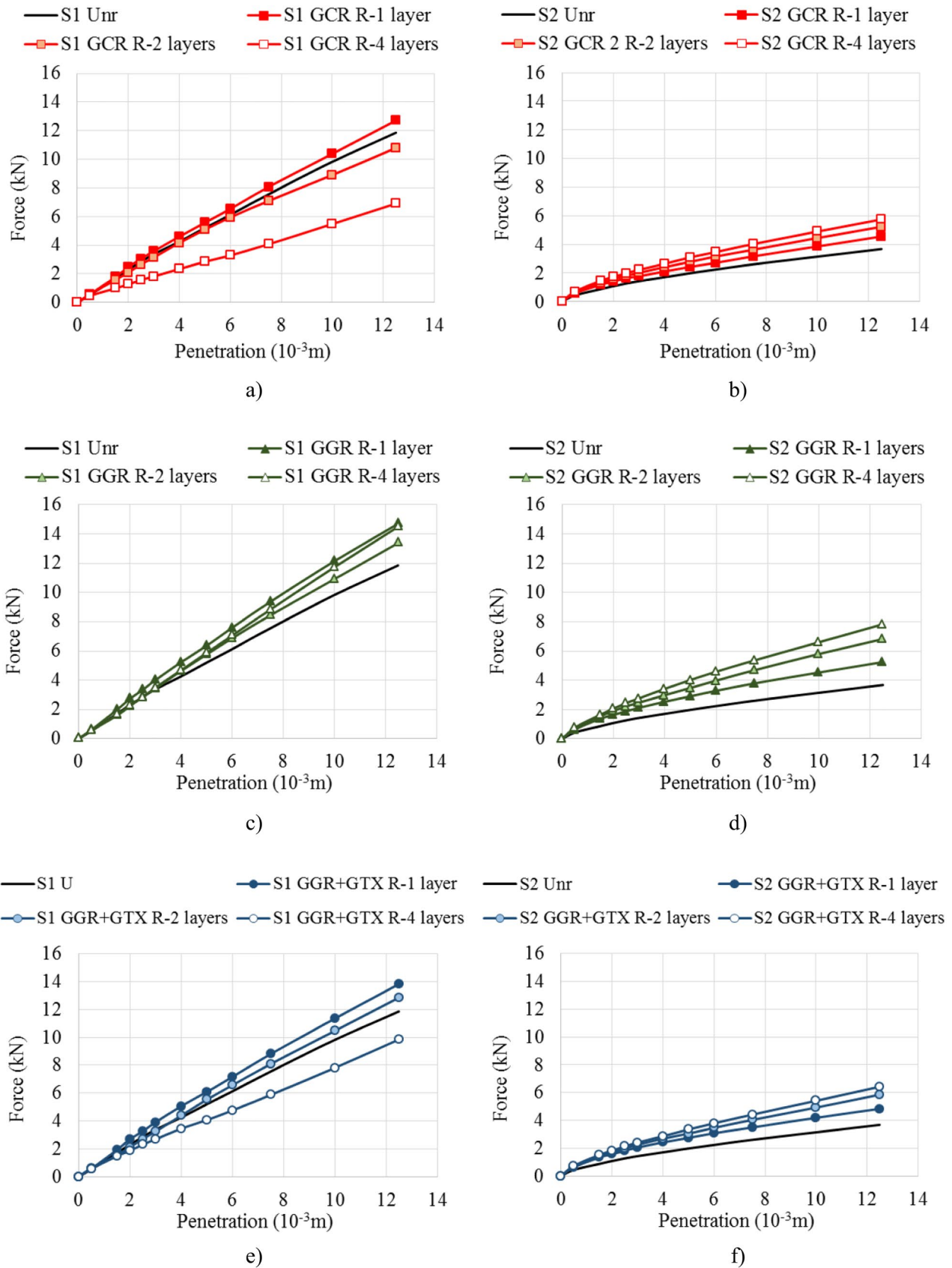


Fig. 9 Force–penetration response of unreinforced specimens and specimens reinforced with different number of reinforcement layers (soil 1—soaked specimens, $w = 11.5\%$, 55 blows; soil 2—soaked specimens, $w = 11.9\%$, 25 blows): **a** and **b** GCR; **c** and **d** GGR; **e** and **f** GGR + GTX

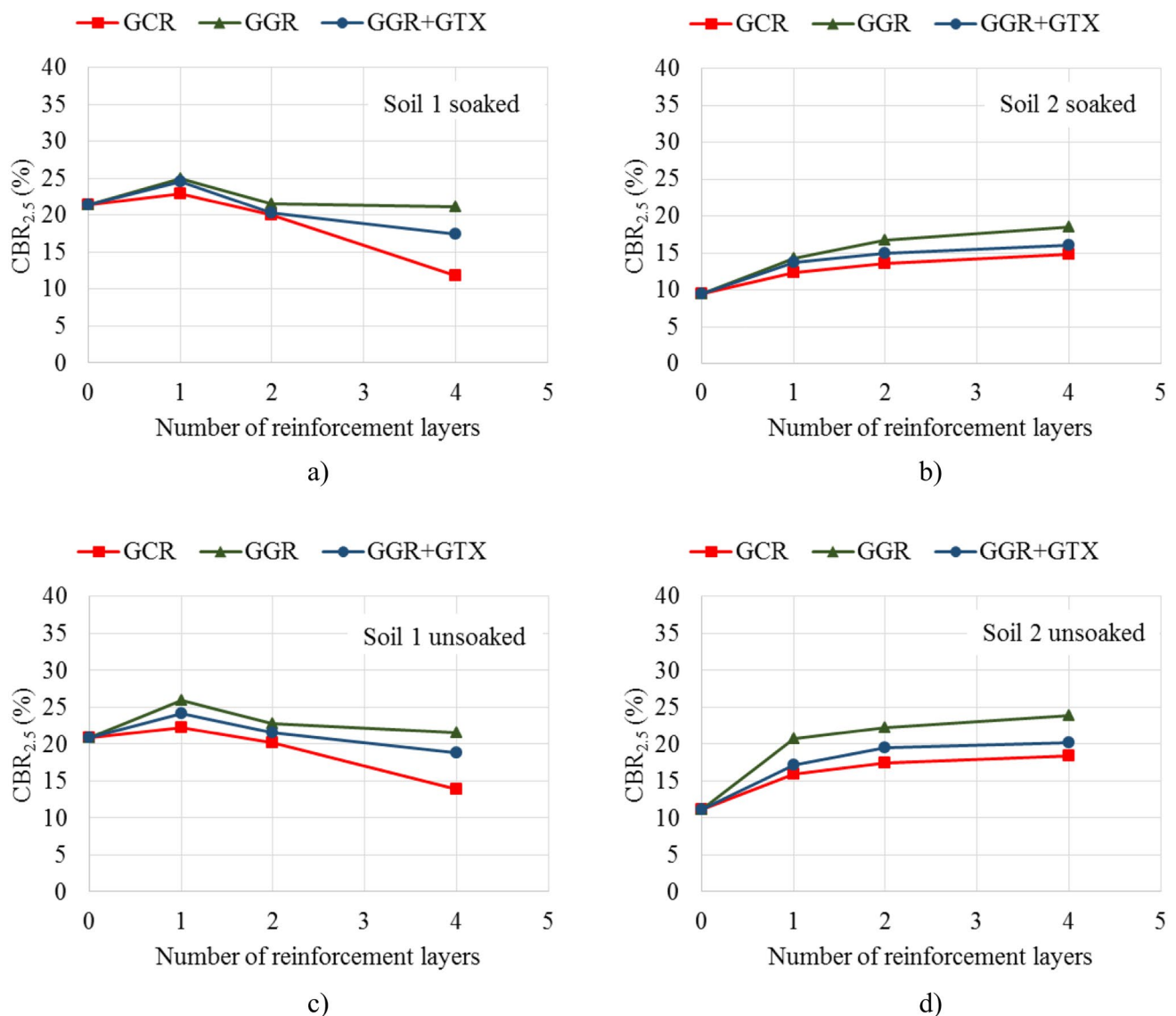


Fig. 10 $CBR_{2.5}$ versus number of reinforcement layers for soil 1 ($w = 11.5\%$, 55 blows) **a** soaked and **c** unsoaked; soil 2 ($w = 11.9\%$, 25 blows), **b** soaked, **d** unsoaked

(classified as GP and SP according to USCS, and including gravel, sand, and fine fractions) reinforced with a nonwoven geotextile (with low stiffness modulus). For the coarser soil (GP), Rashidian et al. [44] concluded that the specimens with two and three layers of reinforcement exhibited lower CBR value than the unreinforced specimen and the specimen with one layer of reinforcement; they attributed the differences to decreased contacts between soil particles due to the geotextile.

Chakraborty and Kumar [43] also analysed circular footings reinforced with two layers of geogrid. From their extensive numerical simulations, the authors concluded that there are critical values for the depths of the first and the second layer of reinforcements (h_1 and h_2 , respectively)

for maximum bearing capacity. Their values depend on the type of soil and the shear stress parameters: h_1 ranges between $0.15d$ and $0.40d$; h_2 ranges between $0.25d$ and $0.45d$; the critical value of h_1 was slightly smaller than the corresponding value for one layer of reinforcement ($u = 0.20d - 0.40d$). Progressively increasing the depth of the first layer of reinforcement for values larger than its critical value, reduced the bearing capacity and the effect of the depth of the second layer on the improvement of the bearing capacity was only marginal [43]. These authors also recommended values for the optimum diameter of the reinforcement, D_r , varying between $2.01d$ and $4.10d$.

The CBR tests with two layers of reinforcement carried out did not meet many of the recommendations by

Chakraborty and Kumar [43]. The mass of soil extended laterally of $1d$ beyond the edge of the footing and $5d$ below the footing, $h_1 = d$, $h_2 = 0.5d$ and $D_r = 3.04d$. Thus, as the depth of the first layer of reinforcement was large and over the maximum range for its critical value, it is likely to have had a detrimental effect on the penetration response and on the bearing capacity of the composite material, and to have led to a marginal contribution of the second layer of reinforcement. The test results (Fig. 9) show that the rigid walls of the mould were able to counteract the reduction in bearing capacity thus expected for most specimens, apart from soil 1 reinforced with two layers of GCR. The lower stiffness of GCR (average value in MD and CMD) relatively to the other reinforcement solutions and its sheet form, restricting the movement of soil particles vertically between layers of soil, may be responsible for the response observed. Soil 2, with smaller particle sizes and larger range of particle sizes than soil 1, can deform and adopt dense packings when restrained laterally by the mould, thus exhibiting larger deformations for similar stress levels.

A similar discussion is extended for the specimens with four layers of reinforcement where the depth of the first layer of reinforcement was $0.5d$ and the spacing between consecutive layers of reinforcement was also $0.5d$. The first layer of reinforcement was still too deep relatively to the optimum depth expected from the corresponding values for one and two layers of reinforcement (always $< 0.4d$). For soil 1, the response of specimens reinforced with sheet reinforcements (GCR and GGR + GTX) was worse than that of the unreinforced soil. This was likely caused by the reduction in bearing capacity expected by the position of the first reinforcement layer and the marginal contribution of the remaining layers, combined with the inhibition of particle movements through sheet reinforcements.

The reduced stiffness and penetration resistance of solutions with higher number of reinforcement layers is not expected in practice. In the field there are no rigid boundaries corresponding to the mould, allowing lateral deformations and mobilisation of soil within a larger area. In addition, field vertical spacing between consecutive layers of reinforcement increases the average number of particles between parallel reinforcement layers, relative to the CBR test. At field scale, the stiffness of the reinforcement (discussed in the following section) also plays an important role.

Type of Reinforcement

The influence of the type of reinforcement in the results was analysed comparing the data for specimens tested under the same conditions (soil 1, soaked, $w = 11.5\%$, 55 blows; soil 2, soaked, $w = 11.9\%$, 25 blows), summarised in Figs. 9 and 10.

For similar test conditions, using GGR resulted in higher CBR and F_{max} . The reinforcement solution with the worst performance was GCR. This trend was observed for all the types of specimens tested under similar conditions (including for different initial water content and number of blows applied during compaction, not shown in Figs. 9 and 10). The different responses observed for the three reinforcement solutions tested may have been caused by several factors related to the geosynthetics: tensile response (strength and stiffness), structure, and soil–geosynthetic interface properties.

The influence of the tensile response of the geosynthetics is difficult to quantify, as the level of mobilisation of the reinforcements could not be assessed. Visual inspections of the specimens after the tests allowed confirming the absence of tensile failure of the reinforcement. The level of deformation in the reinforcement at the end of the tests, which depends on the stiffness of the geosynthetics, could not be quantified. The in-isolation tensile stiffness for 2% strain ($T_{2\%}$, Table 3) of the individual reinforcement types was quantified for both the machine and cross-machine direction. While GGR and GTX have similar tensile responses in both the machine and the cross-machine directions, GCR is highly anisotropic (stiffer in MD). GGR + GTX is likely to exhibit orthogonal isotropy of strength and stiffness, similarly to that of the individual geosynthetics (GGR, GTX), although the corresponding in-isolation stiffness was not measured. The results obtained show that the stiffness anisotropy of GCR may have restricted its mobilisation during the CBR test to some degree and led to larger penetrations for similar loads, compared to GGR and GGR + GTX.

This interpretation of results is based on the conclusions by Nguyen et al. [45]. The authors performed tensile tests of a geotextile exhibiting different strength and stiffness in MD and CMD. The tensile tests were uniaxial (along MD and along CMD) and biaxial (simultaneously along MD and CMD). In the biaxial tensile tests, the load mobilised in both the MD and the CMD were measured; the response of the geotextile exhibited anisotropy of strength and stiffness. The authors concluded that the load–elongation responses for the two directions analysed obtained in the uniaxial and biaxial tensile tests differed and the biaxial response in each direction analysed was stiffer than the corresponding result from the uniaxial tensile test. Then, Nguyen et al. [45] used the average of the uniaxial and biaxial responses in MD and in CMD to interpret the response of reinforced soil specimens in triaxial tests (axisymmetric loading), which was then validated by comparing the observed reinforcement failure strains with the average ultimate strain from the uniaxial and biaxial tensile tests. Assuming that in the CBR test the mobilisation of the reinforcement will be qualitatively similar to that of the triaxial test (axisymmetric

loading), it is expectable that the anisotropy of stiffness influences the response of the reinforced soils samples similarly (qualitatively) to what was observed by Nguyen et al. [45] for triaxial tests.

As discussed in the previous section, the geosynthetic structure may also have played an important role in the response measured. The soil particles could enter the geogrid GGR openings and were able to move vertically through those openings, to readjust their relative positions and to assume a denser state during the test. For specimens reinforced with GCR or GGR + GTX, the reinforcement was a physical barrier to particle movement between adjacent layers of soil (above and below the reinforcement). This effect was particularly important due to the small size of the specimens and small distance between consecutive layers of reinforcement, combined with the presence of the walls of the CBR mould, which restrained lateral deformations of the soil. Combined with its low stiffness, GCR reinforcement exhibited the lowest penetration resistance.

The soil–geosynthetic interaction is responsible for the transference of stresses between the soil and the reinforcement. Therefore, it is likely that such interaction (linked to their structure) affected the response of the reinforced specimens. Often, an interface coefficient is used to characterise the soil–geosynthetic shear strength, defined as the ratio of the shear strength of the soil–geosynthetic interface to that of the unreinforced soil. For example, Ferreira et al. [46] estimated the soil–geosynthetic interface coefficient for some of these geosynthetics confined in soil

1 in direct shear (although for denser specimens): GGR, 0.90 to 0.93; GCR, 0.68 to 0.71. A higher soil–geosynthetic interface strength for GGR than for GCR may have also contributed to a better response of the specimens reinforced with GGR. The difference in soil–geosynthetic interface strength is a result of the different mechanisms: only surface friction for sheet reinforcements; for geogrids also by interlocking of soil particles within the openings of the geogrid. The latter effect restrains the movements of soil particles and can “lock” the soil skeleton up to significant distances above and below the geogrid.

Influence of the Compaction Energy

To analyse the influence of the compaction energy used for preparing the specimens, the results analysed herein refer to specimens of soil 1 compacted, respectively, with 25 or 55 blows (soaked, $w = 11.5\%$): unreinforced and with one layer of reinforcement (GCR, GGR, GGR + GTX). Soil 2 was not compacted with 55 blows, as permanent deformations were induced on the reinforcement during the specimen preparation using that compaction energy (particularly for high water contents).

The property directly affected by the number of blows applied during compaction was the density. Figure 11a summarises the dry density of the relevant specimens of soil 1 compacted using different energies (corresponding to 25 and 55 blows). The specimens compacted with the highest number of blows exhibited the highest dry density (1.0% to 2.4% higher than for 25 blows). The change in

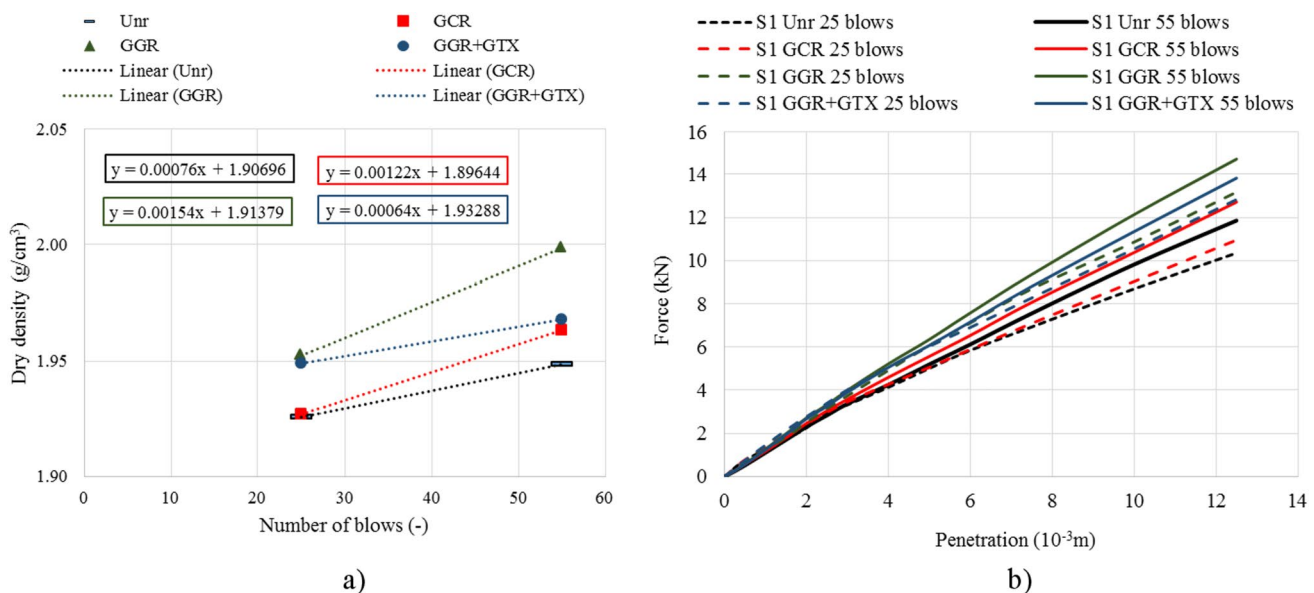


Fig. 11 Influence of the compaction energy used to prepare the specimens (soil 1, $w = 11.5\%$, soaked) unreinforced and reinforced with one layer of GCR, GGR and GGR + GTX: **a** dry density versus num-

ber of blows applied during compaction (25 or 55); **b** load–penetration response of specimens prepared with 25 or 55 blows

dry density of the specimen reinforced with GGR + GTX was similar to the unreinforced specimen (represented by the slope in Fig. 11a). The largest increase in dry density with the compaction energy applied was observed for GCR. For this reinforcement, the dry density of the specimen compacted with 25 blows was similar to that of the corresponding unreinforced specimen. Thus, the disc of

reinforcement may have prevented further densification of the soil within the mould, by limiting movement of soil particles, as discussed before.

Clearly, increasing the compaction energy explains the higher density of the soil, as a larger weight of soil was put in the mould. As expected, a direct consequence of the increased dry density of the specimens observed for

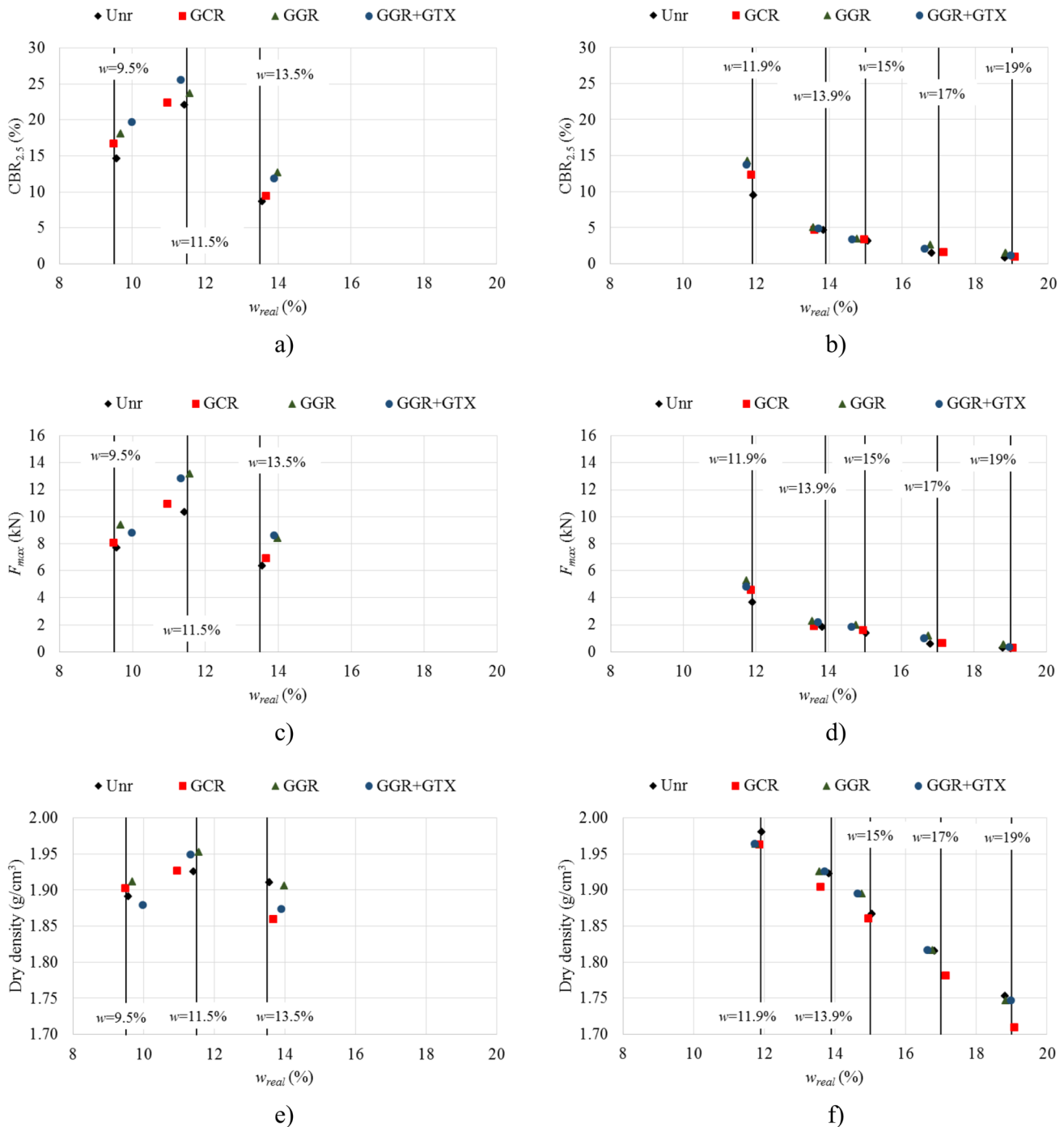


Fig. 12 Influence of the initial water content of the specimens unreinforced and reinforced with one layer of GCR, GGR and GGR + GTX (soaked, 25 blows) on: CBR_{2.5}, **a** soil 1 and **b** soil 2; maximum penetration force, F_{max} , **c** soil 1 and **d** soil 2; dry density **e** soil 1 and **f** soil 2

higher compaction energy was an increased bearing ratio, as illustrated in Fig. 11b, for all reinforcement solutions analysed.

Influence of the Initial Soil Water Content

To assess the degree of sensitivity to the water content of soils, CBR tests can be performed at different water contents [47]. Specimens with different initial water content (w) values were tested: for soil 1, the specimens were prepared with water content of 9.5%, 11.5% (w_{opt}) and 13.5%; for soil 2, the specimens were prepared to the desired water content, w , of 11.9%, 13.9% (w_{opt}), 15%, 17% and 19%. Figure 12 summarises results obtained for soil 1 and 2. The results show that the initial water content influenced the response of the specimens differently for soils 1 and 2.

For soil 1, the optimum water content (11.5%) led to increased performance, for both unreinforced specimens and specimens reinforced with one layer of geosynthetic; specimens with initial water content on the wet side of the optimum value exhibited the worst performance. This is a typical response of a granular soil, with drained response and no (or very limited) plasticity. For soil 2, a different trend was observed, as higher initial water content values always resulted in lower mobilised forces for similar values of the penetration, even for specimens with an initial water content on the dry side of the optimum value ($w = 11.9\%$). This response is typical of a fine soil, loaded under undrained conditions and with plasticity. For both soils, the specimens with higher dry density exhibited higher values of the CBR and of the maximum penetration force (Fig. 12).

The inclusion of the reinforcement layers did not change the trend observed for the unreinforced specimens. The same type of qualitative response was observed for the unreinforced and reinforced specimens (1 layer of geosynthetic), regardless of the type of reinforcement solution adopted. The influence of the reinforcement was greater for specimens prepared with higher values of initial water content, particularly for soil 1. As expected, the influence of the reinforcement for different w values depended on the type of soil: for soil 1 no significant differences were observed for different initial water content values; for soil 2, lower w led to improved $CBR_{2.5}$ and F_{max} .

Influence of the Test Conditions (Soaked Versus Unsoaked)

Soil specimens were tested unsoaked and soaked for 96 h: soil 1 ($w = 11.5\%$, 55 blows); soil 2 ($w = 11.9\%$, 25 blows). As expected, the response of unsoaked specimens was the best in terms of load–penetration curves, CBR and F_{max} ($CBR_{2.5}$ is illustrated in Fig. 10). That difference was greater for soil 2, due to the differences in permeability between

the two soils. The soaked specimens had higher pore water pressures and, thus, lower effective stresses, which then led to lower bearing capacity.

Figures 13 and 14 summarise the results in terms of improvement ratio, IR (Eq. 2), obtained for specimens of soils 1 and 2, respectively, reinforced with GCR, GGR and GGR + GTX with different numbers of reinforcement layers, for both soaked and unsoaked specimens. The results for soils 1 and 2 correspond to different conditions: for soil 1, the water content is w_{opt} and the number of blows used during compaction was 55; for soil 2, the water content is on the dry side of the optimum value and the number of blows applied during compaction was 25.

The improvement ratio was higher for soil 2 than for soil 1, for both soaked and unsoaked specimens. For soil 2, IR is larger for specimens with more reinforcement layers, for all values of the penetration, s , for all test conditions (soaked and unsoaked), and for all the geosynthetic solutions used (GCR, GGR, GGR + GTX). For soil 1, there are some exceptions (discussed before). For soaked specimens of soil 1, in general, the improvement ratio is higher for penetration $s \sim 0.5$ mm; for $s = 1.5$ mm, there is a significant decrease in the improvement ratio, particularly when a larger number of reinforcement layers is used, as discussed before (exceptions, one layer of GGR and of GGR + GTX). For larger values of the penetration and except for specimens reinforced with GGR, namely with 2 and 4 layers of reinforcement, the improvement ratio is practically constant with increasing penetration. For specimens reinforced with GGR, there was an increase in IR for s between 1.5 mm and 6 mm. The trend observed for unsoaked specimens of soil 1 was similar: decrease of IR for $s = 1.5$ mm, although less pronounced than for soaked specimens, with the same exceptions (reinforced with 1 layer of GGR and 1 layer of GGR + GTX).

For soil 2, there was also a decrease in the improvement ratio, but for larger values of penetration s (~ 2 mm) and not as pronounced as for soil 1. For soil 2 specimens reinforced with 2 and 4 layers of GGR, the IR increased for larger values of the penetration, particularly for soaked specimens, while for most of the remaining specimens, the IR was practically constant for $s > 6$ mm. These results show that GGR was particularly effective for large values of the penetration, regardless of the test conditions.

Influence of the Reinforcement on the Penetration Response and on the Stress–Strain Response

The responses of the reinforced soil samples studied, particularly with soil 1, obtained from CBR tests and from triaxial tests are compared and discussed. The stress–strain data from triaxial tests on the same materials reported by

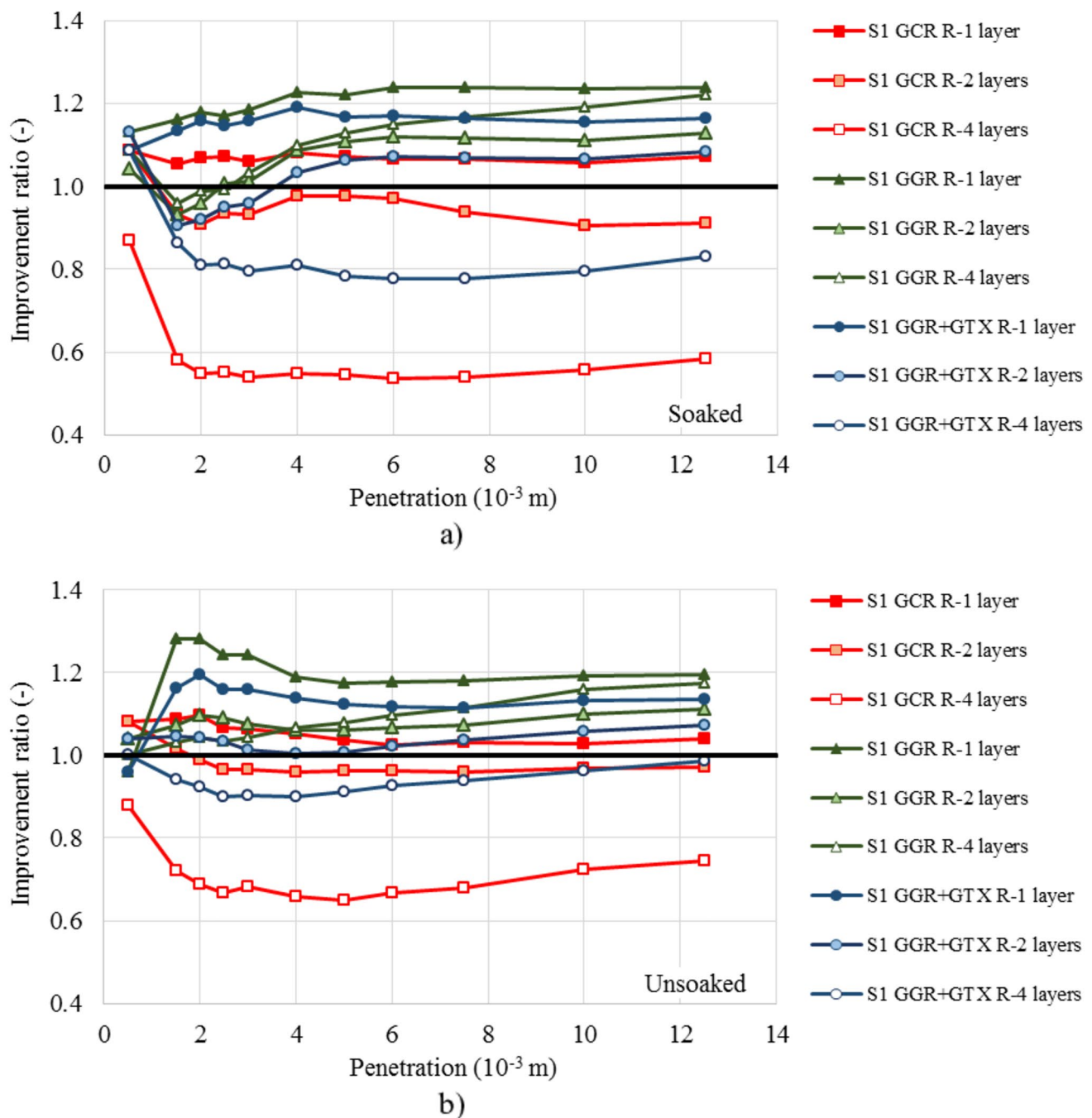


Fig. 13 Improvement ratio of the specimens of soil 1 (S1, $w=11.5\%$, 25 blows) reinforced with different number of layers of geosynthetic (GCR, GGR, GGR+GTX): **a** soaked; **b** unsoaked

Carlos [33] and Carlos et al. [34] are used. However, due to the inherent differences of the tests regarding the setup, specimen preparation and test procedure, there are some differences between the two sets of tests (CBR and triaxial tests), for example: initial water content, density, specimen dimension, and displacement rate. Thus, herein the results obtained from these tests and the effect of a number of factors on the stress–strain response and on the penetration

resistance of the reinforced soils are compared qualitatively only. The data discussed herein from triaxial tests refers to specimens of soil 1 with one reinforcement layer, placed at mid-height of the specimen, or with three reinforcement layers, equally spaced vertically within the specimens and with the second reinforcement layer at mid-height of the specimens (more details can be found in Carlos [33] and Carlos et al. [34]). The triaxial tests included specimens

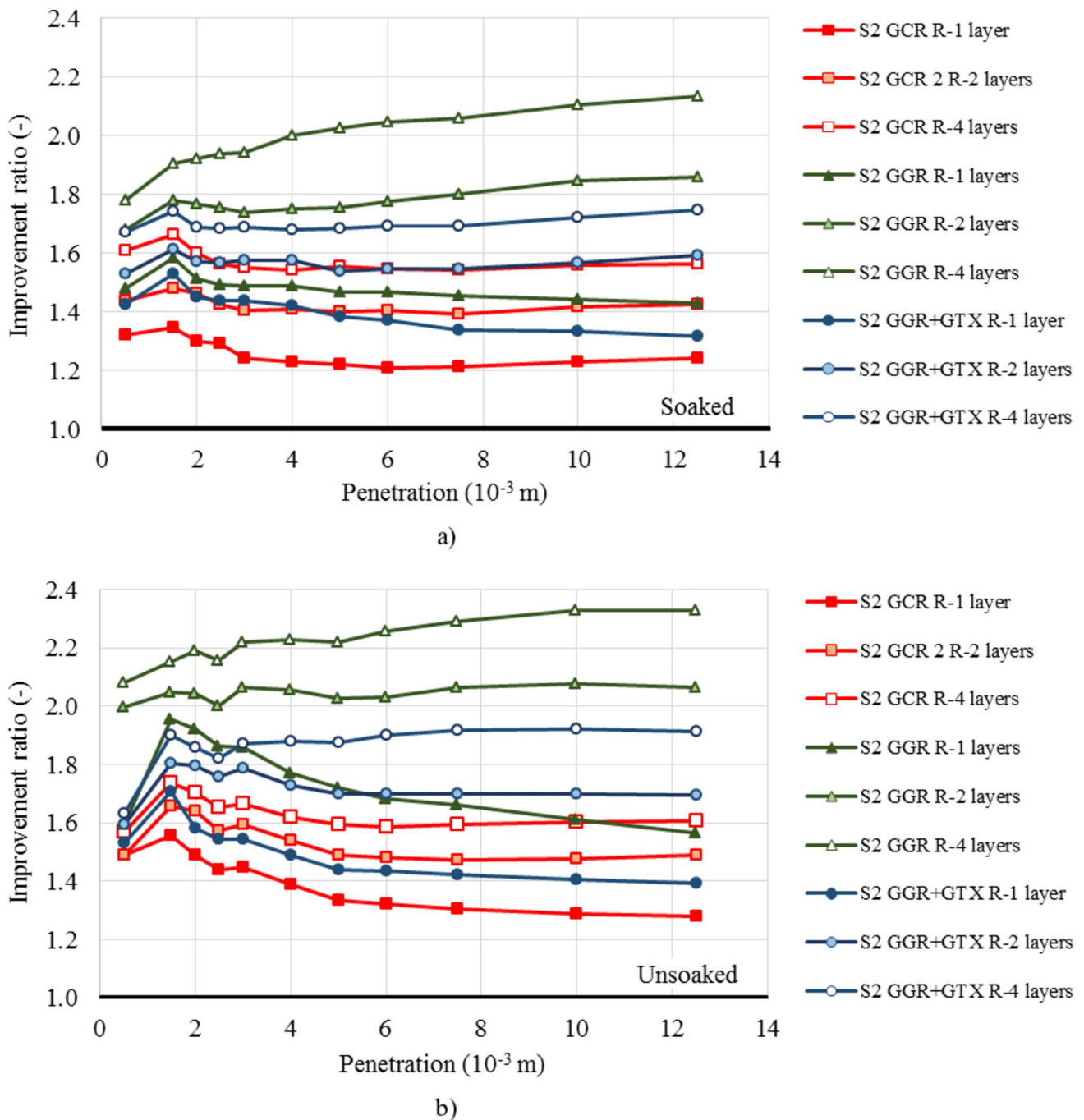


Fig. 14 Improvement ratio of the specimens of soil 2 (S2, $w=11.9\%$, 25 blows) reinforced with different number of layers of geosynthetic (GCR, GGR, GGR+GTX): **a** soaked; **b** unsoaked

prepared with density index (I_D) equal to 53%, 83%, and 97%, and with 0.07 m diameter and 0.14 m high.

Figure 15 summarises strength ratios obtained from triaxial tests by comparing the shear stress of reinforced specimens with that of the corresponding unreinforced soil specimens, from Carlos [33] and Carlos et al. [34]. Figure 15a illustrates the influence of the number of reinforcement

layers (for GCR and GGR) and of the confining stress on the strength ratio for specimens prepared with density index (I_D) equal to 83% for specimens 0.30 m high (similar to the CBR specimens). Figure 15b illustrates the influence of the density index on the strength ratio for different confining stress values of specimens of soil 1 reinforced with 1 layer of GCR, for I_D of 53%, 83%, and 97% for smaller specimens

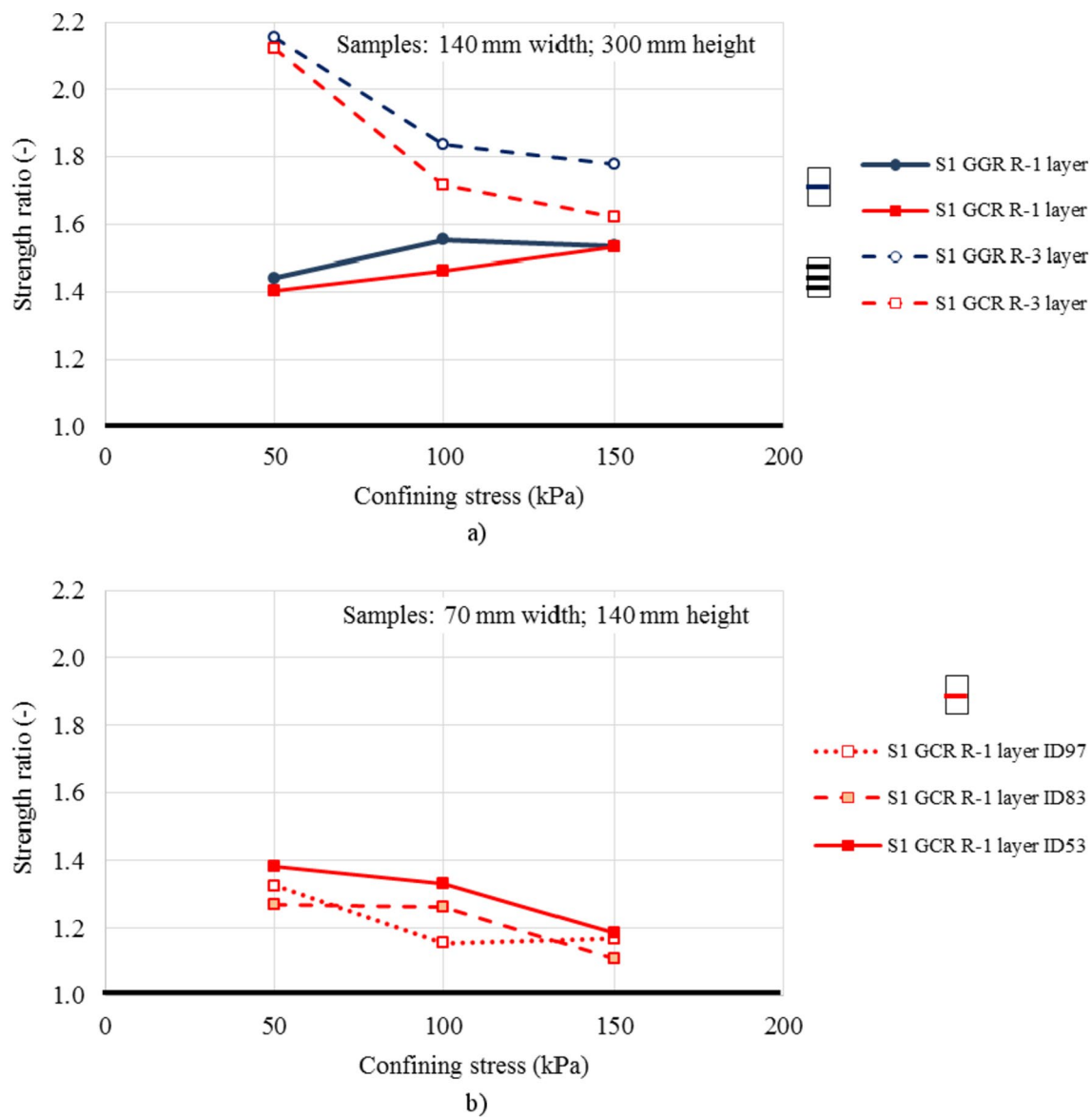


Fig. 15 Results of triaxial tests from Carlos [33] and Carlos et al. [34] for soil 1, expressed as the strength ratio for different values of the confining stress: **a** influence of the number of layers of GCR and

GGR; **b** influence of the density index for specimens reinforced with one layer of GCR

(0.14 m high). The stress–strain response of these specimens (Fig. 15b) and similar unreinforced specimens is illustrated in Fig. 16, where the deviator stress is normalised relatively to the confining stress in the triaxial tests.

Overall, the inclusion of a reinforcement layer had a similar effect on both the stress–strain response and the penetration resistance of the soils. The presence of one reinforcement layer led to increased stiffness and strength of the composite material relatively to the unreinforced soil, as the corresponding values for the improvement ratio (CBR tests, Fig. 13) and strength ratio (triaxial tests, Fig. 15a) were higher than 1. However, increasing the

number of reinforcement layers did not always lead to a further improvement of the mechanical response of the reinforced soil. For soil 1, the CBR and the triaxial tests results exhibited contradictory trends. Increasing the number of reinforcement layers on the coarser soil led to an improved strength ratio (Fig. 15a): the triaxial tests results show that, for the same confining stress, a higher number of reinforcement layers resulted in a higher strength ratio; as the confining stress was increased, the difference in strength ratio between specimens reinforced with one and three layers of GCR or of GGR was reduced; that reduction was particularly important for GCR. Carlos et al. [34] compared

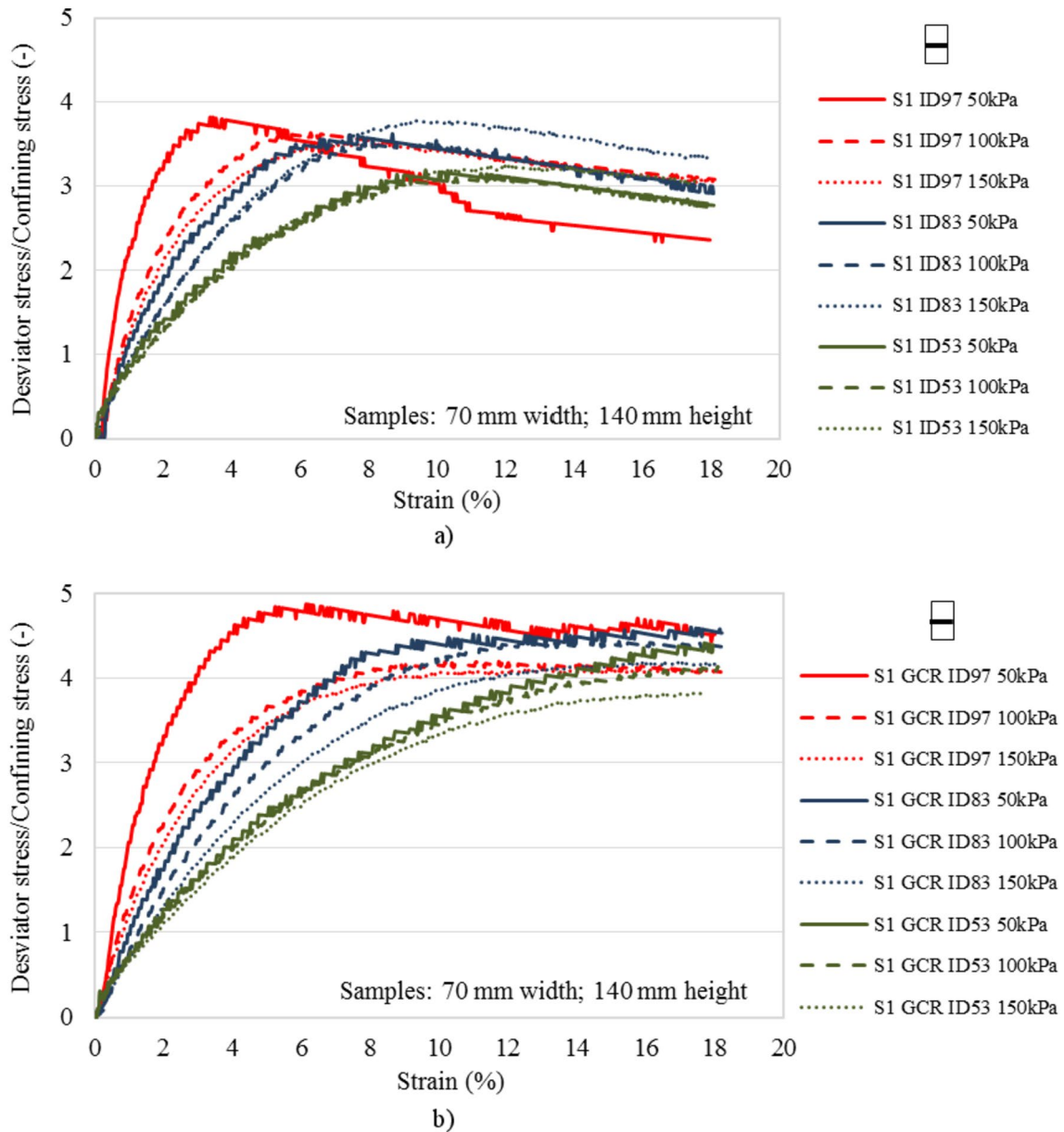


Fig. 16 Results of triaxial tests from Carlos [33] and Carlos et al. [34] for soil 1, expressed as the normalised deviator stress for increasing axial strain for different density index values: **a** unreinforced specimens; **b** specimens reinforced with one layer of GCR

the strength ratio for GCR obtained from specimens 0.14 m high and 0.30 m high and concluded that smaller specimens may lead an overestimation of the benefit from the reinforcement (for both one and three layers), particularly for low confining stress. As discussed by Carlos et al. [34], the height of soil between adjacent layers of reinforcement is key to the performance of the reinforced soil specimens in triaxial tests. This corroborates previous discussions on the CBR tests results. In addition, the CBR tests results showed that, and for soil 1 only, increasing the number of

reinforcement layers of GCR and GGR + GTX an opposite trend was observed: reduction on the penetration resistance. The different responses observed on the two tests (triaxial and CBR) confirm discussions on previous sections, as one of the main differences between the tests (relevant for the present point of discussion) refers to the lateral deformations of the soil (not restricted in the triaxial tests).

The force–penetration and the stress–strain responses (Figs. 9 and 16) show that that GGR was more effective than the other reinforcement solutions. Again, and besides its

high stiffness, the openings of the geogrid are key: allowing movement of soil particles through those openings, while interlocking the soil particles, thus providing an effect similar to an additional confining stress.

The influence of the density index of soil 1 (related to the water content and the compaction energy used in the preparation of the specimens) was also analysed by Carlos [33] and Carlos et al. [34]. Most reinforced specimens did not exhibit a peak response (Fig. 16b), as opposed to unreinforced specimens (Fig. 16a). This shows that GCR (sheet reinforcement) decreased the ability of the soil to dilate, reflecting the confining effect of the reinforcement and its limitation to vertical movement of soil particles within the specimen. Thus, the reinforcement layer reduced strain softening. Specimens with higher density (Fig. 16) exhibited stiffer and stronger responses, for both unreinforced and reinforced soil; however, the strength ratio was not significantly affected by the increase in density index.

Estimates for the Young's Modulus from CBR Data

According to Caicedo [48], in the literature, there are a number of proposals to correlate results of CBR tests with one of the main input parameters of modern mechanistic pavement design methods, i.e. the resilient modulus of the soils. However, these correlations result in a large scatter of values. Herein, the test results were analysed using the most popular correlations between Young's modulus, E , and the CBR suggested by Caicedo [48], including two theoretical relationships, two empirical correlations with the CBR value and two relationships used in pavement design.

The two theoretical relationships used were: Eq. 3, from Magnan and Ndiaye [9] obtained assuming a conical stress distribution below the punch that extends to the perimeter of the mould and a homogeneous distribution of stress below; Eq. 4, derived from the contact theory for non-Hertzian contacts [48], that results in high stress concentration at the edges of the cylindrical punch. Although Eq. 4 applies only for elastic strains, an elastoplastic analysis of the CBR test shows a similar trend [8]: localised plasticity at the edges of the punch, localised high compressive stress under the centre of the punch, and elastic response for most of the soil specimen. Here, p_m is the mean stress under the punch; Δh_e is the elastic displacement of the punch; d is the diameter of the punch; D is the diameter of the CBR mould; L is the height of the specimen; H is the height of the cone (assumed to be similar to the angle of friction of the soil, as suggested by Magnan and Ndiaye [9]):

$$E = \frac{p_m d}{\Delta h_e D} \left[H + \frac{d(L-H)}{D} \right] \quad (3)$$

$$E = \frac{p_m}{\Delta h_e} \frac{\pi d}{4} (1 - \nu^2) \quad (4)$$

Two correlations from the literature have also been applied to the test results, namely Eq. 5, proposed by Heukelom and Klomp [49], and Eq. 6 (d in m) modified from Nielson et al. [50] and Shukla and Sivakugan [51] to incorporate modifications suggested by these authors based on experimental evidence (change in constant):

$$E \text{ (kPa)} = 10340 \text{ CBR} \quad (5)$$

$$E \text{ (kPa)} = \frac{0.75\pi d}{2 \times 0.0254} \frac{(1 - \nu)^2}{1 - 2\nu} 824.1 \text{ CBR} \text{ (} d \text{ in mm)} \quad (6)$$

Lastly, two relationships used in pavement design were adopted (Caicedo [48]): Eq. 7, from the guidelines of the French Centre for Building and Public Works Expertise (CEBTP), developed for African tropical soils; Eq. 8, proposed by the American Association of State Highway and Transportation Officials (AASHTO), in ARA [52]:

$$E \text{ (kPa)} = 5000 \text{ CBR} \quad (7)$$

$$E \text{ (kPa)} = 17600 \text{ CBR}^{0.64} \quad (8)$$

As discussed before, the application of these relationships has limitations; the different values of E obtained for the unreinforced samples reflect some of those limitations (Table 4). The wide range of values obtained for the unreinforced soil (16 MPa to 147 MPa for soil 1 and $s = 2.5$ mm; 8 MPa to 66 MPa for soil 2 and $s = 2.5$ mm) can be explained by the semi-empirical nature of some of the relationships used and because they neglect the effect of parameters representing the compressibility and the shear strength of the soil [48].

The relationships were applied to all test results, including reinforced soil samples (Table 4). The application of Eqs. 3, 4, 5, 6, 7, 8 to the reinforced samples will provide information on an equivalent Young's modulus for the composite material, formed by the soil and the reinforcement layer(s). This allows comparing the overall response of the composite material with that of a soil of equivalent response in the CBR test. The results in Table 4 should be considered together with the discussion on the response of the reinforced samples during the CBR test; E values for samples with 2 and 4 reinforcement layers must be used cautiously, as the vertical spacing between layers is not realistic. For all samples (unreinforced and reinforced) and for the two soils

Table 4 Estimates of the Young's modulus, E , of the different samples tested using equations from the literature (values for penetration of 2.5 mm and assuming Poisson's ratio of 0.3)

| Sample | E | | | | | |
|-------------------------|------------------|------------------|------------------|------------------|------------------|------------------|
| | Equation 3 (MPa) | Equation 4 (MPa) | Equation 5 (MPa) | Equation 6 (MPa) | Equation 7 (MPa) | Equation 8 (MPa) |
| S1 Unr | 16.3 | 20.7 | 147.3 | 33.3 | 71.2 | 96.3 |
| S1 GCR R-1 layer | 17.4 | 22.2 | 158.1 | 66.5 | 76.4 | 100.8 |
| S1 GCR R-2 layers | 15.2 | 19.3 | 137.7 | 57.9 | 66.6 | 92.3 |
| S1 GCR R-4 layers | 9.0 | 11.4 | 81.3 | 34.2 | 39.3 | 65.9 |
| S1 GGR R-1 layer | 19.0 | 24.2 | 172.5 | 72.5 | 83.4 | 106.6 |
| S1 GGR R-2 layers | 16.4 | 20.8 | 148.5 | 62.4 | 71.8 | 96.8 |
| S1 GGR R-4 layers | 16.1 | 20.5 | 146.1 | 61.4 | 70.6 | 95.8 |
| S1 GGR + GTX R-1 layer | 18.6 | 23.7 | 168.9 | 71.0 | 81.7 | 105.2 |
| S1 GGR + GTX R-2 layers | 15.5 | 19.7 | 140.1 | 58.9 | 67.7 | 93.3 |
| S1 GGR + GTX R-4 layers | 13.2 | 16.8 | 119.7 | 50.3 | 57.9 | 84.4 |
| S2 Unr | 8.1 | 9.2 | 65.7 | 14.9 | 31.8 | 57.5 |
| S2 GCR R-1 layer | 10.5 | 11.9 | 84.9 | 35.7 | 41.1 | 67.7 |
| S2 GCR R-2 layers | 11.5 | 13.1 | 93.6 | 39.4 | 45.3 | 72.1 |
| S2 GCR R-4 layers | 12.6 | 14.4 | 102.6 | 43.1 | 49.6 | 76.4 |
| S2 GGR R-1 layer | 12.1 | 13.8 | 98.1 | 41.3 | 47.4 | 74.3 |
| S2 GGR R-2 layers | 14.2 | 16.2 | 115.4 | 48.5 | 55.8 | 82.4 |
| S2 GGR R-4 layers | 15.7 | 17.9 | 127.4 | 53.6 | 61.6 | 87.8 |
| S2 GGR + GTX R-1 layer | 12.1 | 13.8 | 98.1 | 41.3 | 47.4 | 74.3 |
| S2 GGR + GTX R-2 layers | 14.2 | 16.2 | 115.4 | 48.5 | 55.8 | 82.4 |
| S2 GGR + GTX R-4 layers | 15.7 | 17.9 | 127.4 | 53.6 | 61.6 | 87.8 |

studied, the estimates of the Young's modulus, E , ascend when using Eqs. 3, 4, 6, 7, 8 and then 5.

Recommendations for Adapting the CBR Test Mould to Address Some of the Limitations Identified

It is generally assumed that the results of CBR tests have limitations mainly related to the small sample size and the boundary conditions being different from the field conditions. Reinforced soil samples may have additional representativeness issues: (1) the spacing between the reinforced layers in CBR test and in real applications is very different creating unwanted scale effect problems; (2) CBR test procedure was developed for homogenous materials (soils and aggregates) and not for composite materials (such as the reinforced soil).

The analysis and discussion of the CBR tests results confirm that the test method is likely to affect the assessment of the penetration resistance of reinforced soils. Considering the wide use of CBR tests in estimating the resilient modulus of soils, one of the key input parameters of modern mechanistic pavement design methods, some recommendations for

adapting the CBR tests to enable testing of reinforced soil solutions are put forward.

A wider mould should be used, ensuring that the lateral wall is at a distance of $7d$ of the edge of the piston. Thus, a mould with diameter of 750 mm should be adopted.

The reinforcement layout should ensure that the mechanisms forming are representative of field conditions. Therefore, following the recommendations of Chakraborty and Kumar [43], the depth of a single reinforcement layer should be within $0.2d$ and $0.40d$; for two or more layers of reinforcement, the upper layer should be at a depth between $0.15d$ and $0.40d$, and the following layers should have a vertical spacing between $0.25d$ and $0.45d$. The diameter of the reinforcement disc should be $4d$.

The values suggested for the depth and vertical spacings between reinforcement layers still do not replicate distances in field applications. Nevertheless, if the particle sizes are small enough relative to the diameter of the piston, the suggested layout will be able to provide useful information.

In the literature, there have been some attempts at modifying the CBR test setup. For example, Abdi-Goudazi et al. [53] carried out CBR tests with a modified mould, with 250 mm diameter and 375 mm height, and a piston of 50 mm diameter to analyse the influence of the mould size on their results. However, their modified dimensions do not

meet the requirements proposed herein and did not address the limitations discussed.

Future developments of this work should include extensive testing with this improved setup and varying factors such as those analysed in this paper. Further contributions can be obtained by combining such an experimental programme with extensive parametric analysis, validated by the experimental data. Extending the work by Mendoza and Caicedo [8, 16] to reinforced soils and considering the influence of the stiffness and Poisson's ratio of the soil and reinforcements, should allow validating the adequacy of these recommendations for the assessment of the penetration response of soils reinforced with geosynthetics.

Conclusions

In this paper, the effect of geosynthetic inclusions on the penetration resistance of two site-won soils (a granular and a fine soil) was studied using CBR tests, analysing the influence of the reinforcement (reinforcement solutions, number of reinforcement layers), the compaction energy used to prepare the specimens, the initial water content, and the test conditions. The results of the tests carried out allowed to conclude that:

- Overall, the influence of the compaction energy, the initial water content, and the test conditions on response of reinforced specimens (coarse and fine soil) was similar to the corresponding unreinforced soil solutions.
- The reinforcement solutions (geosynthetic, number of reinforcement layers) had a significant impact on the load–penetration responses measured.
- The CBR test was able to capture the beneficial effect of the reinforcement when using one reinforcement layer for both the coarse and the fine soils, and for all reinforcement solutions of the fine soil. However, when the coarse soil was reinforced with two or more layers of sheet reinforcements, the limitations of the CBR test had a significant impact on the results.
- A comparison of the CBR test with that of a circular footing showed that the lateral walls of the mould (restricting lateral deformations of the soil) and the reinforcement layout (depth of the reinforcement and vertical spacing between reinforcement layers larger than their optimum values) may lead to misleading results in the CBR tests.
- These conclusions were qualitatively corroborated by comparisons with the stress–strain behaviour from triaxial tests. For geogrid reinforcement, allowing

particles to move through its openings, the limitations of the CBR test had lower impact on the results, and the test led to an improvement in penetration resistance analogous to the improvement in strength from triaxial tests.

Considering its popularity in pavement design, and to overcome some of the limitations of the CBR test, a modified setup to carry out CBR tests of reinforced soil solutions is proposed. A mould with a larger diameter (up to 15 times the diameter of the piston) is recommended, combined with values for the depth of the first layer of reinforcement and the vertical spacing between consecutive reinforcement layers within recommended limits. This setup aims to ensure that the reinforcement mechanisms mobilised during the CBR test are realistic (though with the inherent limitations of small-scale tests).

Authors Contribution David Miranda Carlos: conceptualisation, methodology, validation, formal analysis, investigation, resources, data curation, writing—review and editing, visualisation, funding acquisition. Margarida Pinho-Lopes: conceptualisation, methodology, validation, formal analysis, resources, data curation, writing—original draft, writing—review and editing, visualisation, supervision, project administration, funding acquisition. Maria de Lurdes Lopes: resources, writing—review and editing, supervision, project administration, funding acquisition.

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Data Availability The data presented in this study are available on request from the corresponding author.

Declarations

Conflict of Interest Maria de Lurdes Lopes is a member of the editorial board of the International Journal of Geosynthetics and Ground Engineering. She was not involved, at any stage, with handling nor peer review of the paper. Financial support has been acknowledged. The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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