

Fibre reinforcement of a uniform sand: stress-penetration and stress-strain responses

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Abstract. Fibres are one of many reinforcement types used to improve the mechanical response of soils, and can be natural, synthetic or waste. This paper studied the influence of polymeric fibres as randomly distributed reinforcement for sand. Fibres have proven useful for soil reinforcement, improving the strength and ductility of sands. Herein, the mechanical response of the sand-fibres composite material was characterised using CBR tests (stress-penetration response) and triaxial tests (stress-strain behaviour). The sand studied is a uniform material with an average particle size of 0.75 mm. Polypropylene fibres were used as reinforcement, with diameter and length of 0.031 mm and 12 mm, respectively. Two different fibre contents were studied (0.5% and 1.0%, in mass). The increase of fibre content led to an improvement of stress – penetration response for larger deformations. However, the CBR tests have some limitations, and the results may not be representative of the response under real conditions. The stress – strain behaviour, in general, was improved with the increase of fibre content, obtaining a better response with 1.0% of fibre content. The fibres need to be mobilised for the effect of reinforcement start to become effective. The soil-fibre interaction mechanism identified was the continuous thin filament fibre reinforcement.

1 Introduction

Soils have very low to no tensile strength. To provide tensile strength to soils, reinforcement is often used. The use of geosynthetic reinforcement allows providing tensile strength along particular directions (often horizontal). However, if the tensile stresses applied to a soil can have varying directions, this type of reinforcement is not the most effective. Alternatively, randomly distributed reinforcements can be used. Particularly, fibre reinforcement has been proved useful for the reinforcement of soils. An additional advantage of fibre reinforcement is that they can be distributed evenly in the soil, allowing for uniform distribution of the reinforcement effect.

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Fibres used in soil reinforcement can be divided into three different categories: synthetic, natural or waste [1]. Synthetic fibres can be made from several polymers, such as polyamide, polypropylene, and polyethylene. These exhibit good durability and resistance [1].

The strength and ductility of sands can be significantly improved by the addition of randomly distributed fibres [2-4]. The mechanical response of fibre reinforced sand depends mostly on parameters relative to the fibres, the soil, the mixture of both materials, and the loading (e.g., confining or normal effective stress) [5].

Regarding the fibres, parameters like the aspect ratio (ratio between the length and the width or diameter of the fibres) [6], stiffness, tensile strength [7], fibre content and orientation [8] can be relevant for the improved mechanical response of the fibre reinforced sand.

The significant soil characteristics include its particle size distribution, particle shape and shear resistance [5]. For the mixture, characteristics such as fibre content and orientation, dimensions of fibres relative to the soil particles, and bulk density of the composite material are considered relevant for the assessment of the mechanical response [5].

Fibres enhance the resistance of a soil, namely by adding tensile capacity and the ability to withstand large deformations, as they occupy voids between soil particles, contributing to interlocking, and they can be mobilised similarly to anchors [9].

In this study, the stress-penetration and stress – strain responses of a sand unreinforced and reinforced with fibres were analysed for two different fibre content values. Two types of tests were used: California Bearing Ratio (CBR) tests and triaxial tests. The results presented herein are part of a wider research and were carried out to validate the approach and procedures for future application to other materials and composites.

2 Methodology

2.1 Experimental programme

This study analyses and compares the influence of different fibre content values (0%, 0.5% and 1.0% of the dry sand mass) on the mechanical behaviour of a uniform sand through CBR and triaxial tests, under the same conditions. In the CBR tests, three specimens were tested for each condition, while in triaxial tests one specimen was tested for each of the three confining pressures adopted. The fibre content used was defined based on data from literature [7, 10, 11].

2.2 Materials

The soil used in this study is a sand, composed mainly of silica (Fig. 1a). The reinforcements used are multifilament polypropylene fibres (Fig. 1b and Fig. 1c).

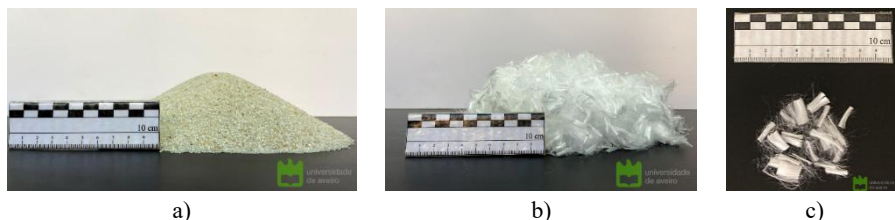


Fig. 1. Materials: a) sand; b) and c) fibres.

The physical, identification, and compaction properties of the soil were characterised. Fig. 2 shows the particle size distribution of the soil (EN ISO 933-1: 2012 [12]). The particle

size distribution provides different parameters: 10%, 30%, 50%, and 60% effective particle size diameters ($D_{10} = 0.54$ mm, $D_{30} = 0.64$ mm, $D_{50} = 0.75$ mm, $D_{60} = 0.80$ mm, respectively); uniformity coefficient, $C_U = 1$; curvature coefficient, $C_C = 1$; percentage of fine particles (smaller than 0.075 mm) equal to 0.03%. Based on this information, it was possible to classify the soil according to the ASTM D2487-11 [13] classification. The sand corresponds to SP - poorly graded sand.

The sand particle density (ρ_s) was obtained through the pycnometer method (ISO/TS 17892-3: 2004 [14]) and $\rho_s = 2.651$ g.cm⁻³. The compaction parameters of the sand were obtained using modified Proctor tests (method C) following the ASTM D1557-07 [15] procedure. The compaction process involves placing 5 layers of material (25 mm high each), compacted with 56 blows (per layer) using a mechanical rammer (weight: 44.52 N; drop height: 457 mm), in a cylindrical mould (diameter: 152 mm; height: 125 mm). The maximum dry density (ρ_{dmax}) and the optimum water content (w_{opt}) of the sand were $\rho_{dmax} = 1.718$ g.cm⁻³ and $w_{opt} = 8.8\%$.

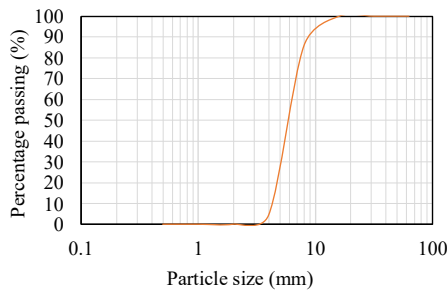


Fig. 2. Particle size distribution of the soil studied.

The polypropylene fibres studied have length (L_f) of 12 mm, diameter of 0.031 mm, density of 0.91 g.cm⁻³ and tensile strength of 380 ± 48 MPa (40 ± 5 cN.tex⁻¹). The fibres have an aspect ratio (fibre length to diameter ratio) of 387; their normalised length and diameter (relative to the average particle size of the sand, $D_{50}=0.75$ mm) are 16 and 0.041, respectively.

The method used to prepare the fibre-reinforced sand mixture was based on Sousa *et al.* (2020) [11], i.e., mixing the fibres and the soil progressively and then adding the water, and mixing it all manually. It is assumed that in the field, the order in which the reinforced soil is mixed (soil, fibres and then water) is the same, but using specific equipment.

The compaction properties of the fibre-reinforced sand were also assessed using modified Proctor tests (ASTM D1557-07 [15], method C). The sand reinforced with 0.5% of fibres (hereinafter, S+0.5%F) had $\rho_{dmax} = 1.717$ g.cm⁻³ and $w_{opt} = 5.7\%$, while the sand reinforced with 1.0% of fibres (hereinafter, S+1.0%F) had $\rho_{dmax} = 1.677$ g.cm⁻³ and $w_{opt} = 2.9\%$.

2.3 Methods

2.3.1 CBR Test

The CBR test, carried out according to ASTM D1883-07 [16], provides the CBR value and a stress – penetration response of materials. The specimens used in the tests were prepared according to method C of the modified Proctor test (ASTM D1557-07 [15]). For each fibre content adopted (0.5% and 1.0% in dry mass of sand), three specimens were prepared with their respective optimum water content. The equipment used in the tests is formed by a loading frame with a mechanical actuator (capacity of 100 kN), a load cell (capacity of 20 kN), connected to a computer for the data acquisition and test control, and a linear variable

differential transformer (LDVT, with a range of 25 mm) connected to a data logger to record the data. The CBR tests were carried out without the soaking process (as the soil studied does not swell), with an axial displacement rate of $1.27 \text{ mm}\cdot\text{min}^{-1}$ (piston diameter: 50 mm) and a constant surcharge at the upper surface of the specimen (applied via three annular plates with a weight of 24.52 N each). Three tests were carried out for each condition: unreinforced sand (US) and fibre reinforcement sand (S+0.5%F and S+1.0%F). The CBR value is determined using Equation (1) as the ratio of the stress measured for a penetration x (2.54 mm or 5.08 mm), σ_x , to that for a standard material (6900 kPa or 10300 kPa, respectively), σ_{xs} .

$$\text{CBR (\%)} = \frac{\sigma_x}{\sigma_{xs}} \times 100 \tag{1}$$

2.3.2 Triaxial Test

Triaxial tests were carried out to evaluate the stress – strain behaviour of both the unreinforced and reinforced sand (0.5% and 1.0% of fibre content). The fully saturated specimens were subjected to consolidated drained tests, performed according to ISO/TS 17892-9:2004 [17]. The height/diameter ratio of the triaxial specimen was $H/D = 2$, with $H = 300 \text{ mm}$ and $D = 150 \text{ mm}$. The confining pressures used in the tests were 12 kPa, 25 kPa and 50 kPa. Each specimen was prepared with five portions of sand or reinforced sand (one per layer), corresponding to 98% of the value of $\rho_{d,max}$ obtained in the modified Proctor test. After mixing the fibres in the same manner as described in section 2.2, the fibre-soil mixture for each layer was placed inside the membrane (within the mould), spread, vibrated with a trowel, and compacted using a manual tamper. The axial strain rate used during the shear phase of the triaxial tests was $0.2 \text{ mm}\cdot\text{min}^{-1}$ and the maximum axial strain reached was 15%.

3 Results

3.1 CBR tests

Table 1 shows the main results obtained from the CBR tests, namely the CBR values for 2.54 mm and 5.08 mm of penetration (and the correspondent coefficient of variation, CV) calculated from the stress – penetration curves obtained for US, S+0.5%F and S+1.0%F (Fig. 3). According to the test standard followed, ASTM D1883-07 [16], in some cases a correction of the curves is needed (if the initial concavity of the curve is upward, the zero point needs to be adjusted). Herein, no correction was made. It was found unnecessary because the complete stress – penetration response was analysed. More, such response may be due to the fibre reinforcement. It must be noted that such correction would influence the CBR values reported.

Table 1. Results of CBR tests.

Parameter	US	S+0.5%F	S+1.0%F
CBR _{2.54} ± CV	13.8 ± 6.7%	10.9 ± 18.8%	11.8 ± 23.4%
CBR _{5.08} ± CV	10.7 ± 9.2%	14.7 ± 5.1%	17.3 ± 12.2%

Fig. 3a, 3b, and 3c show the stress – penetration curves for the different conditions (US, S+0.5%F, and S+1.0%F, respectively). It can be observed that for all specimens, qualitatively

the behaviour was very similar (hardening until a peak is reached, followed by softening). The peak stress became higher as the percentage of fibres increases. For large values of penetration (close to the end of the CBR test), the fibre-reinforced specimens exhibited increased stiffness. This was likely due to the boundary effects of the CBR mould.

The average stress – penetration curves (Fig. 3d) indicate that from the beginning of the test up to the peak stress, the US immediately began to mobilise resistance, increasing the stress up to the peak (which, for the three specimens tested corresponded to a penetration larger than 2.54 mm). The post-peak softening led to a CBR value for 5.08 mm penetration smaller than that for 2.54 mm (Table 1).

For the fibre-reinforced specimens, this behaviour differed. As some level of deformation within the specimen was needed for mobilising the fibres, the initial response (up to ~1.7 mm penetration) was less stiff than for the unreinforced soil. This is illustrated in Fig. 3d and by the values of $CBR_{2.54}$ (Table 1) obtained for S+0.5%F and S+1.0%F, lower than for US. Once the fibres were fully mobilized, the overall response became stiffer than for smaller deformations, until a peak of stress was reached. The CBR value for a penetration of 5.08 mm ($CBR_{5.08} = 10.7\%$ for US; $CBR_{5.08} = 14.7\%$ for S+0.5%F; and $CBR_{5.08} = 17.3\%$ for S+1.0%F) increased with the fibre content, reflecting some of the effect of the reinforcement.

In general, adding fibres to the sand increased the scatter of the CBR values measured (Table 1). This is due to the non-homogeneous nature of the sand-fibre composite. During specimen preparation, the fibres are distributed randomly but as the fibre content increases, fibres tend to accumulate in clumps, creating weaker areas and hindering compaction.

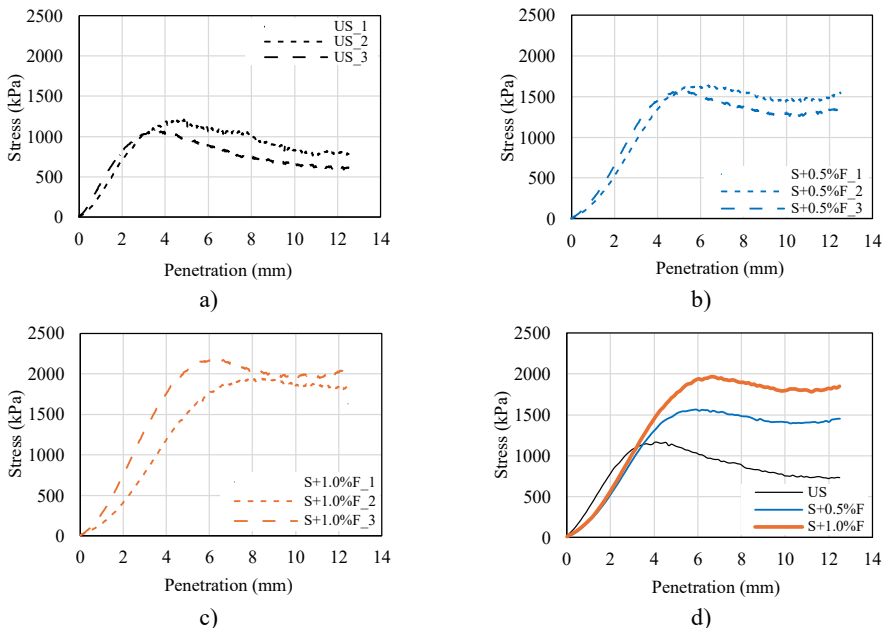


Fig. 3. CBR tests: stress – penetration response for a) US; b) S+0.5%F; c) S+1.0%F; d) average curves (for three specimens) per condition.

3.2 Triaxial test

Fig. 4 shows curves obtained from the triaxial tests: deviator stress – axial strain and volumetric strain – axial strain responses. The tests were stopped at an axial strain of 15%, maximum strain accepted by the equipment (ISO/TS 17892-9:2004 [17]). As a result, most

specimens did not reach critical state (constant stress and volume for increasing axial strain); exception: unreinforced sand (US) and confining pressures of 25 and 50 kPa. The deviator stress – axial strain response was qualitatively similar for all conditions considered: initial response approximately linear, followed by a decrease in stiffness up to a peak in deviator stress; a post-peak softening was observed for all conditions (though more important for the unreinforced sand specimens, US). The volumetric deformation – axial strain response is typical of dense specimens, with an initial slightly contractile response followed by dilatant behaviour.

As expected, for specimens with the same fibre content, higher confining pressure led to higher peak stresses (Fig. 4). For the same confining pressure, the peak deviator stress increased for larger fibre content (Fig. 5). For example, for a confining pressure of 50 kPa the peak deviator stress was 161.7 kPa for US, 181.1 kPa for S+0.5%F and 208.6 kPa for S+1.0%F. This trend was observed for all values of confining pressures analysed (12 kPa, 25 kPa and 50 kPa). The increase in peak deviator stress varied with the confining pressure applied to the specimens S+0.5%F and S+1.0%F, respectively: i) the increase was higher for the lowest confining pressure analysed (12 kPa), 41% and 48%; ii) for 25 kPa, that increase was 16% and 23%; for 50 kPa, it increased 12% and 29%.

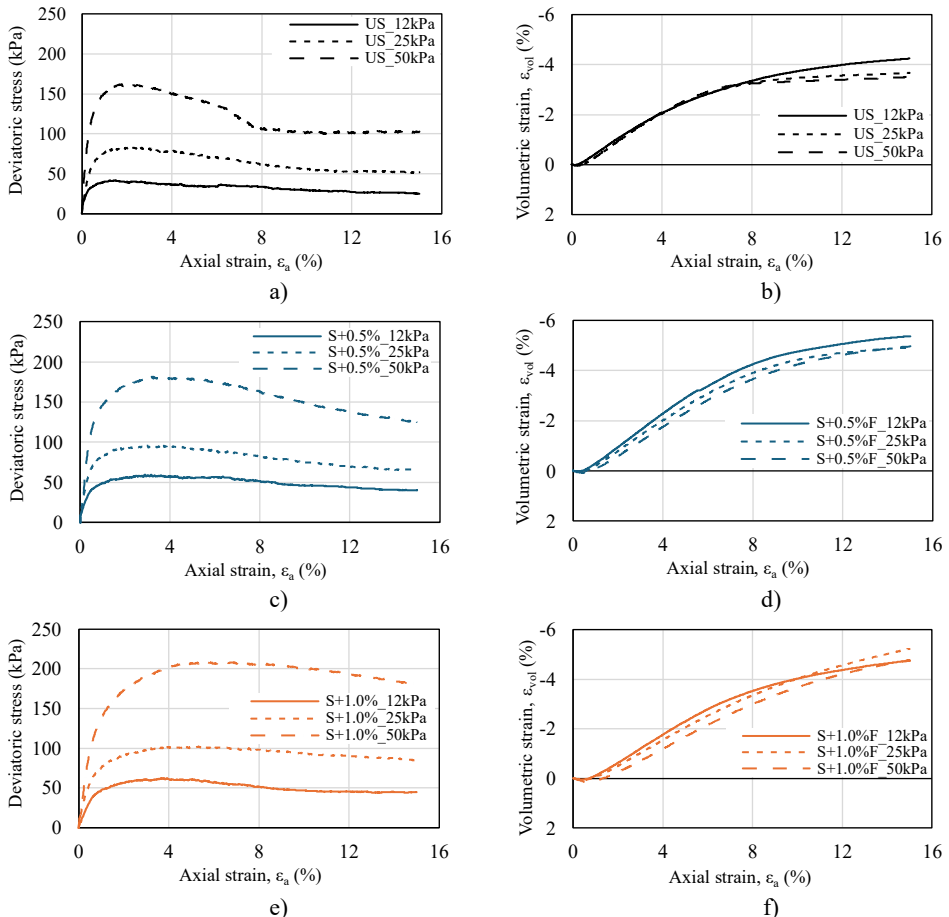


Fig. 4. Triaxial tests: stress – strain response of a) US; c) S+0.5%F; e) S+1.0%F; volumetric strain-axial strain response of b) US; d) S+0.5%F; f) S+1.0%F.

The initial stiffness of the unreinforced sand tended to be larger than that of fibre reinforced specimens; this was particularly clear for a confining pressure of 50 kPa (Fig. 5). This is due to decreased number of contacts between sand particles owing to the presence of fibres; as the fibres are progressively mobilised, the stiffness and shear strength of the fibre reinforced specimens increase, relative to those of the unreinforced sand.

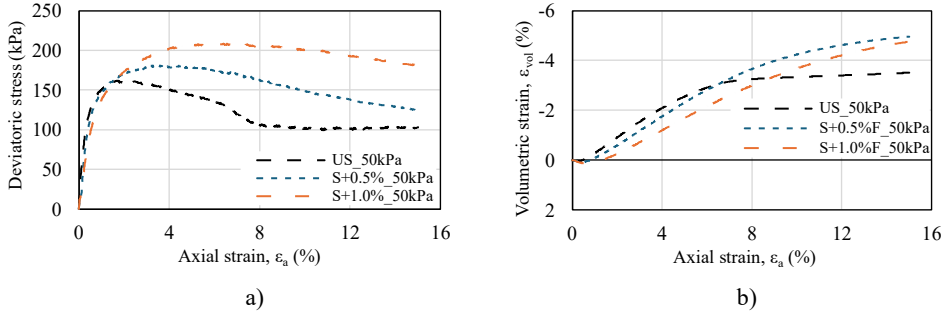


Fig. 5. Triaxial tests (confining pressure of 50 kPa): a) stress – strain response; b) volumetric strain-axial strain response.

Increased confining pressure, led to reduced dilatancy, i.e., decreased volumetric strain, for all conditions tested. Adding the fibres resulted in a delay of the peak, i.e., the peak deviator stress occurred for a higher axial strain (Fig. 4). Thus, overall, the addition of fibres led to a more dilatant response: fibre-reinforced specimens exhibited greater volumetric strain than that of the unreinforced specimens (US); specimen with 0.5% of fibres showed the greatest value for this strain (Fig. 4b, d and f). Part of the voids between sand particles are occupied by fibres, which restrains the ability of the former to displace and occupy them during shearing.

The peak effective friction angle (ϕ'_{peak}) increased as the fibre content increased: $\phi'_{peak} = 37.2^\circ$, US; $\phi'_{peak} = 39.7^\circ$, S+0.5%F; and $\phi'_{peak} = 41.6^\circ$, S+1.0%F. The peak failure envelope was assumed with zero intercept, i.e., no effective cohesion (Fig. 6a).

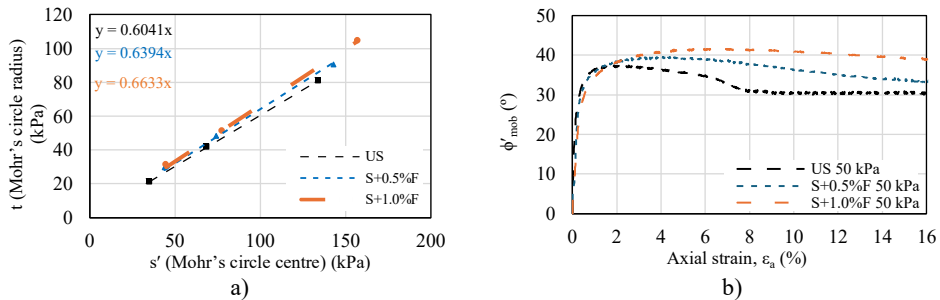


Fig. 6. Triaxial tests: a) peak failure envelope for US, S+0.5%F and S+1.0%F; b) mobilised friction angle for a confining pressure of 50 kPa for US, S+0.5%F and S+1.0%F.

Michalowski [18] and Michalowski and Čermák [7] presented two soil-fibre interaction mechanisms: short-fibre reinforcement; and continuous thin filament fibre reinforcement. Their prevalence is defined as a function of effective diameters of soil particles and fibres. The short-fibre reinforcement mechanism occurs when the fibre diameter (D_f) is larger than the average particle size (D_{50}). The continuous thin filament fibre reinforcement mechanism occurs when the fibre diameter is smaller than $D_{50}/10$ and if the fibre length (L_f) is larger than $10 \times D_{50}$. In this case, $D_f/D_{50} = 0.041$ and $L_f/D_{50} = 16$, which indicates that the soil-fibre interaction mechanism mobilised is likely the continuous thin filament fibre reinforcement,

linked to individual fibre–particle contacts and how fibres fit into the pore space, at particle scale [19]. Such effect was reflected in the angle of friction mobilised during shearing. As an example, Fig. 6b illustrates the mobilised friction angle for a confining pressure of 50 kPa for the three types of specimens studied: US, S+0.5%F and S+1.0%F.

Fig. 7 shows the typical failure modes and the type of failure of each condition tested. In this case, a shear band failure was observed for all specimens. The addition of fibres did not alter the failure modes significantly; they were similar and pronounced, reflecting the values for the angle of friction observed.

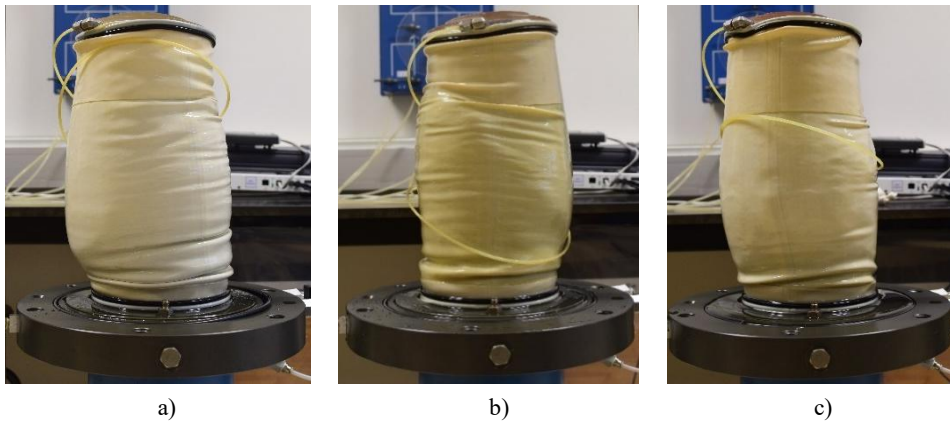


Fig. 7. Triaxial tests: typical failure modes observed for a) US; b) S+0.5%F; c) S+1.0%F.

4 Conclusions

This paper analysed the stress – penetration and the stress – strain behaviour of a uniform sand reinforced with fibres, considering two different fibre content values. Conventional CBR and triaxial tests were carried out and the responses observed were compared to those of the unreinforced sand. Based on the results, the main conclusions are:

- The CBR values for a penetration of 2.54 mm decreased with the introduction of fibres, for both 0.5% and 1.0% of fibre content (in mass). The larger fibre content (1.0%) led to a higher CBR value relative to the lower one (0.5%). This is due to a less stiff behaviour, observed for smaller values of penetration, of the reinforced specimens relative to the unreinforced ones.
- The overall stress – penetration response (CBR tests) of the sand improved with increased fibre content (0.5% and 1.0% in mass), particularly for large values of penetration. For reinforced specimens, the peak stress increased and was reached for values of penetration higher than those for the unreinforced sand and for the reference value of 2.54 mm.
- Adding fibres increased the scatter of stress – penetration responses observed obtained using CBR tests and for low values of penetration.
- The fibre reinforcement did not change the shape of the deviator stress – axial strain response (triaxial tests). The curve included a linear response for lower strains, a decrease in stiffness up to the peak deviator stress, followed by softening.
- The volumetric strain – axial strain response observed is typical of dense specimens (initial slightly contractile response followed by dilatant behaviour).
- At the end of tests (axial strain of 15%) only some unreinforced sand specimens reached critical state (confining pressures of 25 and 50 kPa).

- The overall stress – strain response obtained via triaxial tests improved with increasing fibre content. The best response was obtained with 1.0% of fibres (in mass), reflected by the higher mobilised friction angle.
- The improvement of the different responses observed due to the fibres was only achieved after some level of deformation, necessary to mobilise the fibres.
- The soil–fibre interaction mechanisms observed is likely a continuous thin filament fibre reinforcement reported in the literature. This mechanism is linked to individual fibre-particle contacts and how fibres fit into the pore space, at particle scale. Such effect was reflected in the angle of friction mobilised during shearing.

The CBR test has some limitations, and their results may not reflect realistic conditions. If only CBR values are used to represent the effect of fibre reinforcement, the results can be misleading, as they may suggest that the fibres hinder the response of the sand.

Overall, the fibre reinforcement of the sand studied, using PP fibres, increased the strength and ductility of the sand. This type of fibre reinforcement solution and study can be extended to other soils or alternative granular materials.

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References

1. Y. Yan, M. Huang, X. Qin, Z. Xie, S. Ou, A study on the mechanical behaviour of mixed fiber-reinforced soil. *CSCM*. **20**. e02879 (2024). <https://doi.org/10.1016/j.cscm.2024.e02879>
2. R. L. Michalowski, J. Čermák, Strength anisotropy of fiber-reinforced sand. *Comput. Geotech.* **29(4)**, 279-299 (2002). [https://doi.org/10.1016/S0266-352X\(01\)00032-5](https://doi.org/10.1016/S0266-352X(01)00032-5)
3. S. Lirer, A. Flora, N.C. Consoli, On the strength of fibre-reinforced soils. *Soils and foundations*. **51(4)**, 601-609 (2011). <https://doi.org/10.3208/sandf.51.601>
4. A. Diambra, E. Ibraim, D.M. Wood, A.R. Russel, Fibre reinforced sands: Experiments and modelling. *Geotextiles and geomembranes*. **28(3)**, 238-250 (2010). <https://doi.org/10.1016/j.geotextmem.2009.09.010>
5. O. Ajayi, L. Le Pen, A. Zervos, W. Powrie, A behavioural framework for fibre-reinforced gravel. *Géotechnique*. **67(1)**, 56-68 (2017). <https://doi.org/10.1680/jgeot.16.P.023>

6. N. C. Consoli, L. Festugato, K.S. Heineck, Strain-hardening behaviour of fibre-reinforced sand in view of filament geometry. *Geosynth. Int.* **16(2)**, 109-115 (2009). <https://doi.org/10.1680/gein.2009.16.2.109>
7. R. L. Michalowski, J. Čermák, Triaxial compression of sand reinforced with fibers. *J. Geotech. Geoenviron. Eng.* **129(2)**, 125-136 (2003). [https://doi.org/10.1061/\(ASCE\)1090-0241\(2003\)129:2\(125\)](https://doi.org/10.1061/(ASCE)1090-0241(2003)129:2(125))
8. Y.A. Wang, P.P. Guo, W.X. Ren, B.X. Yuan, H.P. Yuan, Y.L. Zhao, S.B. Shan, P. Cao, Laboratory investigation on strength characteristics of expansive soil treated with jute fiber reinforcement. *Int. J. Geomech.* **17(11)**, 04017101 (2017). [https://doi.org/10.1061/\(ASCE\)GM.1943-5622.0000998](https://doi.org/10.1061/(ASCE)GM.1943-5622.0000998)
9. Y. Wang, J. Liu, Y. Chen, Y. Dong, Z. Liu, Z. Song, X. Ma, Strength assessment of sand stabilized with synthetic polymer and natural fibers. *Bull. Eng. Geol. Environ.* **83(6)**. 1-17. (2024). <https://doi.org/10.1007/s10064-024-03716-8>
10. A. Diambra, A. R. Russell, E. Ibraim, D. Muir Wood, Determination of fibre orientation distribution in reinforced sands. *Géotechnique.* **57(7)**. 623-628 (2007). <https://doi.org/10.1680/geot.2007.57.7.623>
11. W. D. S. V. de Sousa, A. P. F. Santos, S. F. de Sousa Lima, Uso de Fibras de Polipropileno para Reforço de Solo, in Proceedings of the REGEO 2019/ Geossintéticos 2019, São Carlos, São Paulo, Brasil (2020). (*In Portuguese*)
12. European Committee for Standardization (CEN) (2012). EN ISO 933-1:2012 Tests for geometrical properties of aggregates — Part 1: Determination of particle size distribution — Sieving method
13. American Society for Testing and Materials (ASTM) (2011). D 2487-11 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
14. International Organization for Standardization (ISO) (2004). ISO/TS 17892-3:2004 Geotechnical investigation and testing — Laboratory testing of soil — Part 3: Determination of particle density
15. American Society for Testing and Materials (ASTM) (2007). D1557-07 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft³ (2,700 kN-m/m³))
16. American Society for Testing and Materials (ASTM) (2009). ASTM D1883-07 Standard Test Method for California Bearing Ratio (CBR) of Laboratory-Compacted Soils
17. International Organization for Standardization (ISO) (2004). ISO/TS 17892-9:2004 Geotechnical investigation and testing — Laboratory testing of soil — Part 9: Consolidated triaxial compression tests on water-saturated soils
18. R.L. Michalowski, Limit stress for granular composites reinforced with continuous filaments. *J Eng Mech* **123(8)**:852–859 (1997). [https://doi.org/10.1061/\(ASCE\)0733-9399\(1997\)123:8\(852\)](https://doi.org/10.1061/(ASCE)0733-9399(1997)123:8(852))
19. O. Ajayi, L. Le Pen, A. Zervos, W. Powrie, Scaling relationships for strip fibre-reinforced aggregates. *Can. Geotech. J.* **54(5)**, 710-719 (2017). <https://doi.org/10.1139/cgj-2016-0346>