

ISSN 1980-9743
ISSN-e 2675-5475



Soils and Rocks



SOILS and ROCKS

An International Journal of Geotechnical and Geoenvironmental Engineering

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Soils and Rocks publishes papers in English in the broad fields of Geotechnical Engineering, Engineering Geology, and Geoenvironmental Engineering. The Journal is published quarterly in March, June, September and December. The first issue was released in 1978, under the name *Solos e Rochas*, being originally published by the Graduate School of Engineering of the Federal University of Rio de Janeiro. In 1980, the Brazilian Association for Soil Mechanics and Geotechnical Engineering took over the editorial and publishing responsibilities of *Solos e Rochas*, increasing its reach. In 2007, the journal was renamed Soils and Rocks and acquired the status of an international journal, being published jointly by the Brazilian Association for Soil Mechanics and Geotechnical Engineering, by the Portuguese Geotechnical Society, and until 2010 by the Brazilian Association for Engineering Geology and the Environment.

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2020-2024,	43-47 (1, 2, 3, 4)
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ISSN 1980-9743
ISSN-e 2675-5475

CDU 624.131.1

Soils and Rocks

An International Journal of Geotechnical and Geoenvironmental Engineering

ISSN 1980-9743 ISSN-e 2675-5475

Publication of

ABMS - Brazilian Association for Soil Mechanics and Geotechnical Engineering

SPG - Portuguese Geotechnical Society

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The Pacheco Silva Lecture 2024 – Geosynthetics and sustainability: solutions to preserve the environment and reduce the effects of climate change

Ennio Marques Palmeira^{1#} 

Lecture

Keywords

Geosynthetics
Sustainability
Reinforcement
Drainage
Barrier
Climate change

Abstract

Geosynthetics are construction materials that can assume different functions in geotechnical engineering and environmental protection works, such as drainage, filtration, reinforcement, protection and as barriers. They are easy to install, to transport to remote areas and provide engineering solutions that impacts less the environment. This paper presents and discusses the use of these materials in environmental protection works, as well as sustainable solutions using different types of geosynthetics and combinations between them and alternative construction materials such as scrap tires and plastics. Applications in waste disposal, drainage, soil reinforcement, etc. are described and discussed. The use of sustainable geosynthetics such as biodegradable mats for erosion protection and geosynthetics manufactured from recycled plastics and scrap tires is also discussed as well as solutions incorporating geosynthetics to minimize the consequences of climate change. The results from experimental studies and case-histories monitoring presented show that geosynthetics can make a significant contribution to environmental preservation and protection, especially in view of the future consequences of the increasing frequency of extreme events caused by climate change.

1. Introduction

Geosynthetics are established construction materials aiming at different functions in geotechnical and geoenvironmental works such as reinforcement, drainage, filtration, barrier, separation, stiffening and protection, as schematically shown in Figure 1. Some of the advantages of these materials are that they are easy and quick to install and can be transported to remote construction sites. Different products are available capable of meeting the requirements of engineering projects, providing durable and cost-effective solutions. Another advantage of geosynthetics of increasing importance is that they can enable solutions that have less impact on the environment than conventional geotechnical engineering solutions. As an example, Figure 2 compares some environmental impact parameters associated with two different solutions for the filter layer at the base of a pavement (Stucki et al., 2011), one of them a conventional sand filter layer and the other a geotextile layer. The latter solution showed a reduction of more than 90% in the emission of gases that contribute to global

warming, water consumption, acidification, eutrophication and consumption of renewable energy. Over 80% reduction was observed for the consumption of non-renewable energy. Figure 3 presents results of a similar study but comparing environmental impacts from different retaining wall solutions subjected to the same conditions (Damians et al., 2017). In this case, the environmental detrimental effects of a rigid concrete wall and a concrete cantilever wall were compared to those of a geosynthetic reinforced wall. It can be seen that the energy consumption and CO₂ emissions were significantly smaller for the geosynthetic reinforced wall. The results in Figures 2 and 3 are evidence of more environmentally friendly solutions provided by geosynthetic than by conventional geotechnical solutions. Similar conclusions for other geosynthetic applications can be found in Frischknecht et al. (2012), Heerten (2012) and Jacob et al. (2024).

Besides providing better engineering solutions to the environment, geosynthetics can be used in several environmental protection works or can make viable the use of alternative construction materials that would otherwise

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Submitted on March 14, 2025; Final Acceptance on March 24, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.004525>



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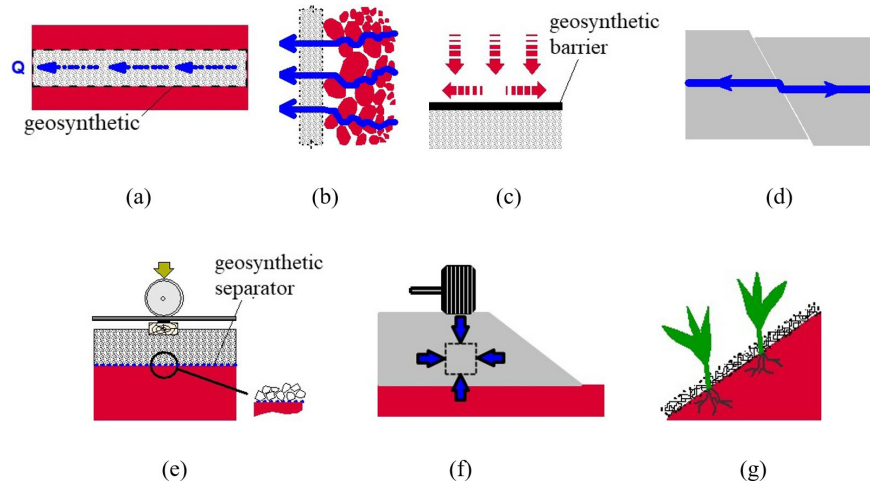


Figure 1. Typical functions of geosynthetics: (a) drainage; (b) filtration; (c) barrier; (d) reinforcement; (e) separation; (f) stiffening; (g) protection.

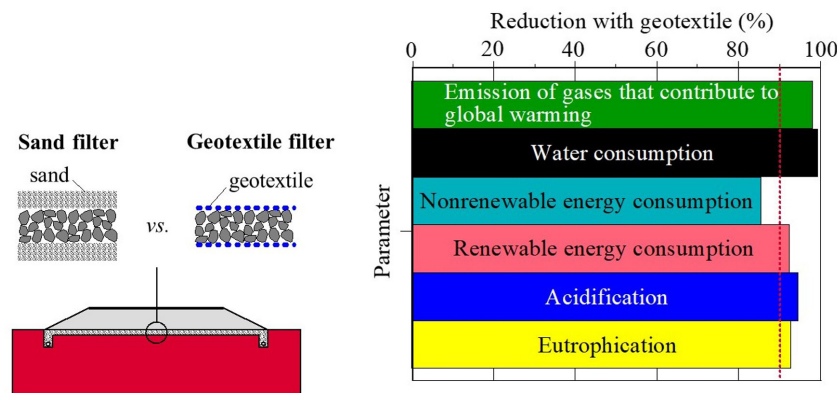


Figure 2. Reduction of some environmental impact parameters with the use of geotextile filter. Source: Stucki et al. (2011).

occupy space in landfills or might be improperly disposed (Palmeira, 2016; Palmeira et al., 2021a; Dabrowska et al., 2023). This paper presents and discusses the benefits to the environment with the use of geosynthetics in different applications, some of them to avoid or mitigate the effects of climate change.

2. Sustainable and environmentally friendly solutions in waste disposal

One of the largest applications of geosynthetics is in waste disposal works, where they can be used for different functions, as schematically shown in Figure 4 in the case of a landfill. Geomembranes and GCLs can be used as barriers, geotextiles as filters and geocomposites as drainage layers; geogrids, geotextiles, geocells and fibres can be used for soil reinforcement etc. Because of the possibility of these multiple functions, geosynthetics can avoid or minimize the use of soils in landfills, reducing the environmental impacts caused by their exploitation and transportation to the site. Besides its traditional use in filtration and drainage, geotextiles can also

be employed to protect geomembranes against mechanical damage (Figure 5a) or to serve as capillary barriers against infiltration of rainwater across cover layers of waste disposal areas (Lima et al., 2018), as schematically shown in Figure 5b. Another environmental application of geomembranes is as vertical barriers to confine regions of contaminated ground, avoiding the lateral spreading of contaminants.

Silva (2014) and Silva & Palmeira (2019) studied the use of nonwoven geotextiles as filters for pre-treatment of the effluents from a landfill. In this case, experimental geotextile panels were used to reduce the pollutant potential of the effluent of the landfill of the city of Cuiabá, MT, Brazil, as shown in Figure 6a. The experimental setup of the research is also presented in this figure, showing a canal (2000 mm long, 500 mm high and 500 mm wide) through which part of the effluent of the landfill was forced to pass. In this canal, one to three vertical panels of nonwoven, needle-punched, geotextiles (mass per unit area, M_A , ranging from 130 g/m² to 1800 g/m² and filtration opening size, FOS , from 0.08 mm to 0.15 mm) were installed to serve as filters for a pre-treatment of the effluent. For comparison purposes, a sand filter ($d_{50} =$

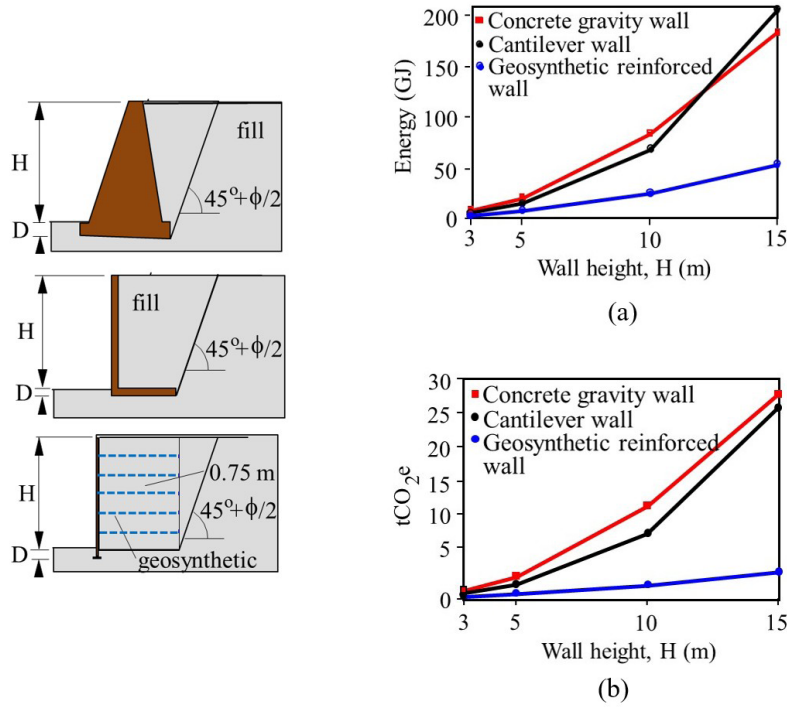


Figure 3. Comparison between environmental impacts from different retaining wall structures: (a) energy consumption; (b) emission of CO₂ (Damians et al., 2017).

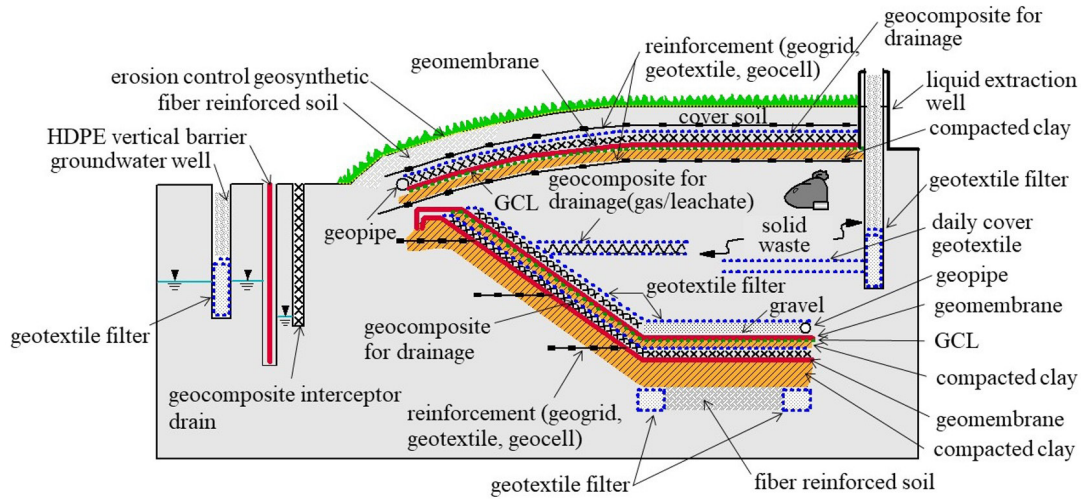


Figure 4. Possible applications of geosynthetics in landfills (modified from Bouazza & Zornberg, 2006).

0.29 mm, $d_{85} = 0.50$ mm, $C_u = d_{60}/d_{10} = 1.7$ and $d_c = 0.048$ mm, where d_n = diameter for which n % of the remaining particles are smaller and d_c = constriction diameter), 500 mm long, was also tested in the same canal. The results in Figure 6b show reductions of leachate chemical (COD) and biological (BOD) oxygen demands reaching up to 40% and 18%, respectively, for heavier geotextile layers. Significant reductions in the presence of heavy metals in the pre-treated effluent with geotextile mass per unit area were also observed (Figure 6c), particularly for zinc (up to 51% reduction).

Figure 6c also shows the reductions in heavy metals in the case of the sand filter and it can be noticed that except for chrome, the reductions observed with the geotextile filter panels were significantly greater. The pre-treatment with geotextiles also reduced the toxicity of the effluent to some extent (Silva & Palmeira, 2019). An additional benefit of using geotextile rather than sand as filter in this type of application is that the geotextile layer may be reused after washing or easily and quickly substituted, besides avoiding the use of a natural filter material. This type of leachate pre-treatment

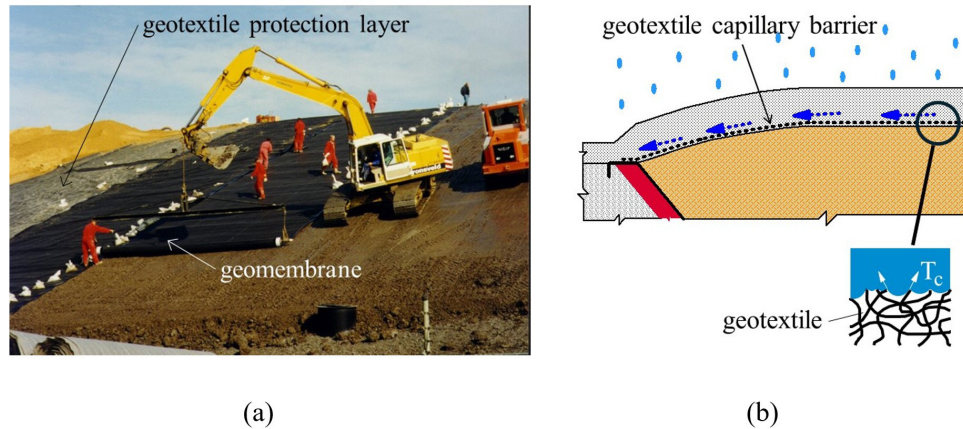


Figure 5. Other uses of geotextiles in waste disposal areas: (a) geomembrane protection; (b) capillary barrier.

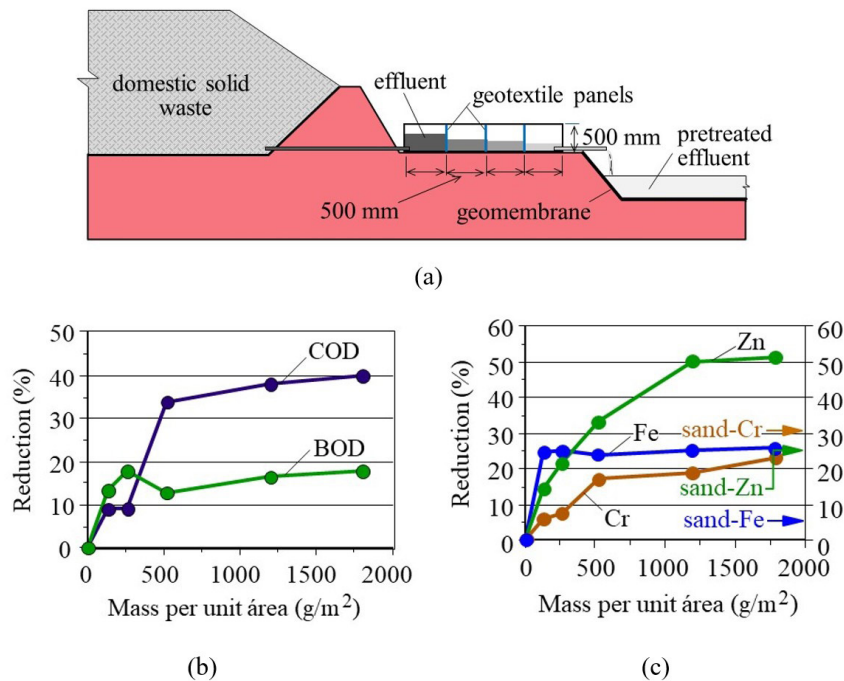


Figure 6. Pretreatment of a landfill effluent with nonwoven geotextile panels (Silva, 2014): (a) experimental setup; (b) reductions of COD and BOD with geotextile mass per unit area; (c) reduction of heavy metals with geotextile mass per unit area.

may be interesting for smaller to medium size communities as a measure to reduce effluent pollutant potential and to reduce the costs of additional effluent treatment processes.

Engineering solutions combining geosynthetics and alternative or low-grade construction materials or that enable the use of wasted materials in geoenvironmental works certainly increase the sustainability of the solutions. Several examples of these combinations can be found in the literature. Junqueira et al. (2006) describe the use of a drainage system in an experimental domestic waste cell consisting of a geotextile filter combined with a drainage layer made of scrap tires (Figure 7). Comparisons between the effluent volumes of this cell and another experimental cell with the same dimensions and conditions, but with a

traditional granular drainage layer, showed no difference in the variation of effluent volume over time.

Palmeira & Silva (2007) investigated the performance of drainage trenches (300 mm wide, 300 mm deep) (Figure 8) consisting of geotextile filters and recycled construction and demolition residues (RCDR) or scrap tire chips for drainage of leachate in large experimental containers (3500 mm × 2500 mm × 1000 mm). The results obtained for different leachate parameters showed the good performance of the alternative drainage system tested (Palmeira et al., 2021a). Kaushik et al., (2017) also investigated the combination of nonwoven geotextile filter encapsulating different sizes of scrap tire chips or mixtures of gravel and tire chips in drainage systems subjected to leachate flow under up to 500 kPa

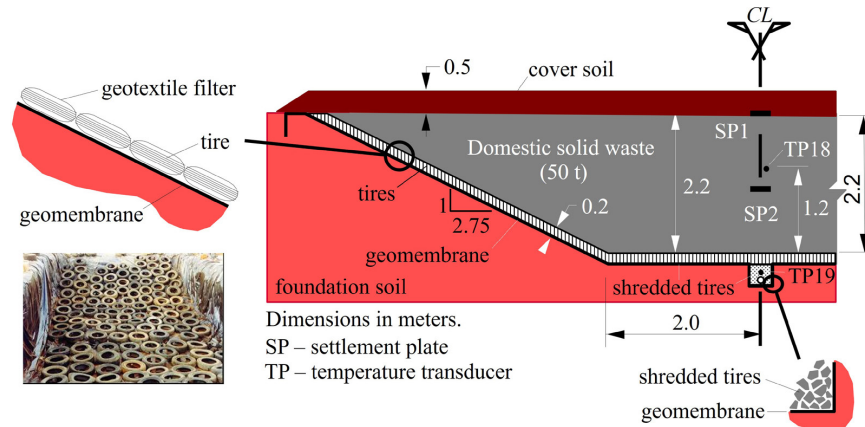


Figure 7. Waste cell with alternative drainage blanket consisting of a geotextile filter and scrap tires (Junqueira et al., 2006).



(a)



(b)

Figure 8. Drainage trenches combining geotextile filter and alternative drainage materials (Silva, 2004): (a) RCDR; (b) scrap tire chips.

vertical stress. According to the authors, these combinations could allow landfill designers and managers to reuse scrap tire chips on a large scale in a more suitable way.

3. Sustainable and environmentally friendly solutions in soil reinforcement

As commented earlier in this paper, geosynthetic reinforced walls and slopes result in less environmental impacts than conventional retaining structures. If alternative construction materials are used in such structures, the additional environmental benefits are twofold: natural building materials will not be used, and a better destination will be given to alternative materials that will not take up space in landfills or be illegally disposed of. An example of the use of alternative construction material associated with soil reinforcement is described in Santos (2011), where two 3.6 m high experimental geosynthetic reinforced retaining walls were constructed using recycled construction and demolition residues (RCDR) as backfill material. Figure 9a depicts the

geometrical characteristics and the instrumentation used in the walls, whereas Figure 9b presents a bird-eye view of the walls at the end of construction. Wall 1 was reinforced with a polyester geogrid (20 mm × 20 mm apertures, tensile stiffness, $J_{5\%}$, of 300 kN/m, maximum tensile strain, ϵ_{max} , of 12%) and Wall 2 was reinforced with a nonwoven, needle-punched, geotextile made of polypropylene ($J_{5\%}$ of 15 kN/m, under in-air conditions, and ϵ_{max} of 70%). The RCDR used as backfill had a d_{50} of 2.5 mm, d_{10} of 0.032 mm, C_u equal to 106, unit weight of 17.6 kN/m³, cohesion of 6 kPa and a friction angle of 41°. Because the walls were constructed on a porous collapsible soil, water reservoirs were constructed at the wall faces to increase the infiltration of water in the subgrade, aiming at enhancing the structural collapse of the foundation soil to investigate the effects of such collapse on the performance of the walls.

Figures 9c and d show the settlement profiles of the wall top surface one year after the end of construction, when the first rainy season in the region had already occurred, and after the artificial inundation of the foundation soil, respectively.

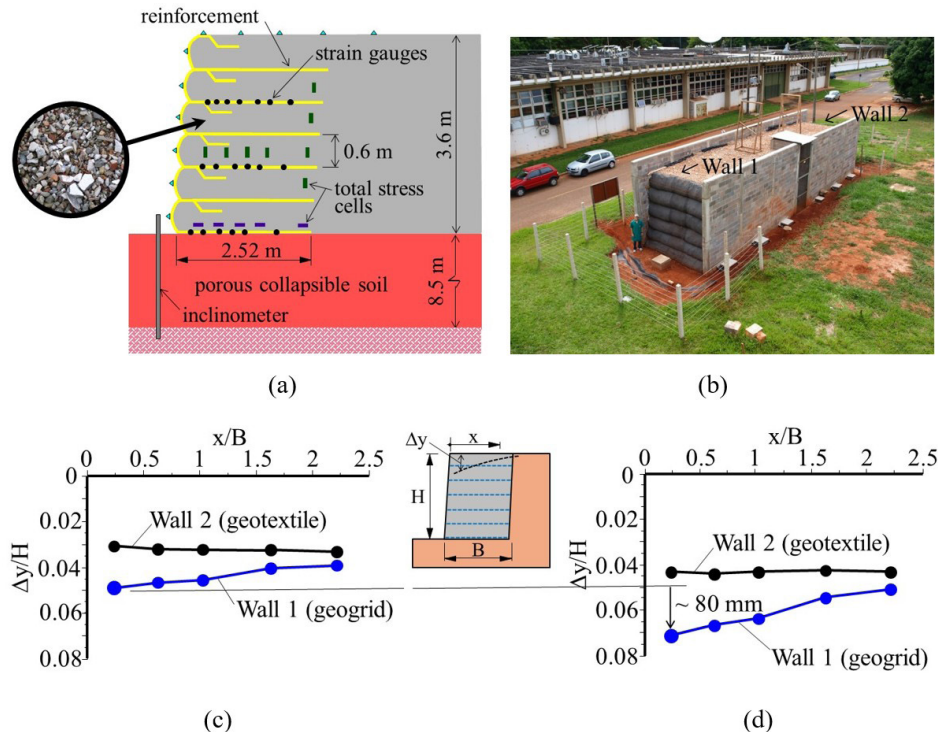


Figure 9. Geosynthetic reinforced walls built with RCD backfill (Santos, 2011): (a) geometrical characteristics and instrumentation of the walls; (b) walls at the end of construction; (c) settlement profile of the wall top 1 year after construction; (d) settlement profile of the wall top after subsoil inundation.

Uniform settlement profiles of Wall 2 can be noted, contrasting with the non-uniform and larger settlements observed for Wall 1. These larger settlements in the latter case were due to the more collapsible nature of the soil underneath Wall 1 in comparison with that underneath Wall 2. Artificial inundation of the foundation soil increased the settlement close to Wall 1 face by 80 mm. Similar settlement mechanisms during rainy seasons have caused damages in retaining structures in the region because of structural collapse of the unsaturated foundation soil, as shown in Figure 10. However, strains in the reinforcement and wall face displacements (the latter particularly for Wall 2) were not significantly influenced by the foundation soil collapse. Bearing in mind typical values of deformation of walls built with conventional backfill materials, the results obtained showed similar magnitudes for the walls constructed with RCD, showing the potentials of using such backfill material in these structures. Additional information on the performance of the walls can be found in Santos (2011), Santos et al. (2013, 2014) and Palmeira et al. (2021a). Other studies investigating combinations of RCD and geosynthetic reinforcements are reported by Fleury et al. (2019), Corrales et al. (2023), Moncada et al. (2024) and Oliveira et al. (2024).

Another sustainable solution involving the combination of geosynthetic reinforcement and alternative construction material is reported by Fernandes (2005) in a railway application

(Victoria-Minas railway). In this case, an alternative geogrid reinforced subballast was employed in substitution to the conventional one. The alternative subballast consisted of a mixture of 50% (in weight) of a local soil (silty sand), 25% of fine ore mining tailings and 25% of the conventional subballast material (sandy gravel), resulting in a subballast material considerably finer than that traditionally used in the region. The reinforcement employed was a polyester geogrid, with 30 mm × 30 mm apertures, tensile stiffness ($J_{50\%}$) of 520 kN/m, tensile strength (T_{max}) of 65 kN/m and maximum tensile strain of 10%. The results obtained in the monitoring of 4 experimental sections subjected to the same traffic as on the real railway revealed that the presence of the geogrid reinforcement improved the performance of the alternative subballast material to the point that it presented deformability similar to that of the unreinforced conventional subballast in the Benkelman Beam tests. Smaller horizontal strains and less breakage of the the grains of the ballast layer were also observed in the experimental sections with the reinforced alternative subballast layer. Additional information on this experiment can be found in Fernandes (2005), Fernandes et al. (2008) and Palmeira et al. (2021a).

Martins (2000) e Martins & Gomes (2002) present the use of ore mine tailings as fill material of a 3-tier reinforced structure in the Fernão Dias highway (BR381), Brazil. In this case-history, each of the three tiers was 6 m high with a 1:2

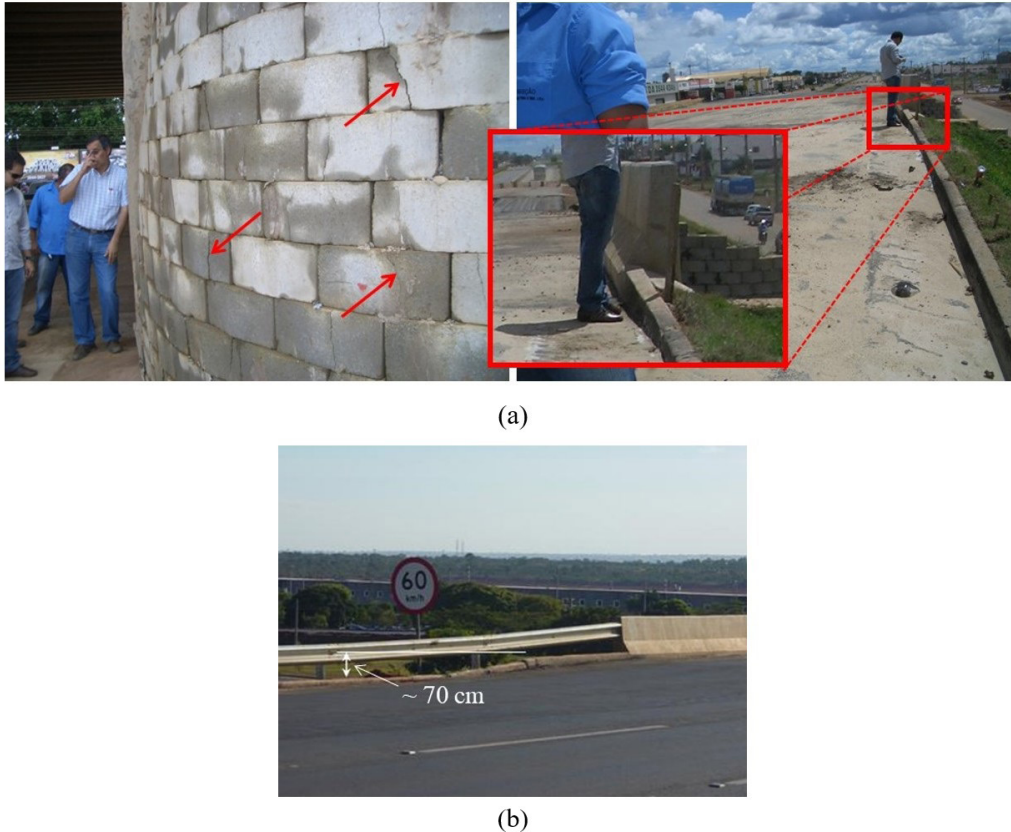


Figure 10. Damages to retaining walls due to structural collapse of the foundation soil: (a) geosynthetic reinforced wall; (b) conventional concrete gravity wall.

(h:v) face batter, resulting in a total reinforced mass height of 18 m in a slope 28 m high, as shown in Figure 11. In the bottom tier, the reinforcement used was a nonwoven geotextile (200 mm reinforcement spacing), made of polypropylene, with M_A equal to 445 g/m², T_{max} of 70 kN/m and ε_{max} of 20%. The middle tier was reinforced with the same geotextile as in the middle tier, but with a spacing of 400 mm between reinforcement layers. The mine tailings fill of the reinforced tiers had a d_{85} in the range 5 mm to 14 mm, d_{50} in the range 0.3 mm to 1.4 mm, 30% to 40% of particles smaller than 0.075 mm, cohesion of 19.6 kPa and 38.9° friction angle. The structure has performed well for 28 years, showing the feasibility of using appropriate mine tailings as a cost-effective backfill material in geosynthetic-reinforced structures.

Combinations of geosynthetic reinforcement and alternative or low-quality fill materials have also been employed in unpaved roads over unstable grounds (Palmeira, 1981; Ramalho-Ortigão & Palmeira, 1982; Góngora & Palmeira, 2012; Mehrjardi et al., 2020; Liu et al., 2025). Palmeira (1981) and Ramalho-Ortigão & Palmeira (1982) describe a pioneer application of geosynthetic reinforcement in Brazil, in which an unpaved road was constructed with a fill material

having 66% of particles passing in the #200 sieve and was reinforced with nonwoven geotextiles. Despite the poor quality of the fill material for this type of application and the type of reinforcement used (only type available in the country at that time), its presence improved the performance of the road and reduced fill consumption between 9 and 24%, depending on the type of reinforcement arrangement.

Góngora & Palmeira (2012) present a study on the performance of unpaved roads on compressible subgrade with RCD as fill material using large-scale model tests. Tests on roads with conventional gravel ($d_{50} = 10.5$ mm, $C_u = 7.1$) as fill were also performed for comparisons. The RCD had a d_{50} of 34 mm and C_u of 6.8. The subgrade had a d_{50} of 0.025 mm, 63% of fines (< 0.075 mm), liquid limit of 39%, plasticity index of 10%, dry unit weight of 14 kN/m³, moisture content of 27.1% and CBR of 4.2%. Three geogrids were used as reinforcement, with values of tensile stiffness ($J_{50\%}$) ranging from 130 kN/m to 1500 kN/m. For a 25 mm rut depth limit at the road surface, the results showed that the geogrid reinforced RCD fill outperformed the conventional gravel, with a number of load repetitions to reach that rut depth 2.2 times greater for the former type of fill. The results also showed the good performance of the reinforced RCD after maintenance of the road surface. Mehrjardi et al. (2020) also observed a good performance

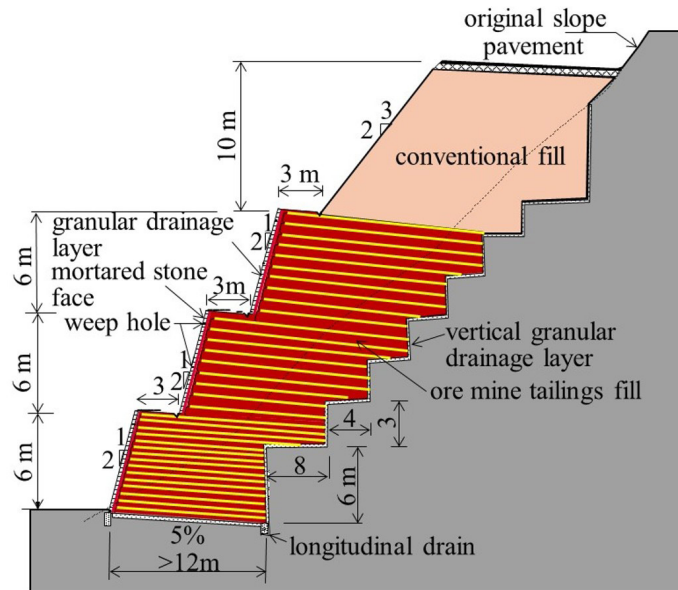


Figure 11. 3-Tier superimposed wall system in BR381 highway (Martins, 2000).

of a RCDW fill reinforced with geocell in large-scale tests on unpaved roads on a compressible subgrade, with similar results of rut depth versus number of loading cycles for the alternative fill material and the reference unreinforced natural aggregate. Liu et al. (2025) describe the use of steel slag as infill material of floating geosynthetic-encased columns in soft clay under cyclic loading, also showing good performance. A summary of the combinations of geosynthetics and alternative construction materials can be found in Vieira (2022).

4. Environmentally friendly solutions in mining

Geosynthetics have a wide range of applications in mining as barriers, filters, reinforcement and in drainage. One increasingly popular application is the use of geotextile tubes and mattresses to dewater mine tailings (Figure 12). This solution can provide a safer method for tailings storage in contrast to traditional tailings dams and this is particularly relevant in Brazil considering recent major disasters involving this type of dam.

Barrantes (2023) and Barrantes et al. (2023) studied the use of geosynthetic tubes to dewater fine grained materials using large-scale laboratory equipment. The dimensions of the tubes tested are presented in Figure 13a, as well as the instrumentation used to monitor tube behaviour. General views of the experiments are presented in Figures 13b (single geotextile tube) and 13c (stack of tubes). The tubes were 800 mm in diameter and 1000 mm long. The geotextile used to manufacture the tubes was a nonwoven, needle-punched, geotextile made of polyester, with a mass per unit area of 200 g/m², tensile strength of 9.8 kN/m, tensile stiffness ($J_{5\%}$) of 11.5 kN/m, maximum tensile strain of 83% and filtration opening size of 0.115 mm. The infill fine-grained material

had a d_{10} of 0.0036 mm or 0.040 mm (depending on the use or not of dispersing agent in the grain size analysis), d_{50} of 0.027 mm or 0.120 mm, d_{85} of 0.063 mm or 0.259 mm, C_u of 8.4 or 3.7, 87% or 29% in weight passing the #200 sieve, liquid limit of 37% and plasticity index of 9%. The tubes were filled with the use of a pump in 1 or 3 filling stages to assess the influence of the number of filling stages on tube performance. Figure 14a shows the variation of tube height with time for one and three stages filling processes. It can be noted that the storage capacity of the tube increases with a larger number of filling stages. The drainage capability of the tubes is revealed in Figure 14b, where it can be noticed that the pore pressure measured by piezometer P2 at the middle of the tube base (Figure 13a) is quickly dissipated after each filling stage. Figure 14c shows the tensile strain distribution along the tube perimeter. As expected, larger strains were obtained for 3 filling stages, although the strains were much smaller than the maximum geotextile tensile strain at failure. Particles that piped through the geotextile during the tests were collected for grain size analyses and the results obtained (Figure 14d) revealed that the diameters of the piped particles were considerably smaller than those of the original infill material and smaller than the geotextile FOS.

Barrantes et al. (2023) also evaluated the accuracy of some available analytical methods for the prediction of geotextile tube behaviour. Figure 15a and b show comparisons between different tube geometrical parameters measured and predicted by the methods by Plaut & Suherman (1998), Lawson (2008) and Guo et al. (2014). In general, predictions and measurements compare well in terms of tube cross-section area, maximum tube width and vertical stress at the base of the tube (Figure 15a). Regarding final tube height, Lawson (2008) and Plaut & Suherman presented satisfactory

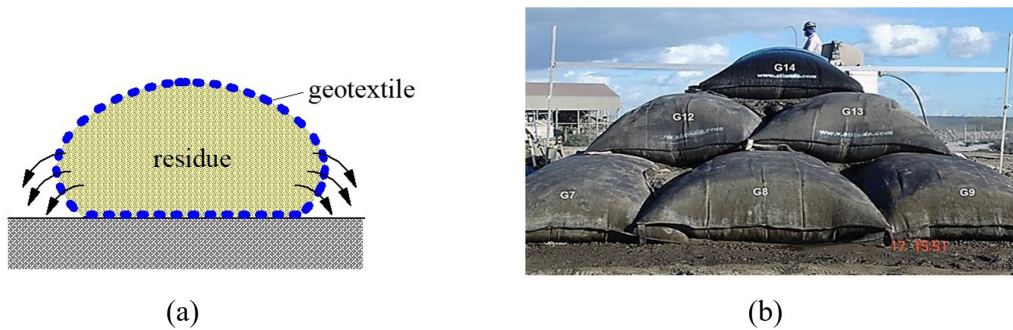


Figure 12. Geotextile tubes for dewatering of mine tailings: (a) isolated geotextile tube; (b) stack of geotextile tubes (courtesy of Prof. M.G. Gardoni).

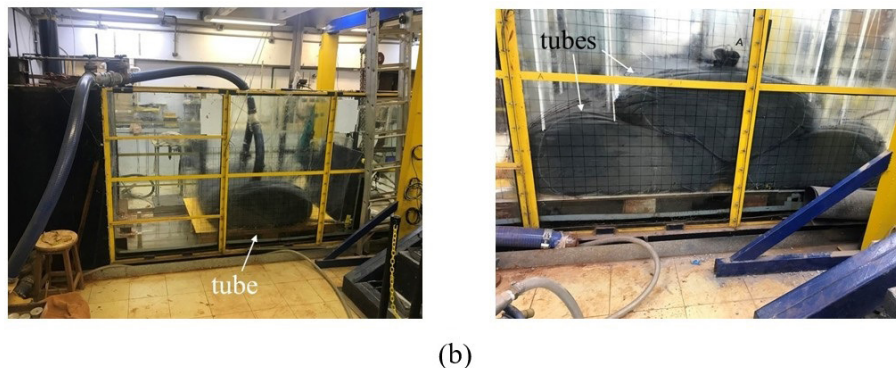
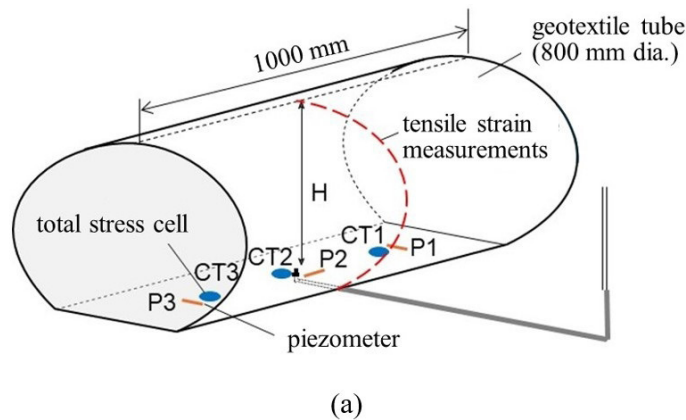


Figure 13. Tests on geotextile tubes (Barrantes et al., 2023): (a) single tube; (b) stacked tubes.

predictions (Figure 15b), but significant deviations between predictions and measurements for the tube base width were observed in the cases of the methods by Guo et al. (2014) and Plaut & Suherman (1998). Despite some deviations, the results obtained are encouraging regarding the potential of those analytical methods for the prediction of geotextile tube behaviour under conditions similar to those employed in the experiments carried out.

The dewatering efficiency of geotextile tubes and geomattresses has also been investigated under field conditions. An example of this is the research reported by Silva (2017) and Morais et al. (2020) on the behaviour of stacked geotextile tubes in the Fazenda Brasileiro gold ore

tailings dam, Bahia, Brazil. The gold ore tailings had 75% of fine particles and a stack of 14 geotextile tubes (2 m wide and 6 m long each) was monitored. The geotextile used to manufacture the tubes was a woven product, made of polypropylene, with a mass per unit area of 554 g/m² and a tensile strength of 100 kN/m. A plan view of the experimental setup is shown in Figure 16a and Figure 16b shows a view of the stacked geotextile tubes. Significant tailings dewatering was observed after 72 hours, when the solids content in the tubes reached 80%, as shown in Figure 16c. This high level of reduction in the moisture content of the tailings was confirmed at the end of the experiment after cutting the geotextile to extract tailings specimens for additional tests

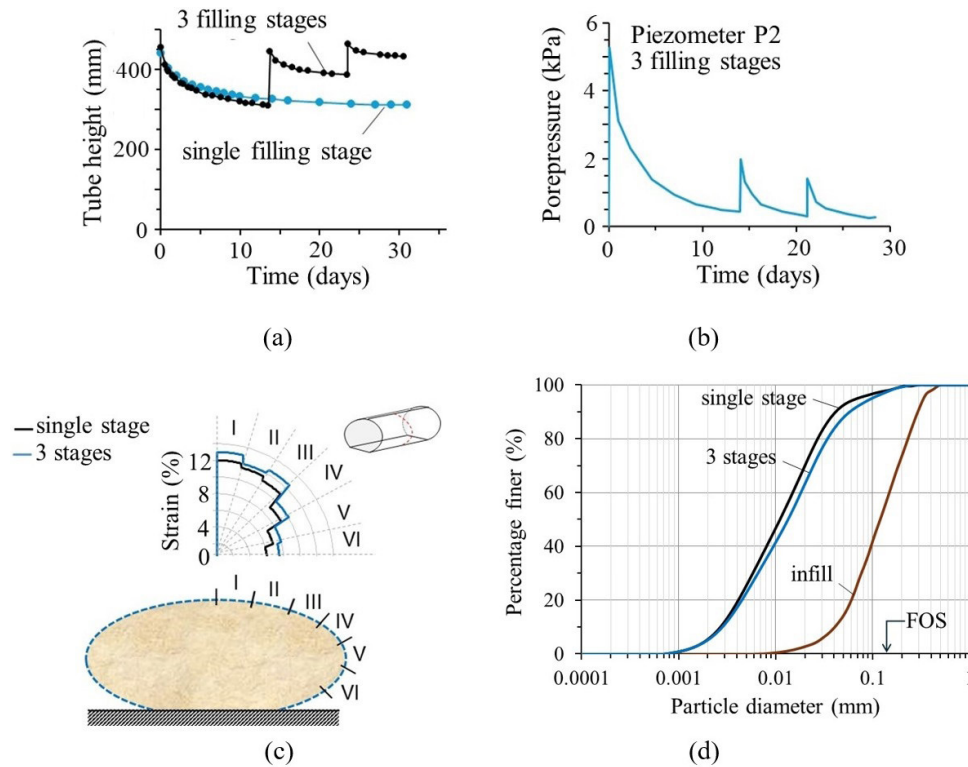


Figure 14. Behaviour of an isolated geotextile tube (Barrantes et al., 2023): (a) tube height versus time; (b) pore pressure versus time; (c) tensile strain distribution along the tube perimeter; (d) grain size distribution of the particles that piped through the geotextile.

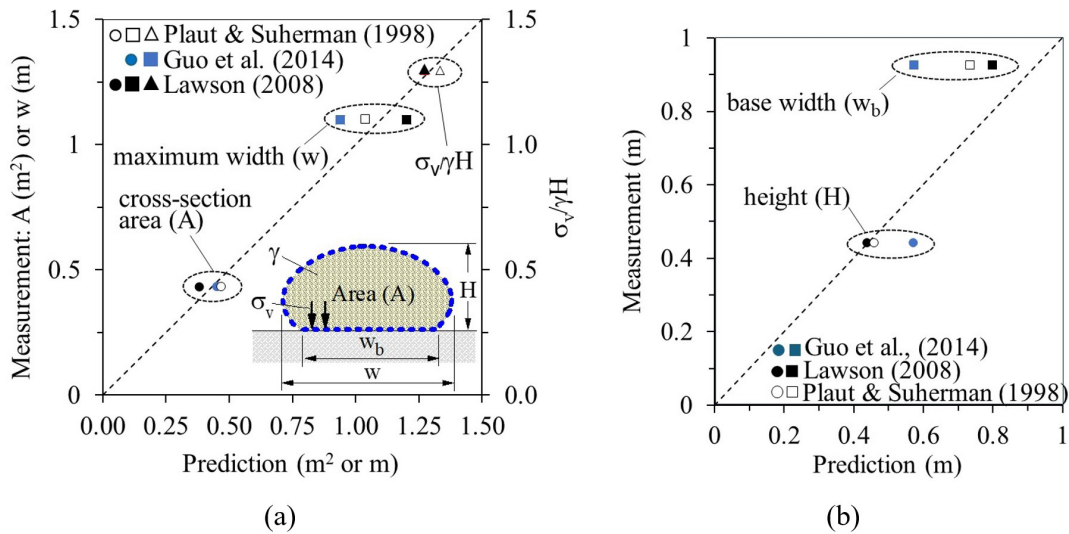


Figure 15. Comparisons between predictions and measurements (Barrantes et al., 2023): (a) tube cross-section area, maximum width and vertical stress; (b) tube height and base width.

(Figure 16d). Further information on the behaviour of the geotextile tubes can be found in Silva (2017), Morais et al. (2020) and Palmeira et al. (2021b).

Geomattresses were used for dewatering of dredged material from the Risoleta Neves dam reservoir (Terratek, 2017a, b; FOS, 2017; Ortigão, personal communication, 2024; Fabel, personal communication, 2024). The silting of

the dam reservoir was a consequence of the failure of the Fundão tailings dam, in 2015, Bento Rodrigues District, state of Minas Gerais, Brazil. The dredged material had a unit weight of 22.5 kN/m³, a friction angle of 37° and a cohesion of 20 kPa. The geomattresses were manufactured with a nonwoven, needle-punched geotextile, made of polyester, with a mass per unit area of 950 g/m². The tensile strength

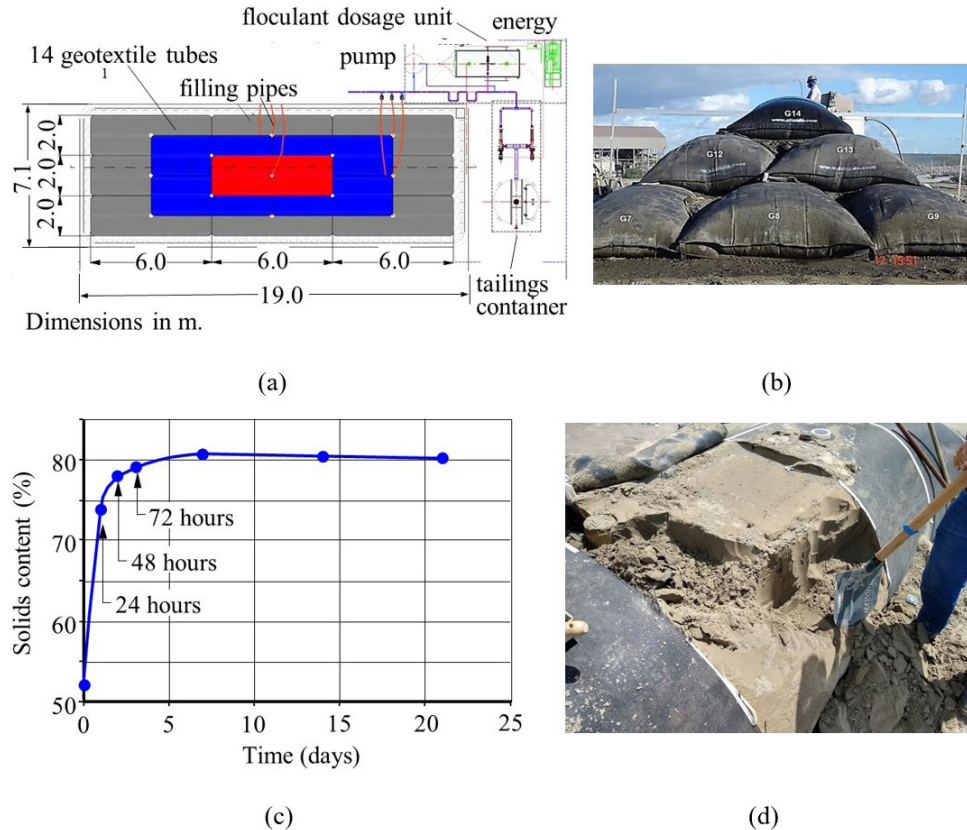


Figure 16. Dewatering of gold ore tailings with geotextile tubes: (a) experimental setup; (b) stacked geotextile tubes (courtesy Prof. M.G. Gardoni); (c) solids content versus time (Morais et al., 2020); (d) view of the tailings at the end of the test (courtesy Prof. M.G. Gardoni).

and the maximum tensile strain of the geotextile were equal to 60 kN/m and 79%, respectively, and its filtration capacity was greater than 500 m³/m²/h. Each geomatress was 18 m wide, 50 m long, 1.8 m maximum height, with a storage capacity of approximately 2350 m³. Figure 17a shows the geometrical characteristics of the geomatresses and Figure 17b shows one of the geomatresses during filling. The geomatresses were stacked, as depicted in Figure 17c. An isolated geomatress was instrumented with 3 vibrating wire piezometers installed at its mid-height and with vibrating wire extensometers at three locations along its top surface (Figures 17b and d). Figure 18 shows a geomatress during the dewatering process.

Figure 19a presents the variation of porepressure at the three measuring points (Figure 17d) versus time. In general, the porepressure increased during the mattress filling stages, but dissipation was fast thereafter, showing the geomatress good drainage capacity. The variation of geotextile tensile strain with time at the middle of the mattress length is shown in Figure 19b. The strain increased during the filling process, reaching a maximum tensile strain of 8.6%, only approximately 11% of the maximum tensile strain of the geotextile. There was some reduction in the geotextile strain after mattress filling as a consequence of drainage and consolidation of the infill material. As in the

laboratory study by Barrantes (2023), the experiments under field conditions have confirmed the excellent performance of geotextile tubes and mattresses for the dewatering of tailings, muds and similar materials.

Another interesting solution related to reducing the environmental impacts due to mining activities is the use of floating silt curtains (FSC) to favour sedimentation of solids in suspension aiming at reducing silting of rivers and lakes, as schematically shown in Figure 20. The curtains change the flow pattern and reduce the water speed to favour significant sedimentation of solids in suspension before they reach important regions or water bodies.

An interesting application of FSC took place in the municipality of Patrocínio, state of Minas Gerais, Brazil. In this case, two FSC's were installed in a sedimentation pond to reduce silting of nearby rivers caused by surface erosion of a mining sterile dump. Figure 21 shows a bird-eye view of the mine sterile dump, which occupies a total area of 9.34 ha, with a maximum height of 137 m, volume of 35.5 Mm³ and 2:1 (h:v) slopes (Macedo, 2023). This figure also shows the drainage ditches that carried the run-off from the sterile dump to the sedimentation pond and from the latter to the Bonito stream.

The two FSC's installed in the sedimentation pond were manufactured from a woven geotextile, made of polypropylene,

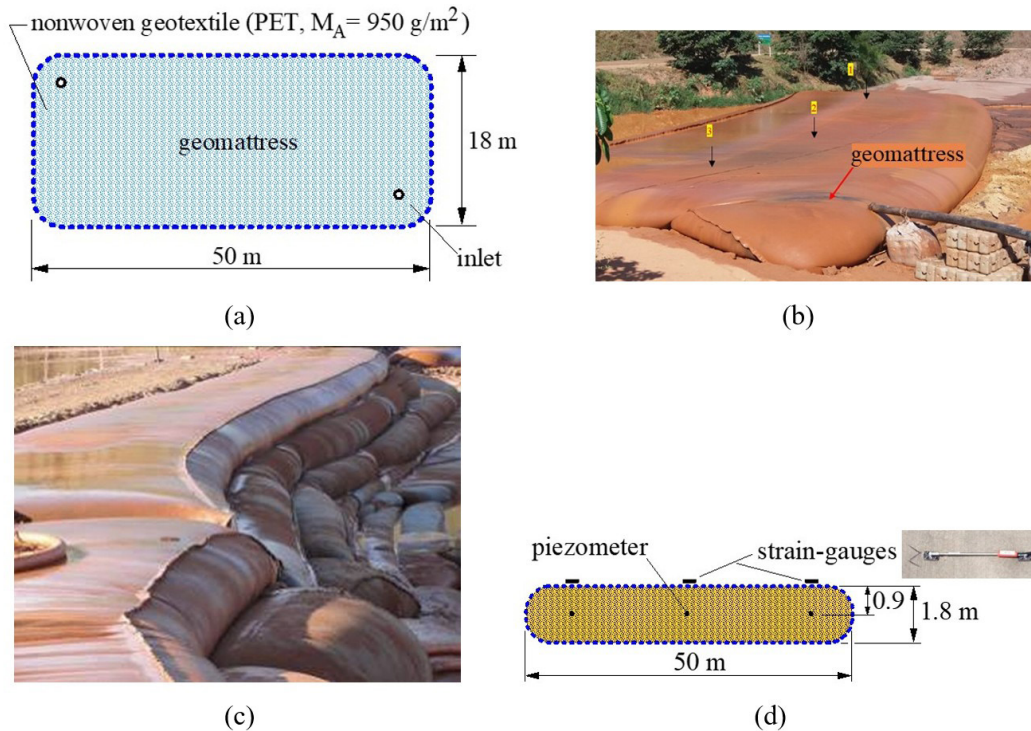


Figure 17. Dewatering of iron ore tailings using geomattresses: (a) geometrical characteristics of the geomattresses; (b) geomattress during filling (courtesy Dr. A. Fahel); (c) view of the stacked geomattresses in the field (courtesy Dr. A. Ortigão); (d) characteristics of the instrumented geomattress.



Figure 18. Geomattress during the dewatering process at Risoleta Neves dam site (courtesy Dr. A.R.S. Fahel).

3 m deep, with a mass per unit area of 440 g/m^2 , tensile strength of 105 kN/m , maximum tensile strain of 10%, normal permeability of 0.2 cm/s and filtration opening size of 0.24 mm . The installation of the floating silt curtains is presented in Figures 22a-c. To reduce even further the discharge of solids in suspension in the nearby stream, a geotextile fence (similar to a silt fence for erosion control) was constructed at the exit of the sedimentation pond (Figure 22d).

Figure 23a presents grain size distribution curves of the sediments downstream and upstream of the curtains, showing the significant reduction in the sedimented particle sizes because of the presence of the curtains (Macedo, 2023). Significant reduction in effluent turbidity downstream of the curtains can also be visualized in Figures 23b and c. It should be pointed out that the performance of the curtains can be influenced by factors such as flow rate intensity, wind direction

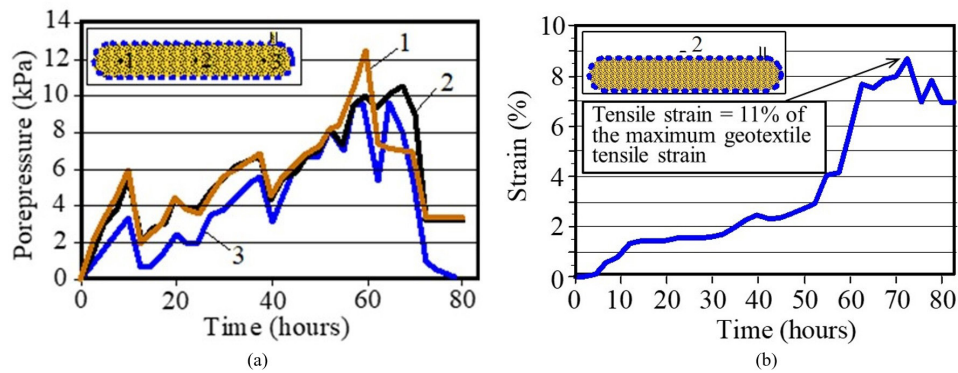


Figure 19. Porepressure dissipation and geotextile strain versus time: (a) porepressure dissipation; (b) geotextile strain at point 2.

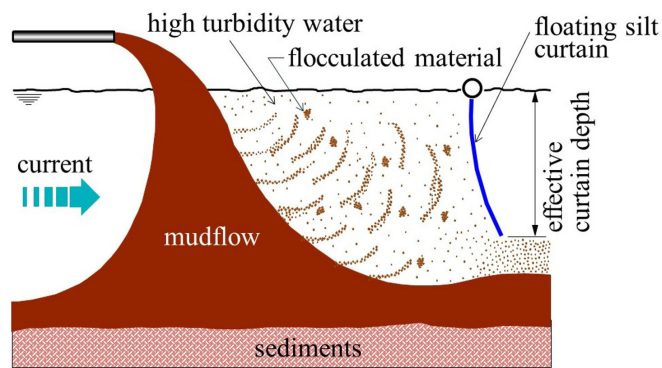


Figure 20. Operating principle of a floating silt curtain (modified from Johanson, 1978).

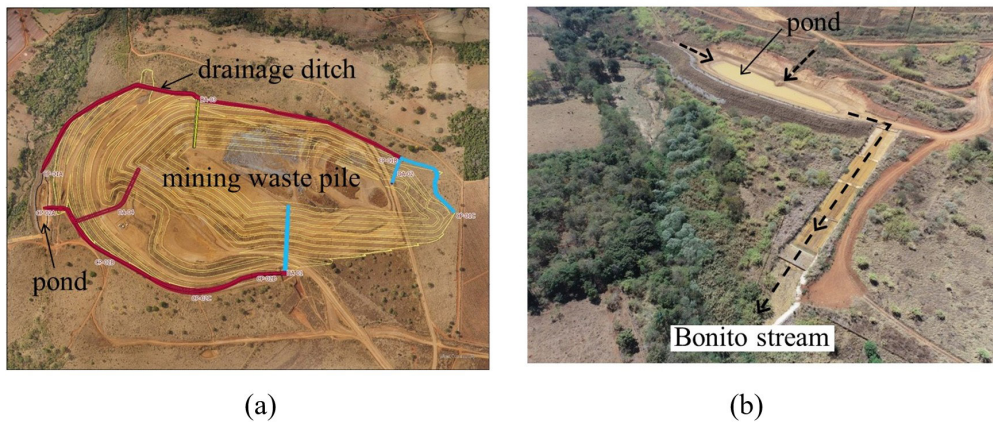


Figure 21. Mine sterile pile and drainage system (modified from Macedo, 2023): (a) mine sterile pile and drainage ditches; (b) sedimentation pond.

and temperature. During the rainy season in the region, under higher flow rate conditions (lotic flow regime) in the pond, the reduction in turbidity reached up to 93% downstream of the curtains. However, little turbidity reductions were observed under unfavourable wind directions and low flow rates (lentic flow regime) in the pond (3.2% to 3.6% reductions downstream of the curtains in comparison with the turbidity upstream of

the curtains). Indeed, low contribution from the presence of the curtains should be expected under lentic or still water conditions, because these conditions by themselves favour sedimentation of solids in suspension. The results obtained showed that the use of floating silt curtains can effectively reduce silting of relevant water bodies. Additional information on this case-history can be found in Macedo (2023).

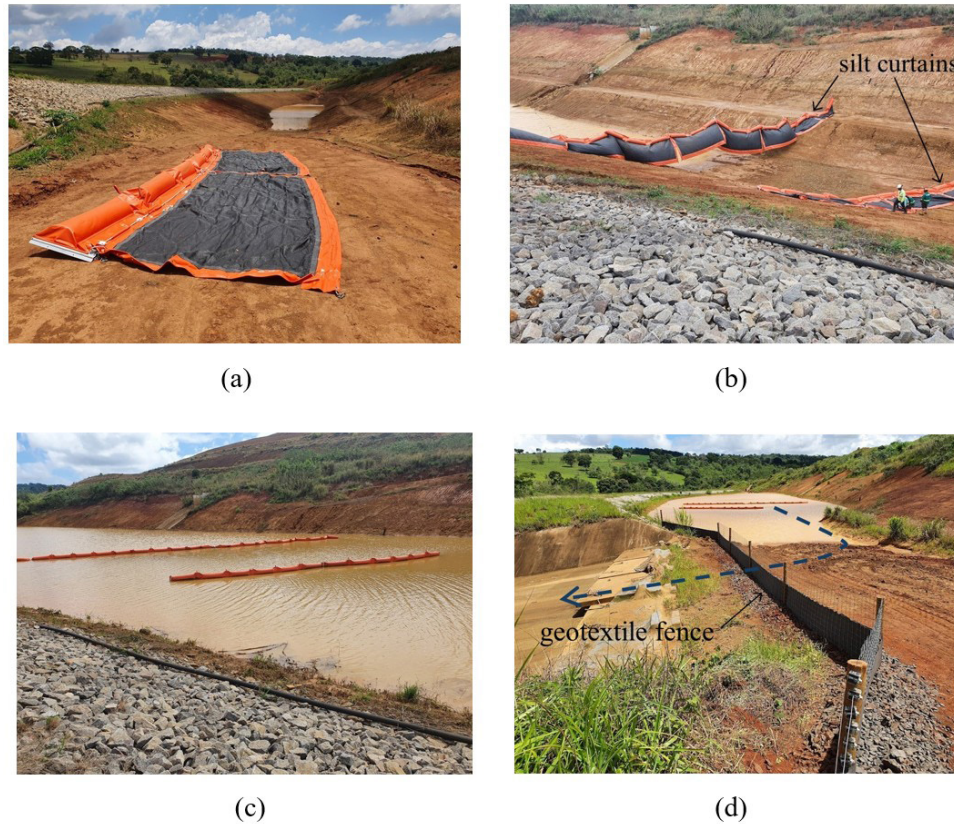


Figure 22. Installation of the floating silt curtains (modified from Macedo, 2023): (a) view of a silt curtain; (b) silt curtains during installation; (c) view of the silt curtains in place with a full pond after a rainy period; (d) silt fence at the exit of the pond.

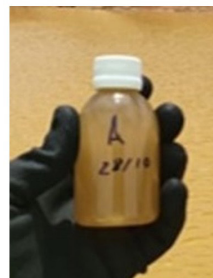
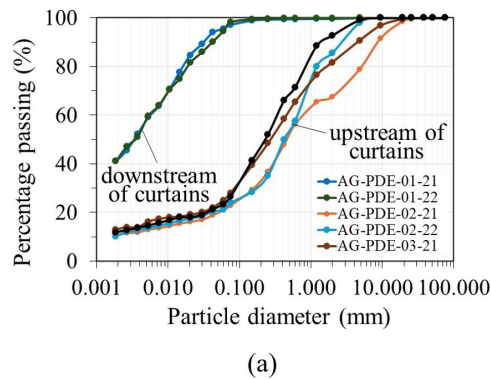


Figure 23. Particle size of sediments and turbidity (Macedo, 2023): (a) grain size distribution downstream and upstream of the curtains; (b) liquid specimen upstream of curtains; (c) liquid specimen downstream of curtains.

5. Sustainable geosynthetics and sustainable solutions with geosynthetics

Some geosynthetics are sustainable in their own right, as they can degrade under natural environmental conditions after they have fulfilled their functions. These are the cases of geosynthetics manufactured from natural fibres (coir, jute, sisal, hemp, palm fibre, cotton etc.) or from biodegradable synthetic fibres. Figure 24 shows some examples of such

products that can present a wide variety of functions, particularly in erosion protection works (Prambauer et al., 2019; da Luz et al., 2021; Melo et al., 2021; Pimentel et al., 2024). They can also function as temporary reinforcements in geosynthetic reinforced structures if they are durable enough (Mathur et al., 2011; Hegde & Sitharam, 2015; Evangeline et al., 2021; Ahirwar & Mandal, 2021). It is estimated that synthetic materials may be successfully substituted by natural fibres and biopolymers in 50% of all applications (Daria et al.,



Figure 24. Examples of geosynthetics manufactured with natural fibres: (a) woven geotextiles from sisal fibres (courtesy J.R.M. Pimentel); (b) geomat for erosion control.

2020). Hegde & Sitharam (2015) describe the use of geogrids and geocells made from bamboo for the reinforcement of a sand fill overlying a clayey subgrade. They observed up to 3 to five times increase in the load-carrying capacity of the system due to the use of the geogrid or geocell made from bamboo. Mujawar et al. (2025) also investigated the use of geogrids made of bamboo to reinforce a sandy subgrade. The results obtained showed the suitability of using geogrids manufactured from bamboo as a sustainable soil reinforcement technique. Evangeline et al. (2021) compared the performance of unreinforced and coir-geotextile (T_{max} of 8.8 kN/m and 8.2 kN/m, MD and CMD, respectively) reinforced roads (Figure 25), observing a reduction in the thickness of the bituminous Macadam overlay with the use of the coir-geotextile reinforcement. Ahirwar & Mandal (2021) also observed good performance of bamboo grid and jute geotextile for the reinforcement of unpaved roads on soft subgrade with 1.9-to-2.3-fold increase in the bearing capacity of the reinforced road-soft subgrade system compared with the unreinforced one.

Melo et al. (2021) show results of large-scale laboratory tests where geomats manufactured from natural fibres were employed to protect a slope from surface erosion caused by intense rainfalls. Reductions of erosion intensity close to 100% were observed with the use of the geomats, particularly for vegetated slopes. Figure 26 presents the results obtained in some of the tests after 15 minutes of rainfall with an intensity of 131 mm/h in terms of reduction in eroded mass versus percentage of vegetation cover on the slope. Even before vegetation cover is present, a reduction of 93% in the eroded mass can be noted. As vegetation coverage increased the efficiency of the protection against surface erosion also increased, reaching 99% reduction in surface erosion. It should be noted that the use of a biodegradable geomat in this type of application may not be the most appropriate choice for short-lived vegetative covers, because the geomat will degrade after some time, leaving the slope unprotected. In these cases, more durable products may be required because the geomat will remain in place, protecting the slope to some extent against the impact of rain drops and facilitating

periodical maintenance of the slope surface with water seeding. Mathur et al. (2011) describe several applications of jute geotextiles in geotechnical works for the protection against surface erosion, drainage and reinforcement.

Conventional geosynthetics products can be combined with other materials to form alternative geosynthetics. This can be interesting in regions where suitable waste materials (PET bottles, scrap tires etc.) are plentifully available (like in landfills, for instance) to be combined with conventional geosynthetics to produce low-cost barrier or drainage systems. Figure 27 shows the combination of a geotextile filter with scrap tires and compressed PET bottles, forming alternative geocomposites for drainage in a landfill and in an urban street in the Federal District, Brazil. Tire shreds have also been used combined with geotextile filters in drainage trenches and blankets in landfills, as mentioned earlier in this paper (Figures 7 and 8). The use of recycled plastic is another sustainable measure to produce low-cost geosynthetics. Hasheminezhad et al. (2025) show comparable performance of unpaved roads reinforced with geosynthetics manufactured from virgin and recycled PET. I should be pointed out that it would be wise to use geosynthetics from recycled plastic in temporary works only, due to the great variety of origins, compositions, quality and properties of recycled plastics, unless the properties of the product considered, obtained in appropriate tests, fulfil the requirements of the design.

Silva (2007) and Silva & Palmeira (2013) tested geocomposites for drainage combining nonwoven geotextile filters with alternative drainage cores. One of the alternative drainage cores investigated consisted of scrap tire strips assembled in a square pattern and the other waste PET bottle caps assembled in a triangular pattern (Figure 28a). Figure 28b shows transmissivity tests on three conventional commercial geocomposites and on the geocomposite made with tire strips. As expected, the compressibility of the geocomposite depends on the core stiffness and as the spacing between the tire strips increases so does its compressibility, resulting in reductions in the discharge capacity of the product along its plane. Nevertheless, the result for the alternative geocomposite with 30 mm spacing between strips compared



Figure 25. Coir-geotextile reinforced road during construction (Evangeline et al., 2021).

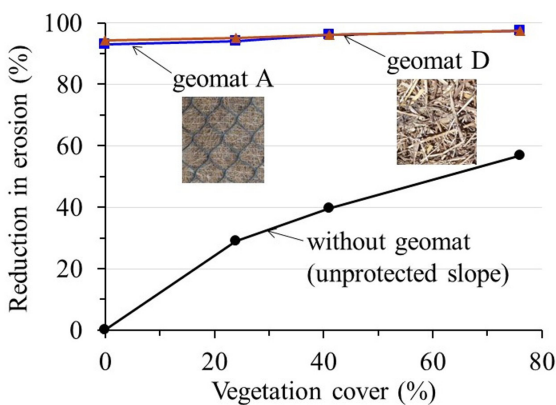


Figure 26. Reduction in slope erosion caused by rainfall with the use of biodegradable geomats (modified from Melo et al., 2021).

well with those of the commercial geocomposites up to a vertical stress of 100 kPa. The same behaviour was observed for the geocomposite with PET bottle caps with 40 mm distance between caps. For the test on the geocomposite with 6 mm distance between caps, the discharge capacity remained constant ($\cong 0.2$ l/s/m) up to 400 kPa vertical stress and similar to those of the conventional commercial products tested (Figure 28b).

The combination of perforated waste PET bottles with a nonwoven geotextile filter in trenches to favour the infiltration of rainwater in the subsoil is described in Silva (2012). This can be a low cost and environmentally friendly solution to reduce floods during rainy seasons in urban areas. Figure 29 shows one of the experimental infiltration trenches tested. Additional information on this type of solution can be found in Silva (2012). Aiming at a better destination of PET bottles, Moghaddas-Tafreshi et al. (2021) present a large-scale laboratory study on subgrade reinforcement combining

PET bottles and geogrid subjected to cyclic loading where lower settlement accumulation rate and a higher resilient (elastic) settlement in the reinforced subgrade were observed compared with the unreinforced one. Dandin et al. (2024) carried out large scale laboratory tests on fly ash backfills reinforced with geocell manufactured from waste PET bottles, showing the feasibility of using this alternative geocell for soil reinforcement.

The use of geocells made of scrap tires as reinforcement in retaining structures and unpaved roads is reported by several researchers (Yoon et al., 2008; Sayão et al., 2009; Hidalgo & Bustamante-Hernández, 2020; Arango, 2023; Susunaga, 2025; Susunaga et al., 2025, for instance). Hidalgo & Bustamante-Hernández (2020) performed large-scale pull-out tests on geocells made of scrap tires, showing the good interaction between the geocell and the surrounding soil. Susunaga (2025) carried out large-scale tests (1200 mm dia. container, 1000 mm high) on unpaved roads with and without scrap tire geocell reinforcement subjected to cyclic loading from a 180 mm diameter platen. The scrap tires were assembled in the pattern shown in Figure 30a to form the geocells. Figure 30b shows the significant improvement in road performance in terms of surface settlement versus number of load cycles of the geocell-reinforced road compared with the unreinforced one.

Viana & Palmeira (2010) mixed different percentages in mass of scrap tire shreds to bentonite to manufacture alternative geocomposite clay liners (GCL, Figure 31a). The presence of the tire shreds may reduce the cost of GCL manufacture but certainly changes some of its properties. The reduction in the expansibility of the alternative GCL is particularly relevant and Figure 31b shows the variation of GCL expansion with time due to submersion in water for two conventional commercial GCLs and an alternative GCL



Figure 27. Combination of geotextile filter with alternative drainage materials: (a) geotextile and scrap tires in a landfill; (b) geotextile and compressed PET bottles in an urban street.

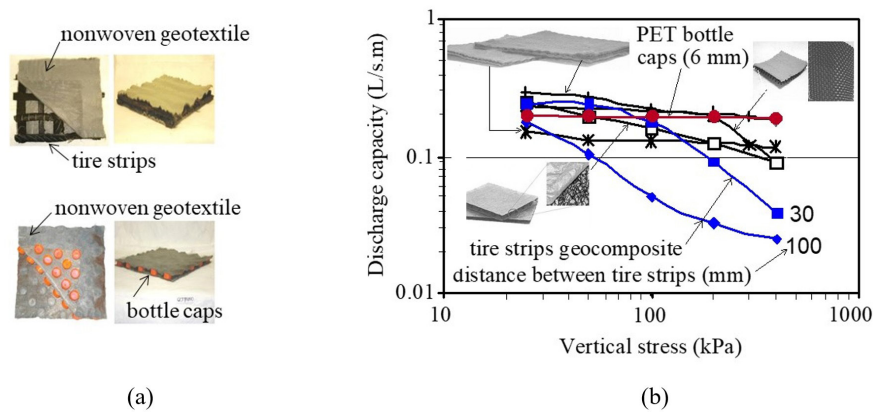


Figure 28. Alternative geocomposites for drainage (Silva & Palmeira, 2013): (a) geocomposites with scrap tire strips or plastic caps drainage core; (b) discharge capacity versus vertical stress-comparison between some conventional and an alternative geocomposite.

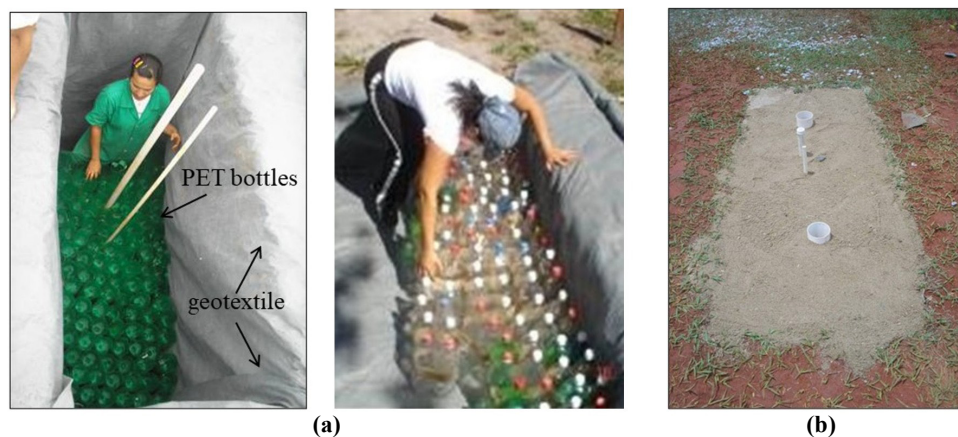


Figure 29. Combination of PET bottles and geotextile filter in experimental infiltration trenches to minimize the consequences of floods (Silva, 2012): (a) during construction; (b) end of construction.

with 50% of tire shreds mixed to bentonite. A reduction of approximately 50% in GCL expansibility can be noted. Despite this limitation, alternative GCL such as the one described may be interesting as low-cost products in temporary barrier applications or as an additional safety barrier layer in landfills, bearing in mind that scrap tires and plastics in general are commonly present in large volumes in landfills.

The examples presented above show how some residues and waste materials can have better destinations than taking up space in landfills or worse, being illegally disposed of in nature. However, the use of these materials in construction must be done with due care, as some of them can be harmful to the environment. For instance, scrap tires are major contributors to the generation of microplastics

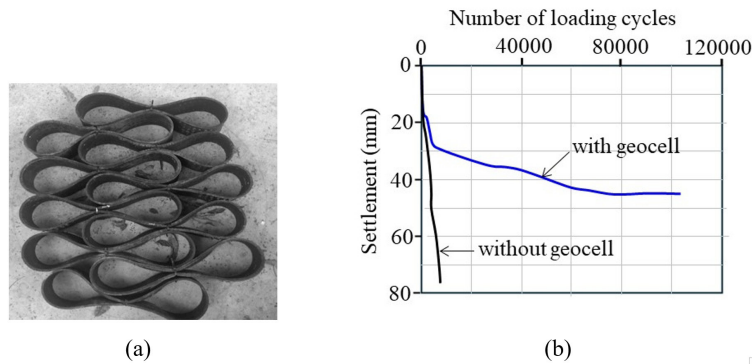


Figure 30. Alternative geocells manufactured from scrap tires (Susunaga, 2025): (a) plan view of the geocell; (b) comparison between road performance with and without geocell reinforcement.

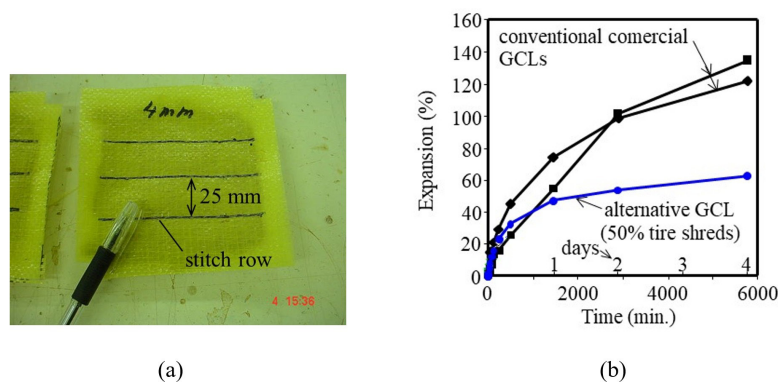


Figure 31. Performance of an alternative GCL (Viana & Palmeira, 2010): (a) view of the GCL; (b) comparison between expansibility of conventional and alternative GCLs.

(Giechaskiel et al., 2024). It should be noted that in most potential geotechnical applications, used tires would be nowhere near as worn to produce microplastic particles as when in contact with a road surface during its operational life. In addition, when used (usually buried) in remote areas, away from important water bodies and infrastructure, the consequences of the generation of microplastics over time can be considered insignificant compared to those of the normal use of tires or other types of plastic waste. Environmental damage is even more negligible when scrap tires and other types of waste and residues are used as construction materials under confined conditions, such as in landfills with properly designed lining systems and effluent treatment plants.

6. Geosynthetic against natural disasters and to minimize climate change effects

Natural disasters can cause significant damage to or destroy existing civil infrastructures. One typical example is the detrimental effects caused by rockfalls and debris flow, which are quite common in mountainous regions. In these situations, the use of soil barriers reinforced with geosynthetics may provide a quick and cost-effective

solution. Descoeudres et al. (1999) and Maheshwari et al. (2024) show that this type of barrier tends to be the one that allows greater energy dissipation in the event of rock impacts. Figure 32a and b show examples of geosynthetic reinforced barriers against rockfalls. Case-histories of this type of solution can be found in Brandl (2010), which shows the significant greater total sum of impacts to barrier cost ratio for complete barrier destruction of reinforced barriers compared with unreinforced ones. Thus, this can be a solution easy to implement for the protection of infrastructures against rockfalls when space is available on the slope.

Other applications of geosynthetics in barrier systems are for the protection of existing infrastructure against floods and tsunamis. The frequency of occurrence of the former has increased significantly in recent decades due to climate change. A recent example of this were the damages caused by the floods due to extreme heavy rains in the southern states of Brazil, particularly in Rio Grande do Sul state, in 2024. Barrier systems such as those presented in Figures 32c and d can be considerably easier and faster to construct than conventional alternatives. Figure 32c shows an example of a dam constructed with geomembrane tubes that can be used for water accumulation during river floods and Figure 32d

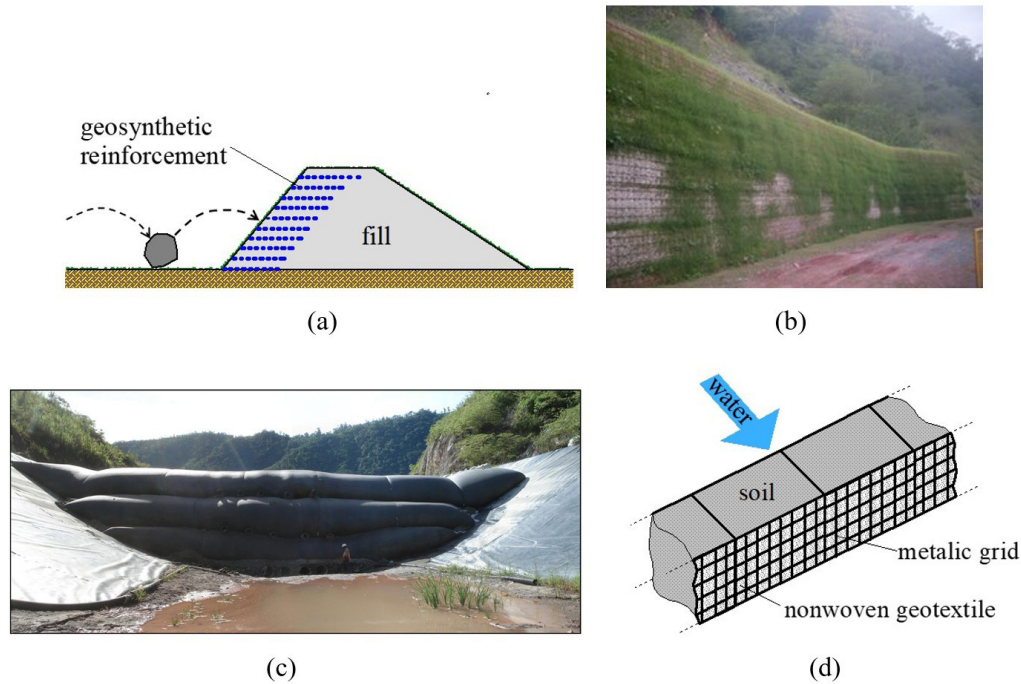


Figure 32. Different types of barriers incorporating geosynthetics: (a) schematic typical cross-section; (b) example of a rockfall barrier (courtesy Dr. P.M.F. Viana); (c) geomembrane tubes stack dam (IGS, 2010); (d) geotextile lined grid boxes for flood control.

depicts a mobile flood protection system consisting of metallic grid boxes lined with nonwoven geotextiles and filled with soil that can be installed along relevant stretches of riverbanks subject to flooding (Brandl, 2010). The same type of protection can be achieved with stacks of geotextile bags, tubes or mattresses. Geosynthetic reinforced dykes can also be used to prevent damage caused by flooding, as described in Heerten & Horlacher (2004), Heerten (2006), Heibaum et al. (2006) and Brandl (2010), for instance.

Geosynthetic reinforced soil barriers and stacks of geotextile tubes can also be employed to protect infrastructures and to avoid or minimize damage to the environment due to failure of tailings dams and reservoirs of pollutant substances, as schematically shown in Figures 33a, similarly to what was described above for the protection against floods. In mining regions, the tailings can be used as fill or infill material in reinforced barriers and geotextile tubes, depending on the nature and properties of the tailings (Figures 33b and c).

The Japanese experience on structures for the protection against tsunamis is certainly applicable in most cases for the protection against mud flows from failures of tailings dams and floods (Mohri et al., 2007; Shindo & Tatsuoka, 2017; Kuwano et al., 2018, for instance). Figure 31d (Shindo & Tatsuoka, 2017) presents the arrangement of a reinforced embankment designed to resist tsunamis. Of relevance is the possibility of erosion of the slopes of embankments such as the one in Figure 31d due to water impact or overtopping. Shindo & Tatsuoka (2017) describe a case-history in which a geocell with concrete-filled pockets (shotcrete) was used

on the slopes for erosion protection (Figures 31e and f). Alternatively, Kuwano et al. (2018) describe the use of geobags and geomattresses to protect the slopes of these barriers against surface erosion.

Because they are typically easy and fast to build, engineering structures incorporating geosynthetics can be used in emergency situations where time for proper action is a key factor. Several other environmental protection-related solutions using geosynthetics can be found in the literature (Pilarczyk, 2000; Koerner, 2012; Palmeira, 2018; Touze, 2021, for instance).

7. Conclusions

Geosynthetics are construction materials that can be used in a variety of applications in geotechnical and geoenvironmental engineering. This paper presented results of research activities and case history performance where geosynthetics were used to make feasible environmentally friendly solutions, which would not be possible with conventional geotechnical engineering alternatives. The main conclusions obtained are as follows.

As the consequences of climate change reach unprecedented levels and are only set to worsen, it is increasingly important to think ahead and investigate engineering solutions that are cost-effective, sustainable and quick to implement in case of emergencies. In this context, geosynthetics bring these advantages together in most situations. In case of flooding, failures or leakages of tailings dams, geosynthetics can be used

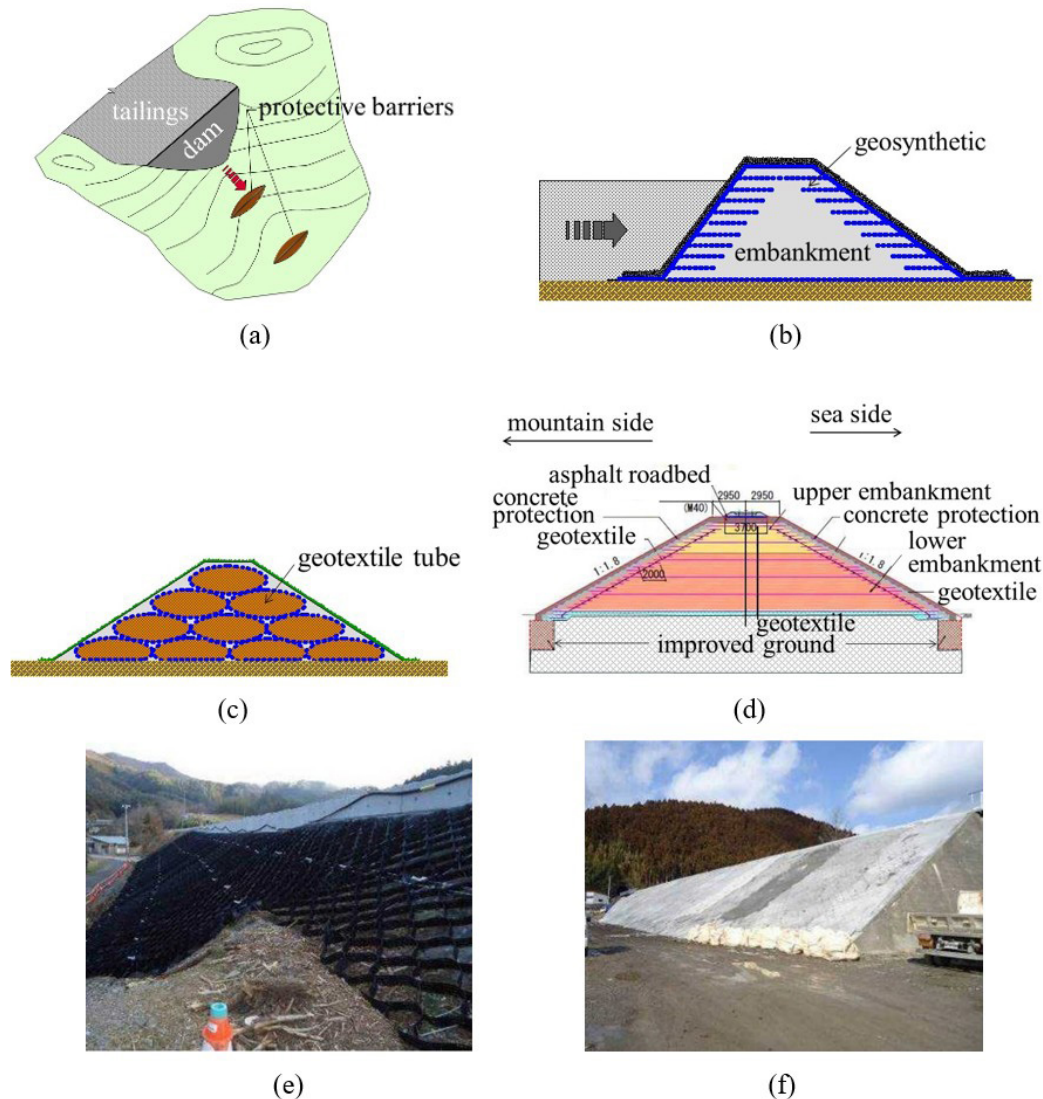


Figure 33. Other applications of protective geosynthetic reinforced barriers: (a) protection against failures or leakage from tailings dams; (b) geosynthetic-reinforced fill barrier; (c) geotextile tubes barrier; (d) tsunami barrier (modified from Shindo & Tatsuoka, 2017); (e) geocell during installation on the slope (Shindo & Tatsuoka, 2017); (f) concrete-filled geocell at the end of construction (Shindo & Tatsuoka, 2017).

to construct barriers in the form of reinforced embankments or stacked geotextile tubes or mattresses to protect existing structures or minimize environmental damage in case of any accident. In the mining industry, if the tailings are not harmful to the environment and have suitable geotechnical properties, they can even be used to construct the barriers, which gives a better destination for such tailings. Floating silt curtains can also be employed to reduce the extent of water bodies affected by silting.

Biodegradable natural and synthetic geosynthetics are expected to be increasingly used in the coming years, especially in temporary works, due to their sustainable nature. Chemically treated natural fibres can extend the service life of natural geosynthetics to the point where they can be used for soil reinforcement in some circumstances.

Reducing waste disposal in landfills through changes in societies habits, recycling and better destination of waste is extremely important. In this case, the use of some wastes and residues combined with geosynthetics may be feasible and sustainable solutions. Examples of this are combinations of mining or recycled construction and demolition residues in reinforced soil structures such as pavements and retaining walls. Another sustainable destination for some waste materials or recycled plastic is in the manufacture of alternative geosynthetic products. Successful examples of the use of scrap tires and PET bottles in the manufacture of geocells or as drainage layers wrapped by geotextile filters have been presented in this paper. These types of alternative geosynthetics may be particularly interesting in regions where waste materials are abundant, such as in landfills. However, it is of utmost importance to

evaluate the potential damage to the environment caused by using waste and residues, depending on their nature, properties and polluting potential. Good engineering judgment must be applied when using waste as construction materials in geotechnical and geoenvironmental works.

It has already been demonstrated that engineering works with geosynthetics can be significantly more sustainable than conventional ones. Due to their easy transport and rapid installation, these materials can be particularly interesting in emergency situations, when the time necessary for a given solution to be implemented is a key factor. However, like any other construction material, the use of geosynthetics must be based on sound engineering principles that also ensure that it will meet the project requirements satisfactorily. Continuous research will certainly not only improve the quality of geosynthetic products, but will also find new fields of application for these materials in sustainable engineering solutions to meet the challenges brought about by climate change.

Acknowledgements

The author is indebted to the following institutions and professionals for the support to this work in different ways: The University of Brasília, National Council for Scientific and Technological Development (CNPq), CAPES Foundation/ Brazilian Ministry of Education, Federal District Research Support Foundation (FAP-DF), Dr. André R.S. Fabel, Dr. Maria G. Gardoni and Dr. José A.R. Ortigão.

Declaration of interest

The author has no conflicts of interest to declare. The author has observed and affirmed the contents of the paper and there is no financial interest to report.

Data availability

The datasets generated analysed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of generative artificial intelligence (GenAI).

List of symbols and abbreviations

d_c	constriction diameter of a sand filter
d_n	diameter for which n % of the remaining particles are smaller
w	maximum geotextile tube width
w_b	width of the geotextile tube base

x	distance along the horizontal direction
y	settlement
A	cross-section area of a geotextile tube
B	retaining wall base width
BOD	biochemical oxygen demand
CBR	California Bearing Ratio
CL	centre line
CMD	cross-machine direction
COD	chemical oxygen demand
C_u	soil coefficient of uniformity
FOS	geotextile filtration opening size
GCL	geosynthetic clay liner
H	height
$J_{5\%}$	secant tensile stiffness at 5% strain
M_A	geotextile mass per unit area
MD	machine direction
PET	polyester
PP	polypropylene
PVC	polyvinyl chloride
RCDR	recycled concrete and demolition residue
SP	settlement plate
TP	temperature transducer
T_c	capillary tension
T_{max}	maximum tensile strength
γ	soil unit weight
ε_{max}	maximum tensile strain
σ_v	vertical stress

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The Milton Vargas Lecture 2023 – Shallow tunnels in cohesionless soils: ground behavior and building damage risks

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Lecture

Keywords

Tunnels
Centrifuge test
Shallow tunnel
Urban tunnel
Numerical modeling
Cohesionless soil

Abstract

The demand for tunnels in densely populated urban areas is growing rapidly to address mobility challenges. Mechanized tunneling is widely adopted in urban environments due to its high productivity and the relatively small ground deformations it induces. However, urban tunneling is highly complex because of the typically shallow depths and interactions with aboveground structures. Therefore, accurately predicting ground deformations induced by mechanized tunneling at the design stage is crucial for assessing potential building damage. To investigate these deformations, a series of centrifuge tunnel tests have been conducted at academic institutions such as the Universities of Cambridge and Nottingham to study the behavior of shallow mechanized tunnels in cohesionless soil. These tests serve as excellent benchmarks for numerical model calibration. Once calibrated to replicate centrifuge test results, numerical models can efficiently analyze a wide range of scenarios at a fraction of the time and cost. This paper investigates ground deformations induced by shallow tunneling in cohesionless soil using numerical models calibrated against selected centrifuge tunnel tests, which encompass varying tunnel diameters, depths, and sand relative densities. The numerical modeling results presented in this paper provide extensive insights into tunnel behavior, illustrating how tunnels respond to different relative densities and depths under tunnel volume losses of up to 5%, approaching failure conditions. Additionally, a comprehensive analysis of ground deformations caused by shallow tunnels in sandy soils and their potential impact on buildings is presented.

1. Introduction

The construction of tunnels in urban areas can induce ground deformations that pose risks to buildings and existing infrastructure. As cities become more densely populated, the demand for tunnels to improve mobility rapidly increases. However, the interaction between tunnels and existing structures in urban environments is highly complex and requires robust numerical modeling for accurate assessment.

Centrifuge tests have been widely used to model tunnel excavation in sandy soils and its interaction with existing geotechnical structures (Marshall, 2009; Farrell, 2011; Zhou, 2015; Franza, 2016; Ritter, 2018; Xu et al., 2021). These tests serve as valuable references for numerical model calibration. Several authors have successfully replicated centrifuge test results through numerical modeling (Boldini et al., 2021; Giardina et al., 2015, 2019; Vitali et al., 2024b,c) and extended the findings to scenarios not explored in physical experiments. Once calibrated, numerical models can efficiently analyze

a wide range of conditions at a fraction of the time and cost required for physical testing.

The reliability of numerical models depends on their ability to accurately predict the stresses and strains generated in the ground during tunnel excavation. Tunnel behavior in clayey soils is often undrained, simplifying the problem as volumetric changes typically do not occur in the short term (Mair et al., 1993). However, in sandy soils, volumetric changes occur, significantly increasing the complexity of the problem (Franza et al., 2019; Vitali et al., 2024b).

The ability of numerical models to capture the volumetric strains of sandy soils at different relative densities is crucial for credible modeling of shallow tunnels in sandy soils. In a previous study conducted by the authors (Vitali et al., 2024b), the NorSand model (Jefferies, 1993) was adopted for this purpose. The agreement between experimental and numerical displacements was excellent for tunnel volume losses up to 2.5%. However, the results presented by Vitali et al., 2024b are not valid for tunnel volume losses exceeding 2.5% or for conditions approaching failure. A key limitation of the

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Submitted on February 13, 2025; Final Acceptance on March 31, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.003225>



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NorSand model was that the numerical model reached convergence before failure, even after full stress relief, which is unrealistic. This issue possibly stems from the NorSand constitutive formulation as implemented in commercial FEM software for geotechnical engineering, such as Midas GTS NX and Plaxis, which do not include a dilation cut-off.

This paper investigates ground deformations induced by shallow tunnel excavation in cohesionless soil for tunnel volume losses up to 5% and conditions approaching failure, extending the scope of the previous study by Vitali et al. (2024b). The study is based on numerical models calibrated against selected centrifuge tunnel tests, which consider varying tunnel diameters, depths, and sand relative densities. The behavior of sandy soil at different relative densities was modeled using the well-known Hardening Soil Model (HSM), a widely used constitutive model for soils. The HSM parameters were iteratively derived from triaxial compression and extension tests available in the literature (Lanzano et al., 2016) and from numerical simulations of the selected centrifuge tunnel tests. The developed numerical models successfully replicated centrifuge tunnel test results for tunnel volume losses of up to 5%, capturing soil behavior near failure conditions.

Using a validated numerical model, a set of numerical experiments was conducted, varying tunnel depth and the relative density of sandy soil. These experiments were designed to assess tunnel behavior as it evolves from small tunnel volume losses to very large ones (i.e., a tunnel volume loss of 5%). The findings offer valuable insights into the response of cohesionless ground to tunneling, including volumetric behavior, induced displacements, and potential impacts on aboveground buildings. Additionally, a practical recommendation is provided for estimating horizontal strains at the surface where the maximum principal strain occurs based on the settlement trough, allowing for a more accurate damage assessment during preliminary tunnel design stages.

2. Numerical modeling approach

Numerical modeling replicating the boundary and loading conditions of centrifuge tunnel tests were conducted. The centrifuge tests were performed at the Universities of Cambridge and Nottingham (Marshall, 2009; Farrell, 2011; Zhou, 2015). The selected centrifuge tests for model calibration encompassed various tunnel depths, diameters, and sand relative densities (D_r of 50% and 90%). These tests used Leighton Buzzard Fraction E sand, with maximum and minimum void ratios of 0.97 and 0.64, respectively (Vorster, 2006). Lanzano et al. (2016) carried out an extensive laboratory campaign to investigate the stress-strain behavior of this sand, and their results were used as a reference for calibrating the constitutive model adopted in the numerical simulations.

The well-known constitutive model for soils, Hardening Soil model, was selected to represent the sand's behavior in the numerical simulations. This model is probably the

most used in Geotechnical Engineering, and it seems to have reached a reasonable balance between sophistication and practicality.

The Hardening Soil Model is an advanced elastoplastic model based on the Mohr-Coulomb failure criterion. It effectively captures key aspects of soil behavior, including stiffness-stress dependence, shear and compression hardening, and elastic unloading/reloading. Its input parameters can be easily determined from conventional laboratory tests. However, it does not capture strain-softening behavior, which is typically observed in dense sands and highly overconsolidated clays under low confinement stresses. It also does not account for changes in soil relative density, unlike constitutive models based on critical state soil mechanics (e.g., NorSand). Consequently, different constitutive model parameters are required for different relative densities of sand. However, this is not considered a limitation in this study since the numerical results from Vitali et al. (2024b) indicate that the magnitude of the volumetric strains in the soil mass induced by tunnel excavation is minor. Therefore, changes in soil relative density surrounding the tunnel are likely negligible.

The numerical modeling calibration was performed iteratively, considering both the laboratory test results presented by Lanzano et al. (2016) and the selected centrifuge tunnel tests. Figure 1 compares triaxial compression test results on dense sand ($D_r = 70\%–80\%$, Figure 1a) and triaxial extension test results on medium-dense sand ($D_r = 50\%$, Figure 1b) with the element test response of the Hardening Soil Model. The figures show the evolution of deviatoric stress (q) and volumetric strain (ϵ_{vol}) with increments of axial strain (ϵ_{ax}). Since the triaxial compression tests on dense sand presented by Lanzano et al. (2016) show a slight variation in relative density, a perfect match with the experimental results is not expected. As one can see, the element tests with the calibrated Hardening Soil Model satisfactorily replicated the triaxial test results. Table 1 presents the selected parameters, along with a brief description of each, for relative densities of 50% and 90%.

Figure 2a shows the mesh and dimensions of the numerical models used for this study. The FEM software Midas GTS NX was adopted for numerical analyses. Three centrifuge tunnel tests were selected for validation of the numerical modeling approach: two on dense sand ($C/D = 1.33$, Farrell 2011, and $C/D = 2.44$, Marshall, 2009) and one on medium dense sand ($C/D = 2.00$, Zhou, 2015), with prototype-scale tunnel diameters of 6.15 m, 4.65 m, and 7.2 m, respectively.

The model's dimensions, mesh refinement, and element selection were based on methodologies and recommendations proposed by Vitali et al. (2018a, 2019c, 2021a, 2024a). These approaches were validated by successfully reproducing results from exact analytical solutions (Vitali et al., 2018b; 2019a,b,d; 2020a, b, c; 2021b; 2022b) and accurately modeling actual case histories (Vitali et al., 2019b, 2022a).

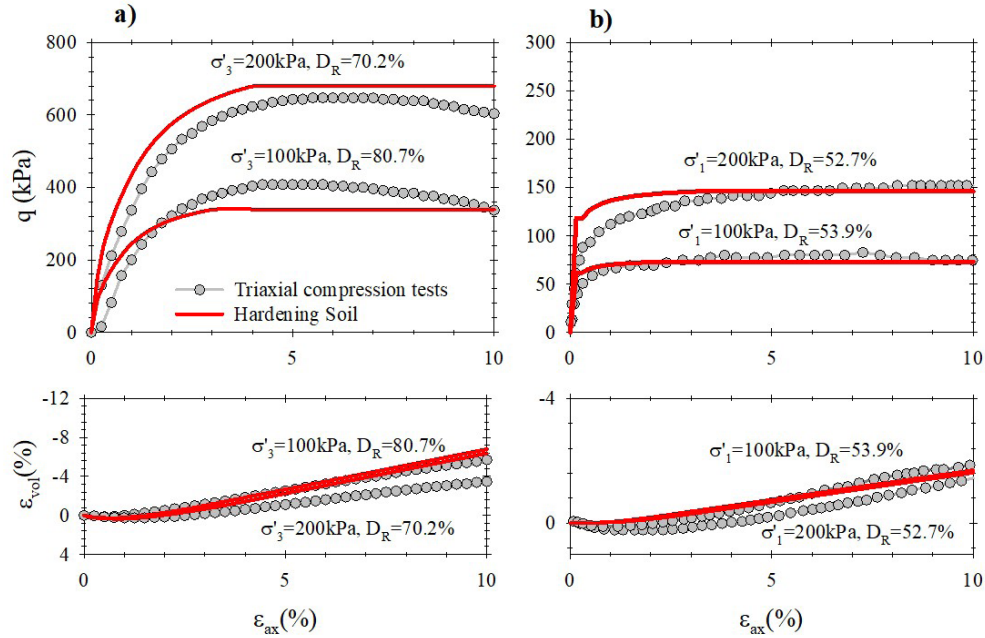


Figure 1. Comparison of the calibrated Hardening Soil model and Lanzano et al. (2016) triaxial test results for (a) drained triaxial compression tests on dense sand; and (b) drained triaxial extension tests on medium dense sand.

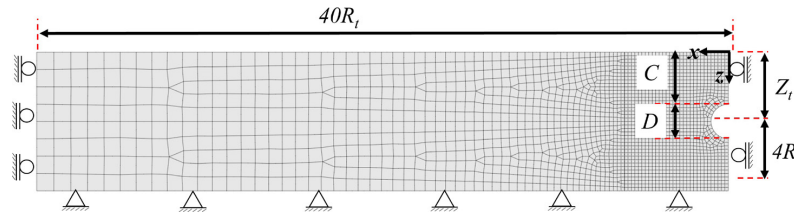


Figure 2. Finite element mesh.

Table 1. Calibrated hardening soil parameters.

Parameter	Definition	$D_r = 50\%$	$D_r = 90\%$
$E_{50,ref}$ (kPa)	Secant Young's modulus in standard drained triaxial test corresponding to p_{ref}	18000	30000
$E_{oed,ref}$ (kPa)	1D compression modulus for primary oedometer loading corresponding to p_{ref}	18000	33000
$E_{ur,ref}$ (kPa)	Unloading / reloading Young's modulus corresponding to p_{ref}	72000	120000
R_f	Failure ratio	0.85	0.85
p_{ref} (kPa)	Reference stress for stiffness	100	100
m	Power for stress-level dependency of stiffness	0.6	0.6
ϕ (°)	Friction angle	35	39
$K_{0,NC}$	K_0 -value for normal consolidation	0.5	0.5
Ψ (°)	Dilatancy angle	6	17
c (kPa)	Cohesion	0	0
γ_d (kN/m ³)	Dry unit weight	14.6	16
ν_{ur}	Poisson's ratio for unloading-reloading	0.2	0.2
OCR	Overconsolidation ratio	1	1

Half of the model geometry was discretized to leverage the problem's symmetry, applying appropriate boundary conditions at the symmetry plane. The lateral boundaries were placed sufficiently far from the tunnel perimeter to eliminate boundary effects. The model's bottom boundary is located at four tunnel radii (4R) from the tunnel center to minimize buoyancy effects, following the recommendation of Vitali et al. (2019c).

The boundary conditions of the centrifuge tunnel tests were replicated in the numerical models. First, the initial stress state was generated using an at-rest earth pressure coefficient (K_0) of $1 - \sin\phi$ (i.e., Jaky's equation). Then, the elements inside the tunnel were deactivated, and nodal forces corresponding to the initial internal pressure of the centrifuge tunnel tests were applied to the tunnel perimeter. Following this step, the displacements were reset, and the imposed nodal forces at the tunnel perimeter were gradually reduced. This reduction in internal stresses at the tunnel perimeter induced stress redistribution and deformations in the surrounding ground. The resulting displacements at the tunnel perimeter were integrated and converted into tunnel volume loss.

Figure 3 presents the evolution of the internal radial stress at the springline normalized with respect to the tunnel's diameter times the soil unit weight ($\sigma_r/\gamma D$) as a function of tunnel volume loss (V_{Lt}). The figure compares results from numerical models and centrifuge tunnel tests. As shown, excellent agreement was observed for the cases where the tunnel cover-to-diameter ratio was 1.33, the relative density was 90% (Figure 3a). Similarly, excellent agreement was found for the case with a cover-to-diameter ratio of 2.0, and relative density of 50% (Figure 3b). For the case with a cover-to-diameter ratio of 2.44, and relative density of 90%, the results showed slight deviations, but the overall agreement remained satisfactory. Notably, most of the stress relief occurred before a tunnel volume loss (V_{Lt}) of 1% to 2% in all three scenarios shown in Figure 3. Beyond this point, large tunnel volume losses occurred with only minor decreases in internal pressure, indicating pronounced nonlinearity and an approaching failure state.

Figures 4, 5 and 6 present the vertical displacements at the surface and at a normalized depth of 0.4 times the tunnel depth ($Z = 0.4 \cdot Z_p$, as shown in Figure 2) for a cover-to-diameter ratio of 1.33 and a relative density of 90% (Figure 4), cover-to-diameter ratio of 2.44 and a relative density of 90% (Figure 5), and cover-to-diameter ratio of 2.0 and a relative density of 50% (Figure 6). These figures show the results obtained from centrifuge tunnel tests and numerical models.

In the centrifuge tests, displacements were measured using particle image velocimetry (PIV) techniques, laser sensors, and linear variable differential transformers (LVDTs). As discussed by Vitali et al. (2024b), Marshall (2009), and Zhou (2015), displacements measured with PIV techniques slightly underestimate those recorded by

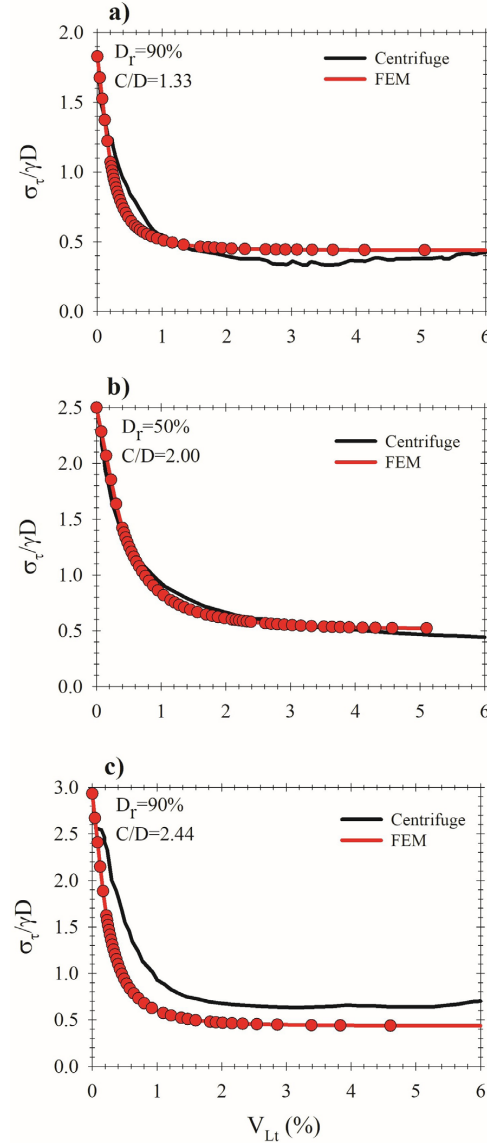


Figure 3. Comparison between centrifuge tests and FEM model, for normalized internal radial stress at springline versus tunnel volume loss, for $C/D = 1.33$ and $D_r = 90\%$ (a), $C/D = 2.00$ and $D_r = 50\%$ (b) and $C/D = 2.44$ and $D_r = 90\%$ (c).

laser sensors and LVDTs, possibly due to friction between the sand and the plexiglass. However, these differences are minor and do not appear to compromise the interpretation of the results.

The results in Figures 4, 5, and 6 are presented with tunnel volume losses up to 5%, which is relatively high given that the expected tunnel volume loss in actual tunnels is typically less than 1%. Both the vertical displacements and the distance from the tunnel centerline are normalized by the tunnel radius. Overall, excellent agreement was observed between the centrifuge tests and numerical models, indicating that the numerical models accurately reproduce the ground deformations observed from centrifuge tunnel tests.

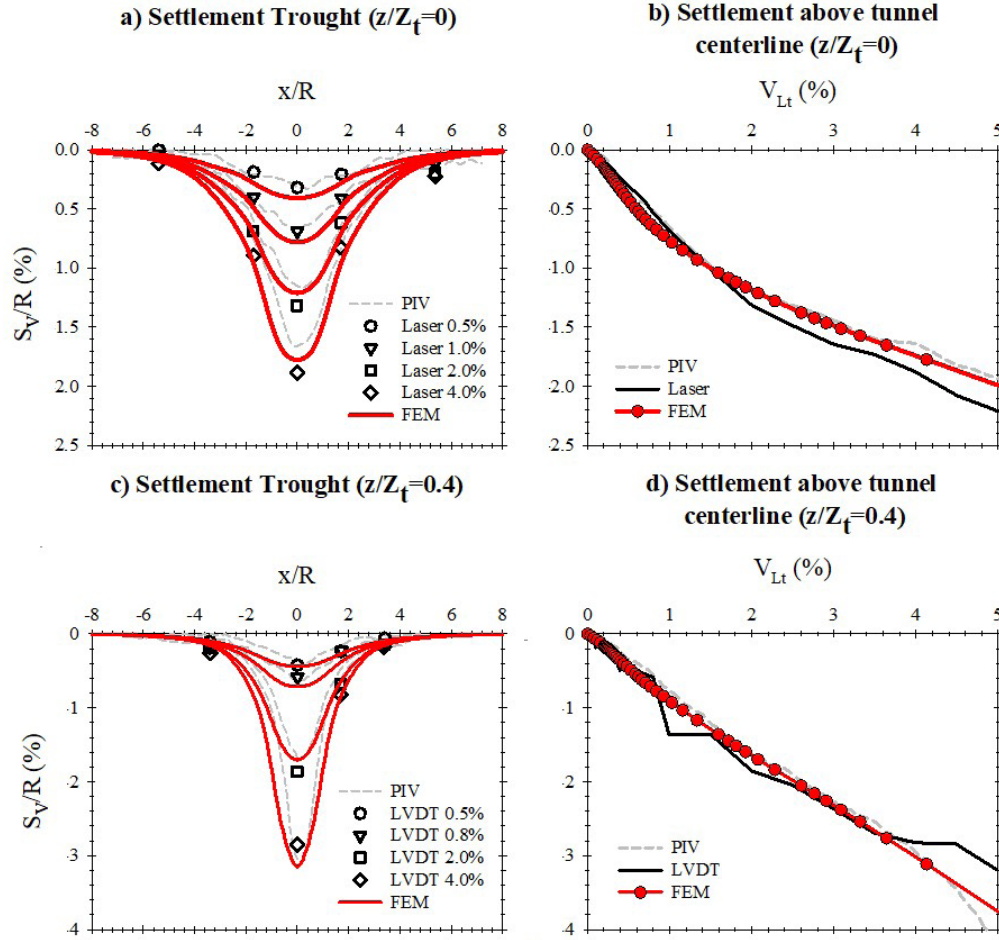


Figure 4. Comparison between FEM results and centrifuge test ($C/D = 1.33$ and $D_r = 90\%$). Settlement trough normalized with respect to the tunnel radius (R) at: (a) the ground surface, and (c) $Z/Z_t = 0.4$; and normalized settlement above the crown at: (b) the ground surface, and (d) $Z/Z_t = 0.4$. The tunnel radius is equal to 3.075 m (i.e., $R = 3.075$ m).

Figures 4a, c, 5a, c and 6a, c show the settlement troughs at the surface and at depth for various tunnel volume losses. As shown in these Figures, the settlement troughs obtained from numerical models are slightly wider than those measured in centrifuge tunnel tests using PIV techniques. However, they align well with the settlement troughs measured by lasers and LVDTs, which are recorded at the mid-length of the strongbox and are, therefore, unaffected by soil friction with the plexiglass interface.

In Figures 4b, 5b, and 5d, which correspond to cases with a relative density of 90%, a nonlinear evolution of the settlement at the centerline above the tunnel is observed as tunnel volume loss increases. As noted by Franza et al. (2019) and Vitali et al. (2024b), dilatancy around the tunnel perimeter during stress relief leads to a nonlinear relationship between settlement and tunnel volume loss by reducing the settlement gradient. Interestingly, this nonlinear behavior is not observed in Figure 4d, which shows settlement at depth for a cover-to-diameter ratio of 1.33 and a relative density of 90%. This may be because the tunnel is too shallow, and

the point above the tunnel at a depth of $0.4 \cdot Z_t$ is too close to the tunnel crown.

In Figures 6b and 6d, which correspond to a relative density of 50%, the vertical displacements at the surface and at depth exhibit a less pronounced nonlinear evolution as tunnel volume loss increases compared to cases with a relative density of 90%. This is expected since the dilatancy is smaller, resulting in more linear behavior. As shown by Vitali et al. (2024b) and Franza et al. (2019), this relationship remains linear in loose sands where dilatancy is minor or nonexistent.

3. Effects of relative density and tunnel depth

The influence of relative density and tunnel depth on the mechanics of shallow tunnel excavation in cohesionless soil is investigated through a series of numerical experiments conducted using the validated numerical modeling approach. A circular tunnel with a radius, R , equal to 5m is analyzed in sandy soil with relative densities, D_r , of 50% and 90%

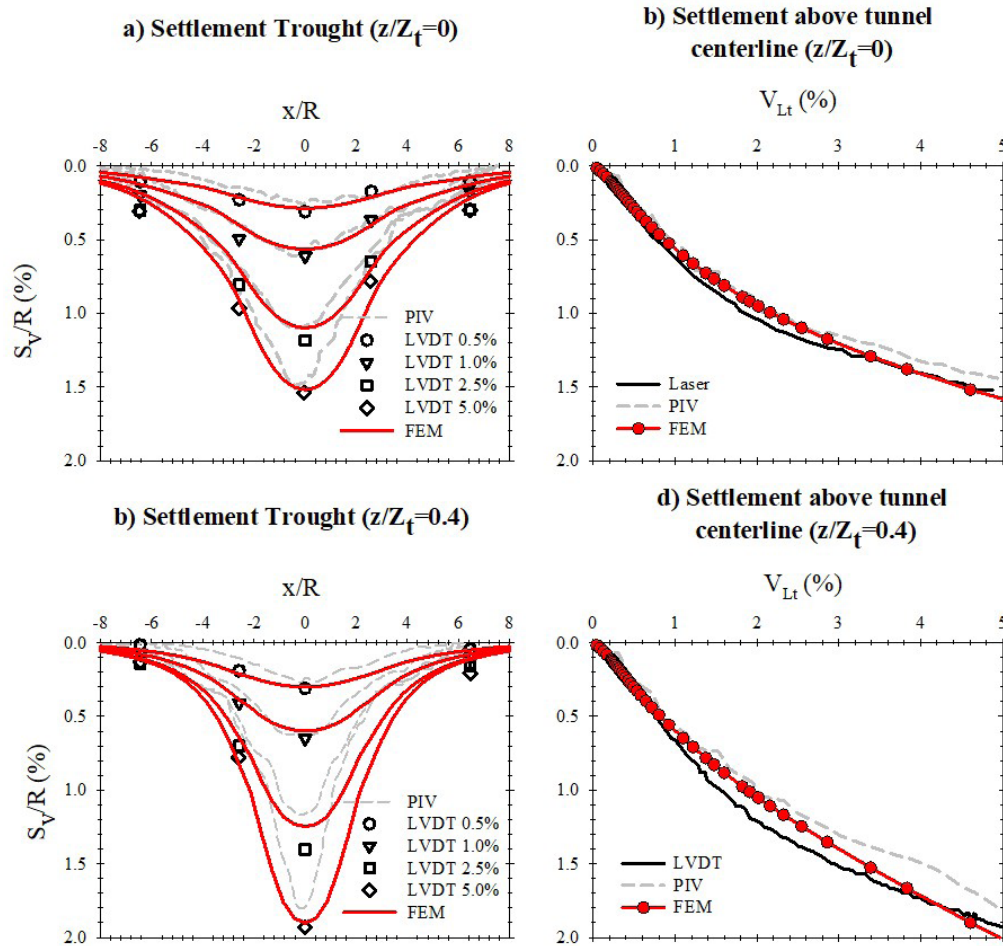


Figure 5. Comparison between FEM results and centrifuge test ($C/D = 2.44$ and $D_r = 90\%$). Settlement trough normalized with respect to the tunnel radius (R) at: (a) the ground surface, and (c) $Z/Z_t = 0.4$; and normalized settlement above the crown at: (b) the ground surface, and (d) $Z/Z_t = 0.4$. The tunnel radius is equal to 2.325 m (i.e., $R = 2.325$ m).

and cover-to-diameter ratios, C/D , of 1.5, 2.0, and 2.5. The selected scenarios are representative of typical shallow tunnels in urban environments.

3.1 Tunnel failure

As discussed in the previous section, the numerical modeling boundary conditions replicate those of the centrifuge tunnel tests conducted by Marshall (2009), Farrell (2011), and Zhou (2015). In these tests, hydrostatic pressure is applied at the tunnel perimeter, with the initial internal pressure at the springline elevation set equal to the overburden stress at that depth. Once stress equilibrium is established, displacements are reset, and the internal pressure is gradually reduced until failure occurs.

Figure 7 illustrates the evolution of normalized internal radial stress ($\sigma_r/\gamma D$) at the springline as tunnel volume loss (V_{Lt}) increases for cover-to-diameter ratios of 1.5, 2.0, and 2.5, with relative densities of 50% (Figure 7a) and 90% (Figure 7b). As shown, the internal radial stress at the tunnel

perimeter decreases towards a constant value as the tunnel volume loss increases. The tunnel volume loss required to reach the minimum internal radial stress depends on tunnel depth and the relative density of the sandy ground. The shallower the tunnel and the denser the soil, the smaller the tunnel volume loss needed to reach the minimum internal radial stress.

Atkinson & Potts (1977) proposed a closed-form solution to determine the minimum internal radial stress required at the tunnel perimeter to maintain the stability of an opening in cohesionless ground. Atkinson & Potts's solution indicates that the required minimum internal radial stress is independent of the cover-to-diameter ratio. Consequently, shallower tunnels require less stress relief (i.e., a reduction in initial internal radial stress) to reach the minimum radial stress, which may correspond to failure. Therefore, although the internal pressure at failure remains the same regardless of the cover-to-diameter ratio, a shallower tunnel will collapse before a deeper tunnel, as the stress relief required to reach failure is smaller.

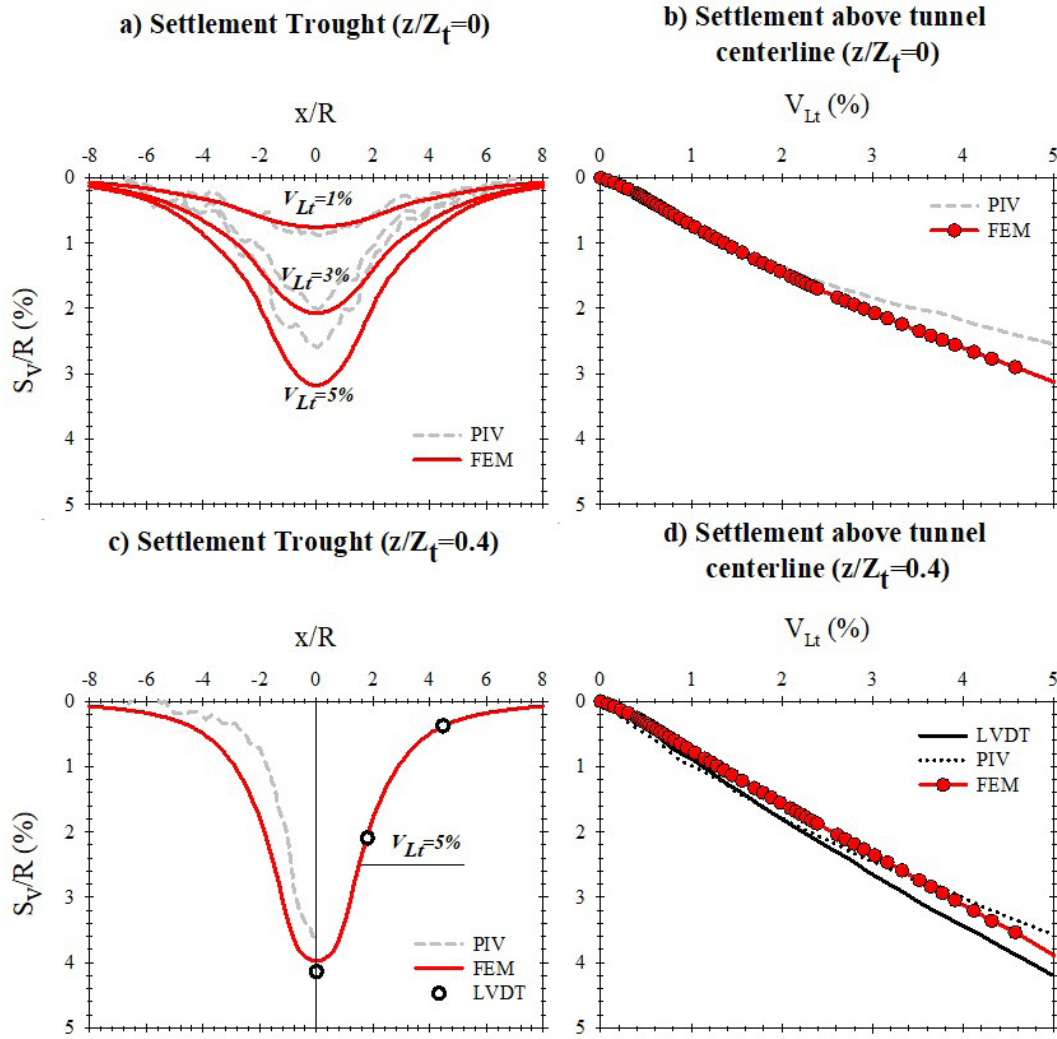


Figure 6. Comparison between FEM results and centrifuge test ($C/D = 2.0$ and $D_r = 50\%$). Settlement trough normalized with respect to the tunnel radius (R) at: (a) the ground surface, and (c) $Z/Z_t = 0.4$; and normalized settlement above the crown at: (b) the ground surface, and (d) $Z/Z_t = 0.4$. The tunnel radius is equal to 3.6 m (i.e., $R = 3.6$ m).

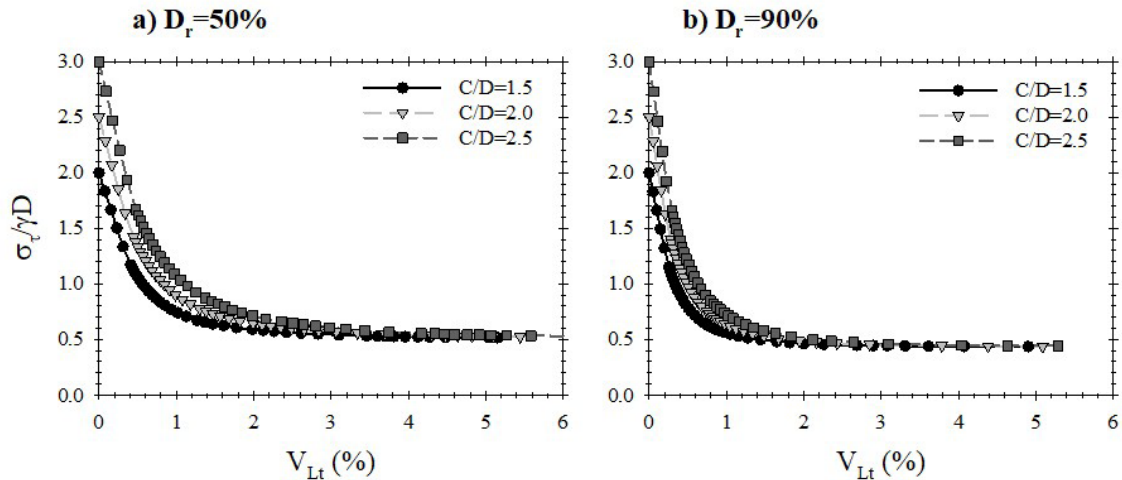


Figure 7. Normalized internal radial stress at springline versus tunnel volume loss, for $C/D=1.5$, 2.0 and 2.5 ; $D_r=50\%$ (a) and $D_r=90\%$ (b).

The normalized minimum internal radial stress depends only on the friction angle, with $\sigma/\gamma D$ equal to 0.52 for a relative density of 50% (friction angle of 35 degrees) and $\sigma/\gamma D$ equal to 0.44 for a relative density of 90% (friction angle of 39 degrees).

For a valid comparison between numerical and analytical solutions, the numerical model must match the boundary conditions of the analytical solution. For example, Vitali et al. (2024a) demonstrated excellent agreement between numerical modeling and analytical solution of Caquot (1934), later extended by Carranza-Torres et al. (2013). In this study, the minimum radial stress does not align with the solution of Atkinson & Potts (1977) due to differences in boundary conditions.

As shown in Figure 7, the minimum radial stress required to maintain stability has already been reached in all studied cases when the tunnel volume loss reaches 5%, indicating opening instability. Figure 8 presents the equivalent plastic strains that develop around the tunnel for all scenarios at 5% tunnel volume loss. The definition of equivalent plastic strain is given by Equation 1:

$$\varepsilon_p = \int \sqrt{\frac{2}{3} \dot{\varepsilon}_{ij} \dot{\varepsilon}_{ij}} dt \quad (1)$$

In all cases, a pronounced accumulation of plastic strain exceeding 5% is observed at the tunnel crown. This results from the boundary conditions imposed in the numerical models, which replicate those of the centrifuge tunnel tests. The observed behavior is consistent with the accumulated shear strains observed in centrifuge tests using PIV techniques (Marshall, 2009; Farrell, 2011; Zhou, 2015).

Notably, a plastic strain zone develops at the ground surface. For $C/D = 1.5$, this zone clearly connects with the plastic strain zone surrounding the tunnel, suggesting the potential for a daylight failure mechanism (Vitali et al., 2024a). However, this connection is less pronounced for $C/D = 2$ and nearly nonexistent for $C/D = 2.5$. Interestingly, although the minimum radial stress threshold is reached in all cases, the numerical models do not exhibit a distinct daylight failure mechanism.

Furthermore, at a relative density of 90%, plastic strain accumulation is more localized at the tunnel crown, whereas at a relative density of 50%, plastic strain is more widely distributed. This difference arises from the more dilatant behavior of denser sands, which enhances the arching effect of stress redistribution around the tunnel, as discussed in detail by Vitali et al., 2024b.

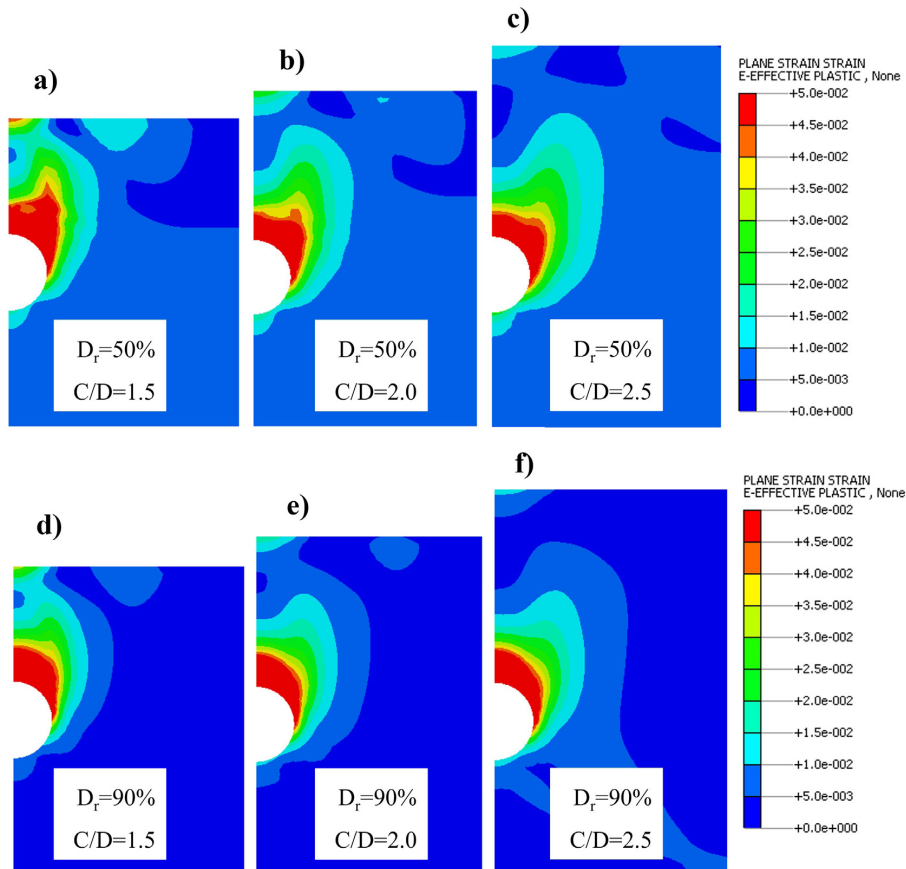


Figure 8. Equivalent plastic strain field for $V_L = 5\%$, for the different cases.

3.2 Volumetric behavior

As discussed by Franza et al. (2019) and Vitali et al. (2024b), shallow tunnels in sandy soils exhibit complex behavior due to volumetric deformations in the ground as tunnel volume loss increases. Consequently, surface settlements and horizontal displacements, and thus the potential for damage to adjacent structures, are closely linked to the distribution of volumetric strains within the sandy soil mass.

Figure 9 illustrates the volumetric strain distribution for cover-to-diameter ratios of 1.5, 2.0, and 2.5, and relative densities of 50% and 90%, considering tunnel volume losses of 1%, 3%, and 5%. Negative volumetric strains indicate dilation, while positive values represent contraction. For a tunnel volume loss of 1%, the volumetric strains surrounding the tunnel are relatively small (i.e., mostly smaller than 0.1%). However, it is possible to observe the onset of an arch-shaped contraction zone forming above the tunnel near

the surface, extending to the side of the tunnel for sandy soil with a relative density of 50%, and a dilation zone at the tunnel perimeter extending from the springline to the tunnel crown for sandy soil with a relative density of 90%.

For large tunnel volume losses (e.g., 3% and 5%), the arch-shaped contracting zone and the dilation zone at the tunnel perimeter become fully developed and well-defined. The shapes of the contraction and dilation zones are similar across the different tunnel depths studied ($C/D = 1.5, 2.0$, and 2.5); however, the volume of the arch-shaped contraction zone increases with tunnel depth, while the size of the dilation zone above the tunnel appears to be only slightly affected by tunnel depth. On the other hand, the dilation zone is significantly larger, and the arch-shaped contracting zone is considerably less pronounced for a relative density of 90% than for 50%, due to the more dilative behavior of denser sands.

The development of the dilative and contraction zones results from the arching of stresses around the opening, as

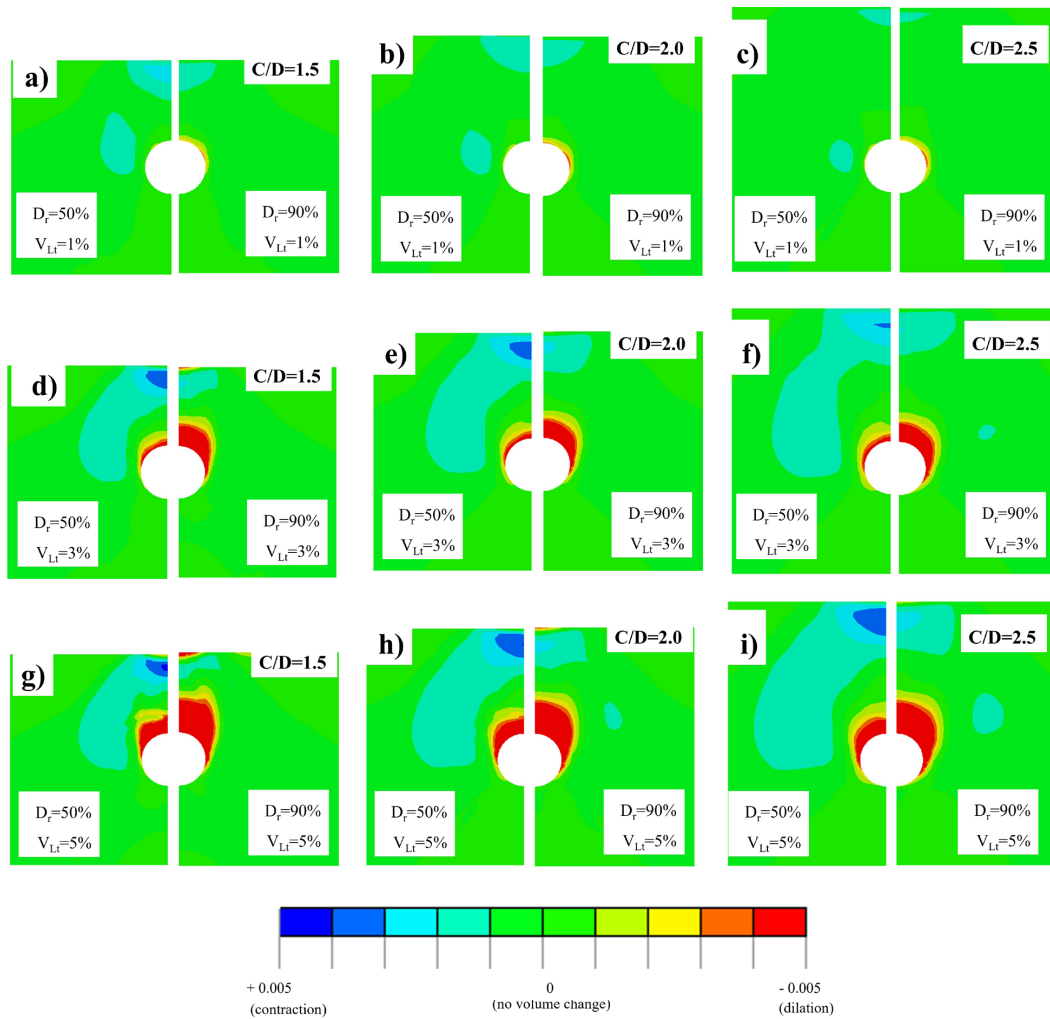


Figure 9. Volumetric strain field for tunnel volume loss equal to 1%, 3% and 5%. $D_r = 50\%$ at left and $D_r = 90\%$ at right. Dilation is represented by negative strain and contraction positive. Figures (a) to (c), (d) to (f), and (g) to (i) show tunnel volume losses of 1%, 3%, and 5%, respectively. The C/D ratios are divided into columns as follows: 1.5 (a, d, g), 2.0 (b, e, h), and 2.5 (c, f, i).

discussed by Vitali et al. (2024b). Within the contraction zone, the mean effective stress increases as tunnel volume loss grows due to stress redistribution, and the shear demand is low. This leads to contraction even in very dense sandy ground (i.e., relative density of 90%). Near the tunnel, the mean effective stress decreases as tunnel volume loss increases and the shear demand is high, causing dilation in this region, even in loose sands. However, the most significant factor appears to be the relative density of the sandy soil, which ultimately determines whether the overall behavior is more dilative or contractive.

Figure 10 compares tunnel volume loss (V_{Lt}) and soil volume loss (V_{Ls}) for different tunnel depths ($C/D = 1.5, 2.0$, and 2.5) and relative densities ($D_r = 50\%$ and 90%). The tunnel volume loss is computed based on displacements at the tunnel perimeter, while the soil volume loss is derived from the settlement trough at the surface. In tunneling practice, volume loss is typically estimated from the settlement trough since tunnel volume loss is unknown. If no volume change occurs, such as in the case of undrained behavior in saturated clays, the tunnel and soil volume losses are equal. However, in sandy soils, which undergo volumetric changes, these volume losses differ, as illustrated in Figure 9.

As shown in Figure 10, the relationship between soil and tunnel volume loss is nearly linear for small tunnel volume losses, with increments in soil volume loss exceeding those in tunnel volume loss (i.e., the tangent of the slope angle is greater than 1). Notably, for tunnel volume losses below approximately 1%, this relationship remains similar regardless of tunnel depth or relative density. As tunnel volume loss increases, the relationship becomes nonlinear, with increments in soil volume loss gradually decreasing

relative to tunnel volume loss (i.e., the tangent of the slope approaches 1). Eventually, the increments in soil volume loss become smaller than those in tunnel volume loss (i.e., the tangent of the slope falls below 1). This transition occurs because, as tunnel volume loss increases, a dilation zone develops around the tunnel perimeter and expands in size and intensity, eventually leading to an overall dilative volumetric response of the sandy ground. For larger tunnel volume losses, tunnel depth, and soil relative density significantly influence the relationship between soil and tunnel volume loss. The shallower the tunnel and the denser the sandy soil, the more pronounced the dilative response of the ground.

3.3 Vertical displacements profile

The vertical displacements along the tunnel centerline above the tunnel crown for all studied scenarios (i.e., $C/D = 1.5, 2$ and 2.5 and relative densities of 50 and 90%) are analyzed in this section. Figures 11a, c, e show the surface settlement to crown displacement ratio ($S_{v,s}/S_{v,crown}$) with tunnel volume loss (V_{Lt}) and Figures 11b, d, f, the normalized vertical displacements (S_v/R) with depth along the tunnel centerline for $V_{Lt} = 1, 3$, and 5% .

As shown in Figure 11, for tunnel volume losses smaller than 1%, the ratio $S_{v,s}/S_{v,crown}$, as well as the vertical displacement profiles observed for relative densities of 50% and 90%, follow the same trend regardless of tunnel depth, however the absolute values of $S_{v,s}/S_{v,crown}$ decrease as the relative depth increases. This is consistent with Figure 10, which shows that the evolution of tunnel and soil volume losses is similar when tunnel volume loss is small. As discussed in the previous section, the development of a

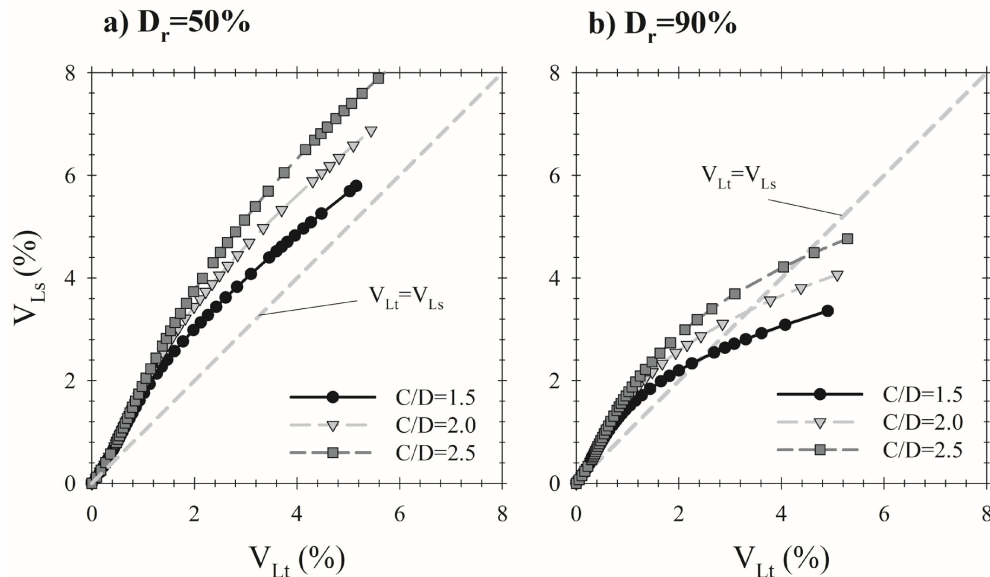


Figure 10. Soil volume loss versus tunnel volume loss, for $C/D = 1.5, 2.0$ and 2.5 ; $D_r = 50\%$ (a) and $D_r = 90\%$ (b).

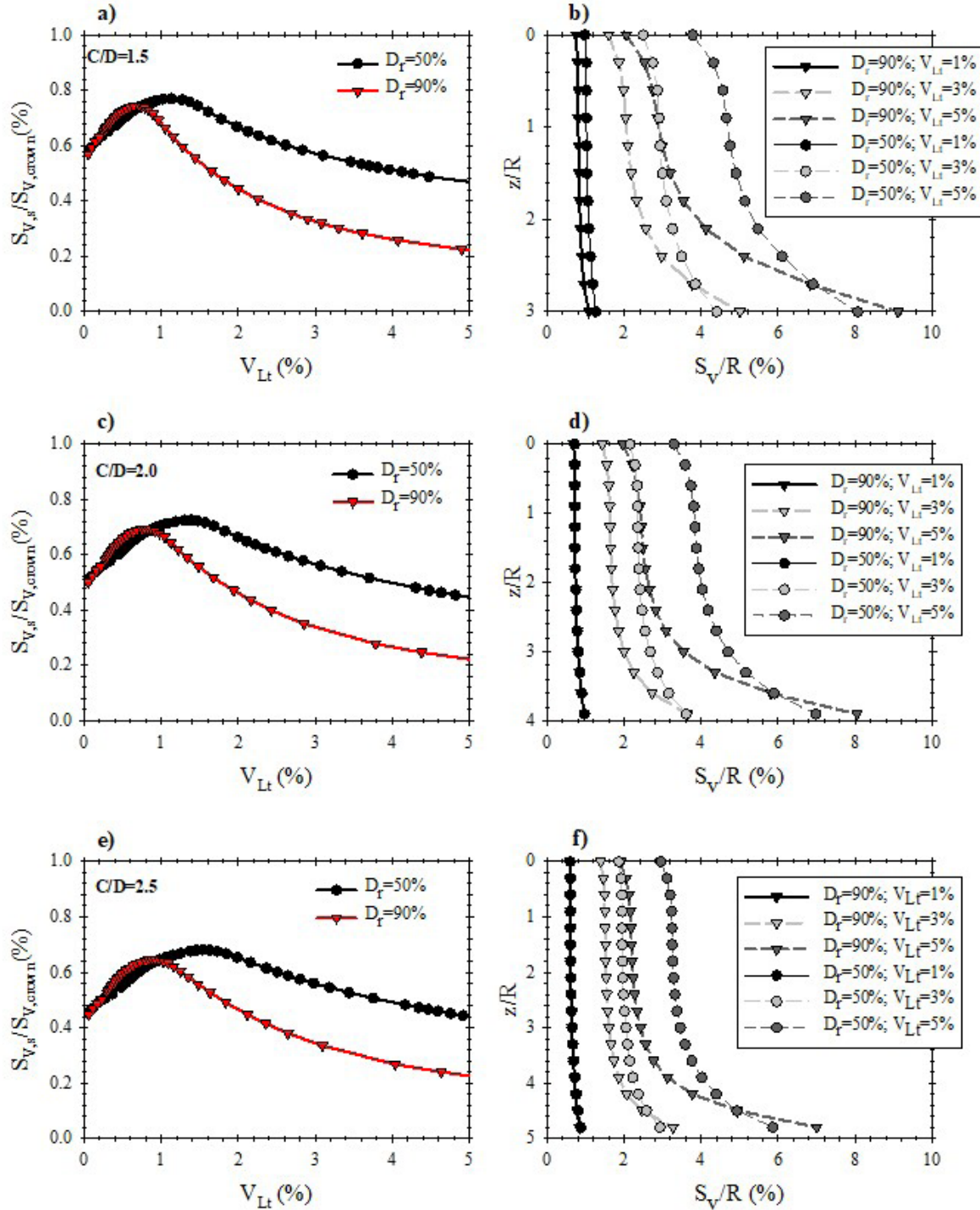


Figure 11. (a, c, e) Ratio between vertical displacements at the ground surface and the tunnel crown with tunnel volume loss; (b, d, f) Normalized vertical displacements versus depth for tunnel volume losses of 1, 3 and 5%. Results for $C/D = 1.5$ (a, b), 2.0 (c, d) and 2.5 (e, f). The tunnel radius is equal to 5m (i.e., $R = 5$ m).

dilative zone around the tunnel perimeter and the transition from contractive to dilative ground behavior require relatively large tunnel volume losses. For tunnel volume losses below 1%, this transition has not yet occurred.

As shown in Figures 11a, 11c, and 11e, the ratio $S_{V,s}/S_{V,crown}$ increases with tunnel volume loss, reaching a maximum value before gradually decreasing as tunnel volume loss continues to increase. This trend is associated with the development

of the dilation zone surrounding the tunnel perimeter, which facilitates stress arching and consequently reduces ground deformations reaching the surface, as discussed by Vitali et al. (2024b). Thus, the observed peak in the ratio $S_{v,s}/S_{v,crown}$ is linked to the formation of the dilation zone around the tunnel perimeter. As shown in Figure 11, the tunnel volume loss corresponding to the maximum ratio $S_{v,s}/S_{v,crown}$ decreases as the sandy soil becomes denser and the tunnel becomes shallower.

Furthermore, Figures 11b, d, and f show that vertical displacements with depth along the tunnel centerline exhibit little variation for small volume losses (i.e., 1%). However, the settlement rapidly increases at depths approaching the tunnel crown for large values of tunnel volume loss (e.g., 3% and 5%). The displacement pattern and significant variation at the tunnel crown are consistent with volumetric strain zones (see Figure 9).

As shown, the volumetric changes in the ground significantly affect how deformations propagate to the surface. From a tunneling engineering perspective, this reinforces the importance of instrumentation readings near the tunnel crown, using devices such as profilometers and deep settlement devices. Meanwhile, surface benchmark readings are directly impacted by volumetric variations in the ground, emphasizing the need for a comprehensive monitoring strategy.

3.4 Surface displacements

Figure 12 presents the settlements and horizontal displacements at the ground surface as a function of the

distance from the tunnel centerline, x , for all studied cases. These cases include cover-to-diameter (C/D) ratios of 1.5, 2.0, and 2.5, with a relative density of 50% shown on the left side of the graphs and 90% on the right side. The presented displacements correspond to tunnel volume losses of 1%, 3%, and 5%.

For small tunnel volume losses (1%), both horizontal and vertical displacements are relatively small and show little variation between soil relative densities. This is consistent with the discussion in previous sections. As tunnel volume loss increases, the displacements at the ground surface also increase, and the differences in vertical and horizontal displacements between relative densities of 50% and 90% become more pronounced.

Deeper tunnels (e.g., $C/D = 2.5$) exhibit wider settlement troughs and smaller maximum surface displacements compared to shallower tunnels (e.g., $C/D = 1.5$). Thus, deeper tunnels are less likely to cause damage to adjacent structures than shallower tunnels.

In tunneling engineering practice, horizontal displacements are often estimated from vertical displacements by assuming the direction of the displacement vector. The horizontal displacement can then be determined using the following Equation 2:

$$S_h = S_v \cdot x / Z_v \quad (2)$$

where S_h and S_v represent horizontal and vertical displacements, x is the offset from the tunnel centerline, and Z_v is the depth

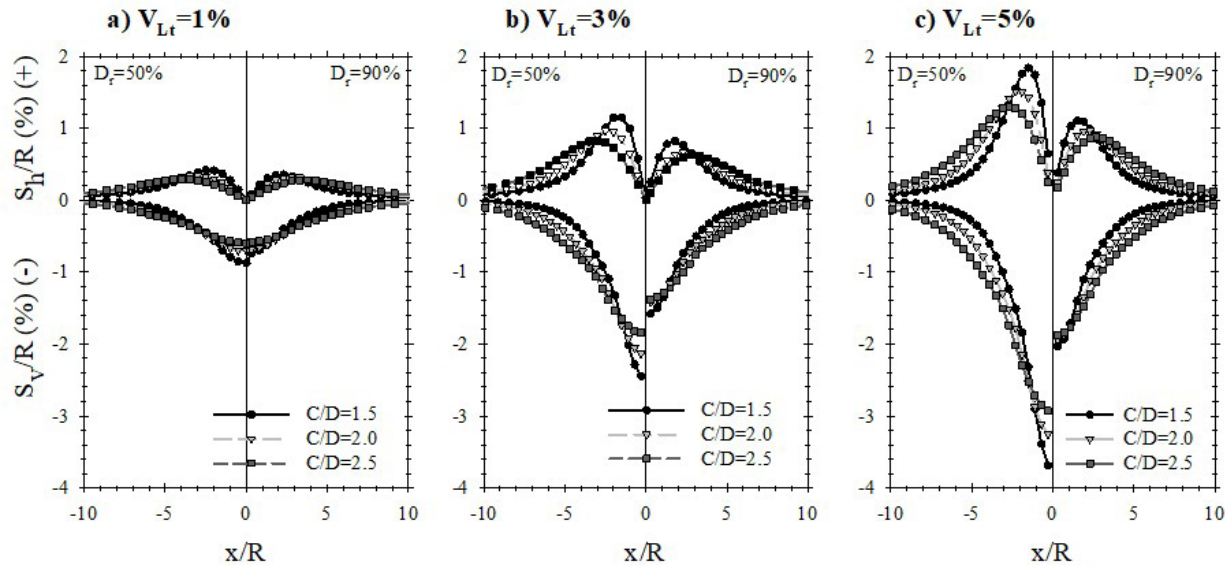


Figure 12. Normalized vertical and horizontal displacements, at the ground surface versus the normalized distance from the tunnel centerline. Tunnel volume loss of 1% (a), 3% (b) and 5% (c), for $C/D = 1.5, 2.0$ and 2.5 ; $D_r = 50\%$ (left) and $D_r = 90\%$ (right). The tunnel radius is equal to 5 m (i.e., $R = 5$ m).

at which the trajectory of the displacement vector intersects the tunnel centerline, as shown in Figure 13a.

O'Reilly & New (1982) suggested that, under constant volume conditions, the displacement vector of any point on the ground surface at a distance x from the tunnel centerline points toward the tunnel's center (i.e., $Z_v = Z_t$).

Figures 13b and 13c show the variation of Z_v with x , both normalized by Z_t and R , respectively. The results are

shown up to absolute values of x/R equal to 6, beyond which vertical and horizontal displacements are small and irrelevant to above ground structures.

Notably, the ratio Z_v/Z_t is always less than 1, indicating that the displacement vector trajectory intersects the tunnel centerline above the tunnel's center. As a reference, the relative depth Z_v/Z_t at the tunnel crown equals 0.75, 0.8, and 0.83 for cover-to-diameter ratios (C/D) of 1.5, 2.0, and 2.5, respectively.

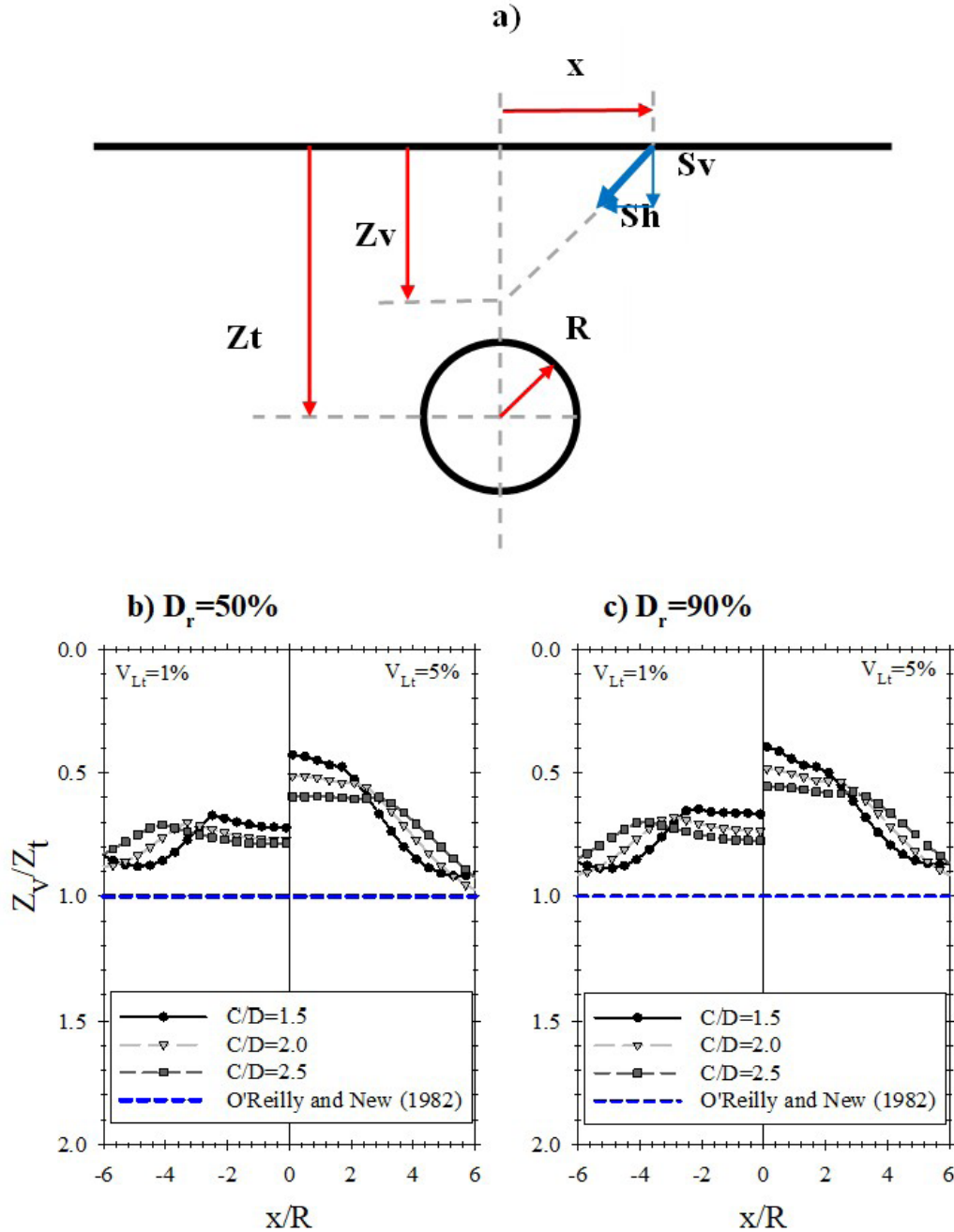


Figure 13. Normalized intersection depth, Z_v , with respect to the tunnel depth, Z_t , versus normalized distance from the tunnel center line, x/R . Tunnel volume loss equal to 1% (left) and 5% (right), for $C/D = 1.5$, 2.0 and 2.5; $D_r = 50\%$ b) and $D_r = 90\%$ c). Comparison with O'Reilly & New (1982). The tunnel radius is equal to 5 m (i.e., $R = 5$ m).

Additionally, the intersection depth Z_v varies significantly with the distance from the tunnel centerline, x , the soil relative density D_r , the cover-to-diameter ratio C/D , and the tunnel volume loss V_{Lt} . The numerical modeling results indicate that assuming a constant Z_v/Z_t ratio to predict horizontal displacements based on settlement may lead to inaccurate predictions in sandy ground.

Predicting horizontal displacements is crucial for assessing damage to aboveground structures. As shown in Figure 13, it is difficult to directly relate horizontal displacements to vertical ones, as various factors influence this relationship. The next section presents a correlation between horizontal strain and angular distortions at the ground surface. Vitali et al. (2024b) demonstrated that estimating the horizontal strains directly from the angular distortion at the surface is a more effective approach for predicting the potential for building damage.

3.5 Potential damage in structures

Assessing the potential for structural damage to adjacent structures due to tunnel excavation is a critical and complex stage in tunnel design. Predictions depend on various factors, such as the geomechanical behavior of the soil mass, excavation geometry, construction sequence, tunnel depth, structural stiffness, conditions of the adjacent structures, and foundation type of adjacent buildings. Given the complexity of the analysis, a staged approach to damage assessment is a more reasonable practice in tunnel design. It begins with a greenfield assessment and increases in complexity only if necessary (e.g., incorporating adjacent structures in the numerical modeling of the tunnel excavation).

Currently, tunnel design heavily relies on numerical modeling, allowing for a complete prediction of displacements and strain fields. Vertical and horizontal displacements obtained under greenfield conditions may be used for preliminary assessment of potential damage to adjacent structures. Assuming that a structure deforms according to greenfield conditions is a conservative and practical approach for preliminary analyses, especially given the uncertainties present in the early design stages. This assumption ensures that potential deformation effects are not underestimated, allowing for safer initial assessments before more refined evaluations are conducted. Rigid structures tend to deform more uniformly, and depending on the foundation type, such as a raft foundation, horizontal deformations may become negligible. It is important to highlight that vulnerable and historic structures, due to simpler construction methods, typically tend to deform under greenfield conditions.

Given both vertical and horizontal displacements, the Son & Cording (2005) criterion provides a practical method for assessing potential surface damage. This approach considers the principal strain (ε_p), directly correlating angular distortion (β) with horizontal strain (ε_h), and there is no need for structure parameters, as shown in Equation 3:

$$\varepsilon_p = \varepsilon_h \cos^2 \theta_{max} + \beta \sin \theta_{max} \cos \theta_{max} \quad (3)$$

where θ_{max} is given by $\tan 2\theta_{max} = |\beta / \varepsilon_h|$.

Using Equation 3, it is possible to quickly determine the maximum principal strains. Son & Cording (2005) also define structural damage levels based on principal strain values. In this study, three damage intervals will be used as reference:

- I. Slight to Moderate: $0.075\% < \varepsilon_p < 0.167\%$
- II. Moderate to Severe: $0.167\% < \varepsilon_p < 0.333\%$
- III. Severe to Very Severe: $\varepsilon_p > 0.333\%$

Figure 14 shows the evolution of the maximum principal strain at the ground surface as tunnel volume loss increases for different cover-to-diameter ratios ($C/D = 1.5, 2.0$, and 2.5) and relative densities of 50% (Figure 14a) and 90% (Figure 14b). The damage classification ranges proposed by Son & Cording (2005) are also indicated. Figures 14c and 14d present the normalized distance from the tunnel centerline, x/Z_t , where maximum principal strain ($\varepsilon_{p,max}$) occurs with the tunnel volume loss for the same cases.

As shown, for a relative density of 50% (medium-dense sandy soil), the principal strain evolves almost linearly with tunnel volume loss. In contrast, for a relative density of 90% (dense sand), the principal strain exhibits a slightly nonlinear evolution as the tunnel volume loss (V_{Lt}) increases due to dilatancy effects, as discussed in previous sections.

The potential for structural damage is significantly lower at higher relative densities and greater depths. Notably, even under large tunnel volume losses in medium dense sand (i.e., $D_r=50\%$), for $C/D=2.5$, the accumulated principal strain does not reach the damage category III, highlighting the potential to avoid surface damage by increasing the tunnel depth.

Regarding the location of the maximum principal strain, $\varepsilon_{p,max}$, Figures 14c and 14d show that the normalized distance from the tunnel centerline with respect to the tunnel depth, x/Z_t , where $\varepsilon_{p,max}$ occurs, is only slightly affected by the sandy soil relative density (D_r) and the cover-to-diameter ratio (C/D). However, it is influenced by tunnel volume loss. As shown, the location of the maximum principal strain tends to shift closer to the tunnel centerline as the tunnel volume loss increases. For small tunnel volume losses, x/Z_t ranges from 0.9 to 1.2, whereas for large tunnel volume losses, it ranges from 0.6 to 0.8.

Figure 15 presents the relationship between angular distortion and horizontal strain corresponding to the maximum principal strain at the ground surface for the investigated cases in this study ($C/D = 1.5, 2.0$, and 2.5 ; $D_r = 50\%$ and 90%). The graph also includes the damage classification zones proposed by Son & Cording (2005), the reference from Vitali et al. (2024b) based on the NorSand model, which suggests a direct relationship between angular distortion and horizontal strain ($\varepsilon_h=0.67\beta$) and the relationship suggested by the results in this study ($\varepsilon_h=0.77\beta$). The results obtained in this study reveal a nearly linear trend, even for high tunnel volume losses, closely matching the predictions by Vitali et al. (2024b).

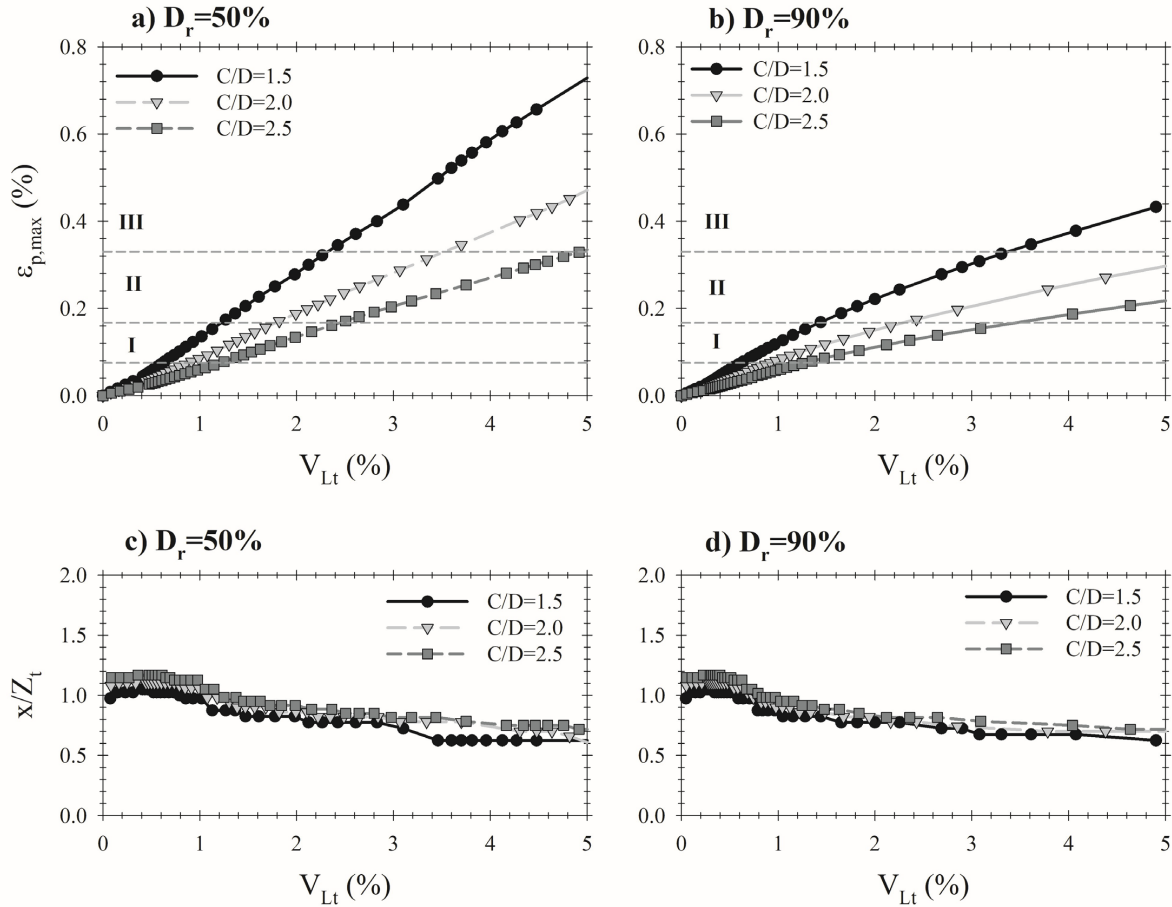


Figure 14. Maximum principal strain ($\varepsilon_{p,max}$) at the ground surface (a and b) and normalized distance to the tunnel centerline with respect to the tunnel depth (x/Z_t) where $\varepsilon_{p,max}$ occur (c and d) versus tunnel volume loss, for $C/D = 1.5, 2.0$ and 2.5 ; $D_r = 50\%$ (a and c) and $D_r = 90\%$ (b and d).

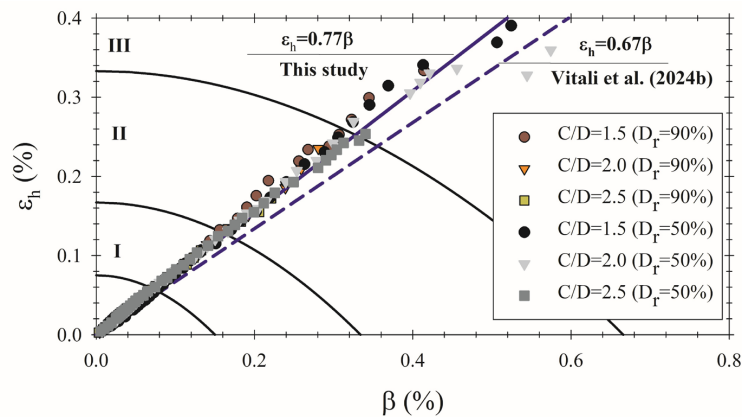


Figure 15. Horizontal strain and angular distortions corresponding to the maximum principal strain at the ground surface. Regions I, II, and III correspond to slight, moderate to severe, and severe to very severe damage.

This direct correlation simplifies the inclusion of horizontal strain in damage assessments. As shown earlier, predicting horizontal displacements from vertical ones alone is challenging due to various influencing factors. In tunneling practice, vertical displacements are typically measured and

approximated using empirical functions like the Gaussian (Peck, 1969; Schmidt, 1969) or the Yield Density curve (Celestino & Ruiz, 1998; Celestino et al., 2000). Incorporating the horizontal strain into damage evaluation can be done by following the practical steps:

- i. Fit the settlement trough using an appropriate function (e.g., Yield Density),
- ii. Calculate angular distortions along the surface;
- iii. Identify angular distortions at $x/Z_t \approx 0.7$ to 1 and estimate horizontal strain using $\epsilon_h = 0.72\beta$ (average from this study and Vitali et al., 2024b).
- iv. Compute the principal strain (Equation 2) and classify the potential damage. Another possibility is to directly input the angular distortion value in Figure 15, eliminating step iii.

4. Conclusions

This study presents a comprehensive investigation of shallow tunnel behavior in sandy ground using numerical modeling that replicates centrifuge tunnel tests. The numerical modeling approach was validated through comparisons with three centrifuge tunnel tests. The numerical models accurately replicated the observed deformation patterns and settlements, closely matching centrifuge test results at varying tunnel depths, diameters, and sand relative densities for tunnel volume losses up to 5%. With the calibrated model, a set of numerical experiments was performed to investigate the behavior of shallow tunnels in cohesionless soil.

It was observed that the minimum internal radial stress required for stability is independent of the tunnel depth. This aligns with Atkinson & Potts's (1977) analytical solution. However, differences in boundary conditions between numerical and analytical solutions must be carefully considered to ensure valid comparisons.

The numerical experiment results show that the soil volumetric response is strongly influenced by tunnel volume loss, tunnel depth, and soil relative density. The numerical experiments show that the sandy ground behavior remains predominantly contractive for small tunnel volume losses, with soil volume loss exceeding tunnel volume loss even in dense sands for tunnel volume losses (V_{Lt}) up to about 1%. However, as tunnel volume loss increases, the overall volumetric behavior gradually transitions from contractive to dilative for the relative densities studied (50% and 90%). Two distinct zones develop: an arch-shaped contraction zone extending from the ground surface to the tunnel springline and a dilative zone above the tunnel crown. These zones are associated with stress arching around the opening and are influenced by tunnel volume loss, tunnel depth, and soil relative density.

Volumetric strains influence the displacements above the tunnel, with contraction and dilation zones significantly affecting how excavation-induced stress relief manifests at the surface. From a tunneling engineering perspective, this underscores the importance of instrumentation near the tunnel crown, using devices such as profilometers and deep settlement devices. Meanwhile, surface benchmark readings are directly affected by volumetric variations in the ground, highlighting the need for a comprehensive monitoring strategy.

Numerical modeling results demonstrate that the depth at which the displacement vector intersects the tunnel centerline (Z_v) varies with tunnel depth, relative density, distance from the tunnel centerline, and tunnel volume loss. The numerical results indicate that assuming a displacement vector trajectory intersecting the tunnel centerline at a fixed Z_v can lead to inaccuracies in estimating horizontal displacements from surface settlements.

The findings of this study support the strong correlation observed by Vitali et al. (2024b) between angular distortion and horizontal strain at the location of maximum principal strain on the ground surface. The nearly linear trend observed simplifies the incorporation of horizontal strain into damage assessments. This study presents a practical approach for estimating horizontal strain by leveraging well-established empirical settlement functions (e.g., the Yield Density function, Celestino & Ruiz, 1998; Celestino et al., 2000) and derived angular distortions. These results contribute to refining damage prediction methods, offering a more reliable approach for integrating horizontal strain effects into damage assessments.

The findings of this research should not be extrapolated beyond the assessed scenarios. For different conditions, the approach presented in this paper is recommended, which involves rigorously calibrating soil behavior using an advanced constitutive model based on high-quality laboratory or field data, along with a detailed numerical modeling approach following well-established procedures.

Acknowledgements

The authors acknowledge the support provided by the research funding agency of the Brazilian Government, CNPq (Conselho Nacional de Desenvolvimento Científico e Tecnológico), Contract 162113/2020- 0: Post-Doctoral Scholarship, and CAPES (Coordenação de Aperfeiçoamento de Pessoal de Nível Superior): Scholarship for Master's Program. The authors also thank the Midas Information Technology Co., Ltd. for providing the license for the Midas GTS NX software used in this research.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Tarcisio Barreto Celestino: conceptualization, data curation, methodology, supervision, validation, writing – original draft. writing – review & editing. Felipe Paiva Magalhães Vitali: conceptualization, data curation, formal analysis, methodology. Osvaldo Paiva Magalhães Vitali: conceptualization, data curation, methodology, supervision, validation, writing – review & editing.

Data availability

The datasets generated and analyzed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols

c	Cohesion
m	Power for stress-level dependency of stiffness
q	Deviatoric stress
z	Depth
C	Cover
D	Diameter
Dr	Relative density
E_{50}	Secant Young's modulus in standard drained triaxial test
E_{oed}	1D compression modulus for primary oedometer loading
E_{ur}	Unloading/reloading Young's modulus
$K_{0,NC}$	K_0 -value for normal consolidation
OCR	Overconsolidation ratio
P_{ref}	Reference stress for stiffness
R	Radius
R_f	Failure ratio
S_h	Horizontal displacement
S_v	Vertical displacement
V_{Ls}	Soil volume loss
V_{Lt}	Tunnel volume loss
β	Angular distortion
γ	Unit weight
ε_{ax}	Axial strain
ε_h	Horizontal strain
ε_p	Principal strain
ε_p	Equivalent plastic strain
ε_{vol}	Volumetric strain
ν_{ur}	Poisson's ratio for unloading/reloading
σ_t	Tunnel pressure – radial stress
ϕ	Friction angle
Ψ	Dilatancy angle

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The 7th Victor de Mello Goa Lecture: Climate resilient design of embankment using geocomposites

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Lecture

Keywords

Intensity-Duration-Frequency curves
Non-stationary
Embankments
Geocomposite
Climate

Abstract

Climate change is driving non-stationary rainfall patterns, intensifying both the frequency and magnitude of extreme precipitation events, which pose significant risks to earthen embankments. Traditional Intensity-Duration-Frequency (IDF) curves, based on stationary climate assumptions, often underestimate future hydrological loads. This study investigates the stability of embankments with and without geocomposite drainage layers under both stationary and non-stationary IDF scenarios, focusing on return periods of 10, 30, 50, and 100 years. Bengaluru, India, a monsoon-affected urban region, serves as the case study, with future rainfall projections derived from the CMIP6 SSP5-8.5 scenario to represent high-emission pathways. Numerical simulations reveal that unreinforced embankments under non-stationary conditions experience rapid pore-water pressure accumulation and a significant drop in Factor of Safety (*FOS*), reaching critical saturation in as little as 12 hours for 50-year return events. In contrast, geocomposite-reinforced embankments exhibit improved drainage, delayed saturation, and maintain *FOS* above 1.5 under stationary scenarios, and up to 18 hours of stability in the 100-year non-stationary case.

1. Introduction

The increasing frequency and intensity of extreme weather events pose significant challenges for infrastructure stability worldwide, especially in regions with pronounced seasonal rainfall. Among such events, heavy rainfall is particularly impactful, threatening the stability of geotechnical structures like embankments, retaining walls, and flood protection systems. These structures are often designed using Intensity-Duration-Frequency (IDF) curves, which estimate rainfall intensities based on historical data across various return periods (e.g., 10, 50, 100 years) and durations (e.g., hourly, daily). IDF curves are a fundamental design tool in hydrological engineering, enabling designers to establish capacity requirements and resilience thresholds for drainage and retention systems (Westra et al., 2014; Her et al., 2020).

Traditional IDF curves, however, assume stationarity—that is, they rely on the premise that historical climate conditions are indicative of future trends. This assumption is increasingly challenged by the evidence of climate change, which disrupts rainfall patterns, alters storm intensities, and increases the occurrence of extreme events beyond historical norms (IPCC, 2021). Recent studies show that rainfall intensity can increase by approximately 7% per degree Celsius rise in global temperature, indicating that IDF

curves based on stationary assumptions may not accurately capture the scale of future rainfall events (Knutson & Zeng, 2018; O’Gorman, 2015). This growing discrepancy between traditional design assumptions and emerging climate patterns necessitates the development of “non-stationary” IDF curves that incorporate climate projections, providing a more realistic basis for infrastructure planning and resilience (Cheng & AghaKouchak, 2014).

Monsoon-affected regions like Bengaluru, India, are particularly vulnerable to these changes. Projections indicate that such areas will experience not only increased annual rainfall but also shifts in seasonal intensity, leading to more frequent and intense short-duration events (Mishra et al., 2020a). Given these projections, the application of stationary IDFs in embankment and flood control system designs may result in underestimating future peak flows and, consequently, oversights in necessary structural capacity. Research reveals that embankments and drainage infrastructure designed with outdated stationary IDFs are at higher risk of saturation, slope failure, and erosion, particularly during unprecedented high-intensity events (Arnbjerg-Nielsen et al., 2013; Nguyen & Chen, 2021).

To address these issues, recent studies advocate for non-stationary IDF curves that incorporate climate model projections, thereby capturing anticipated changes in rainfall

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Submitted on May 23, 2025; Final Acceptance on May 28, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.008425>



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intensity and frequency. By adjusting rainfall estimations to reflect anticipated climate variability, non-stationary IDF curves provide a more accurate framework for designing infrastructure capable of withstanding future conditions. However, while non-stationary IDF curves improve rainfall intensity predictions, the challenges posed by increased pore-water pressure and structural saturation remain significant for embankments. This brings to light another critical component in embankment design: the use of geosynthetic materials, specifically geocomposite layers.

Geocomposite layers, which consist of geotextiles and geonets, have proven effective in enhancing the drainage and stability of embankments during heavy rainfall events. They function both as capillary barriers, limiting water infiltration into deeper soil layers, and as drainage facilitators, allowing excess moisture to be redirected, thereby reducing pore-water pressures (Zornberg & Mitchell, 1994). In embankments and slopes, the integration of geocomposite layers has been shown to significantly delay critical saturation, maintain a higher Factor of Safety (*FOS*), and prevent early onset of slope failure, even under prolonged or intense rainfall conditions (Bahador et al., 2013; Her et al., 2018). These findings highlight the role of geocomposites as a valuable reinforcement technique in infrastructure subjected to high-intensity rainfall, particularly in scenarios where climate change is expected to exacerbate the frequency of extreme weather events.

Despite the increasing evidence of climate change impacts on rainfall intensity, there remains a critical gap in understanding how to best integrate non-stationary IDF curves and geocomposite reinforcement in embankment design to ensure long-term stability. Existing studies have separately examined the benefits of non-stationary IDF curves for hydrological design and geocomposites for soil stability; however, few have analyzed the combined effect of these innovations on embankment resilience under extreme future rainfall scenarios. This gap is particularly relevant for high-risk regions like Bengaluru, where non-stationary climate projections indicate substantial increases in peak rainfall intensities (IPCC, 2021; Mishra et al., 2020b). Without a combined approach, embankment designs may fail to fully address the complex interactions between increased rainfall intensity, pore-water pressure dynamics, and structural stability, leaving infrastructure susceptible to failure under future climate scenarios.

This study aims to bridge this research gap by evaluating embankments reinforced with geocomposite layers under both stationary and non-stationary IDF conditions, with a focus on rainfall return periods of 10, 30, 50, and 100 years. Using rainfall data from Bengaluru, India, and future projections from the CMIP6 SSP585 scenario, we simulate pore-water pressure distribution and Factor of Safety (*FOS*) in embankments with and without geocomposites. This approach allows for a comprehensive assessment of how geocomposite layers can enhance stability under extreme rainfall events anticipated by non-stationary IDF curves. Ultimately, the research provides insights into adaptive design strategies that enhance the

resilience of embankments and other critical infrastructure in a changing climate, emphasizing the need for climate-informed IDF modelling and geosynthetic reinforcement to ensure safe and sustainable infrastructure.

2. Background and literature review

2.1 Intensity-duration-frequency (IDF) curves

IDF curves are statistical representations used to estimate the intensity of rainfall events of various durations (e.g., hourly, daily) and return periods (e.g., 10, 50, or 100 years). These curves are crucial for engineering applications, especially for designing flood control systems, embankments, and other drainage infrastructures (Her et al., 2018). Traditional IDF curves assume that rainfall patterns observed in historical data will continue unchanged into the future, making them “stationary.” However, numerous studies have demonstrated that climate change introduces significant non-stationarity into precipitation patterns, leading to the need for updated, non-stationary IDF curves (Her et al., 2020).

2.2 Impact of climate change on rainfall

Climate change is expected to increase the intensity and frequency of extreme weather events. Projections from General Circulation Models (GCMs) indicate a rise in global temperatures and subsequent changes in hydrological cycles, resulting in more intense and frequent rainfall in many regions (IPCC, 2021). In the Indian context, cities like Bengaluru are expected to experience increased annual rainfall, further complicating drainage, and flood management efforts (Mishra et al., 2020a).

The implications of these changes for infrastructure design are profound. The use of outdated, stationary IDF curves could lead to under-designed drainage systems, resulting in frequent failures or oversizing that is economically inefficient. Non-stationary IDF curves that incorporate climate model projections offer a more accurate representation of future conditions. If rainfall changes over time due to climate change, the curves may provide an insufficient estimate (or an excessive estimate) of future rainfall intensities or depths. Thus, their use could result in under-designed (or overdesigned) engineering projects. However, climate change is an evolving science and forecasting how rainfall patterns and IDF curves will change in response to it remains an imprecise analysis.

Scaling IDF curves using projected future rainfall from global climate models (or a general circulation model) (Her et al., 2020) is an appealing alternative because changes seen in rainfall from the model output between historical and future periods should theoretically include both the changes caused by temperature as well as other aspects of weather patterns that may influence rainfall intensity in a given location.

Projections from the General Circulation Models (GCMs) play a vital role in understanding the future changes in climate. However, spatial resolution at which GCMs are run is often too coarse to get reliable projections at the regional and local scale. Precipitation projections at higher spatial resolution are required for the climate impact assessments. Moreover, precipitation from the GCMs have a bias due to their coarse resolution or model parameterizations. Therefore, for the assessment of the climate change and its impacts on different sectors (e.g., water resources, agriculture), bias-correction is required. Both statistical and dynamical approaches are used for downscaling and bias correction of climate change projections from GCMs. Statistical approaches are based on the distribution and relationship between the observed and projected data for the historical period. On the other hand, dynamical downscaling approaches are based on regional climate model forced with the boundary conditions from the coarse resolution GCMs. Both statistical and dynamical downscaling approaches have limitations. The primary limitation of the dynamical downscaling is related to the requirement of computational efforts to run the regional climate models at higher spatial and temporal resolution. Moreover, dynamical downscaling may not remove the bias in climate variables, which might require corrections based on the statistical approaches. Given these limitations, statistical bias correction approaches are widely used in climate change impact assessments.

2.3 Geocomposite layers in embankment design

Geocomposite layers consist of geotextiles and geonets that are used to manage water flow in embankments. These layers enhance the drainage capabilities of the soil, acting as both a capillary barrier to prevent rapid infiltration and as a drainage layer that facilitates water flow out of the embankment. The inclusion of geocomposites is particularly beneficial in regions experiencing heavy rainfall or in embankments subjected to prolonged periods of saturation (Zornberg & Mitchell, 1994).

Previous studies have shown that geocomposite layers can significantly improve embankment stability by reducing pore-water pressure and maintaining the soil's shear strength (Bahador et al., 2013). This study will extend this research by simulating the effects of geocomposite layers under different rainfall return periods using both stationary and non-stationary IDF curves.

3. Methodology

3.1 Study area and data collection

The study focuses on Bengaluru, India, a region characterized by monsoonal rainfall patterns. The city receives an average annual rainfall of approximately 931 mm,

distributed over 60 rainy days. Historical daily precipitation data from the India Meteorological Department (IMD) for the period 1951–2015 was used to develop the stationary IDF curves. Climate projections were sourced from five Global Circulation Models (GCMs) under the CMIP6 framework, using the SSP585 scenario, which assumes high greenhouse gas emissions until the end of the 21st century.

3.2 IDF curve development

Two sets of IDF curves were developed for this study: stationary and non-stationary. The stationary IDF curves were based on historical rainfall data and represent the traditional design approach. Non-stationary IDF curves were developed using rainfall projections from GCMs for the 2071–2100 period. These projections account for the expected increase in rainfall intensity due to climate change. Daily precipitation at 0.25° can be obtained from the India Meteorological Department (IMD) for the Indian region. Pai et al. (2014) developed gridded daily precipitation for India using station observations from more than 6000 stations located across India. The precipitation captures critical features of the Indian summer monsoon, including higher rainfall in the Western Ghats and northeastern India and lower rainfall in the semi-arid and arid regions of western India. Besides, gridded precipitation captures the orographic rain in the Western Ghats and foothills of Himalaya. The gridded precipitation data from IMD has been used for various hydroclimatic applications. Use gridded observations for bias correction as station data are not available. Outputs (x_m) from 5 CMIP6-GCMs can be used, which are available at different resolutions. Observations for the variable at the resolution of 0.25-degree are obtained from the IMD, for Indian Region.

Precipitations from CMIP6-GCMs are available at different spatial resolutions (Mishra et al. 2020b). For instance, the spatial resolution of the CMIP6 projection varies from 0.7° (EC-Earth3) to more than 2° (CanESM5). Mapped transformation to bias corrects the outputs for the historical period can be done and the SSP585 scenarios for the 100 years period for the precipitation scenarios can be used under r1i1p1f1 initial condition at daily time scale. The scenarios used in the CMIP6 combine Shared Socioeconomic Pathways (SSP) and target radiative forcing levels at the end of the 21st century. Regrid the precipitation data from CMIP6 to 1° spatial resolution to make them consistent. However, the effect of regridding using bilinear interpolation was checked by comparing the gridded datasets against the raw data for all-India mean of precipitation. We did not find any considerable differences in the all-India averaged precipitation and temperature using regridded and raw output from the GCMs.

The average annual rainfall depth for Bengaluru city is 931 mm, spread across 60 rainy days in a year (Ramachandra & Mujumdar, 2009). The stationary and non-stationary IDF

curve for Bengaluru for a return period of 10, 30, 50 and 100 years is shown in Figure 1 and 2, developed utilizing the hourly data from IMD's daily gridded (Pai et al., 2014) data at a spatial resolution of for a period of 100 years. The data is disaggregated for durations less than 1 h and various return period by using Equation 1, which is the most common rainfall disaggregation model of IDF relationship applicable to most of the geographical locations.

$$I(mm/hr) = A * (t + t_0)^B \quad (1)$$

where I is the intensity of rainfall, A , B and t_0 represent the coefficients for each return period (T) in years, t is the duration of precipitation in hours. Tables 1 and 2 show the coefficient of precipitation for stationary and non-stationary IDF.

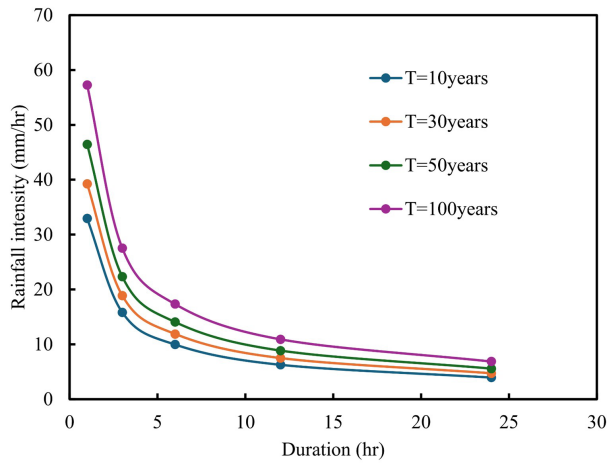


Figure 1. Stationary IDF for Bengaluru.

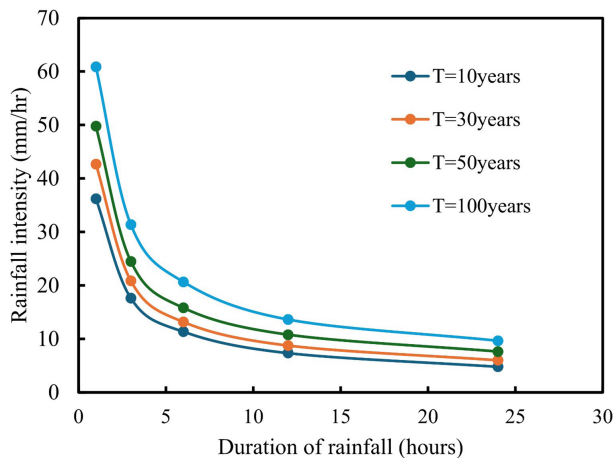


Figure 2. Nonstationary IDF for Bengaluru.

3.3 Embankment model and geocomposite layer simulation

The embankment model was developed using a finite difference method (FEM) simulation to evaluate pore-water pressure distribution and FOS under varying rainfall intensities. Two scenarios were considered:

- An embankment without a geocomposite layer.
- An embankment with a geocomposite layer consisting of a geotextile and a geonet.

The hydraulic properties of the geocomposite were based on experimental data from Gofar & Min Lee (2008), which showed high drainage efficiency and moisture retention capacity. The embankment was subjected to a continuous 24-hour rainfall simulation for each return period.

To represent the variation of suction and effective saturation in the embankment upon rainfall, Van-Genuchten (1980) model was used to present a relation between volumetric water content θ and suction s as given in Equation 2.

$$\theta = \frac{1}{\left[1 + \left(\frac{s}{\alpha} \right)^{n_e} \right]^m} \quad (2)$$

where inverse of α corresponds to the air entry value, $n_e = 1 - 1/m$ and m are the fitting parameters. In terms of non-dimensional form, θ can be expressed as:

$$\theta = \frac{\theta - \theta_r}{\theta_s - \theta_r} \quad (3)$$

where θ represents the volumetric water content at a given suction, maximum and residual water contents are represented

Table 1. Coefficients for stationary IDF.

T (years)	Coefficient A	Coefficient B	Coefficient t_0
10	35.8	-0.795	0.1034
30	46.7	-0.813	0.112
50	58.0	-0.857	0.124
100	63.8	-0.882	0.128

Table 2. Coefficients for Non- stationary IDF.

T (years)	Coefficient A	Coefficient B	Coefficient t_0
10	36.1	-0.745	0.1124
30	47.2	-0.803	0.120
50	58.4	-0.827	0.131
100	64.5	-0.842	0.136

by θ_s and θ_r , respectively. The parameter θ_s corresponds to zero suction condition obtained by extrapolating the SWCC, θ_r represents the residual suction condition and lies below the air-entry value and α represents the inverse of air-entry value.

A range of parameters, detailed in Table 3, was employed to determine the Soil Water Characteristic Curve (SWCC) for both the embankment soil and the geocomposite layer. Although the Barcelona Basic Model (BBM) parameters for the embankment soil were adopted to represent Kaolin, due to limited available data, the SWCC and hydraulic conductivity inputs were sourced from experimental findings on Kaolin soil presented by Gofar & Min Lee (2008) as shown in Figure 3 and 4. The geocomposite layer, consisting of a geonet core flanked by two geotextile layers, was characterized using SWCC and hydraulic conductivity curves, as illustrated in Figures 3 and 4 (Bahador et al., 2013). The geocomposite exhibited a steeper SWCC and higher air entry values (AEV) for both geotextile and geonet components relative to soil, which induces increased suction below the interface and reduced suction above it due to water accumulation (Bahador, 2012). These hydraulic properties enable the geocomposite to function effectively as both a drainage medium and a capillary barrier (Zornberg & Mitchell, 1994). In FLAC modelling, the geocomposite's drainage function was simulated by assigning zero pore pressure at specified nodes, while its mechanical behaviour was represented using a linear elastic model with an elastic modulus of 1.18×10^3 MPa and Poisson's ratio of 0.1 (Bahador et al., 2013). The BBM framework also incorporates shear modulus for geocomposite elasticity, with sensitivity analyses indicating that variations in modulus influence the resulting displacement fields.

In this study, a nonwoven geocomposite layer was incorporated to function as a drainage layer within an embankment undergoing infiltration, as illustrated in Figure 5.

4. Results

4.1 Pore-water pressure distribution using stationary IDF curves

In the case of embankments constructed without the inclusion of geocomposite drainage layers, simulations

conducted under stationary Intensity-Duration-Frequency (IDF) conditions representative of historical precipitation data without accounting for future climatic variability revealed a consistent and progressive accumulation of pore-water pressure throughout the embankment body across all analysed return periods: 10, 30, 50, and 100 years. Under relatively lower return periods (10-year and 30-year events),

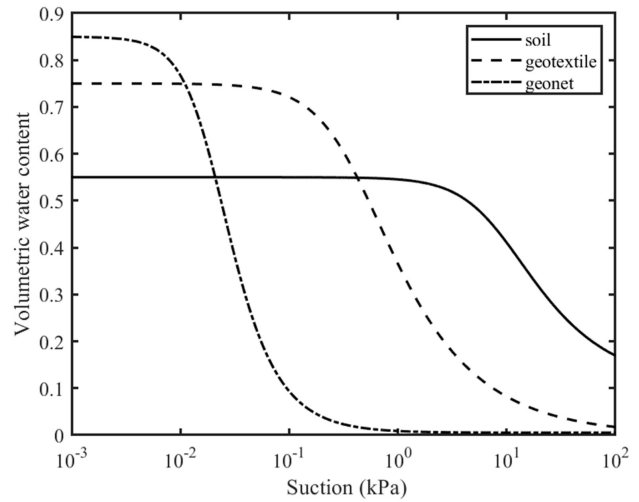


Figure 3. SWCC of soil and geocomposite layer.

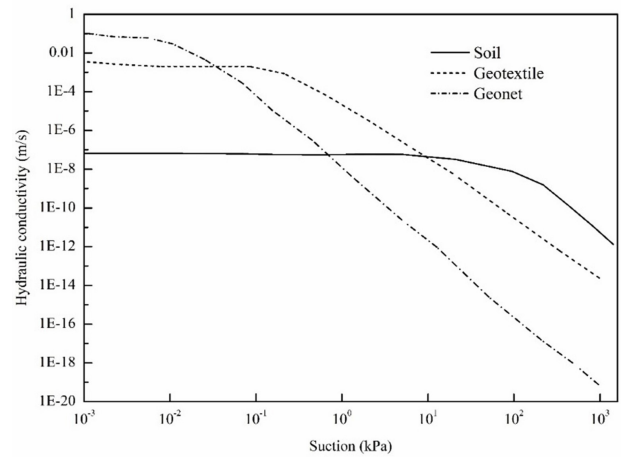


Figure 4. Hydraulic conductivity curves.

Table 3. Hydraulic properties of soil and geocomposite.

Materials	θ_s	θ_r	α (1/kPa)	n_e	k_{sat} (m/s)
Embankment soil (Gofar & Min Lee, 2008)	0.6	0.1	0.014	1.33	6.8×10^{-8}
Geotextile layer (Stormont et al., 2001)	0.75	0	2.577	1.68	2.89×10^{-3}
Geonet (Ramos, 2001)	0.85	0.005	50.251	2.19	1×10^{-1}

the increase in pore-water pressure was moderate. Saturation predominantly occurred within the upper soil horizons due to limited rainfall intensity and duration, as evidenced by the pore-pressure contour distributions in Figure 6a-d. During these lower-magnitude events, the embankment structure exhibited sufficient resilience, with the Factor of Safety (*FOS*) remaining above critical thresholds. Specifically, stability was maintained up to 18 hours in the 10-year return period

scenario and up to approximately 12 hours in the 30-year case before signs of mechanical compromise emerged.

However, under higher-magnitude return periods (50-year and 100-year events), the embankment experienced significantly more adverse hydraulic conditions. The simulated pore-water pressure rose rapidly and penetrated into deeper soil layers, driven by prolonged rainfall infiltration. In the 100-year return period scenario, by the 18-hour mark, nearly the entire

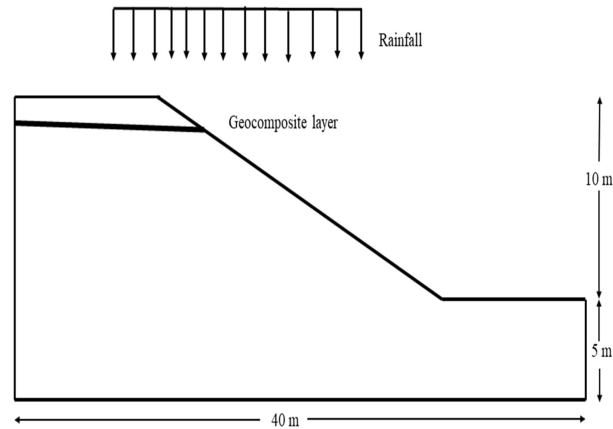


Figure 5. Geometry of embankment with geocomposite layer.

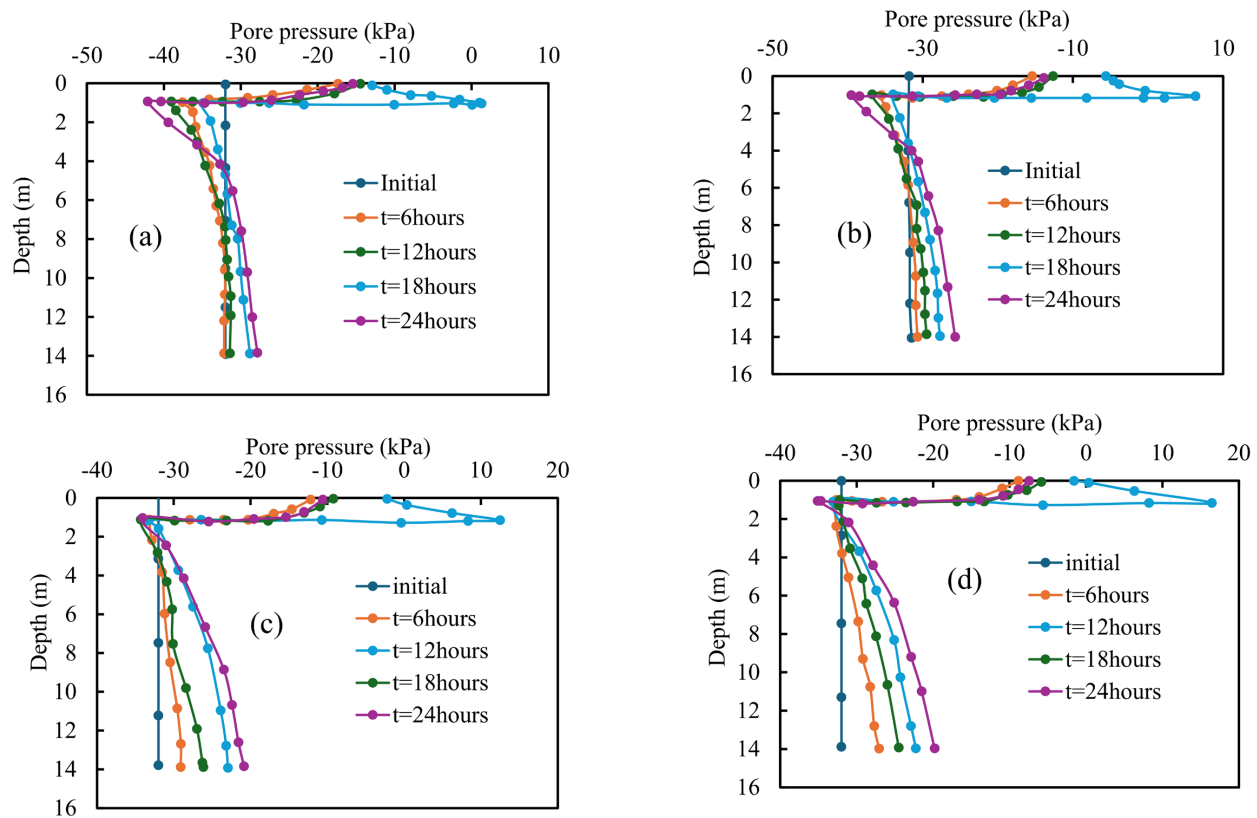


Figure 6. Pore pressure distribution in embankment with geocomposite with: (a) 10 years return period, (b) with 30 years return period, (c) 50 years return period, and (d) 100 years return period.

embankment cross-section exhibited high saturation levels. This hydraulic loading condition induced a substantial loss in matric suction, leading to a marked decline in the effective stress state of the soil matrix. As a result, the *FOS* deteriorated sharply, approaching failure thresholds ($FOS \approx 1.0$) within 24 hours, thus indicating a high probability of structural failure under such extreme yet increasingly probable rainfall events.

In contrast, embankments that incorporated geocomposite layers demonstrated a markedly different hydraulic and mechanical response under identical stationary IDF scenarios. The geocomposite system, composed of a geonet core enclosed between two geotextile sheets, initially acted as a capillary barrier. Due to its high air entry value (*AEV*) and steep Soil Water Characteristic Curve (SWCC), it effectively delayed the downward migration of moisture during the early stages of rainfall infiltration. This barrier effect mitigated the rate of pore-water pressure build-up, particularly within the lower embankment layers, during the critical early hours of the rainfall event. This control was evident across all return periods, with the geocomposite substantially reducing moisture ingress and suction loss during the first 12 hours.

As rainfall persisted, the geocomposite's drainage function became dominant. The permeable structure facilitated lateral and vertical redistribution of water, thus promoting dissipation of excess pore pressure and preserving unsaturated conditions within a significant portion of the soil profile. This dual behaviour capillary hindrance followed by enhanced drainage proved highly effective in maintaining mechanical stability. Notably, even under the 100-year return period scenario, the embankment with geocomposite layers maintained a *FOS* greater than 1.5 after 24 hours of continuous rainfall. This indicates that the presence of the geocomposite significantly extends the critical response time and enhances the overall hydraulic performance and mechanical resilience of the embankment structure under extreme precipitation loading conditions.

4.2 Pore-water pressure distribution using non-stationary IDF curves

Under non-stationary Intensity-Duration-Frequency (IDF) scenarios—derived from future climate projections incorporating anthropogenic warming effects and increased atmospheric moisture content the embankment response exhibited markedly accelerated hydraulic loading, particularly at higher return periods (50- and 100-year events). These curves account for the non-linearity and temporal evolution of extreme rainfall intensity due to climate change, resulting in steeper rainfall hyetographs and higher cumulative precipitation volumes over shorter durations. In embankments constructed without geocomposite reinforcement, this translated into a significantly more aggressive rate of pore-water pressure development. Saturation thresholds were reached substantially earlier compared to stationary cases: within approximately 12 hours for the 50-year return period and as early as 8 hours

for the 100-year scenario, as evidenced in Figure 6a-d. The rapid infiltration under high rainfall intensities induced immediate loss of matric suction and led to a pronounced hydraulic gradient, causing moisture fronts to advance swiftly into the lower embankment strata.

This expedited hydraulic loading reduced effective stress and initiated a sharp decline in the Factor of Safety (*FOS*). For the 100-year non-stationary event, the embankment approached incipient failure conditions within 18 hours of rainfall onset, demonstrating a critical reduction in mechanical stability due to the elevated saturation profile and diminished shear strength of the soil matrix. The vertical propagation of pore pressure fronts under non-stationary rainfall exceeded the rates observed in stationary conditions by over 50%, confirming the inadequacy of legacy IDF-based designs under evolving climate regimes.

Conversely, embankments incorporating geocomposite layers exhibited an improved but not invulnerable response to non-stationary forcing. The geocomposite system, through its capillary barrier effect and enhanced in-plane transmissivity, effectively moderated the upward migration of pore pressure, and facilitated lateral water dispersion. During the initial phase of rainfall exposure (first 12–15 hours), the geocomposite delayed the saturation of deeper layers and maintained lower suction gradients across the soil profile. However, the elevated rainfall intensities characteristic of non-stationary IDF curves still resulted in accelerated hydraulic loading compared to the stationary case. For the 100-year return period, while the presence of the geocomposite ensured that *FOS* remained above the stability threshold of 1.0 up to approximately 18 hours, the system approached hydraulic saturation by the 24-hour mark.

This temporal extension of structural integrity illustrates the functional resilience of the geocomposite under severe climatic stressors, though it also underscores the limitations of passive drainage systems under sustained extreme hydrological inputs. Ultimately, the geocomposite layer served as a critical mitigation component, but its effectiveness diminishes under long-duration, high-magnitude precipitation events anticipated by non-stationary projections, necessitating further optimization of embankment designs for future climate resilience.

4.3 Effects of stationary and non-stationary IDF curves

The analysis was extended to consider the effects of using stationary versus non-stationary IDF curves for modelling rainfall intensities. Stationary IDF curves, which assume historical rainfall patterns remain constant over time, provide a simplified approach. However, as climate change impacts intensify, this assumption can result in an underestimation of future rainfall intensity and frequency, potentially leading to unsafe embankment designs.

When stationary IDF curves were applied, pore-water pressure distributions showed slower but steady increases with rainfall duration across all return periods. For instance,

after 6 hours of rainfall under a 50-year return period, the pore-water pressure remained moderate, and the *FOS* did not drop significantly. However, as rainfall persisted beyond 18 hours, particularly under the 100-year return period, the embankment reached critical saturation levels quicker, indicating a potential failure if the geocomposite layer were absent.

In contrast, using non-stationary IDF curves which account for the projected increase in rainfall intensity due to climate change produced more aggressive pore-water pressure rises, especially in the 50- and 100-year scenarios. Non-stationary IDF curves reflect the dynamic and changing nature of rainfall, particularly in regions like Bengaluru, where future climate projections indicate higher rainfall intensities. As a result, embankments experienced more rapid saturation and higher pore pressures under these conditions. However, the inclusion of the geocomposite layer significantly mitigated these impacts by accelerating drainage and preventing dangerous levels of saturation, even as rainfall intensities increased.

The most dramatic difference between stationary and non-stationary IDF applications was observed during the higher return period events (100 years). Under non-stationary conditions, without geocomposites, the embankment would experience critical pore pressures and fail earlier, as the system would not account for future increases in intensity. The geocomposite proved essential in prolonging the time to failure by reducing saturation rates and maintaining a higher *FOS*, even in extreme rainfall events.

4.4 Factor of safety (*FOS*) comparison across return periods and IDF types

The *FOS* analysis demonstrated notable differences in embankment stability under stationary versus non-stationary

IDF curves, especially in embankments without geocomposite layers as shown in Figure 7a. Under stationary IDF conditions, the *FOS* remained stable across all return periods up to 12 hours but dropped to near failure levels in the 50- and 100-year return periods by the 18-hour mark. In contrast, non-stationary IDF conditions caused a steep decline in *FOS* within the first 12 hours, with failure thresholds reached much sooner for higher return periods, particularly in the absence of geocomposites as shown in Figure 7b.

For embankments reinforced with geocomposite layers, the *FOS* values remained consistently higher across all return periods in both stationary and non-stationary scenarios. The geocomposite layer significantly prolonged stability, keeping *FOS* values above 1.5 for most of the 10- and 30-year return periods and extending stability to 18 hours in the 100-year scenario under non-stationary IDF curves. This suggests that geocomposites are crucial in maintaining embankment stability under extreme future rainfall scenarios.

The *FOS* behaviour across different return periods (10, 30, 50, and 100 years) further highlighted the role of geocomposite layers. For lower return periods (e.g., 10 years), the *FOS* remained high, reflecting the embankment's resilience to short-duration, moderate-intensity rainfall. The geocomposite layer delayed saturation and enabled drainage before critical shear strength reductions occurred. For higher return periods (50 and 100 years), the *FOS* decreased more significantly, particularly under non-stationary IDF curves, where the rainfall intensity was higher than historical norms. However, the geocomposite layer consistently maintained a safer *FOS* by enabling excess water to be efficiently drained and preventing the rapid buildup of pore pressure.

Without the geocomposite layer, embankments subjected to non-stationary IDF curve scenarios would experience a catastrophic reduction in *FOS* during long return periods, often approaching failure thresholds ($FOS = 1$). This finding

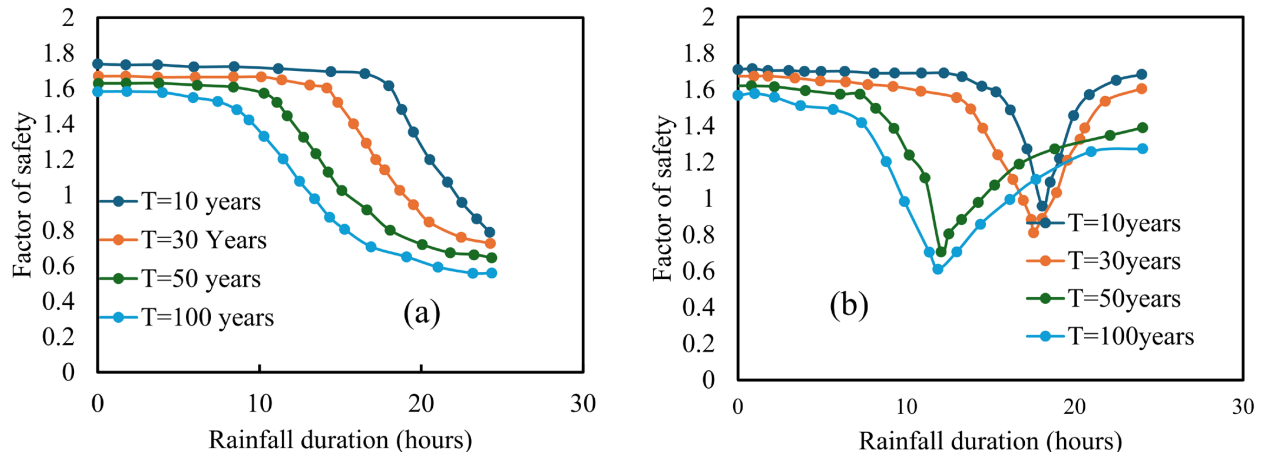


Figure 7. Variation of *FOS* of embankment: (a) at various return periods, and (b) with geocomposite at various return periods.

underscores the importance of factoring in non-stationary IDF's when designing embankments to withstand future climate variability.

4.5 Flux rate calculations

The computed flux rates at the crest of the embankment, as illustrated in Figure 8, reveal a dynamic evolution of infiltration behaviour under transient hydrological loading. Initially, the flux rate exhibited a gradual decline over time, characteristic of decreasing hydraulic gradients as the surface layers approached saturation. This behaviour is consistent with classic infiltration theory, wherein matric suction decreases near the surface with prolonged wetting, reducing the capillary-driven flow component.

A pronounced inflection point in the flux rate curve was observed when the advancing wetting front intersected the geotextile component of the geocomposite system. At this interface, a sudden reduction in vertical flux was recorded, indicating the onset of capillary barrier effects induced by the contrast in pore-size distribution between the embankment soil and the geotextile. This phenomenon is analogous to the mechanism described by Miller & Gardner (1962) in their seminal study on vertical flow through layered soil columns, where a fine-over-coarse stratification imposes a temporary impedance to flow due to higher air entry values in the underlying coarser medium.

Post-intersection, the flux rate stabilized at a lower magnitude, indicative of a redistribution regime dominated by lateral drainage along the plane of the geonet core. The geocomposite's internal transmissivity allowed for rapid in-plane flow, effectively diverting infiltrating water away from the vertical percolation path and mitigating pore pressure accumulation in the underlying soil matrix. This behaviour underscores the drainage capacity of geocomposite systems: by combining high in-plane hydraulic conductivity with capillary tension modulation, geocomposites serve

both to delay vertical saturation and to provide preferential drainage pathways.

Quantitatively, the observed drop in flux magnitude corresponds to the initiation of water storage above the geotextile until the matric potential of the overlying soil exceeds the air-entry pressure of the geotextile. Once this threshold is overcome, vertical flow resumes, albeit at a reduced rate due to redistribution through the geonet. The interplay between capillary forces and hydraulic conductivity within the geocomposite is critical in controlling the timing and magnitude of subsurface saturation, thereby directly influencing the overall hydraulic response and stability of the embankment system under intense rainfall events.

5. Discussion

5.1 Impact of stationary vs. non-stationary IDF curves on embankment stability

The results underscore the limitations of using stationary IDF curves, which rely solely on historical rainfall data. While stationary IDF's can provide reasonable estimates for current rainfall patterns, they do not account for the projected intensification of rainfall events due to climate change. Consequently, designs based on stationary IDF's may underestimate future rainfall intensities, especially for higher return periods (50 and 100 years). This underestimation increases the risk of embankment failure, as observed in the steep declines in *FOS* values under simulated non-stationary IDF conditions. In contrast, non-stationary IDF curves provide a more realistic estimate of future rainfall intensities, especially in climate-sensitive regions like Bengaluru, where rainfall patterns are projected to intensify over the coming decades.

In embankments without geocomposite layers, the non-stationary IDF conditions led to rapid soil saturation and a critical drop in *FOS* within hours. This suggests that traditional embankment designs, relying on stationary IDF's, may not be sufficient for future climate scenarios. The findings align with recent research emphasizing the need to incorporate non-stationary IDF's in infrastructure planning, especially for long-term resilience (IPCC, 2021).

5.2 Role of geocomposite layers in enhancing embankment stability

The incorporation of geocomposite layers significantly improves embankment stability, as demonstrated by the higher *FOS* values across all scenarios. Geocomposite layers provide dual functionality as capillary barriers and drainage facilitators. Initially, they act as moisture barriers, preventing the rapid infiltration of water into deeper soil layers. As rainfall continues, they transition to a drainage function, allowing excess moisture to be directed away from the embankment, thereby reducing pore pressure, and enhancing stability.

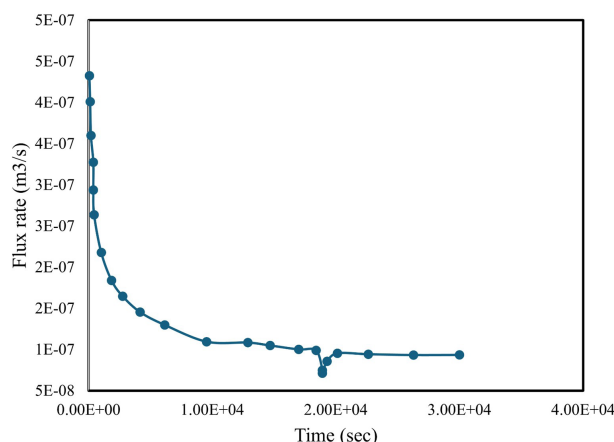


Figure 8. Flux rate with time.

The results from this study show that embankments with geocomposite layers maintain higher *FOS* values, even under the extreme conditions simulated by non-stationary IDF curves. This supports previous findings by Bahador et al. (2013) and Zornberg and Mitchell (1994), which highlighted the effectiveness of geocomposites in enhancing stability in saturated soils. Specifically, the presence of geocomposites delayed critical saturation by up to 6 hours in the 50- and 100-year return periods, indicating that geocomposites provide critical time margins for embankment resilience during prolonged rainfall.

5.3 Implications for infrastructure design under changing climate conditions

The findings suggest that infrastructure design should prioritize the integration of both non-stationary IDF curves and geocomposite layers to accommodate future climate scenarios effectively. Non-stationary IDF curves provide a realistic assessment of future rainfall intensities, which are essential for the accurate estimation of pore pressure and *FOS*. By using non-stationary IDFs, engineers can design embankments that are resilient to future climate variability, thereby reducing the likelihood of failure.

Geocomposite layers emerge as a highly effective means of mitigating the effects of increased rainfall intensity on embankment stability. Their ability to control pore-water pressure distribution and delay saturation under extreme rainfall makes them particularly valuable in regions prone to heavy monsoons and climate-induced rainfall intensification. This study's results align with broader research indicating that geosynthetics, when used in tandem with climate-adaptive IDF models, enhance the resilience of critical infrastructure (Her et al., 2018; Oguz et al., 2024).

6. Conclusions

This study demonstrates the critical need for climate-adaptive strategies in embankment design, emphasizing the limitations of traditional, stationary Intensity-Duration-Frequency (IDF) curves under evolving climate conditions. The findings show that embankments without geocomposite layers exhibit rapid pore-water pressure increases and significant reductions in the Factor of Safety (*FOS*) under non-stationary IDF conditions, with failure thresholds reached much faster in high return periods, such as the 50- and 100-year events. Conversely, the integration of geocomposite layers within embankments proves highly effective in moderating pore pressures, delaying critical saturation, and sustaining higher *FOS* values across all return periods. Specifically, under the 100-year return period with non-stationary conditions, embankments reinforced with geocomposites maintain stability up to 18 hours longer than their unreinforced counterparts. These results underscore two key recommendations for future embankment design:

- Incorporate Non-Stationary IDF Curves: Accounting for projected climate-induced changes in rainfall patterns provides a more accurate basis for infrastructure planning, mitigating under-design risks associated with outdated stationary models;
- Utilize Geocomposite Reinforcements: Geocomposite layers enhance resilience by acting as both drainage facilitators and capillary barriers, significantly extending the time to saturation and improving overall stability during extreme rainfall events.

Declaration of interest

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.

Authors' contributions

Rakshanda Showkat: software, formal analysis, data curation, investigation, methodology, visualization, validation, writing – original draft, conceptualization. G L Sivakumar Babu: writing – review & editing, project supervision.

Data availability

The datasets generated analyzed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

k_{sat}	Saturated permeability
m	fitting parameter
n_e	fitting parameter
s	Matric suction
t_o	coefficient for each return period
t	Duration of precipitation in hours
x_m	Outputs
A	coefficient for each return period
B	coefficient for each return period
AEV	air entry value
BBM	Barcelona Basic Model
<i>FOS</i>	Factor of safety

GCM	General Circulation Model
I	Intensity of rainfall
IDF	Intensity-Duration-Frequency
SWCC	Soil Water Characteristic Curve
α	Air entry value
θ	Volumetric water content
θ_r	Residual water content
θ_s	Maximum water content

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ARTICLES

Soils and Rocks
v. 48, n. 2

Relationship between Point Load Strength Index and Slake Durability Index of sandstones from the Amile Formation, Tansen Group, West-Central Nepal

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Article

Keywords

Slake Durability Index
Point Load Strength Index
Quartzose sandstones
Amile Formation
Correlation

Abstract

Point load strength index (*PLSI*) and slake durability index (*SDI*) are two important indices of rocks used to evaluate strength and durability for various engineering applications. This research investigates these two indices in sandstones from the Amile formation coupled with petrography, aiming to determine the strength and durability of rock, and test for correlations among mineral constituents, texture, *PLSI*, and *SDI* to develop empirical relationships among them. The study provides insights into the geotechnical properties of an important geological formation in the Lesser Himalayan Zone, contributing valuable data for infrastructure development. The methodology involved field sampling, petrographic analysis of thin sections, *SDI* testing over two cycles, and *PLSI* testing on irregular lump samples. Correlations between *SDI*, *PLSI*, water absorption, packing density, and packing proximity were statistically analyzed. Quartzose sandstones have extremely high durability (*SDI*: 99.17–99.78%), very strong to extremely strong compressive strength (*PLSI*: 5.94–14.81 MPa), and low water absorption values (*WAV*: 0–2.17%). Very high positive correlations were found between *SDI* and *PLSI* with $R^2 = 0.804$. Based on this correlation, future research can use the equation $SDI = 0.0712PLSI + 98.727$ to estimate durability from strength for similar sandstones, saving time and resources.

1. Introduction

Rock strength and durability evaluation is critical for assessing geological materials in engineering applications (Hatheway, 2009). Index tests like the Point Load Strength Index (*PLSI*) and Slake Durability Index (*SDI*) offer efficient methods for characterizing rock mechanical properties (Bieniawski, 1975; ISRM, 1985). These tests are valuable for determining construction material quality and predicting rock mass behavior under various conditions (Gokceoglu et al., 2000; Tamrakar et al., 2007).

PLSI provides a simple estimate of rock strength, correlating with uniaxial compressive strength (*UCS*) while requiring less sample preparation (Broch & Franklin, 1972). *SDI* evaluates rock resistance to weathering through repeated wetting and drying cycles (Franklin & Chandra, 1972; Gamble, 1971).

Previous studies on sandstones in Nepal have reported varying *PLSI* and *SDI* values. For instance, (Sharma & Tamrakar, 2023) found *PLSI* values ranging from 2.40 to 11.4 MPa for the Fagfog Quartzite, while (Budhathoki & Tamrakar, 2022) reported *PLSI* and *SDI* values ranging

from 0.024 to 2.16 MPa and 93.23% to 30.37% respectively for sandstones in the Siwalik Group. Sandstones from the Sneinton Formation in England are moderately strong and deformable and durability correlates with quartz content and clay fraction (Bell & Culshaw, 1998). Sabatakakis et al., (2008) reported *PLSI* values for sandstones ranging from 0.5 to 5.7 MPa. Tsiambaos & Sabatakakis (2004) found that the *PLSI* of sandstones varied between 0.5 and 8.4 MPa, correlating strongly with *UCS*. Dhakal et al. (2002) reported *SDI* values for sandstones ranging from 90% to 99% after two cycles, emphasizing the role of mineralogy in durability characteristics. Celik & Kacmaz (2016) observed *SDI* values for various building stones, including sandstones, ranging from 94.8% to 99.6%, demonstrating their high resistance to weathering. Shakoor & Bonelli (1991) found *SDI* values for sandstones between 86% and 100%, correlating with petrographic characteristics and other engineering properties.

The Water Absorption Test, measuring rocks' water absorption, provides insights into porosity, permeability, and degradation susceptibility (ASTM, 2019; Benavente et al.,

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Submitted on August 24, 2024; Final Acceptance on January 13, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.006924>



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2015). Sandstone water absorption values typically range from less than 1% to over 20% (Bell, 2007; Celik & Kacmaz, 2016).

Multivariate correlation is a statistical technique used to analyze relationships among variables by considering three or more variables at once. In the context of rock mechanics, multivariate analysis can reveal how different petrographic, physical, and mechanical properties interrelate, providing insights that may not be apparent from simple bivariate correlations (Shakoor & Bonelli, 1991). For instance, studies by Bell & Lindsay (1999) and Tamrakar et al. (2007) have used multivariate techniques to investigate the influence of petrographic properties. In this research, multivariate correlation is used to explore how petrographic properties along with water absorption relate with *PLSI* and *SDI* of sandstone samples.

The Amile Formation, part of the Tansen Group in west-central Nepal, is distributed well in the hillslopes of the Siddhartha Highway (Figure 1). Understanding the strength and durability characteristics of sandstone is crucial for assessing their suitability as construction materials and comprehending geological factors influencing slope stability. This research aims to determine the *PLSI*, *SDI*, packing density, packing proximity, and water absorption values of sandstones from the Amile Formation, and explore correlations between them.

2. Geological setting of the study area

The Tansen Area is located within the Lesser Himalayan Zone of west-central Nepal. This zone lies between the Main Boundary Thrust (MBT) to the south and the Main Central Thrust (MCT) to the north. The Lesser Himalayan rocks in this region consist of low-grade metasedimentary sequences (Sakai, 1983). The Precambrian rocks in the Tansen Area belong to the Kali Gandaki Supergroup. The younger sedimentary sequence is represented by the Tansen Group, comprising the Dumri Formation, Bhainskate Formation, Amile Formation, Taltung Formation, and the Sisne Formation (Figure 2). The Lesser Himalayan rocks in the Tansen Area exhibit typical structural features like thrusts, folds, and the emplacement of the Lesser Himalayan nappes.

A huge syncline exists in the study area, where the younger sequence of the Tansen Group of rocks is overlain by the older Heklang Formation and the Virkot Quartzite of the Kali Gandaki Supergroup with tectonic contacts. The formations of the Kali Gandaki Supergroup were brought as tectonic klippe. The study area particularly lies in the northern limb of the syncline.

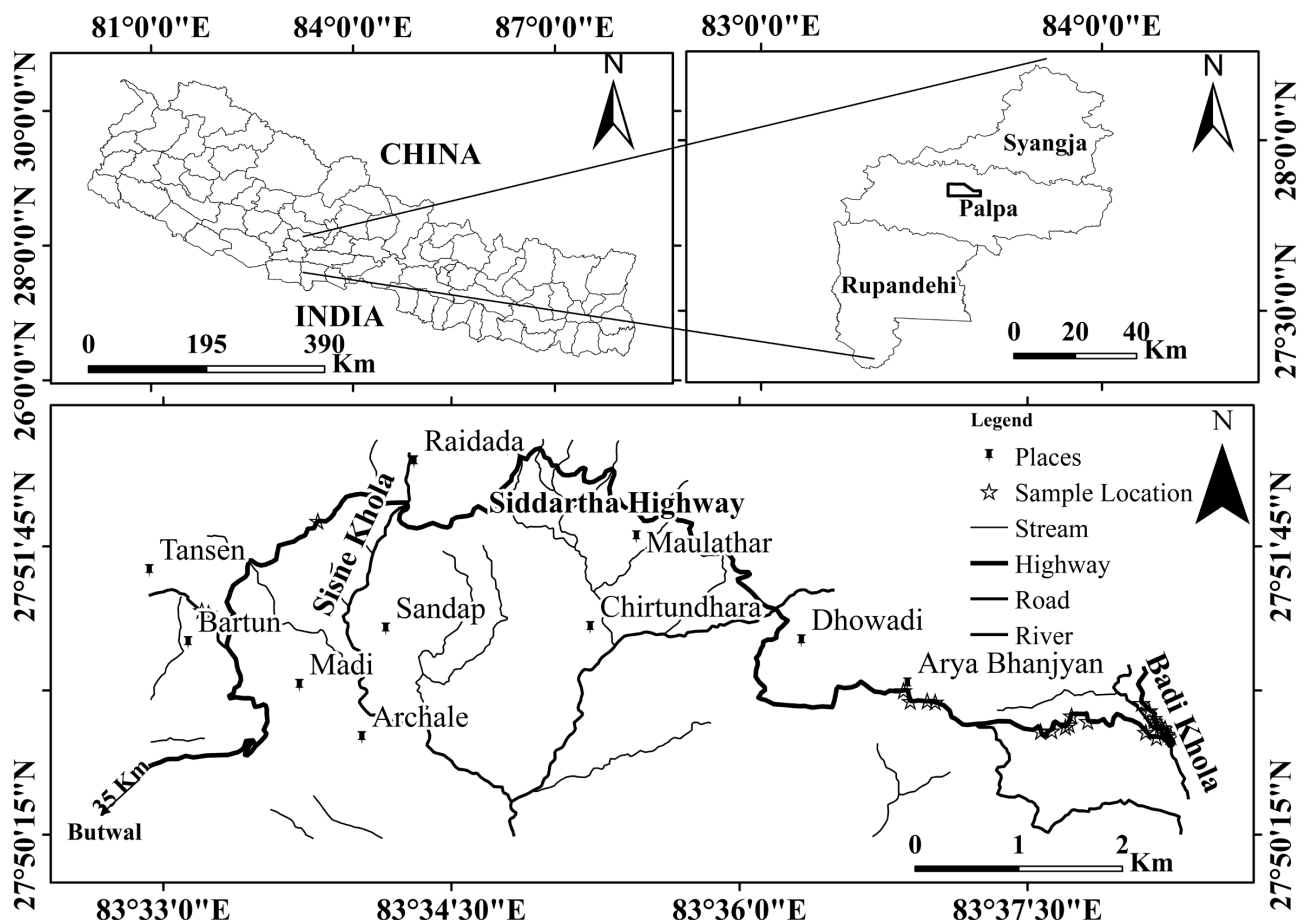


Figure 1. Location map showing study area.

3. Methodology

3.1 Sampling

Twenty-five sampling locations (Figure 2) were selected across the study area. From each location, ten fresh sandstone fragments (40-60g each) were collected for *SDI* testing, and three orthorhombic samples were taken for *PLSI* determination.

3.2 Microscopic examination of sandstone

Thin sections were examined under a polarizing light microscope to analyze rock composition, texture, packing density, packing proximity, and grain characteristics (size, sorting, roundness, and interlocking fabrics). Packing density (*Pd*) and Packing proximity (*Pp*) were measured using the expressions of Kahn (1956):

$$Pd = \{(11 + 12 + 13 + \dots + ln) / L\} \cdot 100(\%) \quad (1)$$

$$Pp = \{(gl + g2 + g3 + \dots + gn / Nc)\} \cdot 100(\%) \quad (2)$$

where: *ln* = length of grains across the traverse line and *L* is the traverse length, and *Gn* = grain-to-grain contact and *Nc* is the total number of contacts.

3.3 Testing for Point Load Strength Index

A point Load Test of irregular lumps ranging in thickness of 25-30 mm was performed following ASTM D5731 02 (ASTM, 2003). The point load strength index $I_{s(50)}$ is calculated using:

$$I_{s(50)} = F \times I_s \quad (3)$$

where: $I_{s(50)}$ = corrected point load strength; *F* = Size correction factor; and I_s = Uncorrected Point load strength index, MPa.

$I_{s(50)}$ was used to calculate the *UCS* of the test sample using an empirical relation (Vallejo et al., 1989).

$$UCS = 17.4 \times I_{s(50)} \quad (4)$$

The value obtained for the *UCS* of the test sample was employed to classify the rock with the reference of ISRM (1981) (Table 1).

3.4 Testing for SDI following ASTM D4644-87

To conduct the test, the ASTM procedure D4644 standard procedure is used. For determining the sandstone's durability, the standard cycle i.e., the second cycle (I_{d2}) of the Slake Durability Index was compared to the classification proposed by Franklin & Chandra (1972) (Table 2). The I_{d2} is obtained from:

$$Id2 = WF / B \quad (5)$$

Table 1. Uniaxial compressive strength index classification (ISRM, 1981).

UCS (MPa)	Strength Classification
0.25 – 1	Extremely Weak
1 – 5	Very Weak
5 – 25	Weak
25 – 50	Medium Strong
50 – 100	Strong
100 – 250	Very Strong
>250	Extremely Strong

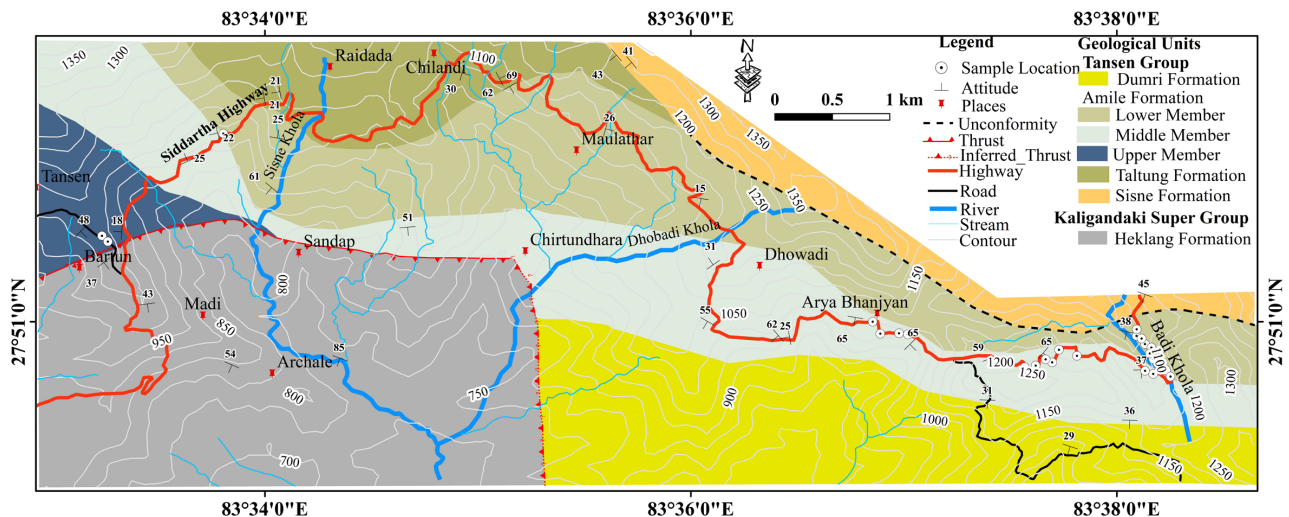


Figure 2. Geological map of study area.

where: I_{d2} = slake durability index (second cycle); WF = mass of oven-dried sample retained after the second cycle, gm; and B = mass of the oven-dried sample before the second cycle, gm.

3.5 Testing for Water Absorption test following ASTM C127

For conducting the test, standard procedure ASTM C127 (ASTM, 2011) was used. The water absorption is calculated using the formula:

$$WAV = ((B - A) / A) * 100 \quad (6)$$

where: WAV = Water Absorption Value; A = Mass of oven-dried sample; and B = Mass of saturated sample.

3.6 Linear regression correlation

Linear regression is a statistical method that analyzes relationships between continuous variables, fitting a linear equation to observed data for predictions and insights. This study uses linear regression to explore the correlation between strength, durability, and factors like water absorption, packing density, and packing proximity.

3.7 Multivariate correlation

Multivariate correlation analyzes relationships among multiple variables simultaneously extending beyond simple bivariate correlations. This study used multivariate correlation to examine relationships between SDI , $PLSI$, WAV , Pd , and Pp . The analysis, conducted in two phases with SDI and $PLSI$ as dependent variables, assessed how various properties collectively influence rock durability and strength, providing a comprehensive understanding of complex interactions among studied rock properties.

4 Results

4.1 Lithological description of Quartose Sandstone in sampling sites

The exposure consists of thick-bedded (40 cm) coarse-grained, moderately weathered, pebble-bearing sandstone with ferruginous molting having pebbles of diameter 3-4 cm interbedded with thick-bedded purple shale. Thin to medium bedded, coarse-grained grey quartose sandstone interbedded with medium bedded, moderately weathered muddy sandstone. Thick-bedded coarse-grained quartose sandstone with hematite and siderite speckles interbedded with reddish-brown siltstone. Medium bedded (10-20 cm) moderately weathered grey fresh siltstone with pisolites and

Table 2. Slake durability index classification (after Franklin & Chandra, 1972).

I_{d2} (%)	Durability Classification
0-25	Very Low
26-50	Low
51-75	Medium
76-90	High
91-95	Very High
96-100	Extremely High

thin-bedded fine-grained dark grey moderately weathered limestone. Thick-bedded coarse-grained white quartzose sandstone interbedded with medium to thick-bedded muddy sandstone, bioturbated muddy sandstone, and carbonaceous shale.

4.2 Description of rock thin sections under Polarizing Microscope

The composition and texture of each sample were determined through meticulous petrographic examination, to understand their potential influence on slaking behavior and rock strength. Thin sections were analyzed under cross-polarized light (XPL) conditions (Figure 3). Figure 3 consists of six different samples: LA1(a) exhibits a poorly sorted Quartz Wacke with the (94.69% quartz, 5.31% feldspar, 16% matrix); LA9(b), a moderately sorted Subarkose arenite with (94.83% quartz, 5.17% feldspar, 3% matrix); MA1(c), Subarkose arenite with lower quartz content and insignificant mica presence than LA9(b) (94.36% quartz, 1.88% rock fragments, 0.36% mica); MA7(d), a poorly sorted Lithic Wacke with a significantly lower matrix percentage than LA1(a) and the third highest percentage of rock fragments in the plate (87.04% quartz, 8.17% rock fragments, 15.37% matrix); UA1(e), a well-sorted Quartz arenite (95.5% quartz, 4.5% feldspar); UA2(f), Quartz arenite with the highest quartz content among the samples with (98.73% quartz, 1.27% feldspar). The detailed compositional result along with the microscopically observable texture of rocks, subsequent rock classification, Pd and Pp is tabulated below (Table 3 and Table 4).

4.3 Point-load strength index ($PLSI$)

The obtained strength index values are presented in a table, where samples from the Lower Middle and the Upper Members are indicated by LA, MA, and UA, respectively. Sample LA10 exhibited the highest strength of 14.81 MPa among all the samples tested (Table 5). In contrast, sample MA9 had the lowest durability with a strength value of 5.94 MPa. According to the calculated value of UCS (Vallejo et al., 1989), the test samples are classified as extremely strong to very strong rock based on Brown (1981), (Table 4).

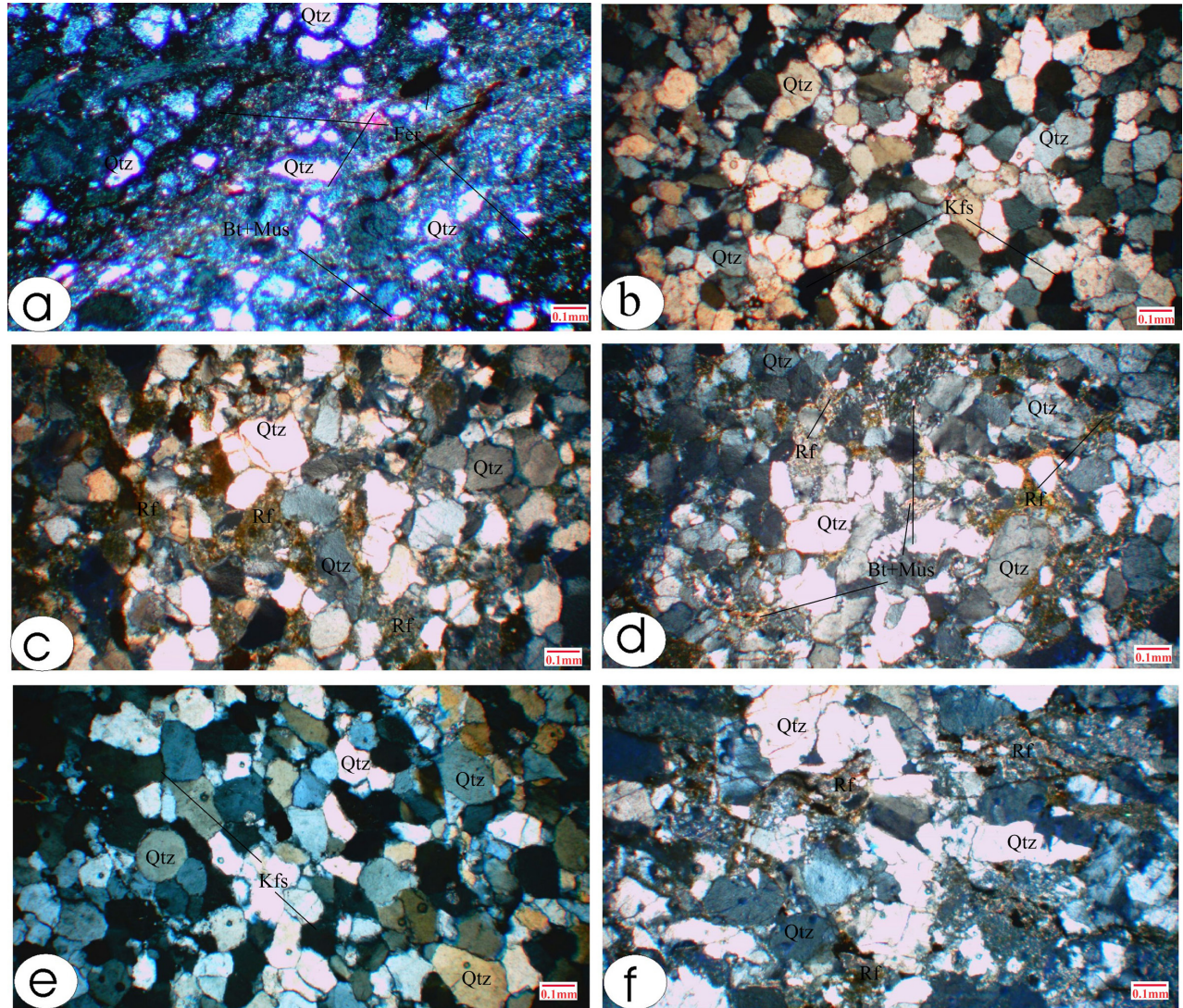


Figure 3. Photomicrograph of Sandstone from The Amile Formation under XPL sample LA1 (a) LA9 (b) MA1 (c) MA7 (d) UA1 (e) and UA2 (f).

4.4 Slake Durability Index (*SDI*)

As per the value of *SDI* of the second cycle, the value ranged from 99.78% (MA1) to 99.17% (LA2, MA5). The test sample lies on the type I disintegration pattern, ASTM D464487 (ASTM, 2001) Method. According to the classification proposed by Franklin & Chandra (1972), these values correspond to an extremely high durability rating (Table 6).

4.5 Water absorption test

The *WAV* ranges from 0 to 2.17% (MA9). According to the ASTM (2011) C 127 specification, the sample $> 3\%$ *WAV* indicates a high effective porosity whereas below 3% indicates a low effect of porosity which is seen in all samples (Table 7).

4.6 Linear regression correlation

The study examined the relationship between *SDI* and *PLSI* by plotting them on a scatter chart and calculating the best-fit line with a 95% confidence level and the relationship coefficient (R^2) (Figure 4). The analysis showed a strong correlation between *SDI* and *PLSI* across all samples, with a coefficient of determination ($R^2 = 0.804$), significantly higher than the critical R-value of 0.337, indicating statistical significance.

A good correlation was found between *PLSI* vs *WAV* ($R^2 = 0.79$), *PLSI* vs *Pd* ($R^2 = 0.88$), *PLSI* vs *Pp* ($R^2 = 0.83$), *SDI* vs *WAV* ($R^2 = 0.74$), *SDI* vs *Pd* ($R^2 = 0.84$) and *SDI* vs *Pp* ($R^2 = 0.78$). All these correlations were statistically significant, reinforcing the connection between these indices and the measured properties (Table 8)

Table 3. Result of the petrographic analysis of sandstone from the Amile Formation.

SN	Clast (%)					Matrix (%)	Cement (%)			Packing density (P_d %)	Packing Proximity (P_p %)	
	Quartz		F	RF	Mica		Others+ Heavies	Ferr.	Sil.			Arg.
	Q_m	Q_p										
LA1	44.57	8.88	3		2	2.59	16	10		12.96	97.7	92.2
LA2	67.7	10.12	1.18			1.00	3		11	6.00	87.2	82.4
LA3	63.92	5.08	5			1.00	3	6	12	4.00	97.0	93.2
LA4	68.7	4.96	1.49			1.85	2	6	9	6.00	95.2	90.3
LA5	73.64	2.32	1.09			1.41	3	8	6	4.54	94.4	89.9
LA6	75.88	2.12	2			1.00	2	11	6		94.5	88.3
LA7	46.36	9.36	3		2	1.86	16	9		12.42	93.7	88.7
LA8	73.22	4.63	1.39			1.4.	3	5	7	4.36	94.9	89.4
LA9	72.73	2.36	4.09			2.00	3	5	7	3.82	93.4	88.7
LA10	69.77	5.12	5.75			1.03		4.15	9	5.18	97.4	91.7
MA1	65.61	9.73	3	1.50	0.36	1.95	7.09	6.76	2	2.00	97.8	90.2
MA2	64.73	2.21	3.47	7.31	0.58	1.00	9.01	5.69	3	3.00	87.0	82.6
MA3	63.09	2.31	5.01	6.00		1.27	14	1.32	2	5.00	87.4	81.5
MA4	58.61	3.49	6.04	5.52		1.05	16	2.29	4	3.00	97.8	91.3
MA5	65.83	2.19	4.21	6.57		1.77	10	2.43	3	4.00	87.4	82.7
MA6	66.43	2.71	3.49	6.69		1.32	12	2.36	2	3.00	88.1	83.0
MA7	61.01	4.83	3.62	6.18	1	1.59	15.37	0.93	1	4.47	93.3	89.0
MA8	56.53	1	6.02	8.00	1	1.38	18	2.38	1	4.69	89.7	83.6
MA9	59.84	1.09	4.74	6.00	1	1.00	15	4.52	2	4.00	87.1	82.6
MA10	64.46	2.18	1.04	8.00	1	1.29	15	1.00	2	4.03	97.4	92.7
MA11	52.65	4.89	2	12.00		1.50	17	3.00	3	3.96	93.2	88.6
MA12	50.98	3.80	3	12.00		1.00	19	4.00	2	4.22	96.1	91.8
MA13	65.54	2.32	2.19	7.00		1.08	13	3.00	2	3.87	87.1	83.7
UA1	82.63		3.89			1.61	1	3.00	6	1.87	97.0	91.5
UA2	82.43	1	1.07			1.64	1	3.00	8	1.86	97.7	91.8

4.7 Multivariant correlation

To create a more accurate predictive model with a higher R^2 , we conducted multiple regression analyses for each property. Each analysis used one test as the dependent variable and the other four as independent variables. We identified significant independent variables based on their P-values, discarding those that were insignificant.

For the prediction of *PLSI*, our analysis revealed that *SDI*, *WAV*, and *Pd* were the most significant independent variables. While single regression analysis showed *Pd* as the best predictor for *PLSI* with an R^2 of 0.88, combining the three significant independent variables in a multiple regression model improved the correlation strength, resulting in an R^2 of 0.916. This R^2 value remained unchanged when only *WAV* and *Pd* were kept as independent variables (Table 9).

In the case of *SDI* prediction, *WAV* and *Pd* emerged as the most significant independent variables. The initial correlation between *SDI* and *Pd* yielded an R^2 of 0.845. To enhance the reliability of the prediction, we performed multiple regression, which slightly increased the R^2 to 0.869. Interestingly, this R^2 value remained unchanged when *PLSI* and *WAV* were included alongside *Pd* as independent variables.

5. Discussion

SDI values for sandstone from the Amile Formation ranged from 99.17% to 99.78% after the second cycle, indicating extremely high durability (Franklin & Chandra, 1972). This range is comparable to high/very high *SDI* values reported for quartzite from the Fagfog Quartzite (Sharma & Tamrakar, 2023), but higher than sandstone from the Siwalik Sub-group (30.37%-93.23%) from Hetauda (Budhathoki & Tamrakar, 2022). Spherical samples generally yielded higher index values than irregular samples (Cicek et al., 2016). The results are similar to, but slightly better than, those sandstone from an underground coal mine at the United Mineral Coal Company Sor-range (97.36%-98.71%) at 4000 ft depth (Ahmed et al., 2022).

Analysis of results showed sample MA1 had the highest *SDI* and LA2 the lowest. This correlates directly with quartz content, where MA1 had high quartz content and LA2 low, consistent with findings from Bell & Culshaw (1998) for sandstone from the Sneinton Formation. All samples exhibited a Type I disintegration pattern after the second cycle.

PLSI values for sandstone from the Amile Formation ranged from 5.94 MPa to 14.81 MPa, higher than those reported

Table 4. Texture and classification of sandstone from the Amile Formation.

SN	Texture				Recalculation (%)			Type
	Size (μm)		Shape	Sorting	<i>Q</i>	<i>F</i>	<i>RF</i>	
	Range	Model						
LA1	50-130	200	Angular	Poorly Sorted	94.69	5.31	0	Quartz wacke
LA2	40-110	180	Sub Angular	Poorly Sorted	98.51	1.49	0	Quartz arenite
LA3	60-140	210	Sub Angular	Moderate	93.24	6.76	0	Subarkose arenite
LA4	70-130	190	Sub Angular	Moderate	98.02	1.98	0	Quartz arenite
LA5	60-130	220	Sub Angular	Moderate	98.59	1.41	0	Quartz arenite
LA6	50-200	280	Sub Angular	Moderate	97.5	2.50	0	Quartz arenite
LA7	40-120	190	Angular	Poorly Sorted	94.89	5.11	0	Quartz wacke
LA8	50-150	230	Sub Angular	Moderate	98.25	1.75	0	Quartz arenite
LA9	50-140	150	Sub Angular	Moderate	94.83	5.17	0	Subarkose arenite
LA10	100-500	600	Sub Angular	Moderate	92.87	7.13	0	Subarkose arenite
MA1	50-220	400	Sub Angular	Moderate	94.36	3.76	1.88	Subarkose arenite
MA2	40-200	260	Sub Angular	Moderate	86.13	4.46	9.41	Sublitharenite
MA3	30-170	200	Sub Angular	Moderate	85.59	6.56	7.85	Sublitharenite
MA4	50-150	300	Angular	Poorly sorted	84.31	8.20	7.49	Feldspathic wacke
MA5	40-130	280	Sub Angular	Moderate	86.32	5.34	8.34	Sublitharenite
MA6	40-140	290	Sub Angular	Moderate	87.17	4.40	8.43	Sublitharenite
MA7	60-250	300	Angular	Poorly sorted	87.04	4.79	8.17	Lithic wacke
MA8	40-110	150	Angular	Poorly sorted	80.41	8.41	11.18	Lithic wacke
MA9	40-230	300	Sub Angular	Moderate	85.01	6.61	8.37	Sublitharenite
MA10	50-200	300	Sub Angular	Moderate	88.05	1.37	10.57	Sublitharenite
MA11	40-210	260	Angular	Poorly sorted	80.43	2.80	16.77	Lithic wacke
MA12	30-200	600	Sub Angular	Moderate	78.50	4.30	17.2	Lithic wacke
MA13	50-180	240	Sub Angular	Moderate	88.07	2.84	9.09	Sublitharenite
UA1	60-220	320	Rounded	Well sorted	95.50	4.50	0	Quartz arenite
UA2	130-520	700	Rounded	Well sorted	98.73	1.27	0	Quartz arenite

for quartzite from the Fagfog Quartzite (2.40-11.4 MPa) by (Sharma & Tamrakar, 2023) and sandstone from the Siwalik Sub-group (0.024-2.16 MPa) by (Budhathoki & Tamrakar, 2022). Based on calculated *UCS* values, samples are classified as extremely strong to very strong (Brown, 1981). Lower *PLSI* values may be influenced by smaller sample sizes. Higher quartz content correlates with higher *UCS* (Sabatakakis et al., 2008), while feldspar can reduce strength (Yusof & Zabidi, 2016). Lower moisture content is associated with higher strength (Aligholi et al., 2017), as observed in samples LA3, LA6, LA10, MA9, and MA11. Samples with higher moisture content (LA9, MA1, MA4, MA6, MA7) showed lower *PLSI* values. Mica presence can reduce rock strength (Yasir et al., 2018), consistent with observations in samples LA1, MA1, MA2, MA7, and MA10. The highest *PLSI* value (sample LA10) corresponded to a lack of mica.

The samples LA1, LA10, UA1, and LA2 have higher *SDI* values and are found predominantly of quartz whereas samples MA2, MA5, MA7, and MA9 have relatively low *SDI* values and are found less quartz content. The sample having a higher *SDI* value has the least amount of matrix whereas the sample having a lower value of *SDI* has more than 15% matrix. Discontinuous matrix sandstones exhibit lower durability than those with continuous matrices (Tugrul & Zarif

1998). The present study found that samples having matrix, rock fragments, and mica may have slightly lower durability. Sandstone rich in smectite and zeolite had higher durability than pyroclastic rocks with just smectite (Dhakal et al., 2002).

Yusof & Zabidi (2016) found that the rock containing a high percentage of quartz gave high strength in this context, the sample UA1 and UA2 with maximum strength is found to have high quartz content whereas samples MA2, MA9, and MA13 with minimum strength is found to have less quartz content. There were no strong correlations between point load and slake durability with mineralogy (Ayakwah et al., 2009).

WAV for sandstone from the Amile Formation ranged from 0% to 2.17%, indicating low effective porosity (below 3%) according to ASTM C 128 specifications. These values are comparable to the low water absorption values reported for quartzite from the Fagfog Quartzite in Nepal (Sharma & Tamrakar, 2023) but lower than sandstone from the Siwalik Sub-groups (1.26% to 17.6%) in Hetauda (Budhathoki & Tamrakar, 2022). Samples with high quartz content showed 0% *WAV*. The low *WAV* indicates high durability and weathering resistance, similar to sandstone from the Sneinton Formation in England (0.3% to 2.5%) reported by Bell & Culshaw (1998), which correlated with high durability and strength.

Table 5. Point load strength index values of sandstones from the Amile Formation.

SN	Thickness avg(mm)	Width avg (mm)	Load (kN)	De (mm)	De ² (mm ²)	I _s (MPa)	F (mm)	I _{s(50)} (MPa)	UCS (MPa)	Class
LA1	29.5	53.4	26.78	44.79	2005.73	13.35	0.95	12.71	221.09	V. strong
LA2	25.5	44.7	11.58	38.10	1451.30	7.98	0.88	7.06	122.85	V. strong
LA3	28.3	47.5	25.22	41.37	1711.55	14.74	0.92	13.53	235.44	V. strong
LA4	26.7	43.8	18.14	38.59	1489.00	12.18	0.89	10.84	188.65	V. strong
LA5	26.5	47.3	18.93	39.95	1595.94	11.86	0.90	10.72	186.56	V. strong
LA6	27.6	48.2	20.42	41.16	1693.82	12.06	0.92	11.04	192.18	V. strong
LA7	26.8	47.5	18.81	40.26	1620.83	11.61	0.91	10.53	183.17	V. strong
LA8	26.2	46.7	18.64	39.47	1557.86	11.97	0.90	10.76	187.18	V. strong
LA9	27.4	45.45	17.34	39.82	1585.60	10.94	0.90	9.87	171.76	V. strong
LA10	27.8	44.7	25.97	39.78	1582.20	16.41	0.90	14.81	257.67	E. strong
MA1	28.3	45.35	22.28	40.42	1634.08	13.63	0.91	12.39	215.60	V. strong
MA2	28.7	48.8	12.3	42.23	1783.25	6.90	0.93	6.39	111.23	V. strong
MA3	26.7	54.1	16.64	42.89	1839.16	9.05	0.93	8.44	146.92	V. strong
MA4	28.5	48.4	24.76	41.91	1756.31	14.10	0.92	13.02	226.57	V. strong
MA5	25.7	45.5	11.92	38.59	1488.86	8.01	0.89	7.12	123.97	V. strong
MA6	28.8	45.8	16.5	40.98	1679.45	9.82	0.91	8.98	156.31	V. strong
MA7	28.6	53.8	22.83	44.26	1959.11	11.65	0.95	11.03	191.94	V. strong
MA8	26.3	45.3	16.25	38.95	1516.92	10.71	0.89	9.57	166.58	V. strong
MA9	27.8	48.7	11.13	41.52	1723.79	6.46	0.92	5.94	103.33	V. strong
MA10	28.4	45.6	26.68	40.61	1648.90	16.18	0.91	14.73	256.37	E. Strong
MA11	26.8	49.6	19.65	41.14	1692.49	11.61	0.92	10.63	185.04	V. strong
MA12	28.9	45.02	21.75	40.70	1656.58	13.13	0.91	11.97	208.25	V. strong
MA13	28.3	49.8	12.45	42.36	1794.43	6.94	0.93	6.44	112.04	V. strong
UA1	27.6	45.4	25.77	39.94	1595.42	16.15	0.90	14.60	254.04	E. Strong
UA2	28.4	47.7	25.27	41.53	1724.83	14.65	0.92	13.48	234.50	V. strong

V. strong = Very strong E. strong = Extremely strong.

Table 6. Result of slake durability index test of sandstones for two cycles.

SN	Initial weight (gm.)	IODW before 1st cycle (gm)	Moisture Content (%)	ODW after 1st cycle (gm)	ODW after 2nd cycle (gm)	I _{d1} %	I _{d2} %	Durability Classification	Disintegration Type
LA1	531.74	530.61	0.21	529.97	529.34	99.88	99.76	EH	I
LA2	507.08	506.81	0.05	504.23	502.6	99.49	99.17	EH	I
LA3	534.62	534.59	0.01	533.52	532.77	99.8	99.66	EH	I
LA4	513.89	512.95	0.18	511.51	510.59	99.72	99.54	EH	I
LA5	512.12	511.96	0.03	510.63	509.66	99.74	99.55	EH	I
LA6	532.2	531.72	0.09	530.44	529.11	99.76	99.51	EH	I
LA7	548.43	547.68	0.14	545.6	544.34	99.62	99.39	EH	I
LA8	529.67	528.8	0.16	526.9	526.26	99.64	99.52	EH	I
LA9	516.42	515.09	0.26	513.44	512.41	99.68	99.48	EH	I
LA10	546.02	545.72	0.05	544.85	544.14	99.84	99.71	EH	I
MA1	541.53	539.84	0.31	539.35	538.65	99.91	99.78	EH	I
MA2	549.09	547.53	0.28	543.75	542.99	99.31	99.17	EH	I
MA3	530.45	528.56	0.36	525.18	523.96	99.36	99.13	EH	I
MA4	519.81	518.14	0.32	517.41	516.64	99.86	99.71	EH	I
MA5	549.87	548.37	0.27	544.64	543.82	99.32	99.17	EH	I
MA6	543.86	542.01	0.34	540.17	539.35	99.66	99.51	EH	I
MA7	540.15	538.2	0.36	535.62	534.38	99.52	99.29	EH	I
MA8	540.92	539.61	0.24	538.05	536.16	99.71	99.36	EH	I
MA9	546.34	545.69	0.12	541.54	541.27	99.24	99.19	EH	I
MA10	511.37	510.87	0.1	509.95	509.34	99.82	99.7	EH	I

Table 6. Continued...

SN	Initial weight (gm.)	IODW before 1st cycle (gm)	Moisture Content (%)	ODW after 1st cycle (gm)	ODW after 2nd cycle (gm)	$I_{d1}\%$	$I_{d2}\%$	Durability Classification	Disintegration Type
MA11	521.58	521.11	0.09	519.6	518.87	99.71	99.57	EH	I
MA12	538.12	537.13	0.18	535.73	535.04	99.74	99.61	EH	I
MA13	491.51	490.31	0.24	487.71	486.73	99.47	99.27	EH	I
UA1	499.2	498.59	0.12	497.89	497.24	99.86	99.73	EH	I
UA2	513.31	512.59	0.14	511.67	511	99.82	99.69	EH	I

Table 7. Water Absorption values of sandstones from the Amile Formation.

S.N.	Dry wt.(gm)	Wet. wt.(gm)	WAV%
LA1	940	940	0
LA2	860	870	1.16
LA3	1500	1500	0
LA4	2910	2920	0.34
LA5	2610	2620	0.38
LA6	2080	2100	0.96
LA7	1080	1090	0.93
LA8	1170	1180	0.85
LA9	1240	1250	0.81
LA10	900	900	0
MA1	750	750	0
MA2	930	940	1.08
MA3	1910	1930	1.05
MA4	1500	1500	0
MA5	690	700	1.45
MA6	1120	1130	0.89
MA7	1270	1280	0.79
MA8	2680	2690	0.37
MA9	460	470	2.17
MA10	1350	1350	0
MA11	1620	1630	0.62
MA12	410	410	0
MA13	1510	1530	1.32
UA1	400	400	0
UA2	840	840	0

Table 8. Correlation between distinct tests under single linear regression.

Correlation	Equation	R^2
<i>PLSI</i> vs <i>SDI</i>	$PLSI = 11.294SDI - 1113$	0.8043
<i>PLSI</i> vs <i>WA</i>	$PLSI = -4.080WA + 13.14$	0.7982
<i>PLSI</i> vs <i>Pd</i>	$PLSI = 0.605Pd - 45.714$	0.8856
<i>PLSI</i> vs <i>Pp</i>	$PLSI = 0.6233Pp - 44.222$	0.8353
<i>SDI</i> vs <i>WA</i>	$SDI = -0.3131WA + 99.677$	0.7455
<i>SDI</i> vs <i>Pd</i>	$SDI = 0.0469Pd + 95.113$	0.8455
<i>SDI</i> vs <i>Pp</i>	$SDI = 0.0479Pp + 95.269$	0.7824

This study explored the relationship between *SDI* and the *PLSI* in sandstone samples, revealing a strong correlation with an R^2 of 0.804, significantly higher than the critical r-value (r

= 0.337), indicating a meaningful relationship. This finding is consistent with Khajevand & Fereidooni (2018) work, which reported a strong *SDI-PLSI* correlation in travertine samples,

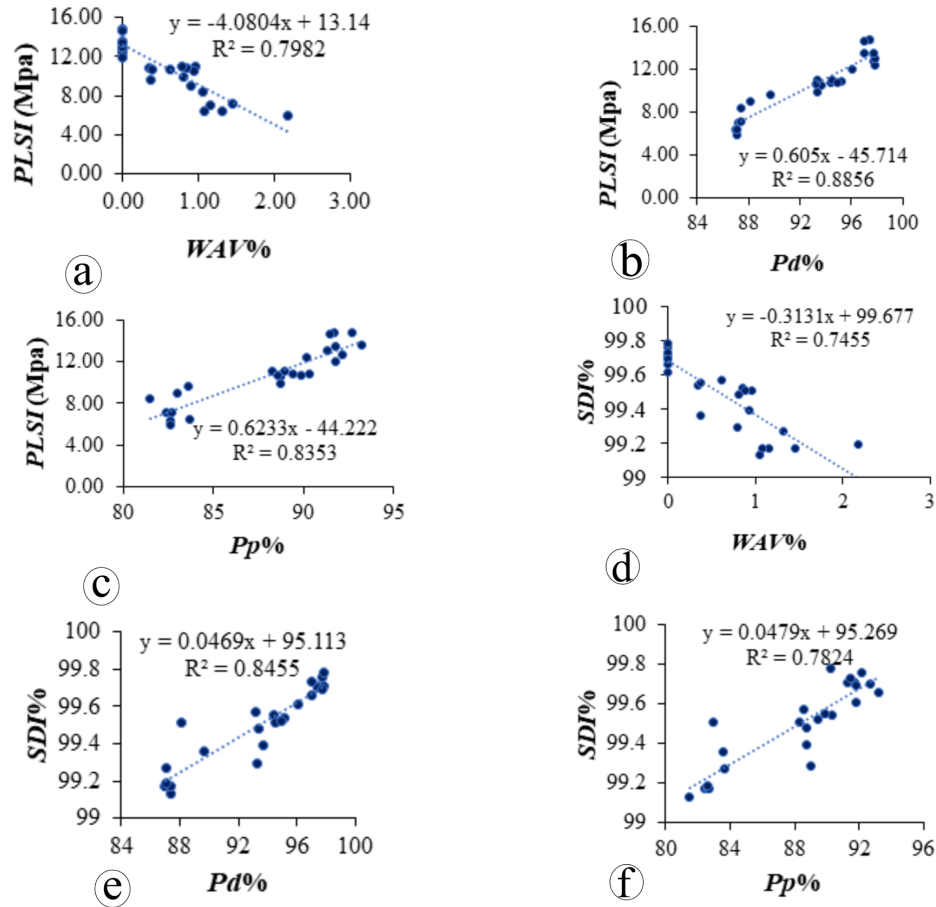


Figure 4. Correlation of best-fitted curve under linear regression (a) $PLSI$ vs WAV (b) $PLSI$ vs Pd (c) $PLSI$ vs Pp (d) SDI vs WAV (e) SDI vs Pd and (f) SDI vs Pp .

Table 9. Correlation between various tests employing significant independent variables under multiple regression.

Dependent Variable	Independent Variable	Equation	R^2	SEE	Critical r
SDI	$PLSI$, Water Absorption, Packing density	$SDI = 96.486 + 0.005PLSI - 0.98WA + 0.032Pd$	0.869	0.082	0.404
SDI	$PLSI$, Water Absorption, Packing proximity	$SDI = 97.480 + 0.022PLSI - 0.109WA + 0.02Pd$	0.848	0.089	0.404
SDI	Water Absorption, Packing proximity	$SDI = 97.010 - 0.153WA + 0.029Pp$	0.840	0.892	0.396
SDI	Water Absorption, Packing density	$SDI = 96.355 - 0.105WA + 0.034Pd$	0.869	0.080	0.396
$PLSI$	SDI , Water Absorption	$PLSI = -605.295 + 6.204SDI - 2.138WA$	0.860	1.051	0.396
$PLSI$	SDI , Water Absorption, Packing density	$PLSI = -73.998 - 0.48SDI - 1.470WA + 0.4064Pd$	0.916	0.833	0.404
$PLSI$	SDI , Water Absorption, Packing proximity	$PLSI = -236.624 + 2.218SDI - 1.663WA + 0.3137Pp$	0.903	0.894	0.404
$PLSI$	Packing density, Packing proximity	$PLSI = -45.584 + 0.662Pd - 0.062Pp$	0.886	0.947	0.396
$PLSI$	SDI , Packing density, Packing proximity	$PLSI = -285.246 + 2.519SDI + 0.510Pd - 0.025Pp$	0.892	0.944	0.404
$PLSI$	SDI , Packing density	$PLSI = -288.944 + 2.557SDI + 0.485Pd$	0.892	0.922	0.396
$PLSI$	SDI , Packing proximity	$PLSI = -530.442 + 5.103SDI + 0.379Pp$	0.871	1.007	0.396
$PLSI$	Water Absorption, Packing density	$PLSI = -27.794 - 1.52WA + 0.423Pd$	0.916	0.814	0.396
$PLSI$	Water Absorption, Packing proximity	$PLSI = -21.409 - 2.002WA + 0.378Pp$	0.898	0.896	0.396

SDI = Slake Durability Index, $PLSI$ = Point Load Strength index, WA = Water Absorption, Pd = Packing density, Pp = Packing proximity, SEE = Standard error of the estimation

and Ahmad et al. (2017), who found an *SDI-PLSI* correlation with an R^2 of 0.812 in sedimentary, metamorphic and igneous rocks altogether 45 samples. However, it contrasts with Liang et al. (2015), who noted a weaker correlation between *SDI* and uniaxial compressive strength (*UCS*) in stronger rocks.

A good correlation was found between *PLSI* vs *WAV* ($R^2 = 0.79$), *PLSI* vs *Pd* ($R^2 = 0.88$), *PLSI* vs *Pp* ($R^2 = 0.83$), *SDI* vs *WAV* ($R^2 = 0.74$), *SDI* vs *Pd* ($R^2 = 0.84$) and *SDI* vs *Pp* ($R^2 = 0.78$). These results align with Yusof & Zabidi (2016) findings, where higher quartz content and *Pd* were linked to greater rock strength. The strong correlations observed suggest that *PLSI*, *SDI*, and other physical properties are interconnected and contribute to the mechanical behavior of sandstone from the Amile Formation. These relationships could be useful for geotechnical assessments, allowing for the estimation of one property based on another.

Multiple regression analysis further improved predictive models. For *PLSI*, combining *SDI*, *WAV*, and *Pd* increased the R^2 to 0.916, while for *SDI*, including *WAV* and *Pd* raised the R^2 to 0.869. These findings, consistent with Yusof & Zabidi (2016), build on previous studies by incorporating multiple parameters, offering a more comprehensive understanding of sandstone properties, which could enhance predictions for geotechnical applications.

6 Conclusion

Sandstone from the Amile Formation exhibited extremely high durability (*SDI* 99.17–99.78%) and very strong to extremely strong compressive strength (*PLSI* 5.94–14.81 MPa). The water absorption test results of sandstone from the Amile Formation indicate very low porosity and high resistance to water penetration (*WAV*: 0–2.17%). Based on the strong correlation between *SDI* and *PLSI* ($R^2 = 0.804$), future research can use the equation $SDI = 0.0712PLSI + 98.727$ to estimate *SDI* from *PLSI* for similar sandstones. *PLSI* is well explained by *WAV*, *Pd*, and *Pp*, whereas *SDI* is explained well by *Pd* and *Pp*. Multiple regression analysis, with an R^2 of 0.869, indicates that *SDI* can be very well explained by *PLSI* in combination with water absorption and *Pd*. Expanding the dataset could improve the study's accuracy.

Acknowledgements

The authors are thankful to the Central Department of Geology, Tribhuvan University and the Department of Mines and Geology, Lainchaur for providing the necessary laboratory facility for conducting the test.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Saurav Poudel: M.Sc. Dissertation (conceptualization, methodology, writing - original draft). Dr. Naresh Kazi Tamrakar: Dissertation Advisor (supervision, writing - reviewing, and editing).

Data availability

All data produced or examined in the course of the current study are included in this article.

Declaration of use of generative artificial intelligence

This work was prepared with the assistance of generative artificial intelligence (GenAI) OpenAI's ChatGPT with the aim of enhancing readability and refining language. The entire process of using this tool was supervised, reviewed and when necessary edited by the authors. The authors assume full responsibility for the content of the publication that involved the aid of GenAI.

List of symbols and abbreviations

ASTM	American Society for Testing and Materials
<i>De</i>	Equivalent core diameter for an irregular rock lump
<i>F</i>	Size correction factor
<i>Gn</i>	Grain-to-grain contact
<i>I_{d1}</i>	Slake durability index (first cycle)
<i>I_{d2}</i>	Slake durability index (second cycle)
EH	Extremely High
<i>I_s</i>	Uncorrected Point load strength index, Mpa
<i>I_{s(50)}</i>	Corrected point load strength
ISRM	International Society for Rock Mechanics
<i>L</i>	Traverse length
MBT	Main Boundary Thrust
MCT	Main Central Thrust
<i>Nc</i>	Total number of contacts
<i>IODW</i>	Initial oven dry weight
<i>ODW</i>	Oven dry weight
<i>Pd</i>	Packing density
<i>PLSI</i>	Point Load Strength Index
<i>Pp</i>	Packing proximity
<i>Q</i>	Quartz
<i>Qm</i>	Monocrystalline quartz
<i>Qp</i>	Polycrystalline quartz
R^2	Relationship coefficient
<i>RF</i>	Rock fragments
<i>SDI</i>	Slake Durability Index
<i>SEE</i>	Standard Error of Estimate
LA	Lower Amile
MA	Middle Amile
UA	Upper Amile

UCS Uniaxial Compressive Strength
XPL Cross-polarized light

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Numerical and experimental analysis of shallow foundations on tropical climate soil using Mohr-Coulomb cap model

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Article

Keywords

Tropical climate soil
Collapsible soil
Shallow foundations
Behavior prediction
Footing loading test

Abstract

The application of loading tests allows us to evaluate the behavior of shallow foundations under real conditions. This article addresses the application of a constitutive model based on Mohr-Coulomb with a cap failure criterion that simulates the compaction of the material and limits its shear strength to describe the behavior of a footing resting on tropical climate soil composed of silt-clay sand. The foundation was tested in terms of axial compression through a static and slowly maintained loading test (SML) until its failure was characterized by significant settlement. A two-dimensional axisymmetric finite element numerical model was used to process the numerical analyses. The numerical model used was able to simulate with high similarity the load-settlement behavior of the footing tested in the loading test, under different water content conditions and preload imposed in the field tests. Additionally, the experimental results facilitated the successful guidance of the numerical analyses, providing parameters to safely analyze similar foundations under other conditions.

1. Introduction

The load versus settlement curve for foundations in tropical climate soils is an important topic to consider in foundation design projects. These soils are typically collapsible, with a metastable structure resulting from their formation process in tropical climate regions. Various methods for predicting the load and settlement behavior of foundations in these soils have already been developed (Barata, 1973; D'appolonia et al., 1970; Kumar et al., 2007; Oh & Vanapalli, 2011; Vesić, 1973). These methods encompass analytical, empirical, and experimental approaches. However, the static load test is considered the most important test for determining the load versus settlement behavior of a foundation. Nonetheless, few load tests are conducted on full-scale prototypes under the same conditions as real constructions, especially in tropical climate soils with matric suction measurements.

To estimate settlement in shallow foundations, solutions based on the theory of elasticity are commonly adopted (Boussinesq, 1885; Poulos, 1968; Milovic, 1992), as well as those that consider factors such as embedding, flexibility, and modulus increase with depth, as proposed by Mayne & Poulos (1999). Additionally, other analytical solutions, like those by Terzaghi & Peck (1963) based on Meyerhof's (1956) work, are used. The effects of suction on shallow foundation behavior were later described using the mechanics of unsaturated soils, according to studies by Alonso et al. (1990), Fredlund et al. (2012), Briaud (2013),

and Giacheti et al. (2019). These studies indicate that the increase in failure stress of footings in unsaturated soils is due to increased suction, with Briaud (2013) explaining differences in considering this resistance increase over the term $0.5 \cdot \gamma \cdot B \cdot N_c$ instead of the term $c' \cdot N_c$.

It is established that tropical climate soils, despite being compressible, have high initial stiffness. This stiffness can be attributed to the effects of drainage, load application speed, and particle bonding (Consoli et al., 1998; Terzaghi et al., 1996; Atkinson, 2000; Poulos & Small, 2000). Therefore, understanding the behavior of shallow foundations in tropical climate soils, considering the inherent aspects of soil matric suction, is crucial.

Understanding the geomechanical parameters of tropical climate soils is essential for comprehending foundation behavior, and these parameters should be obtained through laboratory tests, field tests, and loading tests under natural and flooded conditions. Employing the finite element method (FEM) allows for evaluating foundation behavior under varying water content and compaction conditions. This study presents a modification to Décourt (1999) proposal, enabling the description of foundation behavior in tropical climate soil based on geomechanically parameters, incorporating variables such as stress history and compressibility obtainable in the laboratory.

In the conclusion of this study, the results obtained from empirical and numerical methods are compared with those established in foundation literature. The findings indicate

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Submitted on July 29, 2024; Final Acceptance on January 6, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.004724>



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that soil suction enhances the load capacity and stiffness of the soil beneath the footing. The modified Décourt method demonstrates strong agreement with the loading test results, showcasing its potential as a new alternative for designing footings in popular buildings.

2. Materials and methods

2.1 Experimental site and geotechnical profile

The footing load test was performed at the Experimental Field Mechanics of Soils and Foundations of the Federal University of Uberlândia (CEMSF-UFU), located in the city of Uberlândia, Minas Gerais, Brazil. The characterization of the local subsoil was carried out by collecting deformed samples obtained in simple recognition surveys with a standard penetration test (SPT) up to a depth of 21 meters for basic soil characterization (Figure 1).

The water level was detected at a depth of 10 meters, and the local soil consisted of silty clayey sand, with traces of gravel, belonging to the SC-SM group, according to the Unified Soil Classification System. The uncorrected values provided by the SPT test indicated, according to ABNT (2020), that the soil had a relative density from loose to very loose up to a depth of 19 meters (Figure 2).

Geological surveys revealed that the regional soil formation is linked to the weathering of basalt from the Serra Geral Formation with detrital-lateritic sediments of colluvial origin (CODEMIG, 2017). Therefore, the friction angle was determined by a direct shear test under drained conditions and was equal to 21° (Figure 3a), in accordance with ASTM D3080 (ASTM, 2012), using a displacement rate of 0.10 mm/min and oedometer tests for 1-meter soil (Figure 3b). In addition to the friction angle, the effective cohesion (c') in saturated conditions was determined by a direct shear test. This parameter, in turn, is highly influenced by the variation in suction in tropical climate soils, which depends on the value of the air entry value (u_{wae}) obtained through the Soil Water Characteristic Curve (SWCC), following the method given by ASTM (2016).

From the SWCC (Figure 4), a value of 6 kPa was obtained for the air entry value. Thus, the proposal of Khalili & Khabbaz (1998) was used to determine the portion of apparent cohesion relative to the gain in shear strength due to the relationship between the suction in the soil and the air entry value, presented in Briaud (2013) as Equation 1:

$$\alpha = \sqrt{\frac{u_w}{u_{wae}}} \quad (1)$$

where u_w is the suction present in the soil and u_{wae} is the of air entry into the soil.

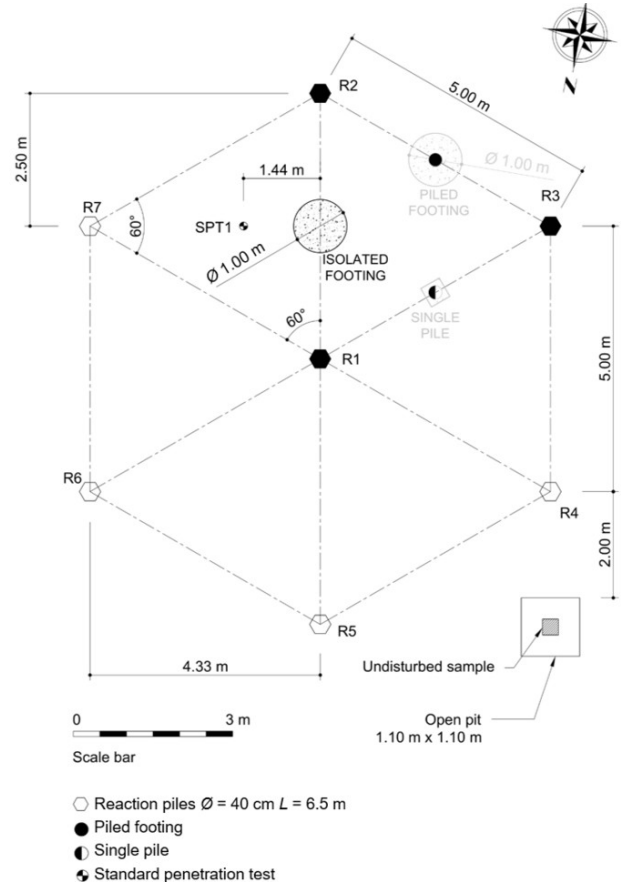


Figure 1. Plan view of the test site.

The suction was measured by tensiometers installed in the field at 0.65 m below the ground surface, varying between 15 and 40 kPa for the rainy and dry seasons, respectively. For the silty clayey sand soil, a suction of 30 kPa was adopted in the analyses.

The compressibility of the surface soil was evaluated by an oedometer test because tropical climate soils tend to collapse when moistened. The magnitude of the collapse that the soil exhibits when flooded was estimated by the collapse index (CP) according to the proposal of Jennings & Knight (1975), as defined by Equation 2:

$$CP = \frac{e_0 - e_c}{1 + e_0} \cdot 100 \quad (2)$$

where e_0 is the void ratio under natural moisture conditions and a vertical pressure of 200 kPa and e_c is the void ratio after flooding under a vertical pressure of 200 kPa, which is taken together with the curve of the oedometer test under flooding.

The authors presented a severity of collapse scale, in which CP values less than 5 are considered moderate. With the oedometer curves at the 9% and 16% moisture levels, the calculated CP was equal to 11, indicating that the severity of

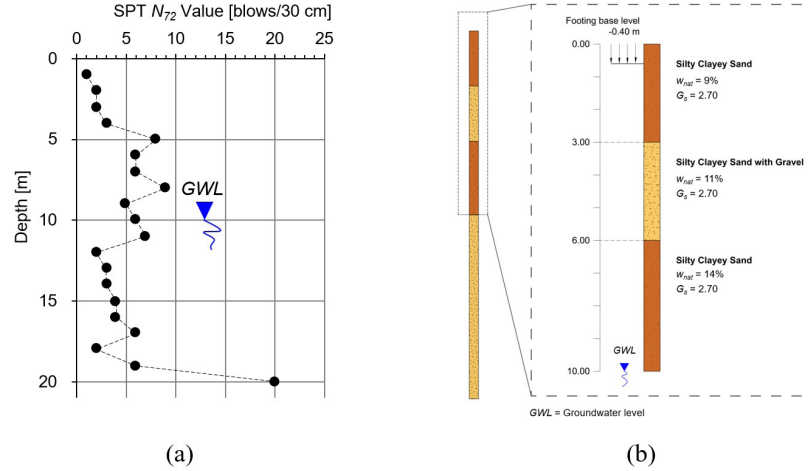


Figure 2. (a) Uncorrected SPT values; (b) geotechnical profile of the experimental site obtained by SPT sounding.

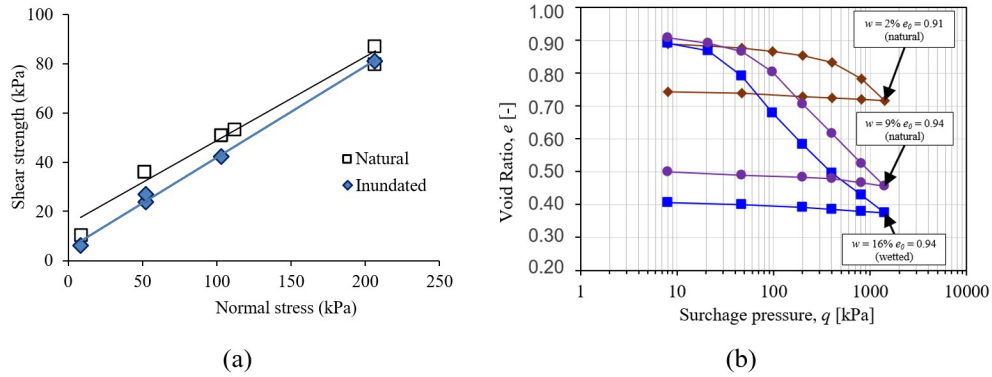


Figure 3. Results of soil characterization tests: (a) direct shear test under natural and flooded condition and (b) oedometer tests for 1-meter soil depth with a log pressure scale under different water contents.

soil collapse was in the category of “severe trouble”. From the oedometer test, several parameters were calculated at different moisture contents that were used in the analyses, including $\lambda = C_c / \ln 10$ and $\kappa = C_r / \ln 10$, in addition to the yield stress (σ_p) estimated by Casagrande construction and the oedometer moduli (Table 1).

For the data from Table 1, the proposal of Alonso et al. (1990) for unsaturated soils describes the increase in the yield stress with increasing suction. This occurs in most Brazilian soils and is known as apparent preconsolidation stress. In addition, the stress values were transformed into the triaxial stress state for the numerical analyses. In a simplified manner, for void ratio from 0.80 to 0.90 and up to a suction interval of 200 kPa, the increase in the yield stress (in kPa) is given by Equation 3:

$$p_p = 1.12 \cdot u_w + 25 \quad (3)$$

where p_p is the yield stress for the triaxial stress state.

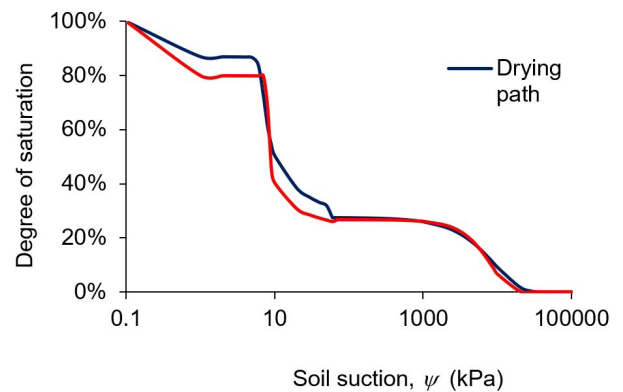


Figure 4. SWCC of local soil described by Fredlund & Xing (1994).

2.2 Loading test setup

The tested foundation is a circular footing with diameter B equal to 1.00 m and a thickness of 0.20 m, this thickness dimension is used in foundations employed in low-income

Table 1. Parameters obtained from the oedometer tests under different water contents.

Oedometer test series	w [%]	e_0 [-]	σ_p [kPa]	C_c [-]	λ [-]	C_r [-]	κ [-]	E_{oed}^a [MPa]	$E_{oed,u}^b$ [MPa]
1	2%	0.91	530	0.212	0.092	0.021	0.009	1.8 - 17	80
2	9%	0.94	85	0.305	0.133	0.039	0.017	1.0 - 4.5	50
3	16%	0.94	32	0.331	0.144	0.054	0.023	0.2 - 1.8	35

^aOedometer moduli calculated for void ratio between 0.85 and 0.65. ^bOedometer moduli calculated during unloading for void ratio between 0.85 and 0.65.


Figure 5. Loading test: (a) under natural conditions (LT-EXP-NAT); (b) under flooded conditions (LT-EXP-FLO).

buildings financed by low-income housing programs. In addition, the current study is part of a larger study with combined foundations.

The footing was designed and constructed using reinforced concrete following recommendations of the ABNT (2014) and by excavation, following the recommendations of the ABNT (2019). The footing base is supported at a depth of 0.40 m below the surface; however, a previous excavation around the footing was performed to eliminate the influence of the lateral friction of the footing with the surrounding soil. Soil samples were collected close to the footing before and after each load test. In the laboratory, the values of water content, void ratio, and matric suction were obtained in accordance with ASTM D7263-21 and ASTM D5298-16 (ASTM, 2016, 2021). For the Experimental Loading Test in Situ (LT-EXP-NAT), the observed moisture content (w) was 8.7%, while the void ratio (e) was 0.78. The suction measurement (u_w), inferred from readings of tensiometers positioned near the tested foundation, was 30. In the case of the Experimental Loading Test Flooded (LT-EXP-FLO), the moisture content (w) was significantly higher, recorded at 17.81%, and the void ratio (e) was 0.62. There was no suction measurement (u_w) in this case, with its value being 0.

A load cell with an accuracy of 0.20 kN was used to measure the load applied during the test. Four dial gages were used for displacement measurements with 0.01 mm precision and 100 mm range, positioned orthogonal on top of the tested footing.

Two loading tests were performed, the first under natural moisture conditions, named LT-EXP-NAT, and the second performed after a 24-h period of soil flooding, named LT-EXP-FLO (Figure 5). The flooding process was induced by supplying water via the surface in order to form a constant water level.

2.3 Existing methods

For the estimation of the ultimate load and settlement, theoretical-analytical and empirical methods have been established in the literature. Such methods are based on parameters sometimes determined in the laboratory, sometimes estimated by in situ tests, which in turn allow correlations.

2.3.1 Ultimate bearing capacity equations

The known solution of Terzaghi & Peck (1963) with the shape factors given by Vesic (1975) is given by Equation 4:

$$q_u = c' \cdot N_c \cdot S_c + \frac{1}{2} \gamma \cdot B \cdot N_\gamma \cdot S_\gamma \quad (4)$$

where c' is the effective cohesion; γ is the unit weight; B is the dimension of foundation width; and $N_c = (N_q - 1) \cot \phi'$, $N_q = 2(N_\gamma + 1) \tan \phi'$, and $N_\gamma = e^{\pi \tan \phi'} \tan^2(45^\circ + \phi'/2)$ are bearing

capacity factors that apply to their respective terms and depend on the friction angle ϕ' .

Briaud (2013) suggests a modification to the proposal of Terzaghi & Peck (1963) to consider the addition of a term that represents the shear strength by the effective stress as a function of the soil suction by Equation 5:

$$q_u = c' \cdot N_c \cdot S_c + \frac{1}{2} (\gamma \cdot B - \alpha \cdot u_w) \cdot N_\gamma \cdot S_\gamma \quad (5)$$

where c' is the effective cohesion; γ is the unit weight; B is the dimension of the foundation; N_c , N_γ , and N_q are bearing capacity factors that apply to their respective terms, S_c , S_γ , and S_q are shape factors as indicated in Vesic (1975) for circular geometry; α is a coefficient defined by Equation 1; and u_w is the matric suction present in the soil.

The bearing capacity factors can also be obtained through an axisymmetric formulation with the lower bound limit analysis as explained in Kumar & Khatri (2011).

2.3.2 Empirical equation

The empirical equation of Décourt (1999) estimates the settlement of foundations in sandy, clayey, or intermediate soils using data from the SPT test. Through the observation of the curves of foundations and tested prototypes, Décourt (1999) presented the following equation Equation 6:

$$\log \left(\frac{q}{q_r} \right) = C \cdot \left(1 + \log \left(\frac{s}{B_{eq}} \right) \right) \quad (6)$$

where B_{eq} is the dimension equivalent to that of a square footing; C is a compressibility parameter equal to 0.426, a value suggested by the author with a variation of + or - 10%; q_r is the reference stress; q is the pressure applied by the foundation; and s is the settlement.

2.4 Upgrade proposed for the Décourt (1999) method

A modification of Equation 6 proposed by Décourt (1999) allows us to make predictions of the foundation behavior in unsaturated soils and to consider the collapse of the soil, a phenomenon also related to increase in stiffness with suction caused by drying in tropical climate soils. To the term q_r of Equation 6, the term $\alpha \cdot u_w \cdot \tan \phi'_{peak} \cdot N_{\gamma(\phi'_{peak})}$, which incorporates the contribution of suction to shear strength, was added, as suggested by Briaud (2013) for the equation of Terzaghi & Peck (1963).

In placing settlement (s) as a stress-dependent variable (q) applied to the footing by Equation 7:

$$s(q) = \frac{B_{eq}}{10} \left(\frac{q}{q_r + \left(\alpha \cdot u_w \cdot \tan \phi'_{peak} \cdot N_{\gamma(\phi'_{peak})} \right)} \right)^{\frac{OCR_{(u_w=0,e)}}{0.426}} \quad (7)$$

where q_r is the reference stress; $OCR_{(u_w=0,e)} = p_v/p_p$ is the relationship between the highest effective vertical stress applied to the soil layer and the corresponding yield stress in the oedometer compression curve under flooded conditions for the same void ratio; u_w is the soil matric suction; $\alpha = (u_{wae}/u_w)^{0.5}$; N_γ is the bearing capacity factor given by Terzaghi & Peck (1963).

Thus, the term α in Equation 7 comprises an asymptotic increase in the bearing capacity with suction and incorporates strength parameters based on the measured friction angle for the compactness of the material.

The term $OCR_{(u_w=0,e)}$ is related to the increase in rigidity already widely observed in soils where the matric suction phenomenon is present, in addition to incorporating the history of stresses imposed on the soil, which also influences the rigidity of the soil.

The proposal also maintains the possibility of considering the shape of the footing, adopting a dimension equivalent to that of a square footing B_{eq} , that is, dispensing with the application of shape correction factors.

2.5 2D FEM numerical models

The numerical models were constructed using the software RS2 from Rocscience. Based on the axisymmetry of the foundation, a two-dimensional geometry was constructed that simulates the three-dimensional space (Figure 6a). Regarding the boundary conditions of the semi space, restrictions were applied to the displacements in the X-direction on the sides and at the base, and the displacements in the X- and Y-directions were restricted. A gradual refinement of the mesh was applied near the footing (Region 1) to improve the convergence of the results (Figure 6b). The elements used in the foundation models were positioned in an installed manner, without the use of interface elements.

Concerning the number of elements, convergence tests were performed to choose the mesh with the lower number of elements (1353) and nodes (2832), which leads to a reduced computational effort. The finite element used is of the triangular type with a total of 6 nodes, 3 of them being middle nodes, and each node has 2 degrees of freedom in the constructed model. The validation of the numerical model applied was carried out from the experimental results obtained in loading tests in the field.

The materials consisted of the concrete represented by the footing (Region 1) and the soil (Regions 2-7). For the soil, the Mohr-Coulomb with cap constitutive model was used, and for the concrete, the Mohr-Coulomb criterion was used.

The numerical loading tests were denoted LT-NUM-NAT and LT-NUM-FLO to identify the condition of the soil tested (natural and flooded), following the nomenclature given for the experimental tests. For each series, loading was divided into 11 stages, and unloading was divided into 4 stages (Figure 6c).

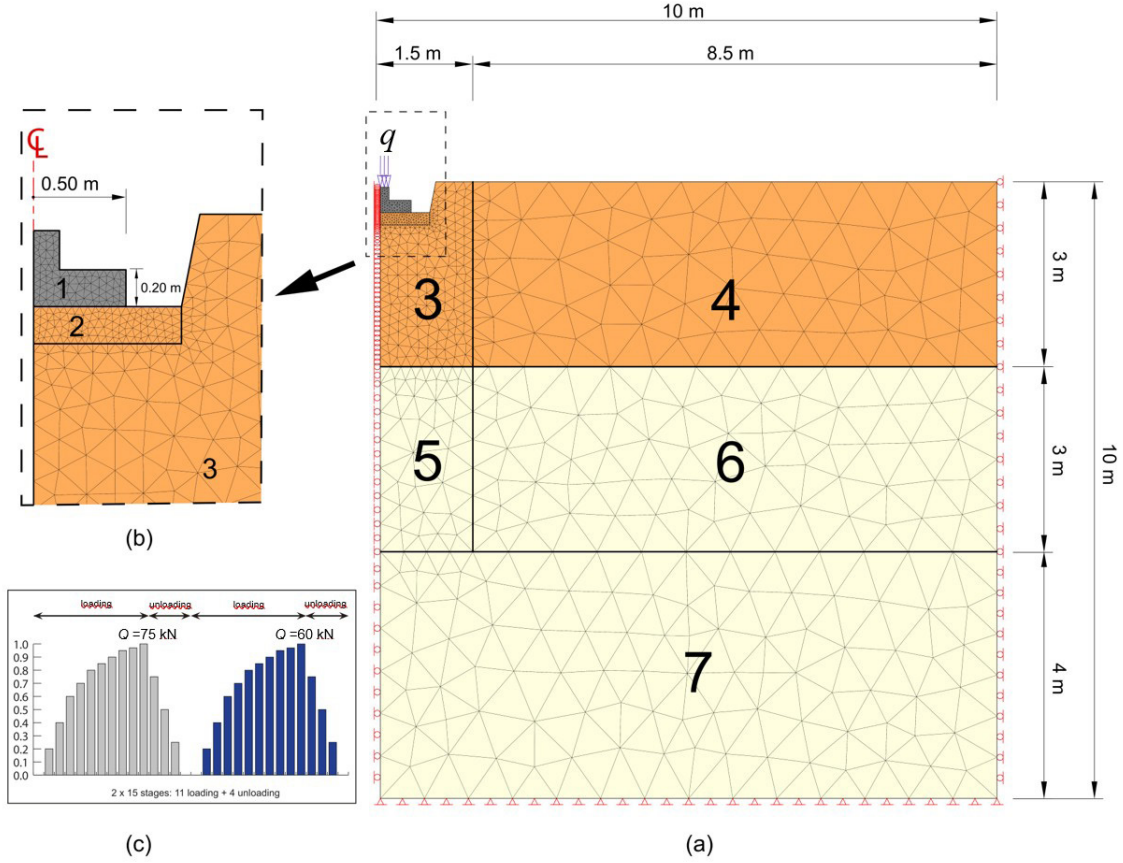


Figure 6. Geometry and details of the 2D axisymmetric model: (a) finite element mesh and boundary conditions; (b) mesh refinement near footing; (c) load stage chart.

2.5.1 Constitutive model of soil

The Mohr–Coulomb with cap constitutive model employed by RS2 makes it possible to simulate an initial state of compaction or densification. The two yield surfaces of interest to the analyses were the Coulomb yield surface and the elliptical cap.

The failure criterion for the Mohr–Coulomb yield surface employed by the RS2 program is given by Equation 8:

$$(\sigma_1 - \sigma_3) + M \left(p - \frac{c'}{\tan \phi'_{peak}} \right) = 0 \quad (8)$$

where c' is the cohesion; ϕ'_{peak} is the peak friction angle; $(\sigma_1 - \sigma_3)$ and p are the differential and mean stresses, respectively; and M is a parameter dependent on the stresses acting on the material.

The parameter p_p , called the yield mean stress, is related to the yield stress (σ_p) determined by the oedometer tests calculated using Equation 3. For LT-NUM-NAT, a suction (u_w) of 30 kPa led to a yield mean stress of approximately 60 kPa. For LT-NUM-FLO, a suction (u_w) of 0 kPa led to a yield mean stress of 25 kPa but considering that there is a

reduction in the soil void ratio due to the load imposed by the LT-NUM-NAT series, the value of 40 kPa was adopted.

The failure criterion for the elliptical surface employed by the RS2 program is given by Equation 9:

$$\left(\frac{(\sigma_1 - \sigma_3)}{M_f} \right)^2 + \left(p - \frac{c'}{\tan \phi'_{peak}} \right) + (p + p_p) = 0 \quad (9)$$

where c' is the cohesion; ϕ'_{peak} is the peak friction angle; $(\sigma_1 - \sigma_3)$ and p are the deviatoric and mean stresses, respectively; M_f is a parameter dependent on the stresses acting on the material; and p_p is the yield stress.

According to a suggestion in the unsaturated soil literature (Briaud, 2013), a way to incorporate the effect of matric suction into the cohesion intercept c is given by Equation 10:

$$c = c' + \alpha \cdot \mu_w \cdot \tan \phi'_{cv} \quad (10)$$

where c' is the effective cohesion intercept, set equal to 1 kPa; $\alpha = (u_{wae}/u_w)^{0.5}$, which for an equal soil suction $u_w =$

30 kPa assumes an approximate value of 0.45; and $\phi' = \phi'_{cv} = 21^\circ$. Therefore, for the unsaturated condition, the cohesion intercept adopted was $c = 9$ kPa (Table 2).

The parameters used in the simulations are shown below (Table 2).

2.5.2 Constitutive model of concrete

The behavior of the reinforced concrete composing the footing was represented by the Mohr–Coulomb model employed by the RS2 program. This representation of the behavior of the concrete references Ardiaca (2009).

The failure criterion applied to the concrete material is given by Equation 11, which in turn is associated with Equation 12 given in EHE-08 (del Hormigón, 2011):

$$\tau = c + \sigma \cdot \tan \phi \quad (11)$$

where c is cohesion, ϕ is the friction angle, and σ is the normal stress.

Another parameter is the tensile strength of the material, which in this case was obtained based on Equation 13 presented in EHE-08 (del Hormigón, 2011):

$$\tau_{md} = \beta \cdot f_{ct,d} + \mu \cdot \sigma_{cd} \quad (12)$$

$$f_{ct,d} = 0.20 \cdot f_{ck}^{2/3} \text{ [MPa]} \quad (13)$$

where β and μ are coefficients with mean values of 0.30 and 0.70, respectively, and f_{ck} is the characteristic compressive strength of concrete, equal to 20 MPa. With the above values, the angle of internal friction (ϕ) was 35° ; the cohesion (c) was 442 kPa; and the design value of the tensile strength ($f_{ct,d}$) was 1473 kPa.

The elastic properties, i.e., the Young's modulus and Poisson ratio, were defined as $E = 23$ GPa and $\nu = 0.20$, respectively, according to EHE-08 (del Hormigón, 2011).

2.5.3 Loading and unloading behavior

The deformation model of the material is elastoplastic and isotropic, using the Young's moduli E and E_{ur} to represent the

rigidity of the soil during loading and unloading, respectively. The values of these moduli were obtained through oedometer tests, according to void ratios measured before each loading test (Table 1). In this sense, these Young's moduli were determined by Equation 14, according to Selvadurai (1979), since the conditions were confined.

$$E = E_{oed} \frac{(1+\nu) \cdot (1-2\nu)}{(1-\nu)} \quad (14)$$

where E is Young's modulus; E_{oed} is the oedometer modulus; and ν is the Poisson's ratio of the soil.

For Poisson's coefficient, a mean value of 0.35 was assumed for the silty clayey sand according to Bowles (1997). Thus, the modulus (E) was equal to approximately 62% of the values obtained in the oedometer test.

3. Results and discussion

3.1 Ultimate bearing capacity

For loose or soft soils, Terzaghi & Peck (1963) recommend that a reduction factor of 1/3 be applied to the tangent of the friction angle, which reduces the friction angle to 21° . The following is a summary of the results (Table 3).

3.2 Loading tests

From the performance of the loading tests, load vs. settlement curves were obtained. These curves are presented individually to demonstrate the sequence of tests performed, natural and flooded (Figure 7).

In Figure 8, the settlements are presented in a normalized manner with the B dimension of the foundation on a semilogarithmic scale to show the range of percentage settlement in which the rigidity of the footing undergoes a marked decrease, that is, starting at 50 kPa.

With the load vs. settlement curves of the footing, it is possible to infer the value of Young's secant modulus (E_s), as shown in BSI EN 1997-2 (BSI, 2007), by means of the theoretical solution for a circular rigid plate based on a homogeneous elastic soil. It is noteworthy that the presence

Table 2. Parameters for the Mohr-Coulomb with cap model.

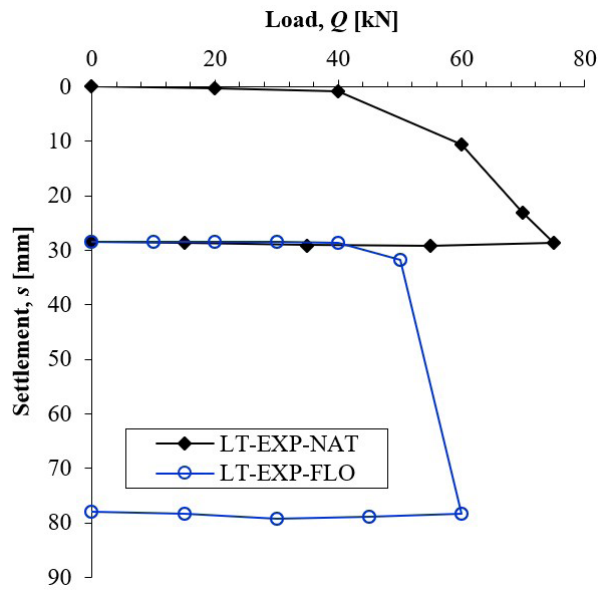
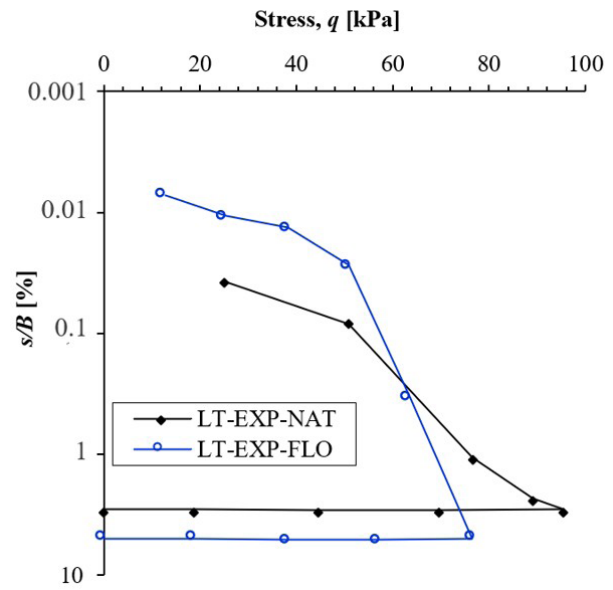
Test series	Regions	Material shear strength					Material stiffness		
		γ	c	ϕ'_{peak}	p_p	$\lambda - \kappa$	E	E_{ur}	ν
		[kN/m ³]	[kPa]	[°]	[kPa]	[-]	[MPa]	[MPa]	[-]
LT-NUM-NAT	2, 3, 4	16	9	21	60	0.12	12	35	0.35
	5, 6, 7	16	10	30	100	0.11	15	--	0.35
LT-NUM-FLO	2, 3, 4 ^a	16	1	21	40	0.15	1	20	0.35
	5, 6, 7	16	10	30	100	0.11	15	--	0.35

^aRegion 4 has the same properties as those defined in LT-NUM-NAT simulation.

Table 3. Ultimate bearing capacity computed.

Author(s)	Soil conditions	Parameter(s)	Equation	q_u [kPa]
Terzaghi & Peck (1963) and Vesic (1975)	Natural	$c = 9$ kPa	Equation 4	236
	Flooded	$c' = 1$ kPa		53
Terzaghi & Peck (1963), Vesic (1975) and Briaud (2013)	Natural	$c' = 1$ kPa; $u_w = 30$ kPa	Equation 5	102
	Flooded	$c' = 1$ kPa; $u_w = 0$ kPa		53

Other parameters: $\phi' = \arctan\left[\left(\frac{2}{3}\right)\tan 29^\circ\right] = 21^\circ$; $N_c = 15.81$; $S_c = 1.45$; $N_\gamma = 6.19$; $S_\gamma = 0.60$; $\alpha = 0.45$; $u_{wde} = 6$ kPa; $\gamma = 16$ kN/m³; $B = 1$ m.


Figure 7. Load-settlement curves obtained for the load test on a circular footing.

Figure 8. Stress versus settlement curves normalized by the footing width B of the loading tests.

of lateritic particles in the soil gives the soil a high initial rigidity. Compared to the oedometer moduli for the same range of void ratio, the values of the initial secant moduli obtained in the field are high due to the effect of soil confinement and the presence of lateritic particles.

3.3 Numerical simulations

The overlap of the load vs. settlement curves obtained numerically with the experimental points indicates significant agreement (Figure 9). Interestingly, the footing exhibits linear-elastic behavior for loads below 50% of the maximum applied load.

In the linear-elastic regime, the contact stresses under the foundation on “sandy soil” are higher at the edge than in the center, which corroborates the observations made in Brown (1969) regarding the behavior of a rigid circular footing and agrees with the theoretical solution given by Boussinesq (1885). In addition, for higher stresses, following

the plastic regime, the normalized stresses tend to be uniformly distributed (Figure 10).

Analysis of the distribution of normalized stresses along the central axis of the footing shows that the numerical results of both simulations are generally similar to the theoretical solutions of Milovic (1992) and Boussinesq (1885) (Figure 11a). Specifically, the solution of Milovic (1992) assumes that the support layer is homogeneous and has a finite length of $3B$, whereas the solution of Boussinesq (1885) supposes such a homogeneous layer with an infinite length. Thus, in both numerical analyses, the results for the stresses in the elastic-linear regime show greater agreement with the solution of Milovic (1992). For stresses greater than 60% of q_{max} , the solution of Milovic (1992) still maintains agreement with the stresses measured in LT-NUM-NAT, while in the case of LT-NUM-FLO, the solution of Boussinesq (1885) is the closest to the numerical results in the elastoplastic regime (Figure 11b). These results agree with those found by

Useche-Infante et al. (2021) that employed experiments and numerical models, observing a graded reduction of the stresses in depth below the footing that become unimportant for depth greater than $2B$.

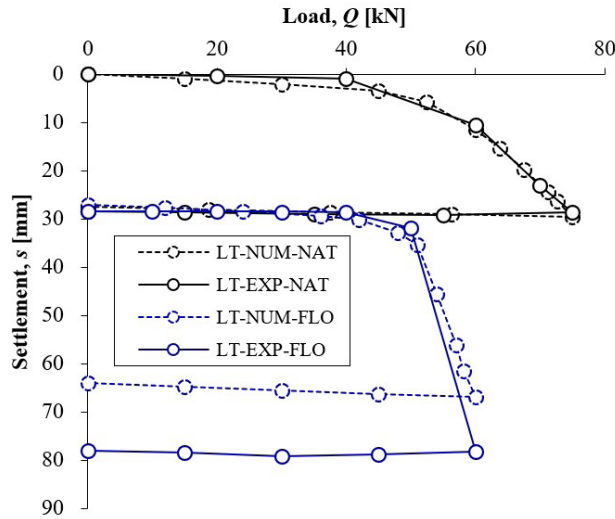


Figure 9. Load-settlement curves of numerical loading tests.

3.4 Upgraded Décourt (1999) method

From Equation 7, the settlement is estimated when the soil is flooded for a current overconsolidation ratio, $OCR_{(u_w=0,e)}$, considering the yield stress, p_p , obtained by Equation 3. Therefore, for the water content condition in which u_w is equal to 0 kPa, this stress is equal to 25 kPa because before the loading test under natural conditions, the soil is subjected only to the stress of its own weight and to the soil suction effect, which is obtained by Equation 3, considering $u_w = 30$ kPa. Therefore, the yield stress, p_p , is 60 kPa and corresponds to the effective stress (p_v). Thus, the value of $OCR_{(u_w=0,e)}$ calculated is 2.4, approaching 2.

Under the flooded soil conditions, the value of the effective stress experienced by the soil is equal to the maximum stress in the load test under the natural conditions ($q = 95$ kPa) added to the stress due to the weight of the soil, which transformed to the triaxial state of stresses, is approximately 98 kPa. In this case, the over consolidation ratio, $OCR_{(u_w=0,e)}$, is equivalent to 4.

According to the geotechnical profile, the N_{72} of the layer on which the footing rests is equal to 1. As the analyzed soil contains several fractions, the reference rate, q_r , is considered equal to $90 \cdot N_{72}$.

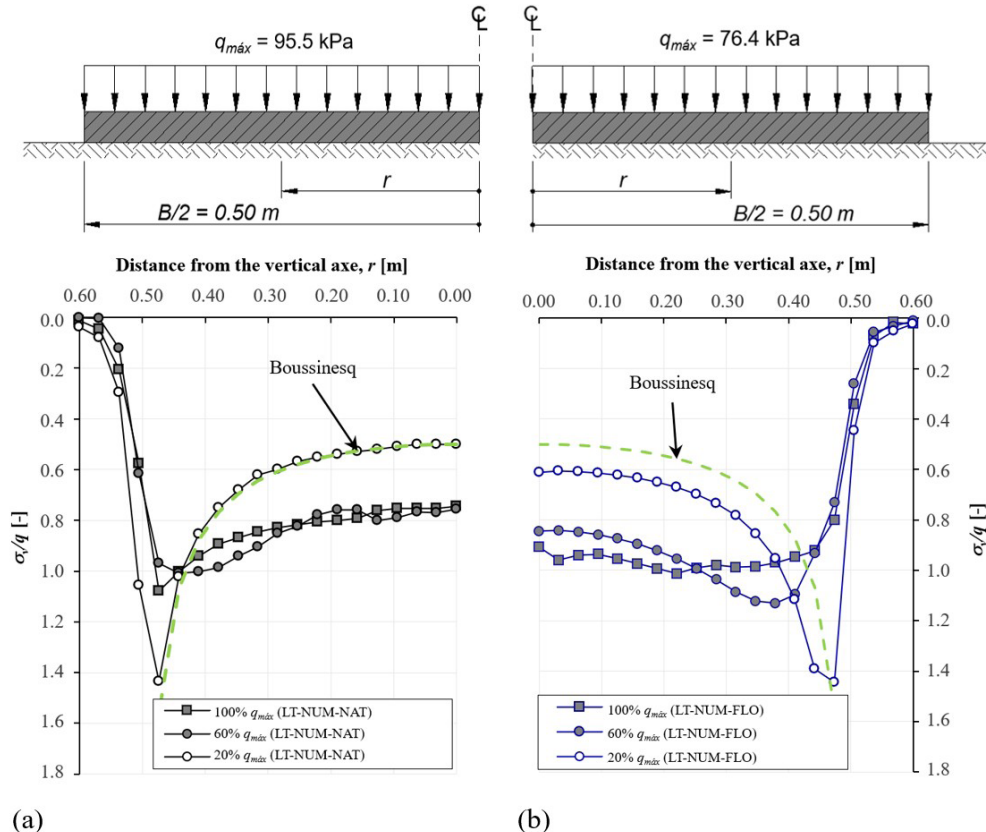


Figure 10. Contact stresses below the circular footing: (a) under natural conditions for the indicated stress levels; (b) under flooded conditions for the indicated stress levels.

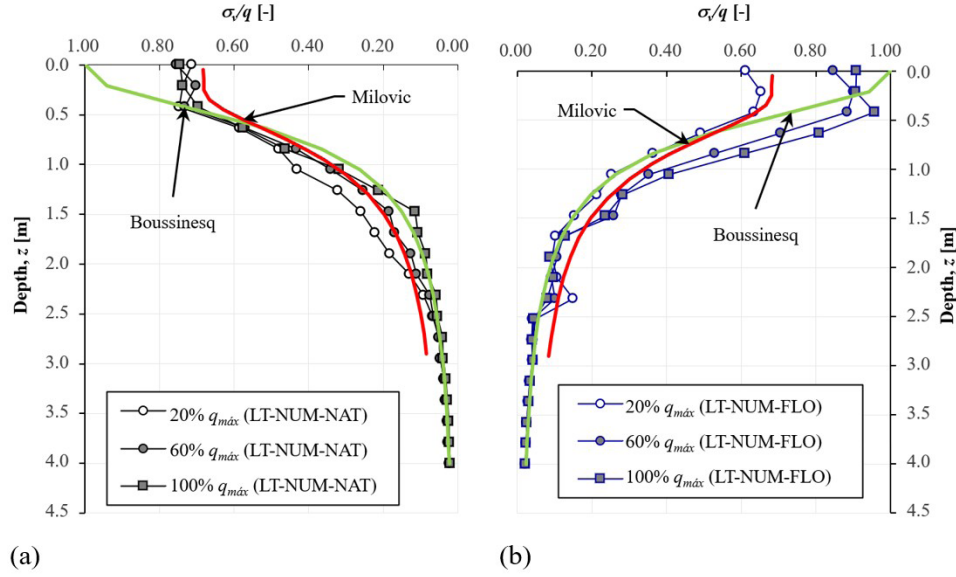


Figure 11. Comparison of the distribution of vertical stresses with depth along the center of the footing: (a) in LT-NUM-NAT; (b) in LT-NUM-FLO.

Table 4. Parameters for the proposed formula.

Soil conditions	$q_r = 90 \cdot N_{\gamma_2}$ [kPa]	u_w [kPa]	$OCR_{(u_w=0,e)}$ [-]	B_{eq} [m ²]	ϕ'_{peak} [°]	N_{γ} [-]	α [-]	$\alpha u_w \tan \phi'_{peak} N_{\gamma}(\phi'_{peak})$ [kPa]
Natural	90	30	2	0.89	21	6.2	0.45	32
Flooded	90	0	4	0.89	21	6.2	0.45	0

The angle of the peak friction (ϕ'_{peak}) used to consider the state of compactness of the material by means of dilation and used to calculate the factor $N_{\gamma}(\phi'_{peak})$ is 21°, given by Terzaghi & Peck (1963), according to Table 4.

The load vs. settlement curves are built with the method originally presented by Décourt (1999) and the modified approach addressed in this article, taking into account the soil conditions and characteristics of the site. Note that in the curve obtained with Décourt (1999), the term $OCR_{(u_w=0,e)}$ was assumed to be 1 since it is normally consolidated soil, without changing the value of the constant (C) defined by the author as equal to 0.426.

In Figure 12, the curves drawn with the modified method of Décourt (1999) agree with the points obtained with LT-EXP-NAT and LT-EXP-FLO. This agreement corroborates the observations reported in Terzaghi et al. (1996) regarding the general appearance of the load vs. settlement curves for loose or dense soils.

3.5 Comparison between the allowable stress results

The admissible stress results obtained by different approaches (experimental, analytical, and empirical) are compared using the settlement criterion derived from the Building Code of

the City of Boston, where the nominal values are 10 mm and 25 mm. Such methodology is recommended by Terzaghi et al. (1996) to define the allowable stress in soils that do not show clear failure, as is the case of the curve displayed in the load test under the natural conditions (Figure 13).

The two series plotted with the modified method of Décourt (1999) were calculated with the term $OCR_{(u_w=0,e)}$ equal to 2 to eliminate the effect of the remaining compaction due to the load test under the natural conditions. Together with results from the modified proposal of Décourt (1999), the points obtained in the loading tests are presented as well as the results obtained by Terzaghi & Peck (1963) with the suggestions given by Briaud (2013) for unsaturated soils.

Regarding the factor of safety (FS), to the stresses corresponding to a settlement equal to 25 mm (σ_{25mm}), an FS equal to 2 was applied, and the same FS was applied to the values of the allowable stresses (q_a) calculated by the analytical proposal of Terzaghi & Peck (1963) with Briaud (2013). For the stresses corresponding to a settlement equal to 10 mm (σ_{10mm}), no FS was applied.

The comparison of the predicted and measured results shows the effect of suction on the admissible load capacity of the footing, in addition to indicating the load gain with increased suction. However, for routine design, it is advisable to restrict

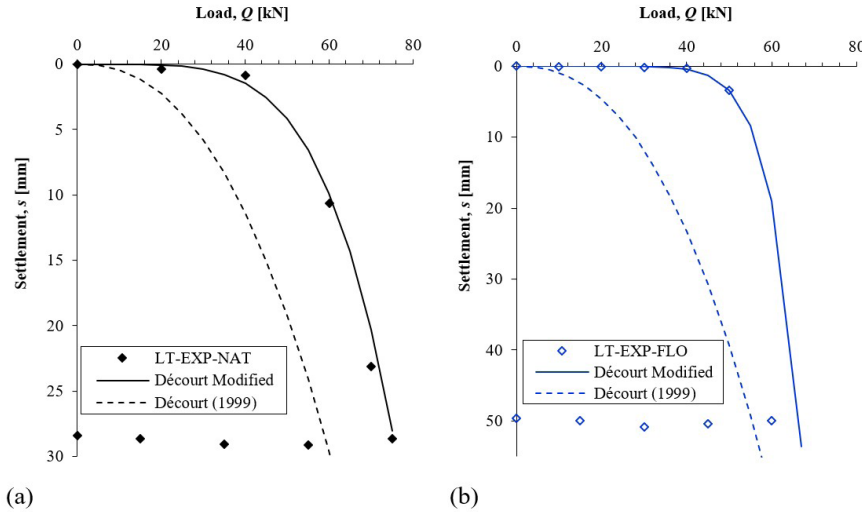


Figure 12. Observed load-settlement curve plotted with Equation 7 by the proposed method: (a) under natural condition; (b) flooded condition.

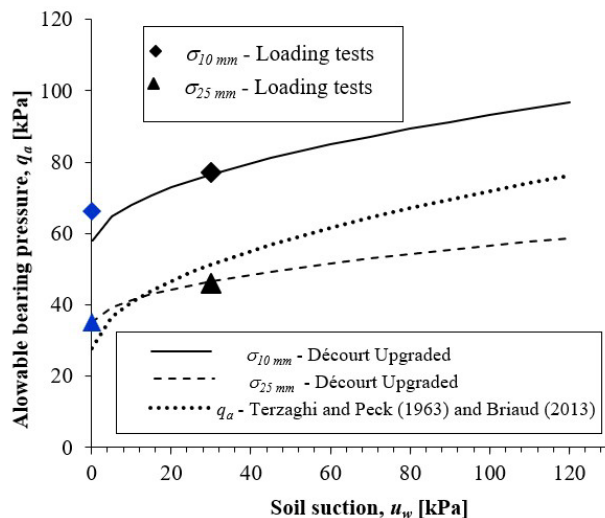


Figure 13. Relationship between soil suction and allowable bearing pressures based on settlement criteria.

this increase to the maximum suction values observed in the in situ, which, for the analyzed soil, did not exceed 40 kPa.

Regarding the admissible stresses, the values calculated by Terzaghi & Peck (1963) and Briaud (2013) are satisfactorily close to the values of σ_{25mm} predicted with the method proposed in this article, especially for suction values below 30 kPa. The stresses indicated by the modified proposal of Décourt (1999) present good agreement with the points obtained in the load tests. The main advantage of the method is the prediction of the load vs. settlement behavior of footings in this type of soil.

4. Conclusions

In this study, aspects inherent to the behavior of a footing on a tropical climate soil subjected to axial compression were

evaluated by means of static and slow load tests, initially in the in-situ condition and later in the flooded condition. Analytical and numerical models were used to predict behavior and extrapolate analyses from field tests.

The loading tests are fundamental for analyzing the behavior of the footing, as well as for validating the numerical model for extrapolation of the results of the studied foundation.

The numerical model used allowed to simulate with high similarity the load-settlement behavior of the footing tested in loading test, in the different conditions of water content and preload imposed in the field tests.

Décourt's (1999) adjusted method, proposed in this article, made it possible to estimate the load vs. settlement curve of the tested foundation with significant precision when compared to the result obtained in loading tests carried out in the field, since the parameters of water content and overconsolidation ratio were incorporated into the original equation.

Acknowledgements

This work was supported by the Coordination for the Improvement of Higher Education Personnel (CAPES) [finance code 001]; the Minas Gerais Research Funding Foundation (FAPEMIG); the Federal University of Uberlândia (UFU) for the acquisition of the software license used in this article. The authors report there are no competing interests to declare.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Eduardo Augusto dos Santos Oliveira: conceptualization, formal analysis, methodology, writing. Jean Rodrigo Garcia: conceptualization, funding acquisition, supervision and review.

Data availability

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

Declaration of use of generative artificial intelligence

This work was not prepared with the assistance of Generative Artificial Intelligence (GenAI).

List of symbols and abbreviations

c	Apparent cohesion intercept
c'	Effective cohesion
c_d	Design value compressive strength of concrete
e_0	Void ratio under natural moisture conditions
e_c	Void ratio after flooding under a vertical pressure of 200 kPa
f_{ck}	Characteristic compressive strength of concrete
$f_{ct,d}$	Design value of the tensile strength
p	Mean stress
p_p	Yield stress for the triaxial stress state / Yield mean stress under flooded condition
p_v	Highest mean stress applied to the soil layer
q	Load applied in soil by footing
q_a	Allowable stress
q_r	Reference stress
q_u	Ultimate bearing capacity
s	Settlement
u_w	Air entry into the soil
u_{wae}	The suction present in the soil
ABNT	Associação Brasileira de Normas Técnicas
ASTM	American Society for Testing and Materials
B	Footing width
B_{eq}	Dimension equivalent to that of a square footing
BSI	British Standards Institution
C	Compressibility parameter equal to 0.426
CAPES	Improvement of Higher Education Personnel
C_c	Compression index
CEMSF	Experimental Field Mechanics of Soils and Foundations
CODMIG	Companhia de Desenvolvimento Econômico de Minas Gerais
CP	Collapse index
CPMTC	Centro de Pesquisa em Mineração e Tecnologia

C_r	Recompression index
E	Young's modulus
EHE	Instrucción del Hormigón Estructural
E_{oed}	Oedometer modulus
E_{ur}	Young's modulus of the soil during loading and unloading
FAPEMIG	Minas Gerais Research Funding Foundation
FEM	Finite Element Method
FS	Factor of Safety
GWL	Groundwater Level
IGC	Instituto de Geociências da UFMG
LT-EXP-FLO	Experimental Loading Test Flooded
LT-EXP-NAT	Experimental Loading Test in Situ
M	Parameter dependent on the stresses acting on the material
M_f	Parameter dependent on the stresses acting on the material
N_c, N_γ	Bearing capacity factors
$N_\gamma (\phi \text{ peak})$	Bearing capacity factor using effective peak friction angle
OCR	Overconsolidation Ratio
$OCR_{(uv=0,e)}$	Overconsolidation ratio between the highest effective vertical stress applied to the soil layer and the corresponding yield stress in the oedometer compression curve under flooded conditions for an equal void ratio
Q	Pressure applied
S_c, S_γ	Shape factors
SC-SM	Clayey Sands – Silty Sands
SML	Slowly Maintained Loading Test
SPT	Standard Penetration Test
SWCC	Soil Water Characteristic Curve
UFMG	Universidade Federal de Minas Gerais
UFU	Universidade Federal de Uberlândia
α	Relationship between the suction in the soil and the air entry value
β, μ	Coefficients
κ	Slope of swelling line
λ	Slope of normal consolidation line
ν	Poisson ratio
σ_1	Major principal stress
σ_{10mm}	Stress corresponding to a settlement equal to 10 mm
σ_{25mm}	Stress corresponding to a settlement equal to 25 mm
σ_3	Minor principal stress
σ_{cd}	Design value compressive strength of concrete
σ	Normal stress
σ_p	Yield stress
τ	Shear strength
τ_{md}	Tensile strength of the material
ϕ'_{cv}	Constant volume friction angle
ϕ'_{peak}	Peak friction angle
ψ	Soil suction

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Comparison of reconstitution methods for mine tailings materials

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Article

Keywords

Reconstitution methods
Compaction curves
Specimen uniformity
Shear strength
Fabric

Abstract

The paper studies the effect of sample reconstitution for three commonly employed procedures, namely static compaction and standard and modified moist tamping methods, in the context of laboratory testing of mine tailing materials. The effect of initial water content and compaction energy on the resulting densities and uniformity of the samples are examined. Equivalencies between different reconstitution methods were established to attain a given target density. Then, the effect of the resulting fabric was investigated by means of isotopically consolidated undrained triaxial tests, on samples prepared with similar densities and water contents but with different reconstitution techniques. The results showed that all reconstitution techniques studied produced samples with acceptable uniformity and good repeatability. Although similar results were obtained across the different methods, the modified moist tamping technique yielded slightly higher undrained peak strengths and exhibited a more pronounced hardening behavior upon reaching the critical state line compared to the static compaction method. In contrast, the moist tamping and static compaction methods produced comparable responses in loose specimens. These results suggest that dynamic and static reconstitution methods may create subtle differences in the fabric of the samples, influencing their behavior under shearing.

1. Introduction

Reconstitution techniques to obtain good-quality specimens for triaxial testing can be challenging depending on the type of material. They are necessary to evaluate the behavior of soils where good quality undisturbed samples are difficult to obtain or the techniques necessary to obtain them are prohibitively expensive; this problem is common in sands, gravels, and some mine tailings. Therefore, many researchers rely on preparing reconstituted samples, with similar properties to the in-situ material, that can be used to obtain engineering parameters. In practice, the most used reconstitution methods (RM) are moist tamping (Castro, 1969; Ladd, 1978), static compaction (Serratrice, 2022; Venkatarama & Jagadish, 1993; Alkiki et al., 2021), air and water pluviation (Miura & Toki, 1982), and slurry deposition (Carraro & Prezzi, 2008). However, although reconstituted specimens can reproduce in situ volumetric conditions (dry or wet density and void ratio), it is much harder to reproduce the structure of the material (mainly fabric in the case of uncemented soils) (Corrêa & Oliveira, 2019) where water

content and the compaction method and energy play an important role (Lambe & Withman, 1969; Sloane & Kell, 1966). Significant effects due to soil's structure have been reported in static or cyclic triaxial tests (Yang et al., 2008; Vaid & Sivathayalan, 2000; Miura & Toki, 1982; Carraro & Prezzi, 2008), although critical state conditions appear to be independent of the initial state, fabric, and testing conditions (Fonseca et al., 2022; Verdugo & Ishihara, 1996; Fourie & Tshabalala, 2005).

It has been identified that the moist tamping method generates a greater contractive response in loose soils with fines compared to other reconstitution methods (Vaid et al., 1999; Thevanayagam et al., 2002; Corrêa & Oliveira, 2019). Some authors also suggest that subaqueous deposition techniques, such as slurry deposition and water pluviation, can better reproduce the in situ soil's structure and the stress-strain response of liquefiable sand deposits, with or without fines, such as alluvial deposits, hydraulics fills, and mining waste deposits (Carraro & Prezzi, 2008), while static compaction and moist tamping can reproduce structures similar to rolled materials (Kuerbis & Vaid, 1988).

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Submitted on April 13, 2024; Final Acceptance on January 14, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.002624>



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One major issue of reconstituted specimens is the possible lack of uniformity, where the lower portion tends to become denser due to the compaction of each successive layer. Consequently, several techniques for evaluating the uniformity of specimens have been reported in the literature, such as digital image analysis (Al-Shibli et al., 1996), X-ray computed tomography (Lee, 1994; Mooney, 2002; Thomson & Wong, 2008), interlayer dyeing (Flores, 2008) and specimen solidification with gelatine (Emery et al., 1972), where variations in the void ratio within the specimen have been identified, that depend on the reconstitution method (Mulilis et al., 1977; Vaid & Negussey, 1988; Vaid & Sivathayalan, 2000; Frost & Park, 2003).

This study aims to compare three commonly employed reconstitution methods in the context of laboratory testing of mine tailing materials. Particularly, the static compaction and the standard and modified moist tamping methods were addressed in this research. Results on specimen uniformities, compaction curves, and isotropic consolidated undrained (CIU) triaxial tests are presented, for different initial void ratios and water contents. They provide relevant insights into the performance of the adopted reconstitution techniques, particularly on the resulting undrained behavior of the samples under shearing, as well as identified limitations and shortcomings.

It is important to note, however, that this study does not address the representativeness of samples prepared using the methods studied here in relation to field conditions. This remains an open problem in geotechnical engineering, which has gained renewed attention following recent large-scale tailing dam failures (Morgenstern et al., 2016; Robertson et al., 2019; Arroyo & Gens, 2021). For hydraulically deposited tailings, water pluviation may appear to be a suitable method for approximating field conditions. However, it is virtually impossible to consistently obtain contractive specimens that exhibit undrained softening (i.e. flow liquefaction) using pluviation techniques (Fourie & Tshabalala, 2005), contradicting the accepted explanations for observed large-scale failures. Furthermore, even when “undisturbed” sampling is feasible, brittleness tends to be underestimated due to the small deformations required to mobilize peak undrained strengths (Reid et al., 2024), which can be attained during sampling. Therefore, at least for now, the preferred reconstitution method is one that offers the greatest control over the initial void ratio, rather than one that best replicates

the in-situ depositional environment (Fourie et al., 2022). On the other hand, for artificially compacted tailings, as in the case of dry stacking (Cacciuttolo Vargas & Pérez Campomanes, 2022), a stronger case can be made for the representativeness of a given reconstitution technique in approximating the compaction method used in the field. In any case, the authors believe that further research on this topic is certainly required.

2. Materials

Two mine tailings materials were considered for this study. The first one (M1) comes from a mine located in the State of Chihuahua, Mexico, while the second one (M4) was obtained from a tailing storage facility (TSF) located in Minatitlán, State of Colima, Mexico. These materials are the byproduct of crushing and processing to obtain silver, gold, and iron. Some physical properties are summarized in Table 1. They exhibit high fine contents, 82 and 44% for M1 and M4, respectively, with basically null plasticity and, therefore, they were classified as ML and SM according to the Unified Soil Classification System (USCS). Figure 1 shows the particle-size distribution of their coarse fraction.

For material M1, specimens were reconstituted using static compaction and modified moist tamping methods. The resulting specimens were assessed through visual inspection, uniformity evaluation, and compaction curves.

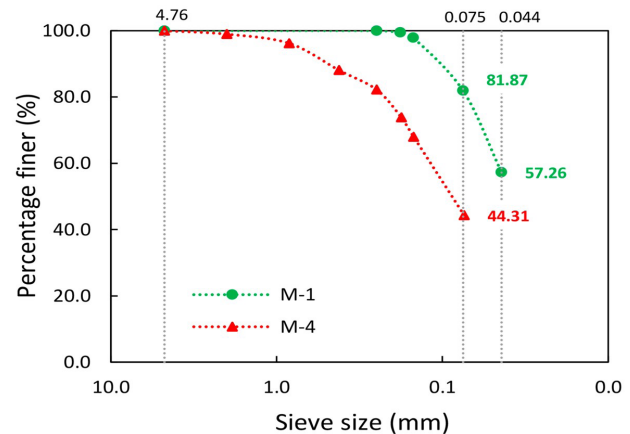


Figure 1. Particle-size distribution for mine tailing M1 and M4.

Table 1. Properties of the mine tailings used in this study.

Sample	Specific gravity, G_s	Liquid limit, LL	Plastic limit, PL	< 0.074 mm	< 0.044 mm	Max. void ratio, e_{u}^a	Min. void ratio, e_{u}^a
-	-	%	%	%	%	-	-
M1	2.67	26.90	-	81.87	57.26	1.57	0.94
M4	3.19	24.87	-	44.31	-	1.89	0.72

^aUsing the methodology proposed by Lade & Yamamuro (1997).

Their behavior under shearing was examined using CIU triaxial compression tests at different initial densities. Similarly, for material M4, specimens were reconstituted with static compaction and moist tamping methods. These specimens also underwent visual inspection and compaction curve analysis, and their shear behavior was assessed through CIU tests under loose conditions.

3. Specimen reconstitution methods

3.1 Static compaction (SC)

The equipment consists of a split mold, usually with an internal diameter of 3.6 cm and height of 8.6 cm, and a tamper, 3.4 cm in diameter, with adjustable mass ranging from 0.2 kg to a maximum of 24 kg (Figure 2). Specimens are compacted in 5 to 10 layers, by placing the tamper on each layer for 10 seconds and repeating the process for subsequent layers. The amount of total dry material ranges between 0.08 and 0.2 kg, and the amount of material for each layer depends on the adopted mass of the tamper and water content. To ensure uniform densities, an undercompaction approach can be adopted (Ladd, 1978) using smaller amounts of soil or changing the mass of the piston for each layer from bottom to top. Generally, using different amounts of soil for each layer is preferred, since it allows for a rapid sample preparation without the need for adjustments in the tamper mass.

3.2 Moist tamping (MT)

The moist tamping method is similar to SC, where a split mold, in this case 3.6 cm in diameter and 8.6 cm in height, is used to compact the soil in layers (from 5 to 10) by means of a tamper of adjustable mass. However, the tamper has a reduced diameter, in this case of 1.5 cm. Therefore, each layer is tamped several times, according to the sequence shown in Figure 3.

The dimensions of the mold and the tamper used here are based on those reported by Ladd (1978). As in the SC method, the amount of dry material for each layer depends on the adopted mass of the tamper and the water content.

3.3 Modified moist tamping (MMT)

Characteristics of the modified moist tamping method are also comparable to the SC method, where similar dimensions of the split mold and the tamper are adopted (Figure 4). However, in this case, compaction energy is provided by dropping a hammer onto the tamper from a specific height. The tamper is generally made of acrylic, which is attached to a steel rod that guides the fall of the hammer (see Figure 4). The rod has an adjustable stop to control the drop height. The drop height, the mass of the hammer, and the number

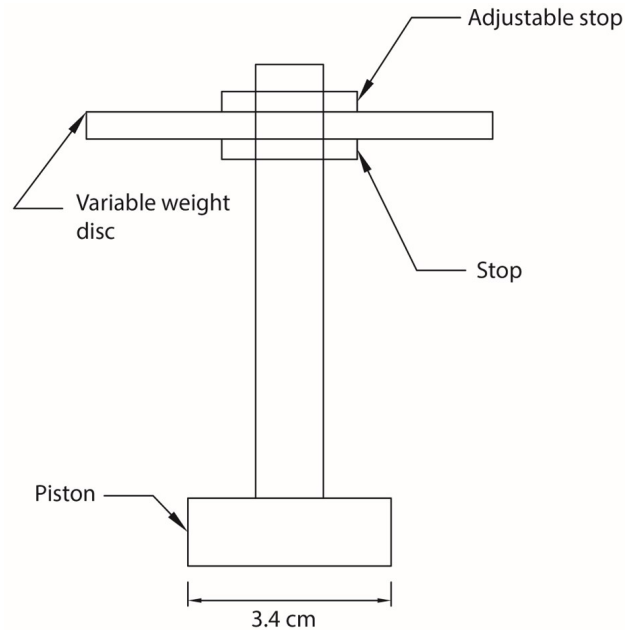


Figure 2. Tamper used in the static compaction method.

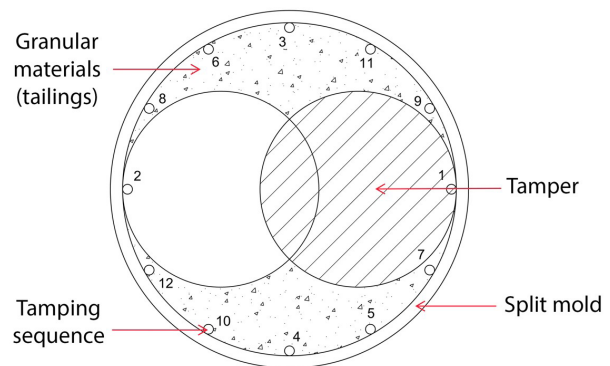


Figure 3. Tamping sequence in the moist tamping method.

of blows can be adjusted to control the compaction energy. The sample is usually compacted in five layers. As in the previous methods, an undercompaction approach can be adopted to improve uniformity, for instance adjusting the drop height (Bradshaw & Baxter, 2007).

3.4 Recommendations for reconstitution

From the application of the aforementioned reconstitution methods, some general recommendations can be drawn regarding its application for mine tailing materials:

- For loose specimens, it is crucial to maintain water content below 15% to prevent the formation of porous and non-uniform structures. As identified by the authors, the maximum water content should be approximately 4 to 5% less than the liquid limit to

avoid water leakage from the mold's bottom during the compaction process;

- Layer thickness larger than 25 mm should be avoided for molds with diameters less than 10 cm (Ladd, 1978);
- Compaction curves should be obtained before selecting a water content for reconstitution, especially if in situ parameters like void ratio or unit weight are not known;
- Density variations of each layer must be identified if an undercompaction strategy is required;
- A visual inspection of the different specimens must be performed;

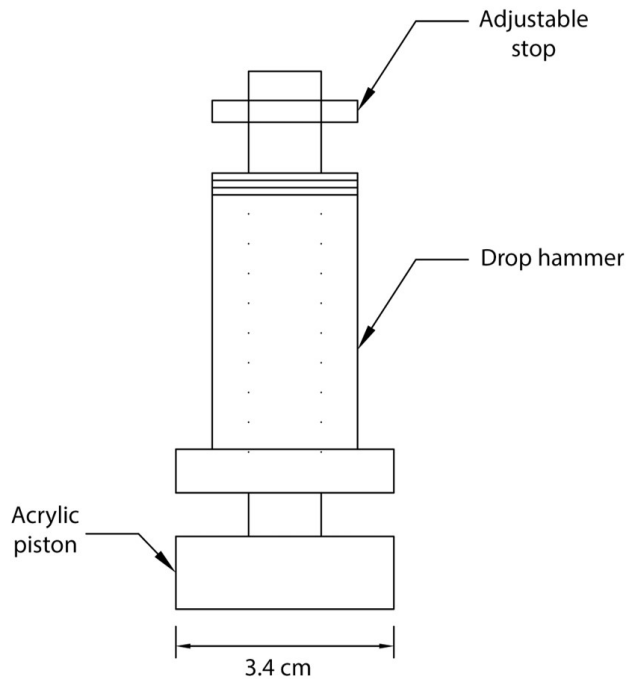


Figure 4. Tamper used in the modified moist tamping method.

- Water contents for reconstitution are generally around 5% for sands and are usually higher for silty and clayey soils. This is due to different water retention capacities, permeabilities, and particle sizes;
- For low water contents, drop heights higher than 3 cm cause material lifting in the MMT method;
- Acetate sheets should be used to prevent the material from adhering to the mold; this problem becomes more significant as the water content is increased;
- Scarification of the top of each layer is required before placing the next to improve friction between the layers;
- Use of a small spirit level during compaction to avoid non-uniform placement, which can generate uneven compaction;
- When very dense specimens are needed, MMT can be more effective using larger hammer masses and drop heights. In the SC method, tamper masses larger than 12 kg make the procedure impractical.

It is also important to mention that the adopted methods did not produce considerable particle crushing during compaction, at least for the masses and drop heights adopted. This was verified by determining the particle-size distribution before and after reconstitution in the most critical cases considered, i.e. a piston mass of 3 kg for the SC method and a hammer mass of 0.127 kg and a drop height of 4 cm for the MMT method. The obtained differences between the two particle-size distributions were only about 1%.

4. Densities achieved

4.1 Static compaction (SC)

Compaction curves from the SC method, for different water contents (w) and tamper masses (TM) are shown in Figure 5a and Figure 5b for materials M1 and M4, respectively.

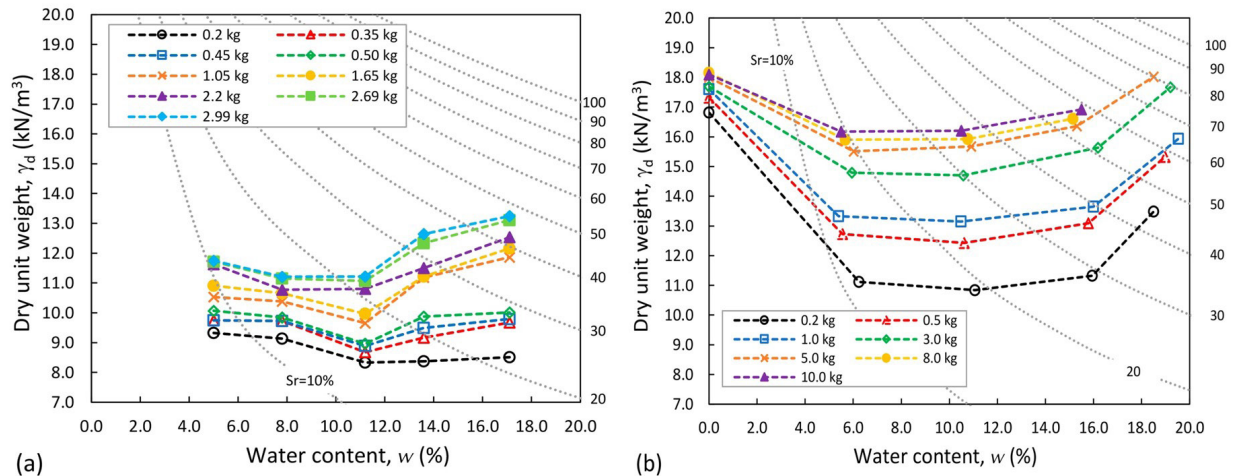


Figure 5. Compaction curves with the SC method for different tamper masses for materials (a) M1 and (b) M4.

The curves have a non-typical shape, where a clear peak value is not obtained within the range of water contents considered; this behavior is common for static methods, and similar findings have been reported by Honda et al. (2003). The initial decrease in the dry unit weight γ_d , observed for both materials, can be attributed to the formation of water menisci between soil grains (thus the development of matric suction) that hinders the rearrangement of particles and favor agglomerations (Mitchell & Soga, 2005).

Water contents selected for specimen fabrication affect the appearance of the specimens (see Figures 6 and 7). High water contents result in non-uniform and porous specimens, regardless of the tamper mass used (Figures 6b and 7b). Conversely, lower water contents, around 5% in this case, produce much more uniform samples (Figures 6a and 7a). Therefore, it is important to consider this factor when selecting a water content for specimen reconstitution for triaxial testing to ensure more uniform specimens.

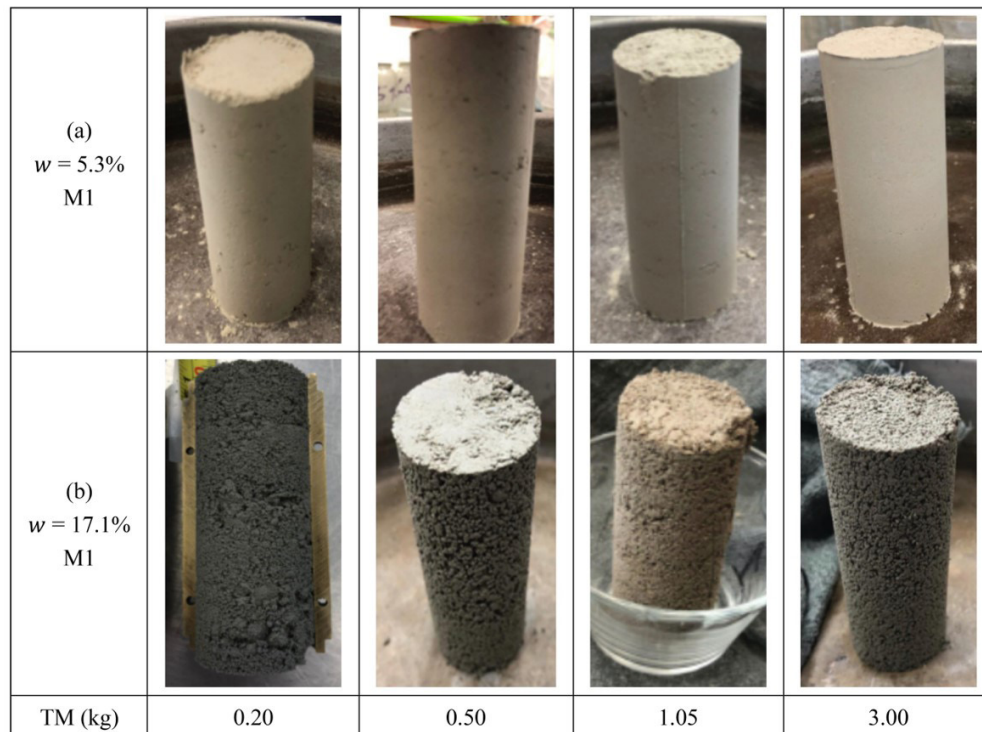
4.2 Modified moist tamping (MMT)

In this case, only material M1 was considered. A constant mass of the hammer was adopted and variations in the drop height and the number of blows were explored. Specimens were compacted in 5 layers. Table 2 summarizes the considered variables and their corresponding compaction energies. Similar compaction energies were obtained by varying the

number of blows and the drop height to investigate whether there is any influence in the way energy is delivered with respect to the achieved dry unit weight γ_d . Water contents between 5 and 12% were considered, based on the results obtained from the SC method.

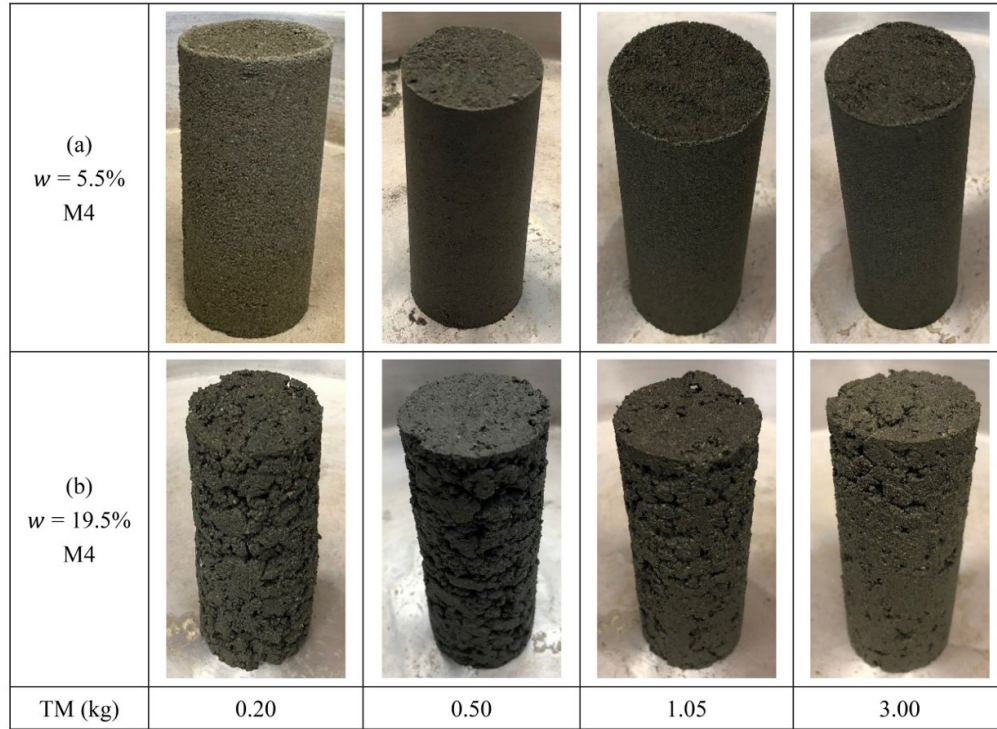
Figure 8a shows the influence of the number of blows N and drop height h on the achieved dry unit weight for $w = 5.3\%$. It can be identified that, for a given drop height, there is a certain number of blows where the effect on γ_d starts to decrease. Similarly, the effect of increasing the drop distance seems to decrease significantly for values larger than 3 cm in this case, at least for a number of blows of 10 or below. However, as shown in Figure 8b, the achieved dry unit weight seems to be approximately independent of the specific combination of N and h , and it is solely a function of the compaction energy E_c .

As shown in Figure 9, this latter conclusion seems to hold for different water contents, where a unique relation between compaction energy and dry unit weight can be identified for each of the water contents considered. Obtained results are presented in Figure 10 in terms of compaction curves for different compaction energies, the latter obtained with different combinations of N and h . As expected, the curves move upward as the compaction energy is increased. They show the typical shape resulting from dynamic compaction, where the dry unit weight first increases with the water content, up to a maximum value, and then decreases.



w : water content; TM: tamper mass.

Figure 6. Specimens with different water contents reconstituted with the SC method for material M1.



w: water content; TM: tamper mass.

Figure 7. Specimens with different water contents reconstituted with the SC method for material M4.

Table 2. Adopted variables and compaction energies used in the MMT method.

Number of blows, N	Drop height, h	Tamper mass, TM	Compaction energy, E_c
-	cm	kg	(kJ/m ³)
5	1	0.128	3.42
10	1		6.84
25	1		17.10
5	2		6.84
10	2		13.68
25	2		34.20
5	3		10.26
10	3		20.52
25	3		51.30
5	4		13.68
10	4		27.36
25	4		68.4

4.3 Equivalence between SC and MMT

To achieve specimens with the same void ratio (e) using both the SC and MMT methods, an equivalence can be established. Figure 11a shows a dual y-axis graph to obtain a certain void ratio as a function of the tamper mass (3.4 cm tamper diameter) for SC (black y-axis) and compaction energy for MMT (red y-axis). From this figure, it can be determined what tamper mass for SC results in a specimen with an equal void ratio corresponding to a certain compaction energy from the MMT method.

To obtain equal void ratios in the SC and MMT methods, the tamper mass and the compaction energy show the relation depicted in Figure 11b, which can be approximated with the polynomial function shown. These results were obtained for M1 with void ratios between 1.2 and 1.8 and for $w = 5.3\%$.

4.4 Equivalence between SC and MT

Before performing the triaxial tests, it was necessary to establish an equivalence between the MT and SC methods to achieve the target density. Trials indicated that a tamper

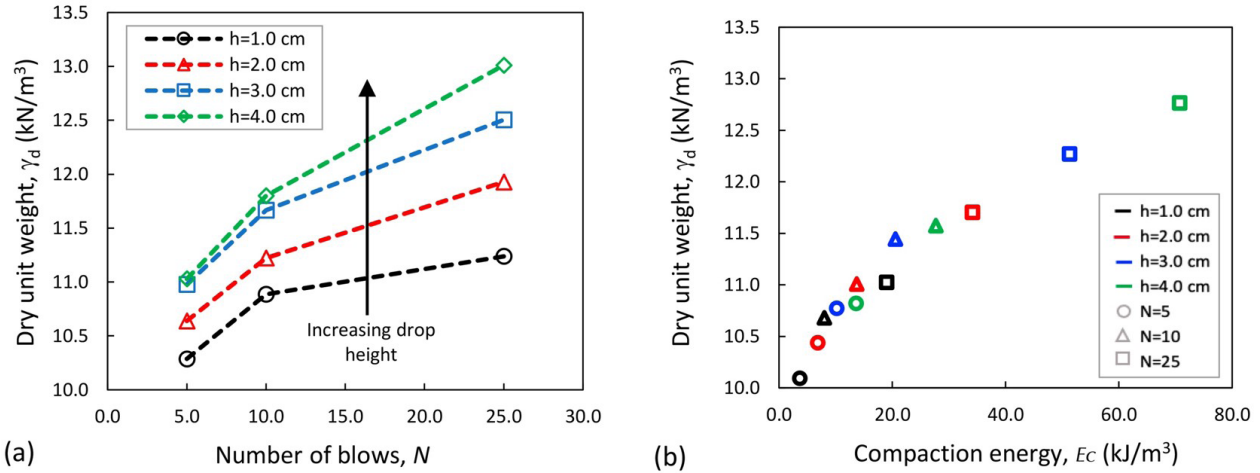


Figure 8. Effect of the (a) drop height h and number of blows N and (b) of the compaction energy on the achieved dry unit weight for $w = 5.3\%$ with the MMT method.

mass of 0.5 kg for the MT method was equivalent to 2.0 kg for the SC method. The compaction curves for reconstituted samples of material M4, shown in Figure 12 are similar and follow the same trend observed with the SC method.

4.5 Interlayer density variations

Simple and inexpensive procedures can be used to verify the uniformity of the reconstitution procedure. Because the most important variations generally occur in the lowest portion of the specimen, a measure of uniformity can be obtained by comparing the density (or void ratio) of the first layer with respect to the average density of the sample. This is achieved by placing a spacer to separate the first layer, allowing for accurate measurement of both the weight and dimensions of the two portions (Figure 13b), thereby enabling independent density determination for each of them. This procedure was applied in selected samples of the material M1, where differences of about 2.5% and 3% with respect to the average density were identified for both the SC and the MMT methods, respectively, on specimens with a void ratio of around 1.38

Another approach consists of dyeing the soil of alternate layers and reconstituting the sample within a translucent mold, so that the thickness of each layer, and therefore its density, can be estimated (Figure 13a) (Flores, 2008). The layer thickness is generally measured at three orientations (0°, 120°, and 240°) so that an average value can be determined and used to estimate the density of each layer. Figure 14 illustrates the variation of void ratio for loose and dense specimens of material M1, prepared using both the SC and MMT methods. The values presented represent the average of two specimens, both showing the same trend. Generally, the lowest two layers tend to be denser. However, for the loose sample prepared using the MMT method and the dense sample using the SC

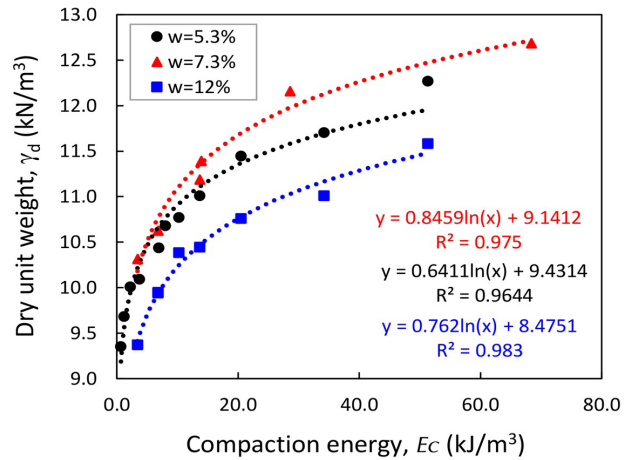


Figure 9. Effect of compaction energy and water content on the achieved dry unit weight with the MMT method.

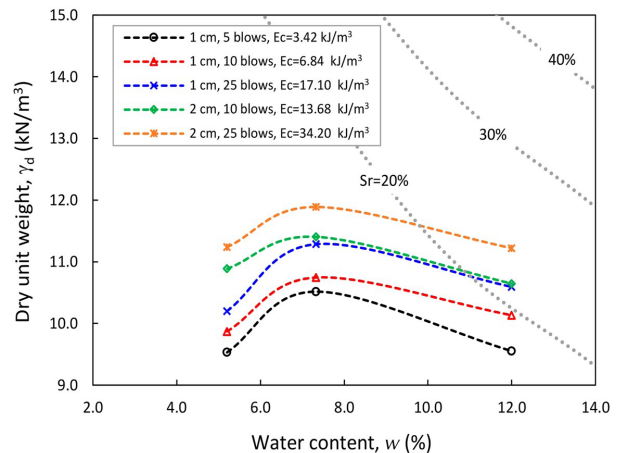


Figure 10. Compaction curves obtained with the MMT method.

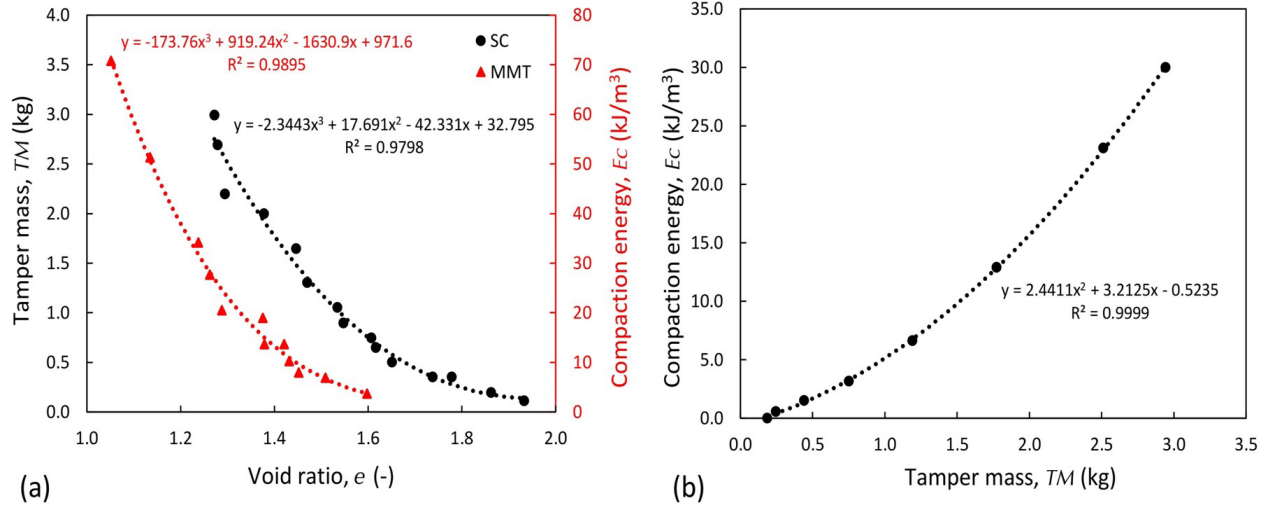


Figure 11. Equivalence between SC and MMT methods. (a) Effect of tamper mass or compaction energy on void ratio, and (b) equivalence between reconstitution methods.

method, the highest density (indicating the lowest void ratio) occurred in the second layer from the bottom. For the MMT method, this could be attributed to wave reflections at the rigid base during the compaction of the first layer, which may hinder efficiency. However, this explanation does not apply to the SC method, leaving a satisfactory explanation still pending. These results contrast somewhat with those reported by Frost & Park (2003), who observed the lowest density in the second layer when using the standard moist tamping method.

Only one specimen exhibited a local deviation in relative density of 10.6% (corresponding to a 7.6% deviation in void ratio) from the average. This occurred in one of the dense MMT samples, where the first layer reached a higher density. Such deviations are common when compacting granular materials in layers, as each successive layer can further densify the layer beneath (Ladd, 1978). Overall, however, deviations from the average remained typically under 7.9% (5% in terms of void ratio), which is usually deemed acceptable (Reid et al., 2024). However, when compared to water pluviation, where relative density variations are reported in the order of 2 to 3% (Vaid & Negussey, 1988), it is evident that the SC and MMT methods yield less uniform specimens.

5. Triaxial testing

5.1 Test procedure

A series of consolidated isotropic undrained (CIU) triaxial tests were conducted on specimens reconstituted with different methods, for different initial void ratios e and consolidation stress σ'_3 , according to the guidelines from ASTM D4767-11 (ASTM, 2020). As described in Section

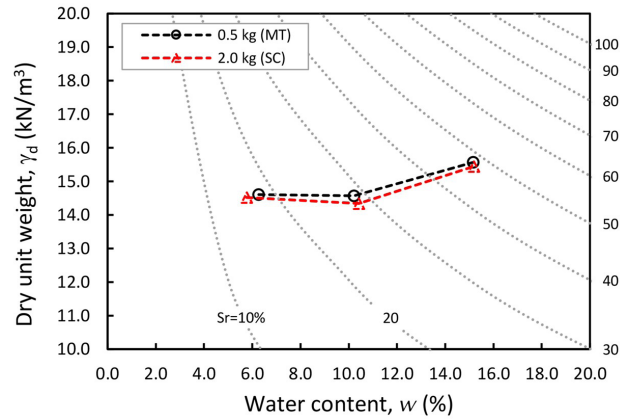


Figure 12. Compaction curves for MT and SC methods, sample M4.

3, specimens were reconstituted in a split mold of 3.6 cm in diameter and 8.6 cm in height, and compacted in 5 layers for MMT and SC, and in 10 layers for MT, using the appropriate tamper for each reconstitution technique.

Specimens were reconstituted with initial water contents ranging from 5% to 10%, as this range yielded the most uniform specimens (see Section 4). Different initial void ratios were considered to assess the effects of density variations among the reconstitution techniques studied. Tests were conducted at effective confining pressures of approximately 100 and 150 kPa for materials M4 and M1, respectively. This level of confinement is readily attained in any tailing storage facilities (TSF), yet it excludes the possibility of particle breakage (Wagner et al., 2024), which was not part of the scope of the present work.

Membrane correction and the use of thin membranes are crucial for very loose specimens, as thicker membranes

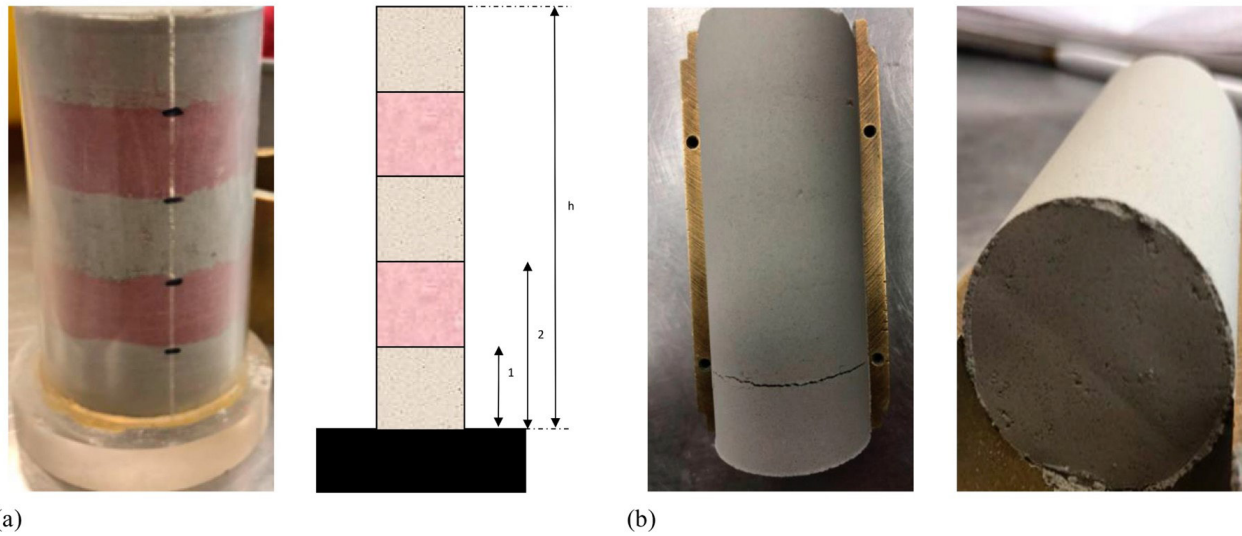


Figure 13. Evaluation of uniformity. (a) Interlayer dyeing method and (b) Last layer spacer method.

can adhere tightly to the specimen, potentially leading to unintended densification. According to ASTM D4767-11, the membrane thickness must not exceed 1% of the specimen's diameter (ASTM, 2020).

For saturation, carbon dioxide (CO_2) was first circulated through the specimen at a low pressure (~ 3 to 10 kPa) for 45 minutes. Since CO_2 is more soluble in water than air, this aids in the saturation process and prevents the use of high backpressures. Then, distilled and de-aired water was circulated at a pressure of 10 kPa (Fonseca et al., 2021). For material M4, a total volume of 170 ml was circulated through the sample, while for M1 the volume was approximately 100 ml. The latter lower value is because of the lower permeability exhibited by material M1, making this process more time-consuming. Finally, specimens were saturated using backpressure until achieving values of Skempton's (1954) B parameter of at least 0.95. Backpressure increments were applied maintaining a constant effective mean stress of about 10 kPa. Samples were sheared by applying a constant axial strain rate of 0.065%/min, which guaranteed pore-pressure equilibration under undrained conditions (ASTM, 2020).

5.2 Comparison between modified moist tamping (MMT) and static compaction (SC)

CIU tests were performed on specimens of material M1, prepared with the MMT and SC methods. Table 3 summarizes some characteristics of the tests performed. All specimens were prepared with an initial water content of 5.3% and for three different target densities (note that test numbers refer to the different density levels). In the case of the MMT method, different final void ratios were obtained using a constant mass of the hammer and varying the drop height and the number of blows, while for the SC method, different tamper

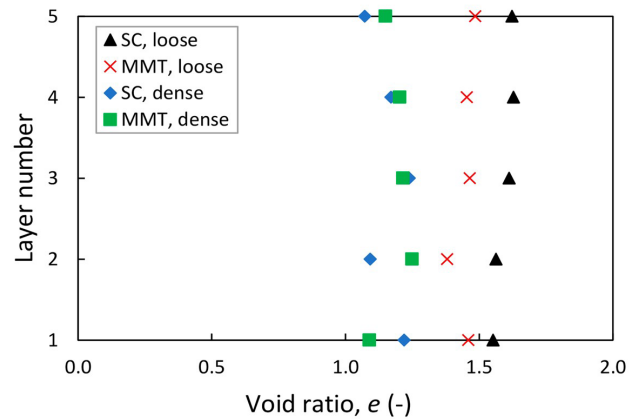


Figure 14. Variations of void ratio per layer with the SC and MMT methods.

masses were adopted. Similar void ratios were obtained with both methods for each density level, which allows for an objective comparison.

For the lowest density, both reconstitution procedures initially achieved void ratios higher than those typically found in situ for mine tailings and sands (> 1.4 ; see e.g. Oliveira et al., 2023). However, these specimens exhibited a significant densification during mounting and saturation. This densification of very loose specimens during the setup has been previously reported in the literature for these RMs (Sladen et al., 1985; Paz, 2015). Membrane effects are also quite significant for these very loose samples and, therefore, membrane correction (Kurbis & Vaid, 1990) is of paramount importance. They also tend to form a noticeable uneven diameter along their height, with the smallest diameter in the center of the specimen.

Table 3. Properties of samples for testing reconstituted with the MMT and SC methods.

Reconstitution method and test number	Tamper mass, TM	Drop height, h	Number of blows, N	Compaction energy, E_c	Layers	Water content, W	Consolidation stress, σ'_3	Initial void ratio, e_i	Final void ratio, e_f
-	kg	m	-	kJ/m^3	-	%	kPa	-	-
MMT-1	0.127	0.01	2	1.37	5	5.3	143.0	1.66	1.11
MMT-2	0.127	0.01	5	3.42			144.2	1.44	1.04
MMT-3	0.127	0.02	10	13.68			143.7	1.10	0.97
SC-1	0.350	-	-	-			140	1.69	1.13
SC-2	0.650	-	-	-			143.1	1.47	1.06
SC-3	2.90	-	-	-			144.1	1.08	0.95

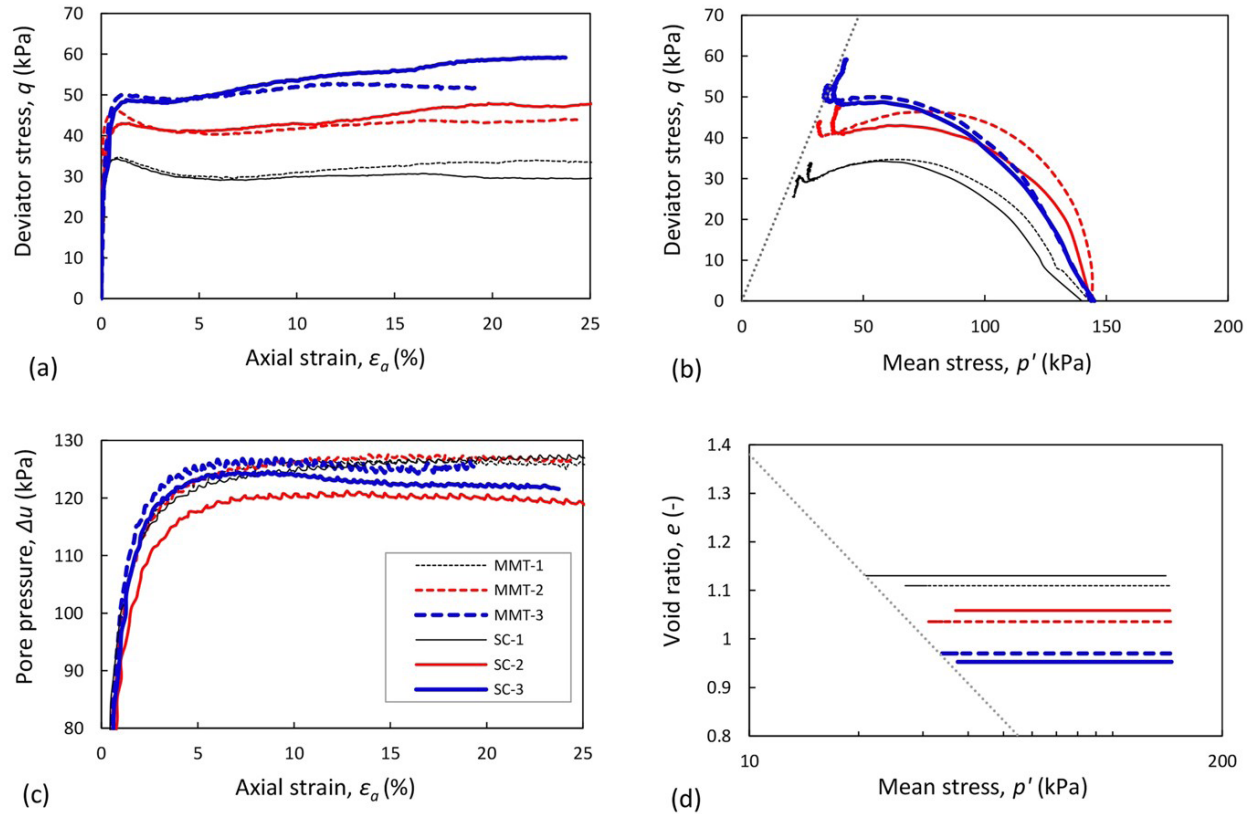
**Figure 15.** Results from undrained triaxial tests on samples reconstituted with the MMT and SC methods. (a) Stress-strain curves, (b) stress paths and, (c) pore-water pressure evolution, and (d) compression plane.

Figure 15 shows the results of the CIU tests in terms of (a) stress-strain curves, (b) effective stress paths, (c) pore-water pressure evolution, and (d) the compression plane. As expected, regardless of the RM or initial density, all samples reached, approximately, the critical state line (CSL) in the p - q (Figure 15b) and compression (Figure 15d) planes.

As previously mentioned, the CSL is known to be independent of the initial state of the sample and the reconstitution procedure (Fonseca et al., 2022). Therefore, under a similar effective confining pressure, there is a significant influence of the initial density, increasing the undrained strength as the void ratio is decreased. All samples showed an initial contractive

behavior, with the accumulation of excess pore-water pressure and, for the lowest and intermediate density levels, a limited softening response. However, on approaching the CSL, all samples showed a slightly dilatant behavior with hardening and, in some cases, with a subtle reduction of water pressures, except for the loosest test with the static compaction method SC-1, which showed a somewhat inconsistent behavior towards the end of the tests. All samples generated similar pore-water pressures, around 120 and 125 kPa, regardless of the initial density or RM. In general, samples from both methods showed similar behavior. However, for the two highest densities, the undrained peak strength before reaching

the CSL is somewhat higher with the MMT method, while the subsequent hardening response is significantly more noticeable in the case of the SC method. Since no particle crushing was identified in either RM, results suggest that, for similar densities and confinement pressures, the SC and MMT methods result in slightly different fabrics.

The loosest samples showed quite similar behavior between both RM methods, with virtually the same peak undrained strength. This might be due to the very small drop height and small number of blows (see Table 3), which makes the two procedures tend to be analogous.

5.3 Comparison between moist tamping (MT) and Static compaction (SC)

Two additional CIU triaxial tests were performed on specimens of the material M4 to assess differences between the MT and SC methods. Both specimens were prepared with an initial water content of 10% and tested under an effective confining pressure of 100 kPa. The same saturation procedure, described in Section 5.1, was also adopted

here. Table 4 summarizes some characteristics of the tests performed. It can be identified that quite similar void ratios were achieved for both reconstitution procedures, allowing for an objective comparison.

Figure 16 shows the obtained results in terms of (a) stress-strain curves, (b) effective stress paths, and (c) pore-water pressure evolution. Unlike the behavior of the tests in Figure 15, both samples exhibit flow liquefaction, with a distinct undrained peak strength and a subsequent abrupt softening response until reaching a total liquefied condition, i.e. a null effective mean stress. Furthermore, both tests showed quite similar behavior in all plots, only with a slightly smaller undrained peak strength in the case of the SC method. In general, it can be stated that, for similar densities and confinement pressures, both the SC and MT methods result in analogous samples. These results suggest that, for these tamper-based RMs, differences in fabric occur between static and dynamic compaction procedures and not between variants of the static methods.

It is important to note that the strain-stress curves (Figure 16a) and stress paths (Figure 16b) exhibit small

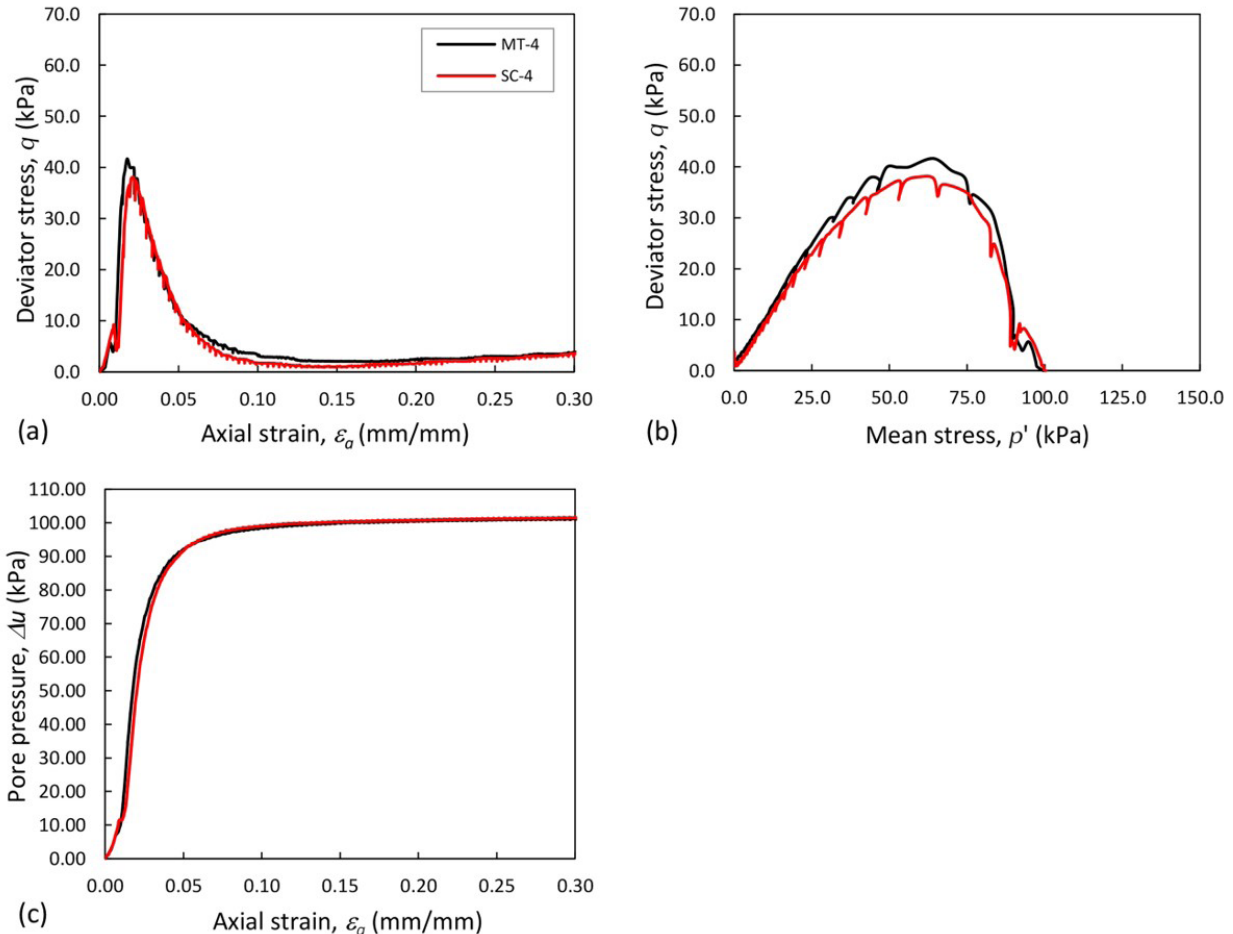


Figure 16. Results from undrained triaxial tests on samples reconstituted with the MT and SC methods. (a) Stress-strain curves, (b) stress paths and, (c) pore-water pressure evolution.

Table 4. Properties of samples for testing reconstituted with the MT and SC methods.

Reconstitution method and test number	Tamper mass, TM	Layers	Water content, w	Consolidation stress, σ'_3	Initial void ratio, e_i	Final void ratio, e_f
-	kg	-	%	kPa	-	-
MT-4	0.5	10	10.0	99.58	1.073	0.890
SC-4	2.0	10	10.0	100.33	1.105	0.899

oscillations. This behavior can be attributed to the use of a servo-controlled equipment, which enables highly precise data acquisition, with a load cell precision of around 2 kPa. The high resolution of the system highlights minor fluctuations, resulting in the observed oscillations.

6. Conclusions

This paper presented the results of an experimental program aimed to assess three commonly employed reconstitution procedures, namely the static compaction and the standard and modified moist tamping methods, in the context of testing mine tailing materials. The following conclusions can be drawn from this study:

- In general, the methodologies assessed are readily applied and do not generate particle crushing, at least for the conditions studied. Very loose to medium dense specimens can be formed, and under compaction strategies can be applied to reduce non-uniformities;
- The modified moist tamping method resulted in specimens that tend to have somewhat higher undrained peak strengths compared to the static compaction method, while the latter showed a more noticeable strain hardening response on reaching the CSL. These results suggest that somewhat different fabrics are attained between static and dynamic compaction procedures. However, these differences become less important for looser specimens, where the dynamic methods require very small drop heights and a low number of blows, which tend to make them analogous to a static compaction;
- In the case of the modified moist tamping method, the way compaction energy is delivered does not seem to have a significant effect on the resulting density;
- Due to its simplicity and ease of use, the static compaction method provides an appealing alternative for the reconstitution of loose to medium dense tailing samples, showing analogous behavior to the moist tamping method;
- However, the initial water content represents a major factor in the resulting fabric of the reconstituted specimens. Higher water content facilitates the formation of lumps, resulting in porous and non-homogenous specimens. Maximum water contents of around 4 to 5% less than the liquid limit of the fine fraction resulted in adequate compaction and reasonable uniform specimens.

Acknowledgements

The authors would like to thank Fresnillo PLC and Minera San Julián, in particular Ing. Juan José Torres and MEng. Carlos Omar Vargas, who provided the tailings material from the M1 and Minera Peña Colorada who provided the tailings for M4 for this research.

The financial support of the National Council for the Humanities, Science and Technology (CONAHCYT) of Mexico and the Soils Mechanical laboratory of the Institute of Engineering are gratefully acknowledged.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Mauricio Bernal López: conceptualization, data curation, investigation, visualization, writing – original draft, methodology. Luis Gerardo Cruz Flores: conceptualization, data curation, investigation, methodology, visualization, validation, writing – original draft, methodology. Eduardo Botero Jaramillo: supervision, funding acquisition, writing – review & editing, project administration, resources. Miguel Ángel Mánica Malcom: supervision, funding acquisition, writing – review & editing, project administration, resources. Osvaldo Flores Castrellón: supervision, funding acquisition, project administration, resources.

Data availability

All data produced or examined in the course of the current study are included in this article.

Declaration of use of generative artificial intelligence

This work was prepared with the assistance of generative artificial intelligence (GenAI) [ChatGPT] for the purpose of identifying grammatical errors and improving the clarity of wording in certain parts of the text. The entire process of using this tool was supervised, reviewed, and edited by the authors as necessary. The authors take full responsibility for

the content of the publication, including any sections where GenAI tools were utilized.

List of symbols and abbreviations

e	Void ratio
e_f	Void ratio prior to shearing
$e_{\max}$	Maximum void ratio
$e_{\min}$	Minimum void ratio
h	Drop height
p'	Mean effective stress
q	Deviator stress
w	Water content
ASTM	American Society for Testing and Materials
B	Skempton's pore pressure coefficient
CIU	Isotropic consolidated undrained tests
CO_2	Carbon Dioxide
CSL	Critical state line
E_c	Compaction energy
G_s	Specific gravity
LL	Limit liquid
M1	Mine tailings from Chihuahua.
M4	Mine tailings from Colima
ML	Low plasticity silt
MMT	Modified moist tamping
MT	Moist tamping
N	Number of blows
PL	Plastic limit
RM	Reconstitution method
SC	Static compaction
S_r	Degree of saturation
SM	Silty sand
TM	Tamper mass
TSF	Tailing storage facilities
USCS	Unified Soil Classification System
γ_d	Dry unit weight
ε_a	Axial strain
σ_3	Consolidation effective stress
Δu	Excess pore water pressure

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Methodology for correcting spectral ratio absorption estimates for source wave travel paths in DST

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Article

Keywords

Downhole seismic testing
Site characterization
Absorption estimation
Fermat's principle
Spectral ratio technique
Iterative forward modeling

Abstract

Downhole Seismic Testing (DST) is utilized for estimating low-strain shear damping ratio (η_s). Estimates of low strain η_s values are very important for predicting and assessing ground amplification during earthquakes. The low strain η_s estimates also provide for a reference of laboratory tests such as the resonant column test. The Spectral Ratio Technique (SRT) is one of the most utilized analysis techniques for processing seismic data to obtain η_s . The SRT is applied in the frequency domain where the spectral ratio of two data sets and relative arrival times are calculated to obtain the quality factor. The quality factor is inversely proportional to the damping ratio. One of the limitations of the SRT is that it doesn't account for the raypaths of the source waves. In DST the raypaths are of particular importance due to the fact the source waves will refract as they travel from the surface down to the seismic receiver. This paper outlines a technique, so-called FMDSM-SRT, which facilitates adjusting estimated SRT absorption DST values so that source wave raypaths are accounted for. It is of paramount importance to first implement newly optimal estimation algorithms on extensive test bed simulations prior to processing real data sets. This paper outlines the results from processing a challenging test bed simulation with the FMDSM-SRT algorithm.

1. Introduction

Downhole Seismic Testing (DST) such as Seismic Cone Penetration Testing (SCPT) (ASTM, 2019; Campanella et al., 1986) is a commonly used geotechnical technique for measuring in-situ ($<10^{-5}$) shear (V_s) and compression (V_p) wave interval velocities. The in-situ interval velocities form the core (Shear Modulus (G), Poisson's Ratio (μ) and Young's modulus (E)) of mathematical theorems to describe the elasticity/plasticity of soils and they are used to predict the soil response (settlement, liquefaction or failure) to imposed loads (whether from foundations, heavy equipment, earthquakes or explosions) (Finn, 1984; Andrus & Stokoe II, 2000; Ishikara, 1982). Baziw (2002) outlined that it was of paramount importance to take into account raypath refraction (Snell's law and Fermat's principle) when processing DST data sets for estimating interval velocities. Baziw (2002) also gave the mathematical details of a newly developed algorithm, the so-called Forward Modelling Downhill Simplex Method (FMDSM), which considered Fermat's principle when estimating DST interval velocities. Baziw & Verbeek (2012, 2014, 2018) published results from processing DST data sets using the FMDSM algorithm where special attention was given to identifying critical soil layers and tomographic imaging. Masters et al. (2024) utilized the

FMDSM technique when processing an extensive amount of offshore DST datasets. In Masters et al. (2024) paper it was demonstrated that it was necessary to take into account Fermat's principle when estimating interval velocities within the top 5m to 10m.

Another important geotechnical parameter estimated with DST is the low-strain shear damping ratio (η_s) (Stewart & Campanella, 1991, 1993; Redpath & Lee, 1986). Stewart & Campanella (1993) state that the DST low-strain η_s is very important for predicting and assessing ground amplification during earthquakes and serves as a reference for medium to large strain η_s values derived from laboratory test such as the resonant column test. The Spectral Ratio Technique (SRT) is the most widely used method for determining DST low strain η_s estimates (Stewart & Campanella, 1991, 1993; Redpath & Lee, 1986; Rebollar, 1984). A major limitation in the application of SRT for processing DST data sets is that the raypath of the source waves is not taken into account. This can lead to significant errors in the estimated damping ratio values.

Baziw & Verbeek (2019, 2021) developed the so-called FMDSMAA algorithm for estimating η_s values from DST seismic data. This algorithm accounts for source wave travel paths when estimating η_s values from peak particle accelerations (maximum amplitudes) of DST seismic traces

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Submitted on September 23, 2024; Final Acceptance on January 23, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.008024>



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and it also provides for error estimates. Baziw and Verbeek demonstrated the implementation and performance of the FMDSMAA (AA is the acronym for Absorption Analysis) algorithm by considering challenging test bed simulations and processing real data sets. It was shown that the algorithm was able to obtain accurate absorption estimates which demonstrated the algorithm's correctness. This paper outlines modifications and additions to the FMDSMAA algorithm so that estimates of absorption from the Spectral Ratio Technique (SRT) are corrected for DST source wave travel paths. In general terms, the FMDSMAA is modified where instead of minimizing the difference between synthetic source wave amplitude ratios and recorded values, the difference between synthetic spectral ratios and measured values are minimized. This technique is referred to as the so-called FMDSM-SRT algorithm.

2. Absorption analysis and the spectral ratio technique

The amplitude of a seismic wave propagating in soils decays due to geometric spreading (due to the change in wave front), apparent attenuation (due to mode conversion, reflection-refraction at an interface) and material losses (intrinsic attenuation or absorption). In general terms, the soil profile acts as both an attenuator and a low pass filter as the seismic wave travels through it (Aki & Richards, 2002; Gibowicz & Kijko, 1994; Sheriff & Geldart, 1982). The signal amplitude A within a homogeneous medium at distance x from the source is related to the amplitude A_0 at distance x_0 by:

$$A(x) = A_0 (x_0 / x)^n e^{-\alpha(x-x_0)} \quad (1)$$

In Equation (1) it is assumed that the decay is due to only geometric spreading and absorption (α in Equation (1)). The Quality Factor, Q , is related to the absorption coefficient as follows:

$$Q = \pi / \alpha \lambda \quad (2)$$

In Equation (2) λ is the source wave's wavelength. The Quality Factor is a desirable term to define the absorption of a medium because it is nondispersive. The logarithmic decrement, δ , and fraction of critical damping or damping ratio, η_s , are expressed as:

$$\delta = \ln \left(\frac{\text{amplitude}}{\text{amplitude one cycle later}} \right) = \alpha \lambda = \pi / Q \quad (3)$$

$$\eta_s = \frac{\delta}{2\pi} = \frac{1}{2Q} \quad (4)$$

In geotechnical engineering the shear damping ratio is typically defined as $\eta_s = 1/(2Q)$. The unit of δ is nepers (Np).

Attenuation from refraction at a layer boundary is quantified by the transmission coefficient. The amplitude decay term in Equation (1) $(x_0/x)^n$ corresponds to geometric spreading where $n = 1$ based upon the conservation of the energy flux of a traveling seismic wave and spherical divergence. Spherical divergence is given as:

$$E_r = \frac{E_0}{4 r^2} \quad (5)$$

where r is the expanding radius from the source, E_0 is the initial source energy and E_r is the energy at distance r from the source. The amplitude of the source wave decays proportional to the distance r from the source as outlined as follows:

$$A_r = \sqrt{\frac{E_0}{4\pi r^2}} \quad (6)$$

Some researchers have noted that the amplitude of the seismic wave generally does not decay as $1/r$ where r is the travel distance, and this is why the exponent n is incorporated into Equation (1).

The apparent attenuation due refraction at an interface is quantified by the transmission coefficient. The transmission coefficient quantifies the loss of energy when transitioning from layer 1 to layer 2 is outlined below in Equation (7) for horizontally polarized shear wave (SH).

$$T_{12} = \frac{A_2}{A_1} = \frac{2Z_1}{Z_1 + Z_2} = \frac{2\rho_1 V_1 \cos\theta_1}{\rho_1 V_1 \cos\theta_1 + \rho_2 V_2 \cos\theta_2} \quad (7)$$

In Equation (7) T_{12} is the transmission coefficient for the source traveling moving from layer 1 to 2, A_1 is the amplitude of incident wave, A_2 the amplitude of refracted wave, ρ_i is the medium density of layer i , θ_1 denotes the incident angle, θ_2 is the refraction angle, V_1 is the medium velocity of layer 1, and V_2 is the medium velocity layer 2 (note: $Z_i = \rho_i V_i$ is the acoustic impedance).

The ability to solve the absorption coefficients demands that several assumptions be made regarding the properties of the medium of propagation. These include the assumptions that the medium is an attenuating, velocity dispersive, causal, and linear system. Some properties of seismic medium absorption is proportional to frequency, absorption is inversely proportional to velocity, absorption is proportional to pore saturation, absorption is proportional to porosity in saturated rocks and soils (e.g., absorption is greater in clean sands than shales), absorption is dependent upon pore fluid type (e.g., absorption is greater for brine-watered saturated rocks than methane saturated rocks), and absorption is inversely proportional to formation pressure.

There are several mathematical techniques which facilitate the derivation of absorption from seismic time series. The most popular methodology is the spectral ratio

technique (SRT) (Rebollar, 1984; Campillo et al., 1985; Blakeslee et al., 1989; Kvamme & Havskov, 1989; Stewart & Campanella, 1993; Howie & Amini, 2005).

2.1 Spectral ratio technique equation derivation

The mathematical representations of an elastic and anelastic plane wave in the depth domain (z) and frequency domain ($\omega = 2\pi f$) are outlined as follows:

Elastic Plane Wave:

$$u(z, \omega) = u(0, \omega) e^{ikz} \quad (8)$$

Anelastic Medium:

$$u(z, \omega) = u(0, \omega) e^{ikz} e^{-\alpha(\omega)z} \quad (9)$$

Parameter k in Equation (8) is the wavenumber (Sheriff & Geldart, 1982). Equation (9) implies that anelastic attenuation is a linear phenomenon. As was previously stated, α denotes the absorption and is defined as:

$$\alpha(\omega) = \omega / 2V(\omega)Q(\omega) \quad (10)$$

where $V(\omega)$ - dispersive velocity and $Q(\omega)$ - dispersive quality factor

Experimental data have shown that Q is almost identically constant with respect to frequency over a large bandwidth (10mHz to 10MHz) (Toksoz et al., 1979). This bandwidth encompasses frequencies encountered in seismic wave propagation. Equation (9) can also be rewritten in terms of a complex wave number as

$$u(z, \omega) = u(z_0, \omega) e^{ik'(\omega)(z-z_0)}, \quad (11)$$

$$k'(\omega) = \omega / V(\omega) + i\alpha(\omega)$$

Substituting Equation (10) into (11) and taking real parts (to have physical meaning) gives

$$\ln \left| \frac{u(z, \omega)}{u(z_0, \omega)} \right| = \frac{\omega(z-z_0)}{2V(\omega)Q} = -\frac{\omega T_T(\omega)}{2Q} \quad (12)$$

In Equation (12) $T_T(\omega)$ is the travel time difference between depths z and z_0 . The arrival time difference is strictly

a function of frequency; however if the distance ($z-z_0$) is not to large and $V(\omega)$ is not to dispersive, then $T_T(\omega)$ will be approximately constant with respect to frequency; therefore, Q can be determined from the dominant frequencies of the wave under study, the relative arrival time and the natural logarithm of the spectral ratio by Equation (12) as follows:

$$Q = -\frac{T_T}{2 \times SRS}, \quad SRS = \ln \left| \frac{u(z, \omega)}{u(z_0, \omega)} \right| / \omega \quad (13)$$

The propagating waves are also affected by non-intrinsic amplitude variation (i.e., frequency independent) such as geometric spreading. The non-intrinsic amplitude variations are lumped into the intercept term of Equation (12) to give

$$\ln \left| \frac{N_z u(z, \omega)}{N_{z_0} u(z_0, \omega)} \right| = -\frac{\omega T(\omega)}{2Q} + \ln N' \quad (14)$$

where : $N' = \frac{N_z}{N_{z_0}}$

In general terms, Equation (14) indicates that attenuation due to geometric spreading and apparent attenuation are included in the intercept term and do not affect the intrinsic Q estimation derivation provided they are independent of frequency.

A significant problem with implementing the SRT on DST data sets is that the SRT assumes the source waves travel along the same path as shown in Figure 1. Source wave trajectories adhere to Fermat's principle, meaning the raypath travels along the trajectory that minimizes the travel time between points. This results in raypath refraction as shown in Figure 2. The schematic illustrated in Figure 2 outlines a DST investigation where seismic source waves are generated at the surface. The source waves travel down into the soil profile where they are recorded by seismic sensors vertically located at increasing depths. Parameter X denotes the source radial offset from the vertically installed seismic sensors. Parameters D_i , V_i and α_i denote the depth of the seismic sensor, interval velocity and absorption value for soil layer i , respectively. Finally, in Figure 2 d_{ij} is the travel path for each source wave j within layer i . As is shown in Figure 2, the source waves refract at the soil interfaces according to Snell's law and Fermat's principle.



Figure 1. Assumed source wave travel paths when implementing SRT.

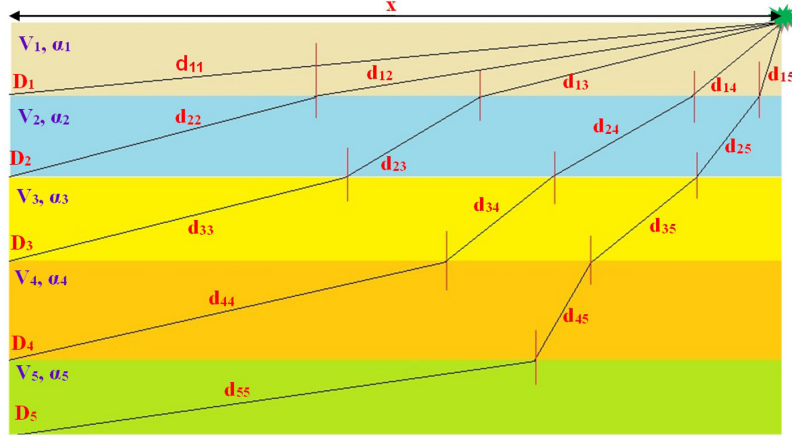


Figure 2. Near surface DST where SRT assumptions are not valid.

3. FMDSM-SRT algorithm

Baziw Consulting Engineers Ltd. (BCE) developed the so-called FMDSM-SRT algorithm for correcting DST SRT absorption estimates for source wave travel paths. The FMDSM-SRT utilizes iterative forward modeling (IFM) for parameter estimation. IFM is based on iteratively adjusting the parameters until a user specified cost function is minimized. The IFM technique implemented by the FMDSM-SRT algorithm is the downhill simplex method (DSM) developed by Nelder & Mead (1965). The DSM searches for the minimum of the costs function by taking a series of steps, each time moving a point in the simplex away from where the cost function is largest. The simplex moves in space by variously reflecting, expanding, contracting, or shrinking. The simplex size is continuously changed and mostly diminished, so that finally it is small enough to contain the minimum with the desired accuracy.

Equations (15) to (18) define the expected intrinsic source amplitudes for traces recorded at depths D_1 , D_2 , D_3 , and D_4 for the source raypaths illustrated in Figure 2.

$$A_1 = e^{-\alpha_1(d_{11}-x_0)} \quad (15)$$

$$A_2 = e^{-(\alpha_1(d_{12}-x_0)+\alpha_2d_{22})} \quad (16)$$

$$A_3 = e^{-(\alpha_1(d_{13}-x_0)+\alpha_2d_{23}+\alpha_3d_{33})} \quad (17)$$

$$A_4 = e^{-(\alpha_1(d_{14}-x_0)+\alpha_2d_{24}+\alpha_3d_{34}+\alpha_4d_{44})} \quad (18)$$

In general terms, the intrinsic amplitudes recorded at each subsequent DST depth of acquisition are mathematically expressed as follows:

$$A_i = A_0 e^{-\left(\alpha_1(d_{1i}-x_0) + \sum_{j=2}^i \alpha_j d_{ji}\right)}, \quad j \geq 1, i > 1 \quad (19)$$

If you then consider the ratio of the amplitudes, whether in absolute terms or globally normalized, the unknown amplitude initial A_0 drops out of the set of equations. The equation describing the intrinsic amplitude ratio estimated DST absorption values from the measured seismic traces is outlined below.

$$\frac{A_{i+1}^m}{A_i^m} = e^{-\alpha_{mi}(D_{i+1}-D_i)} \quad (20)$$

where α_{mi} is the estimated SRT absorption value for interval i and D_i denotes the total travel distance for raypath i .

SRT absorption estimates α_{mi} are obtained from the real data sets by substituting Equation (10) into Equation (13)

$$\alpha_{mi} = -\ln \left| \frac{u(D_{i+1}, \omega)}{u(D_i, \omega)} \right| / (D_{i+1} - D_i) = \text{SRS} * \Delta\omega / (D_{i+1} - D_i) \quad (21)$$

Based on the above the proposed FMDSM-SRT algorithm for estimating absorption coefficients can then be described as follows:

- Utilizing the standard interval velocity FMDSM technique (Baziw, 2002; Baziw & Verbeek, 2012, 2014), obtain estimates of V_i and d_{ij} .
- For the depth increments under analysis determine the measured SRT absorption estimates (α_{mi} - using Equation 21) from the recorded DST traces each depth increment.
- Implement FMDSM-SRT algorithm to calculate the synthesized amplitudes with Equation (19) based on assumed absorption coefficients, whereby the absolute difference between the measured and synthesizes amplitude ratios is iteratively minimized.

$$\min_{\alpha_i} \left\{ \sum_{i=1}^{n-1} \left| \frac{A_i}{A_{i+1}} - e^{-\alpha_{mi}(D_{i+1}-D_i)} \right| \right\} \quad (22)$$

where n is the number of layers or absorption coefficients to be estimated.

4. FMDSM-SRT test bed simulation

The test bed simulation outlined in this paper uses the same one outlined by Baziw & Verbeek (2019). However, in this case measured SRT absorption values are utilized instead of PPAs. This was done for consistency and comparison of results with the FMDSMAA algorithm. Table 1 below outlines the FMDSM-SRT test bed simulation working parameters. Columns 1 and 2 of Table 1 are the assumed depths and associated source wave arrival times of the simulated DST. Column 3 of Table 1 contains the estimated interval velocities with the depths and arrival times input into the FMDSM

algorithm, which accounts for raypath refraction. Column 4 of Table 1 outlines the true absorption values for each soil interval. Column 5 of Table 1 contains the wavelength of the source wave as it travels through the soil layers of varying interval velocities ($\lambda = V/f$). Column 6 of Table 1 outlines the quality factor obtained by implementing Equation 2. Finally, column 7 of Table 1 contains the associated estimated measured SRT absorption values for each depth increment. The SRT measured absorption values were calculated by implementing Equations 19 and 23. As is evident from reviewing column 7, there are numerous nonsensical absorption values such as the negative values at 3m and 5m.

$$\alpha_{mi} = -\ln \left| \frac{A_{i+1}}{A_i} \right| / (D_{i+1} - D_i) \quad (23)$$

Table 2 and Figure 3 outline the output of the FMDSM-SRT algorithm after inputting the values given in Table 1, and the raypaths determined by the FMDSM algorithm

Table 1. FMDSM-SRT test bed example parameters ($f = 100$ Hz).

Depth [m]	T_r [ms]	V [m/s]	α [1/m]	λ [m]	Q [1/ Np]	α_m [1/m]
1	17	131.5	0.34129	1.32	7	N/A
2	25	97.1	0.64708	0.97	5	0.94071
3	28	176.3	0.162	1.76	11	-0.02294
4	34	142.8	0.275	1.43	8	0.27871
5	37	249.6	0.05244	2.5	24	-0.01041
6	41	222.5	0.07431	2.23	19	0.05774
7	46	189.1	0.1278	1.89	13	0.12536
8	49	303.2	0.03701	3.05	28	0.02556
9	54	193.7	0.10137	1.94	16	0.10001
10	58	242.2	0.06177	2.42	21	0.05933
11	62	243.3	0.05869	2.43	22	0.05661
12	65	322.1	0.03048	3.23	32	0.02760

Legend: see List of symbols and abbreviations. V is derived from the FMDSM algorithm. α_m is obtained from Equation (23).

Table 2. FMDSM-SRT test bed example results.

Depth (m)	α (1/m)	α_m (1/m)	α / α_m percent difference (%)	α' (1/m)	α / α' percent difference (%)	Amplitude ratio error residual
1	0.34129	N/A	N/A	0.31725	7.3	N/A
2	0.64708	0.94071	37	0.6353	1.8	2.62E-06
3	0.162	-0.02294	266	0.15293	5.8	7.86E-07
4	0.275	0.27871	1.3	0.27181	1.2	6.07E-07
5	0.05244	-0.01041	299.1	0.04996	4.8	1.33E-06
6	0.07431	0.05774	25.1	0.0733	1.4	1.40E-07
7	0.1278	0.12536	1.9	0.12729	0.4	5.77E-08
8	0.03701	0.02556	36.6	0.03653	1.3	4.64E-07
9	0.10137	0.10001	1.4	0.10118	0.2	1.01E-06
10	0.06177	0.05933	4	0.0616	0.3	4.28E-06
11	0.05869	0.05661	3.6	0.05857	0.2	4.78E-06
12	0.03048	0.02760	9.9	0.03038	0.3	1.36E-06

Legend: see List of symbols and abbreviations. α' is obtained from the FMDSM-SRT algorithm. α_m is obtained from Equation (21).

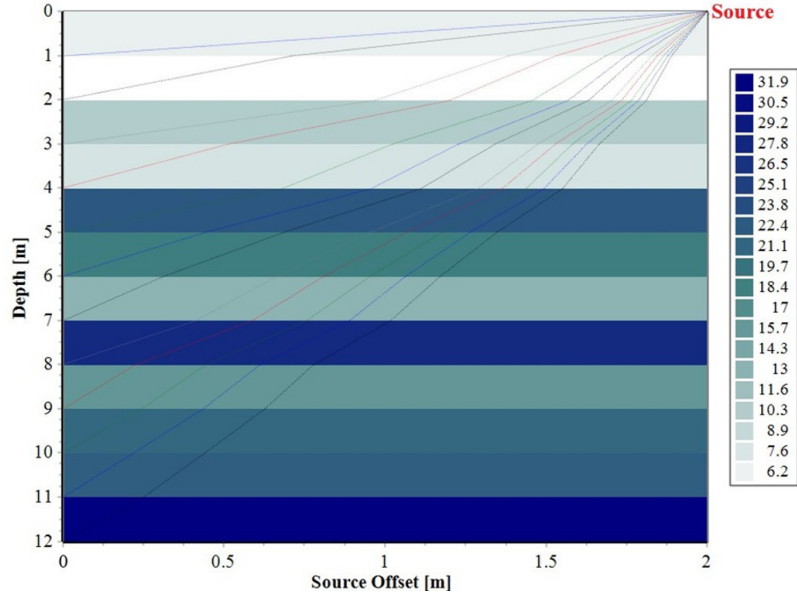


Figure 3. Output from the FMDSM-SRT algorithm illustrating source wave raypaths and estimated Q values.

which are illustrated in Figure 3. Figure 3 also illustrates the estimated FMDSM-SRT algorithm Q values. Column 4 of Table 2 outlines the very large percent differences between the true absorption values and the DST measured SRT values. For example, depth intervals to 2m to 3m and 4m to 5m have percent differences of 266 and 299, respectively. Column 5 of Table 2 contains the FMDSM-SRT estimate, α' , while column 6 outlines the percentage differences between the true absorption values and the FMDSM-SRT estimate. As is evident from the present differences outlined in Table 2, the FMDSM-SRT was able to correct the DST SRT values for the source wave raypaths and obtain accurate estimates of the true absorption values. Column 7 of Table 2 outlines the low FMDSM-SRT error residuals.

5. Conclusion

The low-strain DST η_s estimates are very important for predicting and assessing ground amplification during earthquakes. In addition, DST η_s estimates provide for a reference of laboratory tests such as the resonant column test. RCT can suffer from various disadvantages such as sample disturbance and sample preparation which can shift the estimated η_s values. The RCT results can be adjusted so that the low-strain RCT η_s estimates agree with the low-strain in-situ DST η_s estimates. The most popular methodology for estimating DST is the spectral ratio technique. The major shortcoming of the SRT on DST data sets is that the SRT assumes that the source waves travel along the same path. This assumption can result in very large errors in the η_s estimates. This paper has outlined the mathematical details of a new technique, the so-called FMDSM-SRT algorithm, which corrects the estimated DST SRT η_s estimates for source wave

raypaths. The effectiveness of the FMDSM-SRT algorithm was demonstrated by a challenging test bed simulation. It is of paramount importance that optimal estimation algorithms first carry out sufficiently challenging test bed simulations for algorithm verification prior to implementation on real data sets.

Declaration of interest

The author has no conflicts of interest to declare. The author has observed and affirmed the contents of the paper and there is no financial interest to report.

Data availability

The datasets generated analyzed during the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was not prepared with the assistance of Generative Artificial Intelligence (GenAI)

List of symbols and abbreviations

k	wavenumber
u	plane wave
x_0	source wave reference distance
A_0	source wave reference amplitude
DST	downhole seismic testing
G	shear modulus

Q	quality factor
PPA	peak particle acceleration
RCT	resonant column test
SRS	spectral ratio slope
SRT	spectral ratio technique
T	transmission coefficient
T_T	travel time
V	source wave velocity
α	absorption coefficient
α'	FMDSM-SRT estimated absorption coefficient
α_m	SRT measured absorption coefficient
δ	logarithmic decrement
η	damping ratio
λ	source wave's wavelength

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Numerical analysis of the behavior of piled rafts and pile groups in the presence of pile defects

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Article

Keywords

Numerical analysis
Piled raft
Pile foundation
Pile defect
Foundation engineering

Abstract

This article aims to evaluate the behavior of piled rafts and pile groups subject to the presence of defective piles. To simulate the defect, it was considered that a foundation pile is shorter than projected. Seven static load tests performed in 2011 in the city of João Pessoa, in north-east Brazil, were used as the basis of analysis. These load tests were performed on an isolated raft, in 3 piled rafts and in 3 pile groups. In this study, the ELPLA® software, based on a hyperbolic iterative model, was used. Initially, modeling was performed for the original situations with intact foundations, and the results were compared with previous experimental results. Afterwards, numerical modeling was performed considering four defect levels, which correspond to a decrease in the pile length. According to the results obtained, it could be observed that, due to the presence of the defect, there was a reduction in the stiffness and load capacities of all the foundations analyzed, the occurrence of differential settlements in the foundations, and an increase in the load absorbed by the raft in the piled rafts.

1. Introduction

Increasingly intense occupation of urban space can be observed as a historical trend in Brazilian cities. This dynamic reduces the space available to construct to such an extent that buildings are built on smaller plots of land, becoming taller and leaner. As a result, the demands imposed by the superstructure on the foundation system become increasingly high and complex, which makes it necessary to compatibilizer designs and make use of more efficient, better performing foundation systems to support these demands, ensuring greater safety and cost reduction.

In current foundation engineering practices, some simplifications are allowed when designing a pile foundation system. One hypothesis used to facilitate the study and modeling of the problem is that the bearing superficial block (or raft) only has the function of transmitting superstructure loads to the piles, serving as a transition element between them. In this case, the contribution of the block to the load capacity of the foundation is not considered.

Given the simplification mentioned above, commonly adopted in design, in a foundation system constituted by

a pile group, the load capacity is provided by the stresses distributed by the pile shaft (lateral frictional resistance) and the stresses distributed by the pile tip (tip resistance). In a piled raft, the contribution of the superficial element to the load capacity of the system is considered. Thus, the load capacity is given by the lateral friction and the pile tip resistance, as well as by the stresses distributed by the raft base (superficial element).

Although it is expected that piled rafts may lead to a cost reduction and efficiency in the foundation's performance, there is still a noticeable restriction when using them. One explanation for this is that piled raft modelling is more complex, involves more interactions than conventional pile group modelling and there is relatively little experience, particularly in Brazil. Another explanation may lie in the fact that the Brazilian foundation standard, NBR 6122 (ABNT, 2019), although allowing the use of piled rafts, does not explicitly include this modality in its text, and designers generally want to ensure a conservative resistance in the foundation (designing traditionally), hence safeguarding against structural element defects or unpredictable variability in soil characteristics. Studying the foundation behavior

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Submitted on November 18, 2024; Final Acceptance on May 6, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.001124>



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when there are defects is important because it can evaluate the important role that the superficial element (raft) plays in the system's performance in these situations.

This paper aims to numerically evaluate the behavior of piled rafts and pile groups, based on sandy soils typical of northeastern Brazil, in the presence of pile defects, using the ELPLA® software, version 10.1. The defect analyzed in this article was the presence of a pile shorter than the designed length, which is a common occurrence in foundation pathologies. For instance, Klingmüller & Kirsch (2004) found that 21% of the pathologies, in 25 years of integrity tests in Germany, were caused by insufficient foundation length. Numerical modeling was performed considering variations in the length of a pile in order to verify its influence on the stiffness, load capacity and load mobilization mechanisms of the foundations.

2. Piled rafts: case studies of pile defects

A piled raft is a type of mixed foundation, that is, it results from joining a shallow foundation (block/raft) with a deep foundation (piles). The performance of a piled raft foundation is the result of the efforts mobilized by the pile tip and shaft, as well as the raft base.

The good performance of a foundation depends on the geotechnical conditions of the subsoil and the integrity of the structural elements. According to Poulos (2001), the favorable situation for using piled rafts corresponds to soil profiles consisting of stiff clay and/or compact sand, especially in the most superficial layers of soil. However, even under favorable conditions, the piled foundations may be subject to defects that affect their behavior. Depending on the type and magnitude of the defect, its presence can lead to the collapse of a foundation system.

Defects in piled foundations can be of geotechnical or structural origin. According to Poulos (1999), defects of geotechnical origin are related to the presence of compressible material layers and low resistance adjacent to the pile, residues in the pile tip, and presence of bentonite mud in the pile shaft, among others. Structural problems are related to lower strength concrete in pile regions, necking in the pile shaft, damaged areas, smaller dimensions in structural elements, among others. In this article, distinct performance scenarios were simulated where there is a shorter than expected pile, which is configured as a situation of structural defect and/or construction pathology, as described in chapter 3.2.

Various studies can be cited in scientific literature that address piled foundations subject to the presence of defective piles. The following studies can be mentioned as examples: Poulos (1997), Abdrabbo (1997), Poulos (1999), Xu (2000), Kong & Zhang (2004), Poulos (2005), Cordeiro (2007), Cunha et al. (2007), Zhang & Wong (2007), Cunha et al. (2010), Leung et al. (2010), Freitas Neto (2013), Albuquerque et al. (2017), Alva (2017), Cordeiro (2017),

Xu et al. (2019), Freitas Neto et al. (2020), Yan et al. (2020) and García et al. (2023).

Regarding the most recent studies, Freitas Neto et al. (2020) carried out numerical (finite element method) and experimental studies, on a full-scale case, in a raft with a defective pile founded on a tropical soil in Brazil. The defect considered was the necking of the shaft, with the subsequent decrease in structural strength and stiffness of the shaft. The authors noted that the defect in the pile resulted in a decrease in the load capacity and the safety factor of the foundation. Yan et al. (2020) also carried out experimental, centrifuge, and numerical studies, using the finite element method, to investigate the lateral load capacity performance of a pier supported by defective piles. García et al. (2023) investigated, through experimental and numerical methods, the effects of defective piles in horizontally loaded piled raft foundations. The results show that the presence of a defective pile increases the raft tilting.

3. Materials and methods

3.1 Experimental program

Soares et al. (2015), from the D.Sc. thesis of Soares (2011), conducted four Standard Penetration Tests (SPT) and seven slow static load tests on isolated raft, piled raft and pile group foundations in the city of João Pessoa, north-east Brazil (Figure 1). An isolated raft, and piled rafts with one, two and four piles were tested, as well as a pile group also consisting of one, two and four piles. Thus, overall, the author executed 14 hollow auger piles whose length and diameter were 4.5 meters and 0.30 meters, respectively. The dimensions of the block/raft positioned at the top of the piles were 1.55 m x 1.55 m x 0.85 m. Details about the execution of the hollow auger piles can be seen in Soares et al. (2015). Figure 2 shows the position of the tested foundations, as well as the location of the SPT's holes.

Based on the results of the SPT's, Soares et al. (2015) established the representative subsoil profile, with the mean values of the penetration resistance index (N_{SPT}) along the depth (Figure 3). It is observed that the subsoil is typically sandy.

3.2 Numerical modelling

In the present article, the ELPLA® software version 10.1 was used, which presents the possibility of working specifically with piled rafts. A continuous model was considered, in which the soil is represented by a layered medium, with isotropic and elastic behavior and a non-linear hyperbolic model was adopted. The calculation method used in the software was typical for rigid piled rafts. Although the software developer provides details on the calculation method in the manual available on the website, the following is a description of the most relevant aspects of the numerical tool. The main

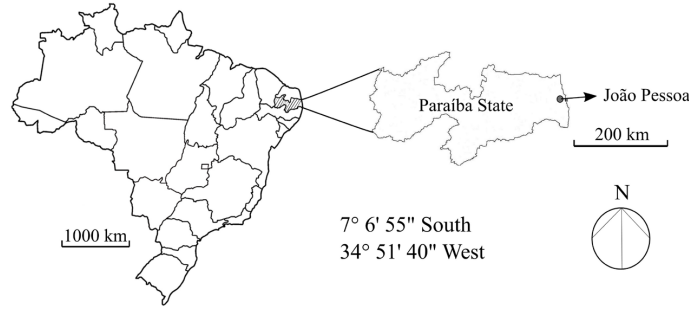


Figure 1. Location of the city of João Pessoa, Brazil.

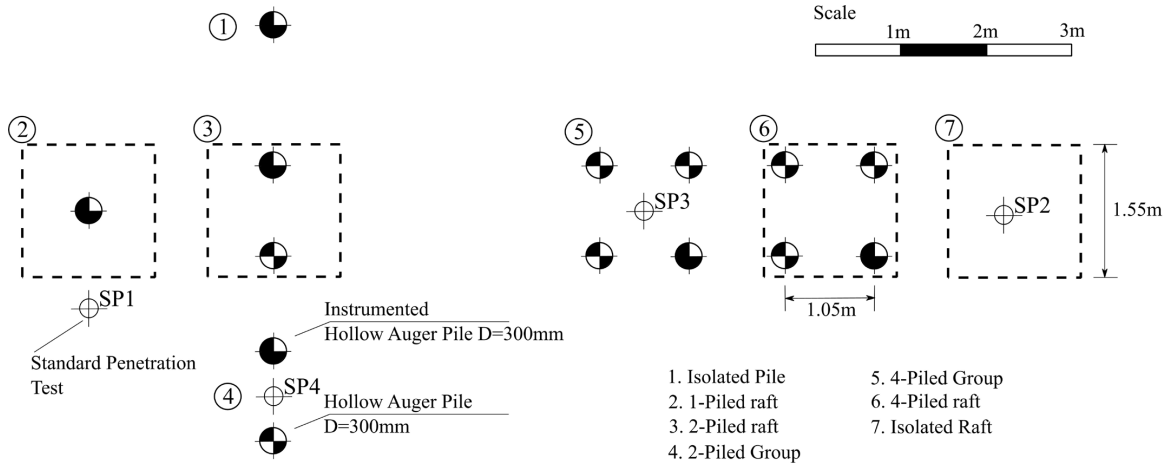


Figure 2. Position of tested foundations and SPT's (Soares et al., 2015).

motivation for using the ELPLA 10.1® software is because of its reduced processing time and the fact that it is easy to use. The ELPLA 10.1® software presents a calculation model in which the raft and the piles are divided into nodes whose number is defined by the user. Thus, the software establishes a system of equations that relates, at each node, the contact force (Q) to the soil stiffness (k) and settlement (w). For the general foundation scheme shown in Figure 4, the relationship between settlements and contact forces of the piled raft can be written in the form of a general compacted matrix as follows (Equation 1):

$$\{Q\} = [k] \cdot \{w\} \quad (1)$$

where: $\{w\}$ is the settlement vector; $\{Q\}$ is the contact force vector; and $[k]$ is the soil stiffness matrix of the piled raft.

In the analysis of rigid piled rafts, the Equation 2 expresses the settlement (w) in each raft node or in a pile, which have coordinates (x_i, y_i) in relation to the centroid of the raft geometry, as a function of the rigid body translation of the piled raft (w_c) and the rigid body rotations (θ_x and θ_y) of the piled raft about the axes of the raft geometry centroid.

$$\{w\} \approx [X]^T \{ \} \quad (2)$$

where: Δ is the vector of translation w_c and rotations $\tan\theta_x$ and $\tan\theta_y$, and $[X]^T$ is the matrix of $\{1, x_i, y_i\}$, where x_i and y_i are coordinates of node i .

For equilibrium, the sum of the forces applied to each node, as well as the sum of the moments due to these forces, must meet the equilibrium equation (Equation 3):

$$\{N\} = [X] \cdot \{Q\} \quad (3)$$

where: $\{N\}$ is the vector of the resultant forces and moments acting on the piled raft.

Substituting Equations 1 and 2 in Equation 3, the Equation 4 is obtained. Solving this equation, the values of w_c , $\tan\theta_x$, and $\tan\theta_y$ are determined. Thus, from Equation 2, the settlements are found.

$$\{N\} = [X] \cdot [k] \cdot [X]^T \cdot \{\Delta\} \quad (4)$$

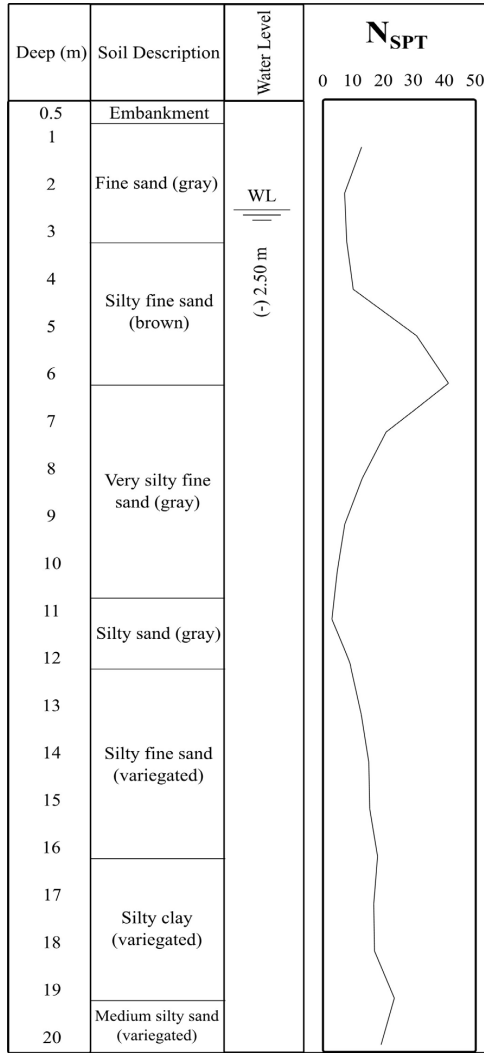


Figure 3. Subsoil profile with N_{SPT} values along the depth (adapted from Soares et al., 2015).

Substituting Equation 2 in Equation 1, it is possible to determine the pile loads and the contact forces, using the Equation 5:

$$\{Q\} = [k] \cdot [X]^T \cdot \{\Delta\} \quad (5)$$

After the initial linear analysis, it is possible to determine the settlements $\{w\}$ (Equation 4) and the nodal contact forces $\{Q\}$ (Equation 5) in the piled raft. The nonlinear analysis of the piled raft is based on a hyperbolic formulation through an iterative procedure.

From the computed settlements and contact forces, the nodal stiffness (k_e) at all nodes on the raft and on pile heads is determined using Equation 6.

$$k_e = \frac{Q}{w} \quad (6)$$

The pile stiffness is modified by Equation 7:

$$k_e = \frac{1}{\frac{1}{k} + \frac{w}{Q_l}} \quad (7)$$

where: k is the soil stiffness and Q_l is the limit pile load.

The soil stiffness matrix $[k]$ is converted to the stiffness matrix $[k_e]$. Thus, after the nonlinear analysis of the piled raft, New settlement values $\{w\}$ are determined. With the new settlement values, new contact force values $\{Q\}$ are determined (equation 6). The procedure described is repeated until there is convergence in the generated settlement values.

Regarding the dimensions of the model, in the horizontal domain the software considers only the dimensions of the raft, while in the vertical domain, a dimension of 3 times the length of the pile was adopted. It is noteworthy that the software's mathematical model only considers the soil beneath the raft foundation, as shown in Figure 4, where the soil and the piles are divided into nodes. Thus, the lateral soil region outside the raft is not taken into account in the mathematical procedure, and therefore, it was not necessary to define a horizontal domain wider than the raft for the model. Figure 5 shows the dimensions of the model.

In this article, initially, numerical analysis was performed for foundations in the intact condition, which were tested by Soares et al. (2015) through static load tests, as discussed in chapter 3.1. These initial analyses were carried out to compare the results of the ELPLA® software with the experimental results and evaluate the convergence of the results, to then expand the analysis considering the presence of defective piles in the foundation. Considering such non-standard piles, the influence of the defect and its magnitude on the load/settlement curves of the foundation systems, the differential settlement, as well as the strength redistribution and load capacity were evaluated. In the present work, the defect considered in the analysis of all the piled foundation models was the presence of a pile shorter than initially predicted, which is 4.5 meters.

For each pile foundation model analysed, the presence of one defective pile and four defect levels were considered, namely: $D_{11\%}$ - 11% reduction in the pile length (4 m defective pile); $D_{33\%}$ - 33% reduction in pile length (3m defective pile); $D_{56\%}$ - 56% reduction in pile length (2 m defective pile); and $D_{78\%}$ - 78% reduction in pile length (1m defective pile). Table 1 presents all the cases considered in this study. Overall, 31 cases were considered, 7 in scenarios with intact piles and 24 in scenarios considering piles with different defect magnitudes. Standard pile groups (PG) differ from piled rafts (PR) in this table, by simply considering or not the raft contact with the superficial soil.

Figure 6 shows the position of the defective pile under the connecting structural element (block/ raft) for the 2-pile and 4-pile foundations. In the case of the 1-piled block/raft, the defective pile represents the unique pile in the foundation system.

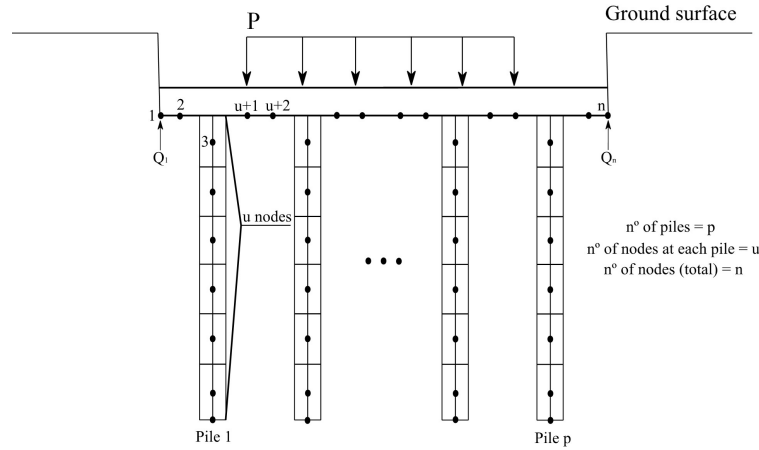


Figure 4. General foundation scheme.

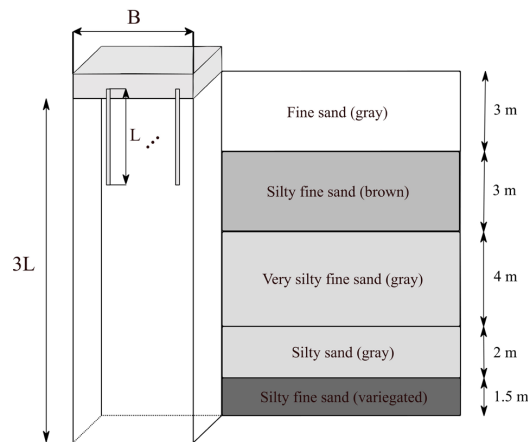


Figure 5. Dimensions of the model.

Table 1. Cases considered in this study.

Intact cases		Defective cases		
R	-	-	-	-
PR ₁	PR ₁ D _{11%}	PR ₁ D _{33%}	PR ₁ D _{56%}	PR ₁ D _{78%}
PR ₂	PR ₂ D _{11%}	PR ₂ D _{33%}	PR ₂ D _{56%}	PR ₂ D _{78%}
PR ₄	PR ₄ D _{11%}	PR ₄ D _{33%}	PR ₄ D _{56%}	PR ₄ D _{78%}
PG ₁	PG ₁ D _{11%}	PG ₁ D _{33%}	PG ₁ D _{56%}	PG ₁ D _{78%}
PG ₂	PG ₂ D _{11%}	PG ₂ D _{33%}	PG ₂ D _{56%}	PG ₂ D _{78%}
PG ₄	PG ₄ D _{11%}	PG ₄ D _{33%}	PG ₄ D _{56%}	PG ₄ D _{78%}

N.B.: R: Raft; PR_i: Piled raft with *i* piles; PR_iD_{j%}: Piled raft with *i* piles and a pile with *j*% shorter length; PG_i: Pile group *i*; PG_iD_{j%}: Pile group *i* and a pile with *j*% shorter length.

Based on the results of the SPT sampling conducted by Soares et al. (2015), shown in Figure 3, the necessary soil parameters for numerical modeling were obtained, according to the following formulations: Modulus of elasticity “ E_s ” (Teixeira & Godoy, 1996 - 1996 - $E_s = \alpha \cdot K \cdot N$, α and k are coefficients that depend on soil type; N is the penetration

resistance index obtained in the SPT); Friction angle “ ϕ ” (Teixeira, 1996 - $\phi = \sqrt{20N + 15}$, N is the penetration resistance index obtained in the SPT); Specific weight “ γ ” (Godoy, 1972 apud Cintra et al., 2011); Poisson’s ratio “ ν ” (Godoy, 1972 apud Cintra et al., 2011). Figure 7 shows the simplified geological profile of the subsoil, with the soil

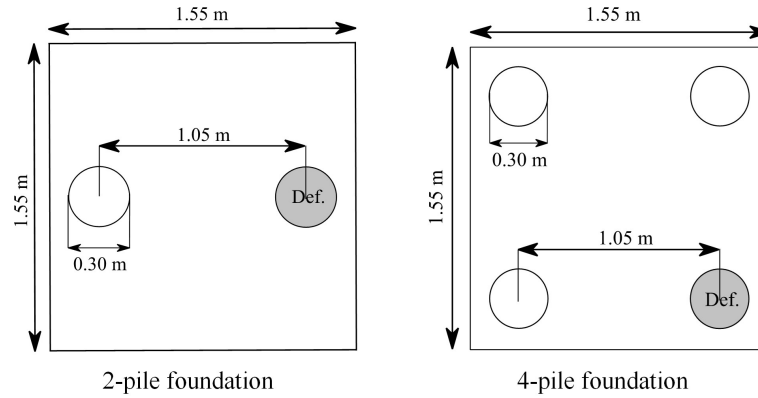


Figure 6. Position of defective pile.

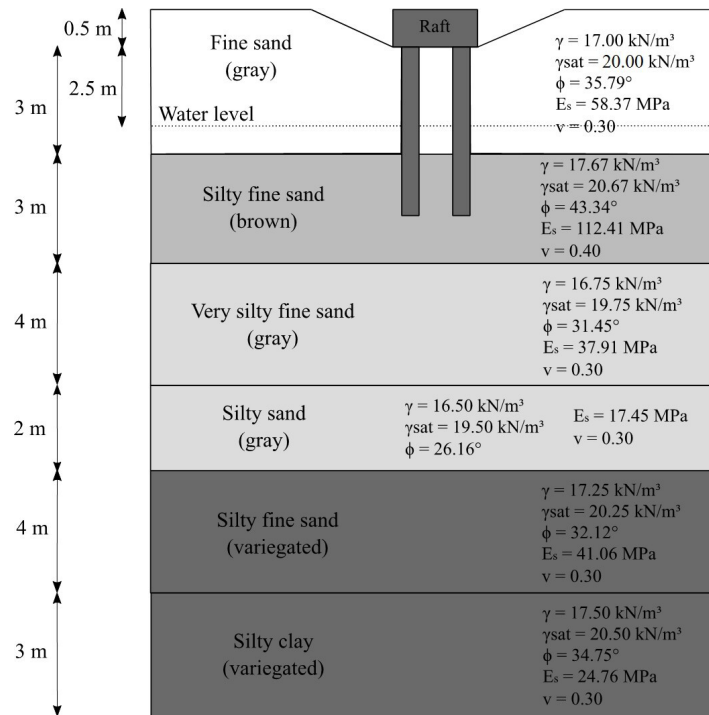


Figure 7. Representative profile and soil parameters.

resistance and deformation parameters per layer. The concrete parameters of the foundation elements were obtained from the Brazilian standard NBR 6118 (ABNT, 2014) using formulations as a function of concrete strength, which was 14.8 MPa (average of the strengths obtained through compression tests on specimens). The remaining parameters were defined as: Modulus of elasticity ($E = 21.50$ GPa); Specific weight ($\gamma = 25.00$ kN/m³); Poisson's ratio ($\nu = 0.20$).

4. Analysis and results

In order to verify the software and input parameters shown in Figure 7, the load tests performed by Soares et al. (2015) were numerically modeled. Figures 8 shows the load versus settlement curves, either in numerical and experimental

terms, for the isolated raft and for the foundations with one, two and four piles, respectively. To determine the conventional load capacity (P_{max}), the criteria proposed by Décourt (1993, 1995) and the British Standard (BSI, 2015) were adopted, which indicate that the conventional load capacity for pile foundations is equivalent to the load corresponding to a settlement of 10% of the nominal pile diameter (D), i.e. 30 mm in this case.

Based on the load capacity (P_{max}) a Global Safety Factor (GSF) can be applied to obtain the allowable load (P_a). In this study, a GSF equal to 2 was considered, which is the minimum value recommended by the Brazilian National Standard NBR 6122 (ABNT, 2019) for deep foundations (for conventional pile groups, hence assumed herein equal for piled raft types as well). It is important to note that the

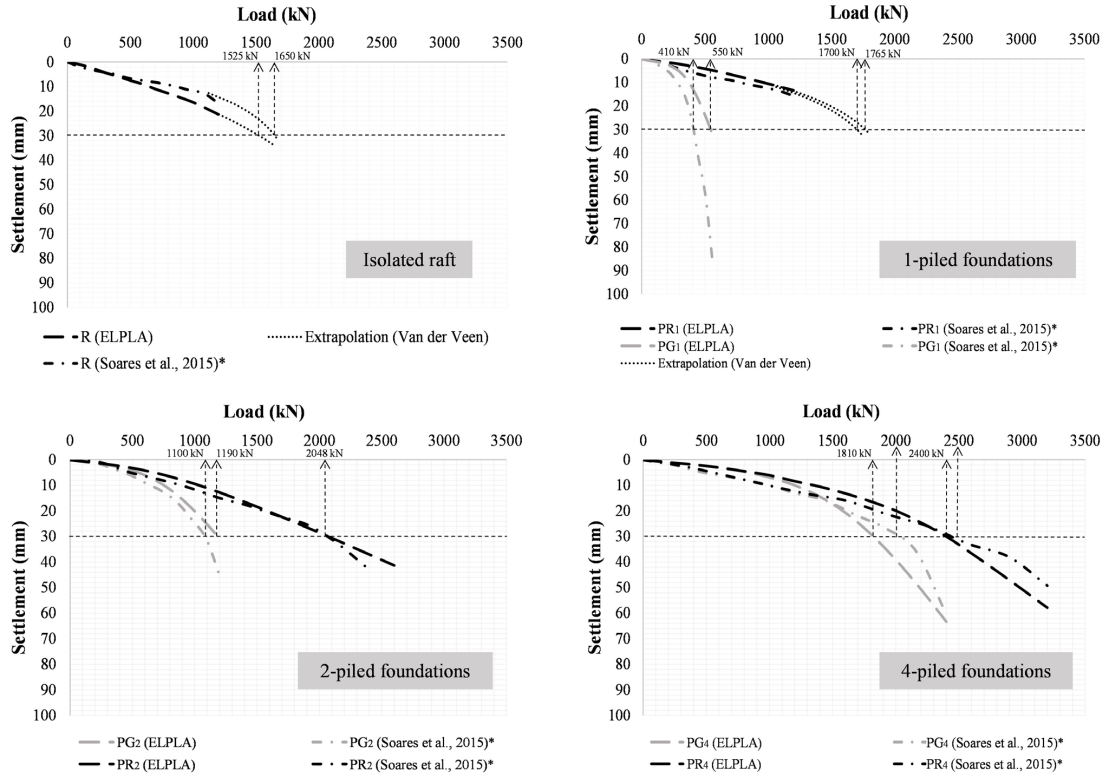


Figure 8. Load/settlement curves (*Experimental Results).

isolated raft and the 1-piled raft did not reach the 30 mm level of settlement, therefore for these types of foundation the load versus settlement curve was extrapolated through the method of van der Veen (1953) to the desired level of settlement.

Table 2 shows the load capacities (P_{max}) and allowable loads (P_a) for all foundation types analyzed using the experimental model proposed by Soares et al. (2015), and the numerical model used in this study. In addition, the load capacities obtained by the two models are compared and it can be seen that the P_{max} variations (δP_{max}) were less than 10% in all the foundations analyzed, except in the PG_1 foundation, where there was a variation of 34.1%. It is also observed that the P_{max} variations were smaller in the piled rafts than in the pile groups, when comparing the numerical and experimental model. The numerical model represents a simplified simulation of what occurs in the field, as verified by the experimental model. In the ELPLA® software, the soil and the piles are represented by nodes, as observed in Chapter 3.2. This may explain the discrepancies found between the numerical and experimental results. However, in general, it can be assumed from Figure 8 that the numerical and experimental load versus settlement curves exhibit similar behavior and converge with each other.

From a global view, considering the results of Figure 8 and Table 2, it can be concluded that the initial geotechnical parameters (Figure 7) led to an adequate closeness between

numerical and experimental results, even though some slight divergence was found for the PG_1 case. By evaluating the numerical and experimental load/settlement curves for displacements equivalent to the allowable load (load to which the foundation will be subjected in the field), a greater proximity between the curves was observed, which demonstrates that it is feasible to use the geotechnical parameters of Figure 7 in the modelling process.

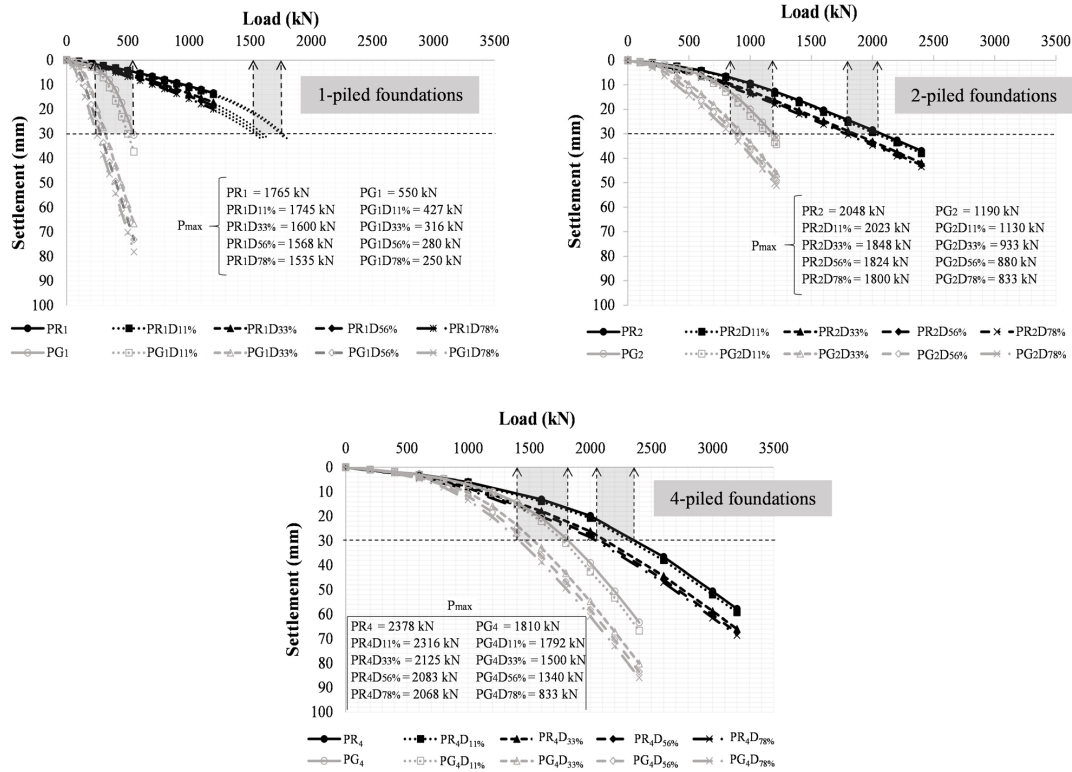
After carrying out the comparative analysis of the ELPLA® 10.1 software with Soares et al. (2015) experimental results, modelling was performed considering the presence of a defective pile whose length was shorter than initially predicted. Four defect levels (D11%, D33%, D56%, and D78%) were considered in which the pile length gradually decreases. Figure 9 shows the load versus settlement curves and the load capacities (P_{max}) obtained for the 1-, 2- and 4-piled rafts and pile groups in the above situations. It can be observed that, as predicted, the presence of the defect promotes a reduction in P_{max} and in the stiffness of all the foundations analyzed, as they assume higher settlements in relation to the intact situation, for the same load levels applied.

The load capacity of foundations obtained in Figure 9 are depicted in relation to the level of defect in the pile. In addition, Figure 10 shows the percentage reductions in the load capacity of defective foundations when compared to the intact case (ΔP_{max}), for each defect level in the pile. It can be seen that the ΔP_{max} values are higher in the pile groups

Table 2. Load capacities and allowable loads obtained through ELPLA® and the experimental model.

Foundation	ELPLA®		Soares et al. (2015) (experimental)		Comparison of numerical and experimental models $\delta P_{max} (\%)*$
	P_{max} (kN)	P_a (kN)	P_{max} (kN)	P_a (kN)	
R	1525.0	762.5	1650.0	825.0	7.6%
PR ₁	1765.0	882.5	1700.0	850.0	3.8%
PR ₂	2048.0	1024.0	2048.0	1024.0	0%
PR ₄	2400.0	1200.0	2500.0	1250.0	4.0%
PG ₁	550.0	275.0	410.0	205.0	34.1%
PG ₂	1190.0	595.0	1100.0	550.0	8.2%
PG ₄	1810.0	905.0	2000.0	1000.0	9.5%

N.B.: $\delta P_{max} = |P_{max, experimental} - P_{max, ELPLA}| / P_{max, experimental}$

**Figure 9.** Load/settlement curves and load capacities in defective situation.

than in the piled rafts, which is expected, since in the latter case, the raft is mobilized in order to compensate for the loss of the defective pile resistance, which promotes a lower reduction of P_{max} of the foundation. In the case of pile groups, redistribution in mobilized loads does occur, with migration from the defective pile to other piles of the system.

Figure 10 also shows that the greatest loss of load capacity was 55% and occurred in PG1D78%, that is, in the group with 1 pile that was 1 m in length. In the piled

raft under the same condition (PR1D78%) the loss of load capacity was 13%, which demonstrates again the positive influence of the raft on the performance of the foundation.

Through Figures 9 and 10, it can be observed that there is a sharper decrease in load capacity and stiffness in all analyzed cases when transitioning from defect D11% to D33%, that is, when the pile length is reduced from 4 m to 3 m. This occurs because, when the pile is shortened to 3 m, it becomes entirely embedded in the fine sand (gray) layer,

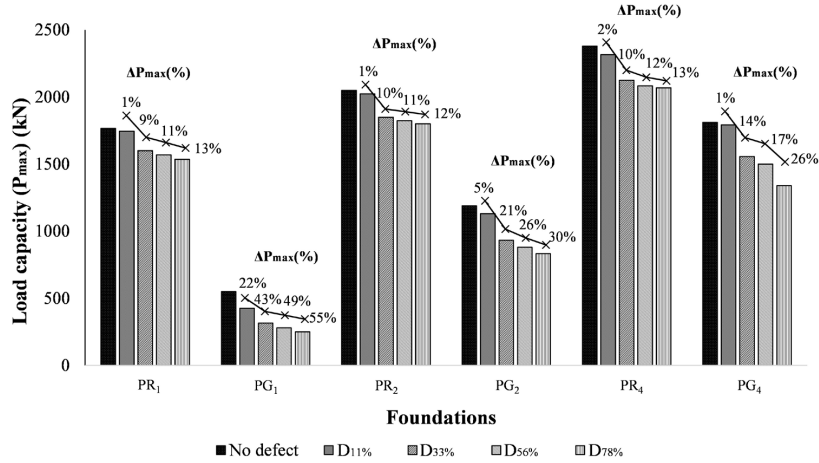


Figure 10. Load capacities of foundations (P_{max}) and their percentage reductions (ΔP_{max}) as a function of the defect level at the pile.

as seen in Figure 6, which has significantly lower resistance and deformability parameters compared to the underlying soil layer, silty fine sand (brown).

Considering the load capacities of all foundations in either intact situation or in the defective four cases, aspects of safety at the rupture of the foundations can be assessed in regard to the defect in the pile. For this, a safety factor was determined for each foundation according to the occurrence of the defect (SF_d), using equation 8:

$$SF_d = \frac{P_{max}}{S} \quad (8)$$

where: P_{max} is the defective foundation capacity and S is the load applied to the foundation. P_{max} varies according to the defect level in the foundation (as the defect level increases, P_{max} decreases, as seen in Figure 10). It was assumed that the applied load S corresponds to the allowable load of the foundation in the intact condition, numerically assessed, that is, 882.5 kN, 1024 kN and 1200 kN for piled rafts with 1, 2 and 4 piles, respectively and 275 kN, 595 kN and 905 kN for pile groups with 1, 2 and 4 piles, respectively. Thus, in a certain foundation, as the defect level increases, the SF_d decreases, as P_{max} decreases with the increase in the defect. In this case S remains constant, as it is equivalent to the allowable load of the foundation in intact condition.

Figure 11 shows SF_d variations as a function of the defect level in the foundations. It can be observed that when there is no defect, SF_d is equal to 2, which is the expected ideal situation, of intact foundation. When there is a defect, SF_d is less than 2 and decreases as the defect level increases. It can be seen that the group with 1 pile of 1 m in length (PG1D78%) represented the most critical situation, in which SF_d assumed the value of 0.91 and, therefore, indicated a rupture of the foundation.

In Figure 10, it was already noted that this case was the most critical, as it assumed the greatest reduction in

load capacity (55%), and this directly interferes with the calculation of the safety factor. The group with a pile of 2 m in length (PG1D56%) is also in a situation of imminent rupture, in which SF_d assumed the value of 1.02. Analyzing the piled rafts under the same conditions (PR1D78% and PR1D56%), it can be observed that the SF_d values are 1.71 and 1.75, respectively, which ensures a satisfactory reserve of resistance even in the presence of the defect. Therefore, the influence of the raft in ensuring the stability of the system is notorious, especially in those cases where there is only one pile in the foundation (SF_d ranged from 0.91 in PG1D78% to 1.71 in PR1D78%, which represents an increase of 88% in the safety factor as a function of the raft-soil contact). Analyzing foundations with 2 and 4 piles at the same defect level, it can be observed that the SF_d varies from 1.4 in the PG2D78% to 1.76 in the PR2D78% (which represents an increase of 26% in SF_d) and from 1.48 in the PG4D78% to 1.74 in the PR4D78% (which indicates an increase of 18% in the safety factor as a function of the raft-soil contact). It can be observed, therefore, that with lesser piles in the foundation, the greater the raft-soil contact contribution, as well as the system's performance.

When loads are applied on a piled raft, one part of the load is mobilized by the raft and the other by the pile(s). Figure 12 shows, for the 1-, 2- and 4-piled rafts, that as the loading level in the foundations increases, the raft will mobilize more load while the piles will have a smaller contribution to the load capacity. This fact was also observed by Lee et al. (2015) in their studies with piled rafts in sand. Alshenawy et al. (2016) also observed that the piles mobilize less load with the increase of loading when the load-settlement behavior is nonlinear. It can also be observed that as the magnitude of the pile defect increases, the raft becomes more mobilized, absorbing more load and tending to compensate for the loss of performance of the defect pile.

As it was done for the determination of SF_d in Figure 11, it was considered that in each foundation a load equivalent

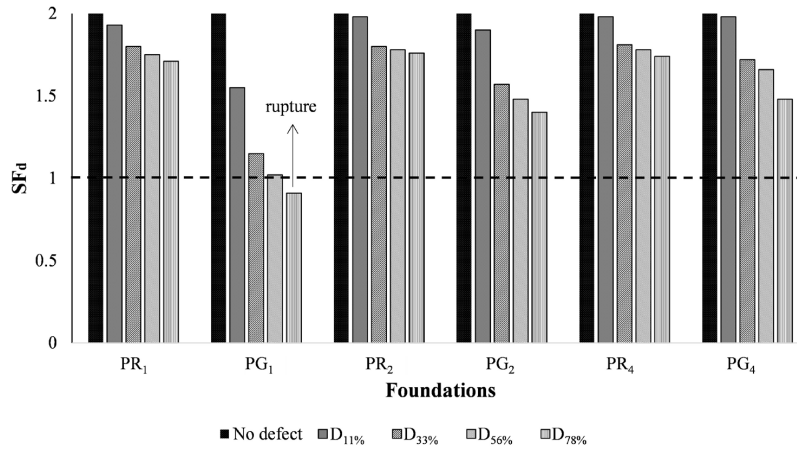


Figure 11. SF_d variation for foundations depending on the level of defect.

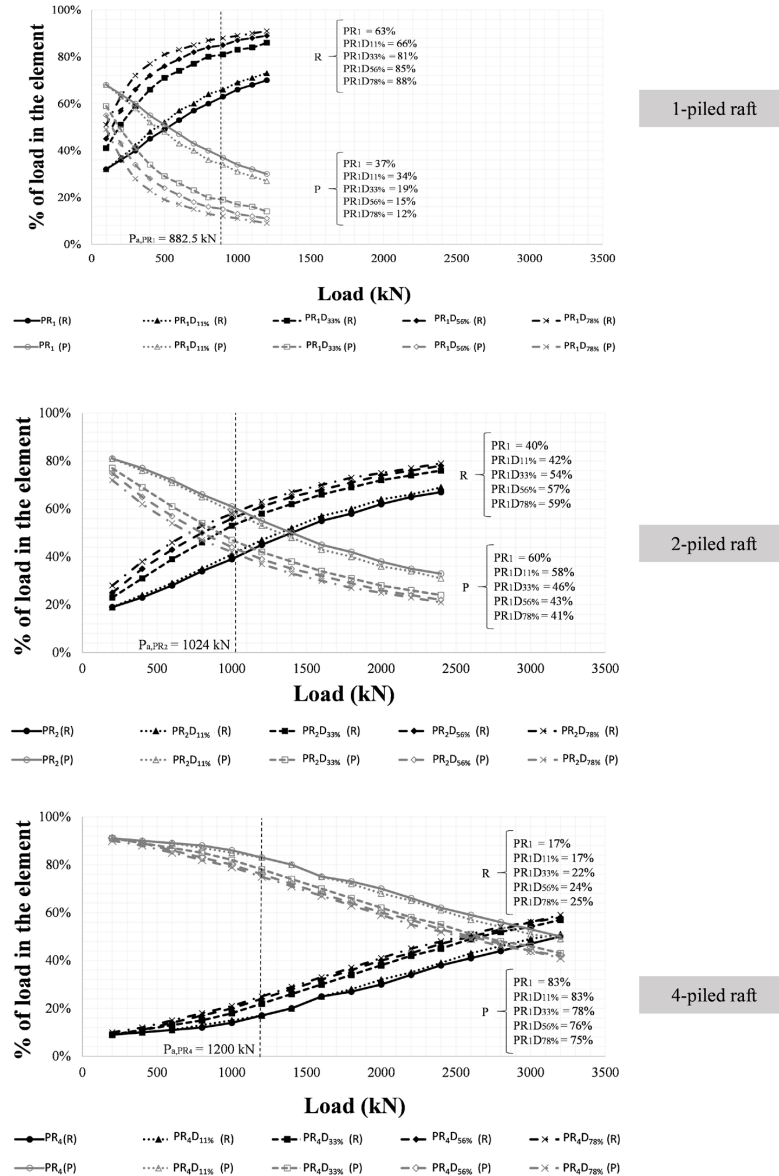


Figure 12. Load mobilization on raft (R) and piles (P) during loading.

to its allowable load in the intact situation (P_a), numerically related, should be applied, that is, 882.5 kN, 1024 kN and 1200 kN for piled rafts with 1, 2 and 4 piles, respectively. Thus, for this level of applied load, Figure 12 highlights the percentage of the load mobilized by the raft (R) and the piles (P) for each defect level, in foundations with 1, 2 and 4 piles, respectively. It can be seen that, in PR1, the raft mobilizes from 63% of the applied load, in the intact situation, to 88%, in the situation with D78% defect, that is, there was an increase of 25% in the load absorbed by the raft in function of the defect. In PR2, this increase was 19% (the raft mobilized 40% of the applied load, in the intact situation, and started to mobilize 59%, in the situation with D78%). Finally, in PR4, there was an increase of 8% in the load absorbed by the raft due to the defect (the raft absorbed 17% of the applied load, in the intact situation, and started to absorb 25%, in the situation with D78%). This fact demonstrates again the importance of the raft on the behavior of the defective foundation and corroborates with aforementioned discussions, regarding the increase of raft contribution with decrease of piles in the system.

The presence of a defective pile in the pile groups and 2- and 4-piled rafts results in differential settlements

when loading is applied. The superficial element (block/raft) rotates towards the defective pile which can be measured by calculating the angular distortion (β). Figure 13 shows how the angular distortion in the superficial element varies during loading for the 2- and 4-piled foundations at different defect levels. It is observed that the angular distortions increase with the increase of loading and the defect level in the pile. Also, it is noticed that the angular distortion values are higher in the pile groups than in the piled rafts.

It is considered again that in each foundation a load equivalent to its allowable load in the intact situation (P_a), numerically calculated, is applied, that is 1024 kN for PR2; 595 kN for PG2; 1200 kN for PR4 and 905 kN for PG4. In this case, Figure 13 specifies, for this load level, the angular distortion values as a function of the defect in foundations with 2 and 4 piles, respectively. In this regard, Bjerrum (1963) established boundary distortion values for a foundation as a function of associated damage.

Based on this, it can be observed that the foundations in pile groups PG2D33%, PG2D56%, PG2D78%, PG4D56% and PG4D78% assume angular distortion values of more than 1/100, which results in a situation of high structural damage to buildings and a high potential for rupture. In

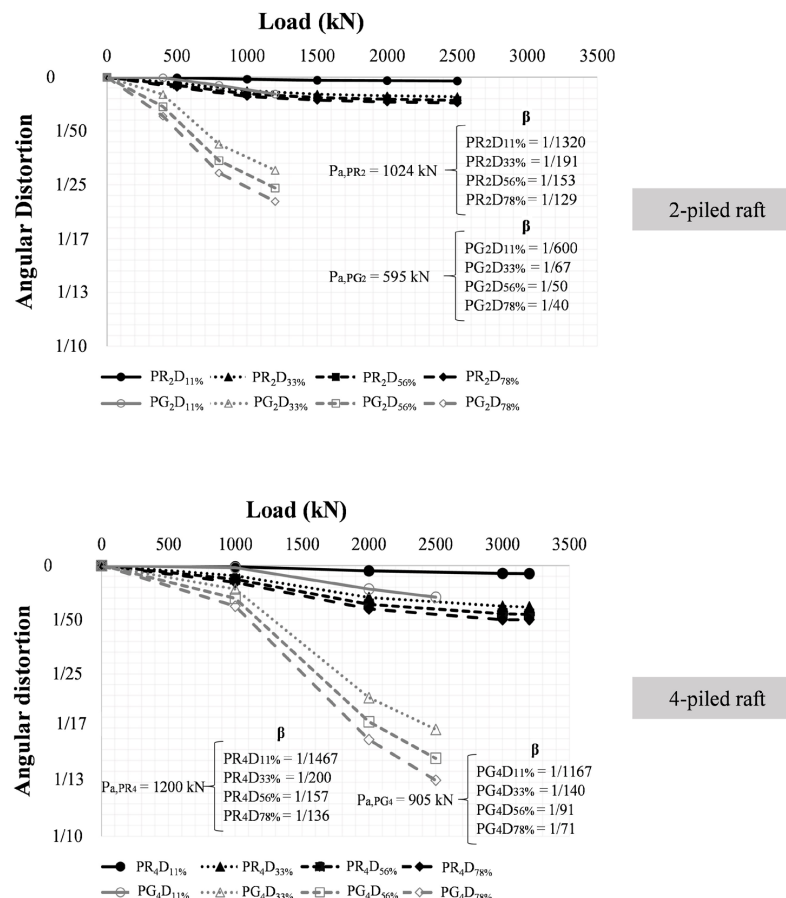


Figure 13. Angular distortion values (β) during loading.

piled raft foundations, the greatest angular distortion found was in PR2D78%, assuming a value of $1/129$, which also represents a situation of structural damage in buildings. In the pile groups, the most critical situation was that of PG2D78%, which assumed a value of $1/40$. That is, although it still assumes values that would induce the structure to pathological manifestations, the design of the piled raft foundation performed better than the pile group foundation.

5. Conclusions

Considering the analyses performed using the ELPLA® software, version 10.1 regarding the stiffness and load capacity of the intact foundations, it can be observed that the results obtained were in agreement with the experimental results of Soares et al. (2015). Therefore, it is argued that tools with simplified calculation methods, such as the one used in this article, should be encouraged because they attend to the needs of everyday engineering practice, which require speed, agility, and simple models to obtain engineering results.

The main conclusions of this study are presented below:

- When considering the percentage reductions of 11%, 33%, 56% and 78% in the length of the pile, a reduction in the system's stiffness was observed, and higher settlements appeared for the same applied load levels, as well as a reduction in the load bearing capacities of the foundations. The most critical situation for the piled rafts occurred in PR1D78% (1-piled raft with 78% reduction in pile length), which had a 13% reduction in load capacity, while for the pile groups, the case with a 78% shorter pile (PG1D78%) had a 55% lower load capacity than the intact situation.
- Considering the application in the foundations of a load equivalent to the allowable load of the intact situation (P_a) obtained numerically, the safety factors could be calculated as a function of the presence of the defect (SF_d), relating the load capacity of the defective foundation and the applied load. It could be observed that there was a reduction in the SF_d with an increase in the defect level and, in the PG1D78% situation (group with 1 pile 78% shorter), the rupture was seen, as the SF_d assumed the value of 0.91. The importance that the raft-soil contact represents in the safety of the foundation was notorious, because in all cases of defects the presence of the raft led to an increase in SF_d .
- Regarding the mobilization of loads between elements of the piled rafts, an increase was observed in the mobilization of the superficial element (raft) with the increase in the load and with the increase in the defect level. For a load level equivalent to the allowable load of the foundation with no defect, in the PR1D78% (1-piled raft with 78% reduction in pile length) the raft will mobilize more than 85% of the

applied load, characterizing the foundation system's performance as typical of a piled raft (with partition of load among raft and piles). The importance of the raft in foundation behavior is well known.

- In the 2- and 4 piled- foundations, in situations where there is a pile defect, there were differential settlements that were evaluated by determining the angular distortions. It could be observed that the angular distortions increased when the load and the defect level increased. For a load equivalent to the allowable load of the foundation without defects, in the piled raft foundations the greatest angular distortion found was in PR2D78% (2-piled raft with a 78% shorter pile), which assumed a value of $1/129$, while in the pile groups the most critical situation was that of PG2D78% (2-pile group with a 78% shorter pile) with a value of $1/40$. Although the raft-soil contact has promoted a reduction in the angular distortion of the foundation, from $1/40$ to $1/129$, it still represents, in this case, structural damage in buildings.

Based on the analyses performed and the results obtained, it could be concluded in general terms, from the present exercise, that the existence of structural defects in a pile changes the behavior of the group foundation, reducing its stiffness and load capacity. It was clear that piled raft types behave in a better way than conventional pile groups, when a defect is present in one of the piles.

Acknowledgements

The authors are grateful for the support provided by Brazilian organizations that promote research CNPq (National Council for Scientific and Technological Development) and CAPES (Coordination for the Improvement of Higher Education Personnel). This research was also carried out (among other groups) with support from the foundation and in situ testing research group of the Univ. of Brasília, GPFees (<https://rpcunha.wixsite.com/gpfees>).

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Felipe Carlos de Araújo Leal: conceptualization, methodology, visualization, writing – original draft, writing – review & editing, funding acquisition, resources, supervision. Osvaldo de Freitas Neto: conceptualization, methodology, visualization, writing – original draft, writing – review & editing, funding acquisition, resources, supervision. Wilson Cartaxo Soares: conceptualization, methodology, visualization,

writing - original draft, writing - review & editing, funding acquisition, resources, supervision. Roberto Quental Coutinho: conceptualization, methodology, visualization, funding acquisition, resources, supervision. Renato Pinto da Cunha: visualization, writing – original draft, writing - review & editing, supervision.

Data availability

All data produced or examined in the course of the current study are included in this article.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

k	Soil stiffness
w	Settlement
D	Pile diameter
E_s	Modulus of elasticity
GSF	Global Safety Factor
N_{SPT}	Penetration resistance index
P_a	Allowable load
P_{max}	Load capacity
Q	Contact force
S	Load applied to the foundation
SF_d	Safety factor with the occurrence of the defect
SPT	Standard Penetration Test
β	Angular distortion
γ	Specific weight
$\delta P_{max} P_{max}$	variations
ν	Poisson's ratio
ϕ	Friction angle
ΔP_{max}	Percentage reductions in the load capacity of defective foundations when compared to the intact case.

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Analysis of the incorporation of sludge from a water treatment plant (WTP) into soil for use in impermeable layers of a sanitary landfill

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Article

Keywords

WTP sludge
Waterproofing layer
Landfills

Abstract

The collection and treatment of water are crucial to ensure the provision of safe drinking water for the population. Raw water is treated using physical and chemical processes to produce treated water for distribution to cities, but the processes also generate waste. The waste from Water Treatment Plants (WTP) is known as WTP sludge. Disposing of WTP sludge in an environmentally friendly manner is crucial. In order to minimize the environmental impact, waterproof barriers are designed in landfills to prevent the contamination of underground water resources provoked by the percolation of generated liquids. This study assessed the geotechnical feasibility of using WTP sludge as a waterproofing material in landfills. The benefits of this process include a decrease in soil extraction from quarries and the environmentally sound disposal of waste. The approach is based on the growing demand for sustainable practices in the treatment and disposal of waste, while also seeking a technical alternative to partially replace soil in geotechnical structures, aiming for resource conservation. The study characterized the sludge from the WTP and conducted tests on compaction, hydraulic conductivity, and oedometer consolidation. Two different fractions of the soil-sludge mixture (2.5% and 5%) were used, with moisture contents ranging from 17.62 to 46.71%. The results showed that the addition of sludge either maintained (2.5%) or improved (5%) the permeability characteristics of the residual soil under study. In optimal conditions, the permeability was approximately 10^{-9} m/s, which renders it suitable for use in waterproofing layers for landfills.

1. Introduction

Since the 21st century, urbanization has grown all over the world. Unfortunately, population growth in urban areas has been largely unplanned, leading to the construction of homes in irregular areas. As a result, the lack of basic infrastructure for the population, the importance of implementing improvements to the basic sanitation system, and the availability of drinking water became evident (Rangel, 2005).

According to research carried out for the World Bank by Kaza et al. (2018), around 1.4 billion tons of municipal solid waste (MSW) are generated worldwide every year, which represents 1.27 kg per capita per day.

In addition to MSW, sludge from water or sewage treatment processes is also disposed of in landfills or dumps. In these treatment processes, substances are added to promote the agglutination of particles, which then settle as sediment, resulting in water treatment plant (WTP) sludge (Rodrigues

& Holanda, 2013). The characteristics of WTP waste depend on the quality of the raw water, the chemicals used, and the treatment technology applied. Studying the composition of sludge is important to improve understanding of the resulting waste and establish reuse methodologies. Its disposal is a major environmental problem, as around 1 to 5% of the volume of raw water treated results in solid content, i.e. WTP sludge (AWWA, 1978; Katayama, 2012).

On the other hand, the efficiency of landfills is closely related to the performance of the layers used to waterproof them. To minimize the infiltration of leachate into groundwater, it is necessary to use layers with low hydraulic conductivity and high shear strength (Dixon et al., 1999).

The lining layers in landfills, whether they are bottom layers, daily covers, or final covers, have multiple purposes such as isolating the waste from the external environment, draining percolate and gases, and separating the landfill into cells. The layers can be made of synthetic materials (such as

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Submitted on June 7, 2024; Final Acceptance on May 6, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.004524>



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geosynthetic barriers), of natural materials, such as compacted soils, of mixtures of soil with other low permeability materials, or even a combination of geosynthetics and compacted soils. Therefore, using WTP sludge for geotechnical purposes, such as mixing it with soil for bottom waterproofing layers, daily cover, and final cover for landfills, can be a technically promising solution for its disposal, provided that the relevant geotechnical parameters are respected, and the behavior of the soil-sludge mixture proves to be suitable for use in landfill layers.

Daniel & Koerner (1993) and Sharma & Reddy (2004) studied the use of different types of soil as cover layers and concluded that for a compacted soil cover system, the material used must meet certain criteria to achieve good performance, such as: having low saturated permeability ($k_s \leq 10^{-7}$ m/s) to minimize water infiltration into the residual material layers below the cover; proper soil compaction to ensure that the compaction energy is evenly distributed across all soil layers; and the compacted cover layer must not crack.

The study by Rahardjo et al. (2017) used compacted soil to minimize rainwater infiltration into a landfill in Singapore. The results demonstrated good performance of this cover system, minimizing leachate movement to the surrounding area.

In another study, Rahardjo et al. (2018) identified that the presence of concrete particles reduced the water retention and storage capacity of surface soil by 45% compared to soil without concrete particles. In addition to the low water retention capacity, the mixture also exhibited a lower permeability coefficient, which was due to the finer concrete particles in the mixture. In research on the use of another type of waste, Ou et al. (2022) presented experimental results showing that the reuse of residual slag from tunnel excavations, in the appropriate proportion, can be recycled as one of the components of injection material for tunnel engineering, demonstrating performance similar to the original grouting injection material mix.

Gonçalves et al. (2017) point out that using WTP sludge in landfill layers to be justified requires the geotechnical criteria for landfill waterproofing such as adequate granulometric distribution and low permeability. If the parameters of the mixture are found to be adequate, its use has the advantage of not reducing the useful life of the landfill and allowing for the decrease of soil borrow areas for waste containment operationalization.

Although still considered scarce, there has been an observed increase in the number of studies focusing on the geotechnical properties of WTP sludge in recent years (Bosco et al., 2021).

To investigate potential applications of sludge and avoid improper disposal, studies have explored the possibility of using WTP sludge, as well as wastewater treatment plant (WWTP) sludge and leachate treatment plant (LTP) sludge, as a construction element, either associated with soil or not.

Regarding the application of WTP and WWTP sludge, Castilhos Junior et al. (2012) conducted physicochemical characterizations of both types of sludge collected from the treatment plants in Jurerê Internacional, located in Florianópolis, Santa Catarina. The authors also characterized mixtures of sludge with quicklime, for chemical stabilization, and clayey soil. They concluded that, although WTP and WWTP sludge are not suitable for direct use as cover material, their mixtures with soil and lime met geotechnical requirements for application in landfill cover layers.

The study by Mazzutti et al. (2023) indicated that for the WTP sludge from the water treatment plant in Frederico Westphalen - RS, the soil-sludge mixtures studied could be used as construction layers in landfills. Notably, the mixture with 30% sludge addition was identified as the best for application in base and cover layers. Samples of this mixture achieved the required permeability coefficient for landfill applications and exhibited the highest unconfined compressive strength among all samples.

Knierim et al. (2023) conducted a technical feasibility analysis of using WTP sludge-soil mixtures from Santa Maria - RS in landfill layers, based on minimum permeability coefficient values established by various standards. The authors concluded that the mixture incorporating 15% WTP sludge performed best, as it met the most stringent permeability requirements, making it suitable for application in landfill base and cover layers.

Scapin (2021) analyzed samples of WTP sludge-soil mixtures from Santa Maria - RS under chemical, physical, and biological parameters. The author constructed experimental cells with municipal solid waste and, after analyzing the waste mass inside the cells and the generated leachate, concluded that the sludge did not increase environmentally harmful parameters. Scapin (2021) concluded that the sludge could be used in base and cover layers in landfills, with 15% sludge incorporated into the layers.

This paper presents the initial results of the characterization of WTP sludge obtained from the WTP located in the city of Pinhais, in the state of Paraná, Brazil, aiming to contribute to the technical literature by employing a sludge that has not been studied so far for applications of this nature. It is noteworthy that the research focused on the use of the raw material for geotechnical applications without the need for pre-processing. For the characterization of the material, tests commonly used in soil characterization were employed.

2. Materials and methods

The research comprised three stages, according to the following flowcharts. In the first stage, the soil was characterized (Figure 1); in the second stage, the sludge was characterized (Figure 2) and in the third stage tests were carried out with the soil, sludge, and mixtures in the percentages in terms of 97.5% soil - 2.5% sludge (raw) and 95% soil - 5% sludge (raw) (Figure 3). The proportions were defined

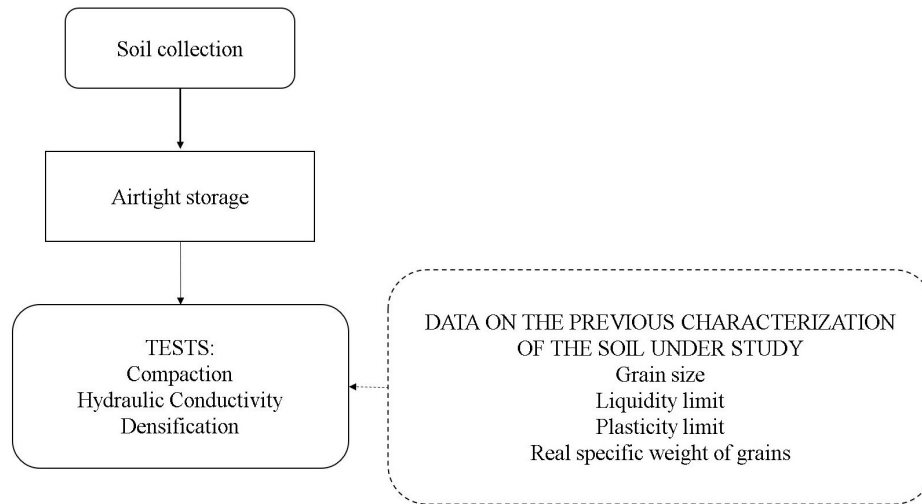


Figure 1. Soil research flowchart.

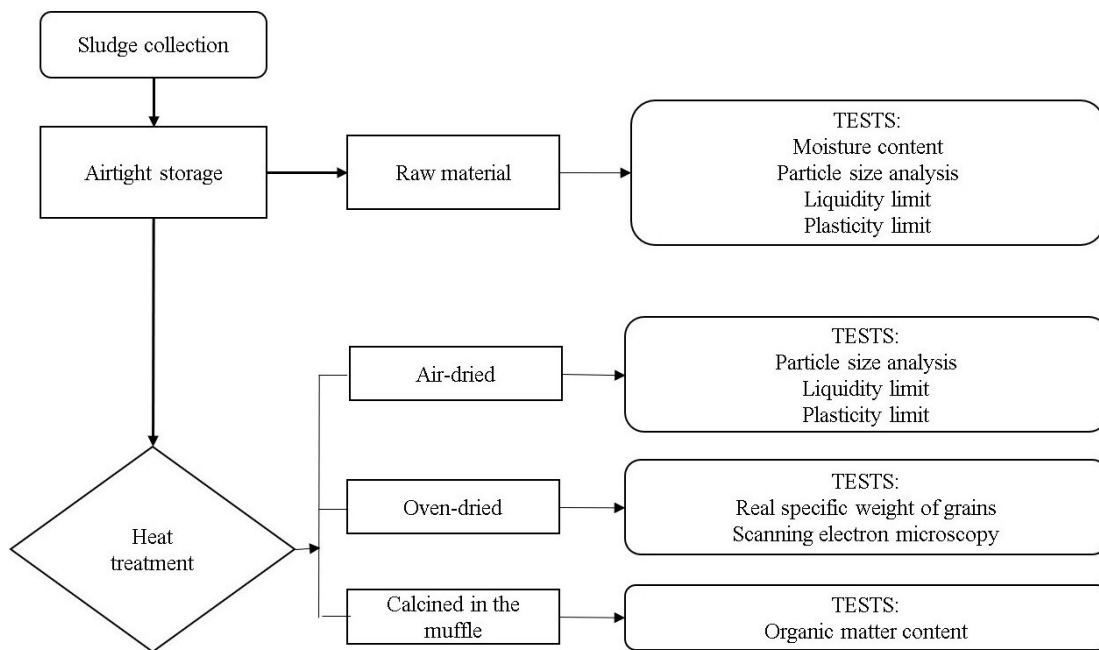


Figure 2. Sludge research flowchart.

considering the objective of analyzing the behavior of the soil with lower sludge incorporations, aiming to observe the influence of the addition of sludge on the properties of the natural soil in quantities lower than those usually reported in the literature on the subject. As observed by Montalvan (2016) and Gonçalves et al. (2017), similar studies using proportions ranging from 15% to 100% were observed in the literature. It is important to emphasize that the proportions used in the mixtures were calculated in terms of the dry mass of the components, but that the incorporation of sludge into the soil occurred in the raw state, leading to the use of larger

masses of waste due to the high moisture content intrinsic to the sludge in the disposal state.

The collected soil was mixed with the sludge to form a waterproofing layer for the embankment, and its behavior was analyzed through laboratory tests. The soil samples studied were provided by the Experimental Field for Geotechnical Studies in Ponta Grossa (*Campo Experimental de Estudos Geotécnicos de Ponta Grossa*), which is predominantly silty-clay and typically found in the Campos Gerais region. Its characterization has been reported in recent publications for different geotechnical applications (Anibele et al., 2020;

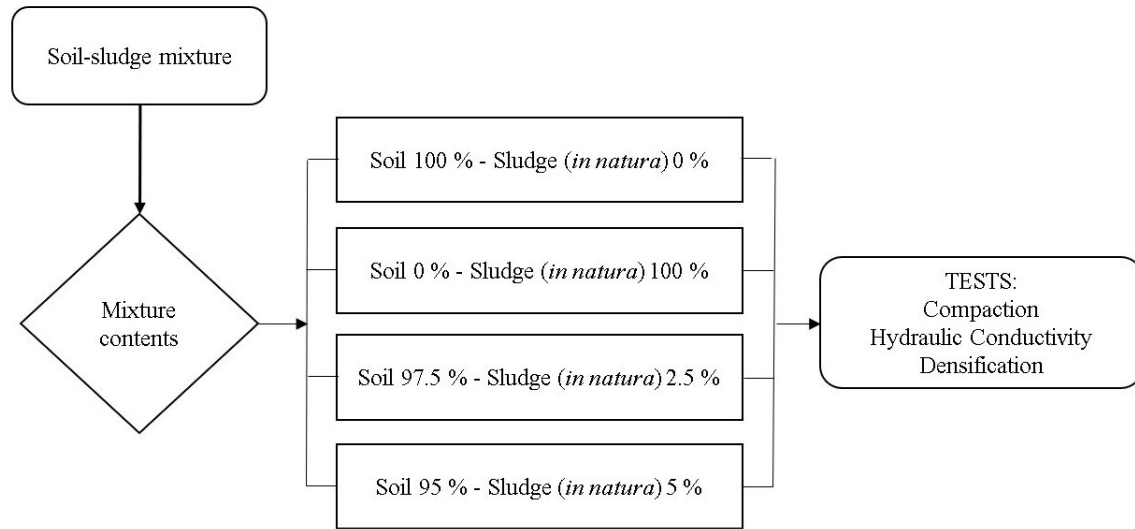


Figure 3. Flowchart of research on soil-sludge mixtures.

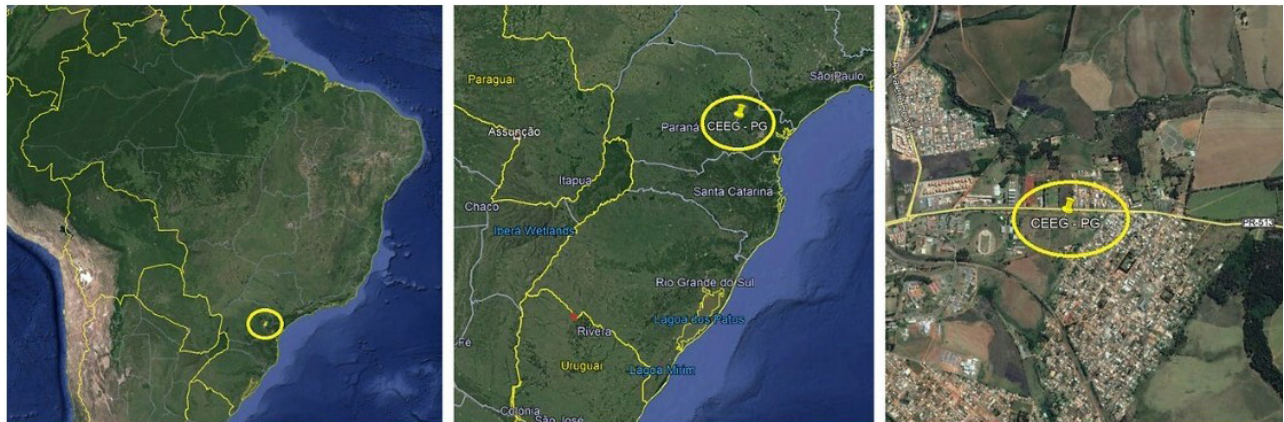


Figure 4. Experimental Field for Geotechnical Studies in Ponta Grossa location.

Pereira & Vargas, 2019; Tonus et al., 2022). The geotechnical tests conducted to characterize the natural soil followed the guidelines of NBR 6457: Soil Samples - Preparation for Compaction Tests and Characterization Tests (ABNT, 2016a).

Centrifuged sludge from the Iraí WTP, located in the municipality of Pinhais, Paraná, was used in this study (Figure 4). This plant supplies water to over 1 million inhabitants of the city of Curitiba and the metropolitan region (Oliveira, 2010). At this WTP, the raw water undergoes complete treatment through a process that includes flotation, coagulation, flocculation, filtration, disinfection, pH correction, and chlorination. The decanters have been replaced by floto-filtration, which is a physical separation process based on the dissolved air flotation process. In this process, water saturated with air is mixed with the flocculated water in an expansion chamber that precedes the floto-filters (flotation in the top layer of the filters).

The sludge is scraped from the surface and dehydrated through centrifugation. The resulting sludge used in the research had an average solids content of 22.58% and was in a pasty state. It underwent physical-chemical and geotechnical characterization to determine its potential as a substitute for dry matter when mixed with soil. The geotechnical characterization included moisture content tests (NBR 6457/2016), granulometric analysis (ABNT, 2016d), actual specific weight (DNER-ME 093/1994), Atterberg limits (liquidity and plasticity limits) (NBR 6459/2016 and 7180/2016), and organic matter content (ABNT, 2022). To determine the moisture content and specific weight of the grains, the sludge underwent drying in an oven at 60 ± 5 °C and 105 ± 5 °C. This allowed for observation of variations in these parameters while considering the possible presence and influence of organic matter. Atterberg limits tests were conducted on sludge samples prepared under

three conditions: air-dried, oven-dried at 60 ± 5 °C, and unprocessed.

Particle size analysis of the sludge was conducted on two samples: raw sludge and dried sludge using the Bettersizer 2600 Particle Size Analyzer. Chemical characterization was performed on three crumbled and dried sludge samples in an oven at 60 ± 5 °C, using Scanning Electron Microscopy (SEM) to obtain images and point chemical analysis through Energy Dispersive Spectroscopy (EDS).

The soil and sludge samples were characterized individually before being mixed in two different proportions: 97.5% soil - 2.5% sludge (raw) and 95% soil – 5% sludge (raw). The purpose of this study was to investigate the effect of sludge addition levels on the geotechnical behavior of the mixtures. Compaction, hydraulic conductivity, and oedometer consolidation tests were conducted for both percentages of soil-sludge mixture, as well as for samples of sludge alone.

The objective of this phase was to compare the characteristics of the soil-sludge mixtures with those specified for landfill-cover soils.

3. Analysis and results

Tests carried out on samples taken at different depths, according to DNER-ME 093 (DNER-ME, 1994), revealed that the actual specific weight of soil grains ranges from 26.3 to 28.5 kN/m³. The consistency characteristics, obtained through tests defined by NBR 6459 (ABNT, 2016b) and NBR 7180 (ABNT, 2016c), shows in the Table 1, indicate that the soil's liquidity limit (LL) falls between 31% and 43%, characterizing it as a non-plastic soil due to the lack of a plasticity index (Anibele et al., 2020).

After characterizing the soil, we proceeded to analyze the WTP sludge. As sludge is composed of suspended solids from raw water treatment, it is expected to contain a certain percentage of organic matter. Therefore, we dried sludge samples in an oven at 60 ± 5 °C and 105 ± 5 °C to compare the results. Regarding moisture content, the sample yielded values of 342.87% when dried in an oven for 24 hours at 60 ± 5 °C and 368.66% when dried in an oven for 24 hours at 105 ± 5 °C.

As there was a variation in moisture content between the two drying temperatures, it is considered that there is an organic matter content being volatilized at higher temperatures. Since the aim of the study was to use natural sludge without undergoing a drying process, the moisture content value of 60 ± 5 °C drying, applicable to soils with organic matter according to NBR 6457/2016, was adopted.

The specific gravity values of the grains were 20.51 kN/m³ for samples dried in an oven at 60 ± 5 °C and 21.10 kN/m³ for samples dried in an oven at 105 ± 5 °C. This difference in moisture content and the low specific gravity values, when compared to soils, may indicate the presence of a higher amount of organic matter, which is susceptible to volatilization at higher temperatures. This point is reinforced when evaluating the result of the organic matter content test, which indicated 51.72% when the samples were dried in a muffle oven at 700 °C for 2 hours. The obtained data at this stage of the characterization is consistent with previous studies by Roque et al. (2021), Montalvan (2016), and Scapin (2021).

O'Kelly (2016) found that the specific mass of WTP sludge solids decreases with an increase in organic matter content. Therefore, a high percentage of organic matter leads to a decrease in the specific mass value. Furthermore, it is expected that higher temperatures will result in greater solubilization of organic matter. The values obtained for the sludge from the Iraí WTP are consistent with the expected behavior found in the literature.

Particle size analysis was performed using a Bettersizer 2600 particle size analyzer with two samples of sludge: raw and dried at room temperature. The curves for these two types of samples were chosen to better visualize the impact of drying on waste particle size (Figure 5).

The granulometry curve of the dried sludge shows a shift to the right compared to the raw sludge. This reaction may be due to the sludge's behavior during water loss. As the sludge dries, its particles agglutinate, forming larger, strongly cemented lumps, resulting in a granular texture (Montalvan, 2016).

Regarding the consistency limits of the sludge, both tests were conducted using raw sludge and sludge dried at room temperature. The LL values obtained were 100% for sludge dried at room temperature and crumbled, and 487% for raw samples.

Regarding the plasticity limit (LP), only the raw sample exhibited plasticity with an LP value of 392%. This indicates a Plasticity Index of 95%, which indicates that the raw sludge is highly plastic. These values are comparable in magnitude to those found in other studies of sludge in this state, such as Roque et al. (2021). Plastic properties are influenced by moisture content, particle shape, particle size, and the chemical and mineralogical composition of the material. According to Scapin (2021), the sludge loses its plastic characteristics during the drying process, indicating changes in other factors besides moisture content.

Table 1. Results of characterization tests carried out in the soil (Anibele et al., 2020).

Sample	Actual specific weight (kN/m ³)	Liquidity limit (%)	Plasticity limit (%)	Plasticity index (%)
Soil	26.3 - 28.50	31 - 43	-	NP

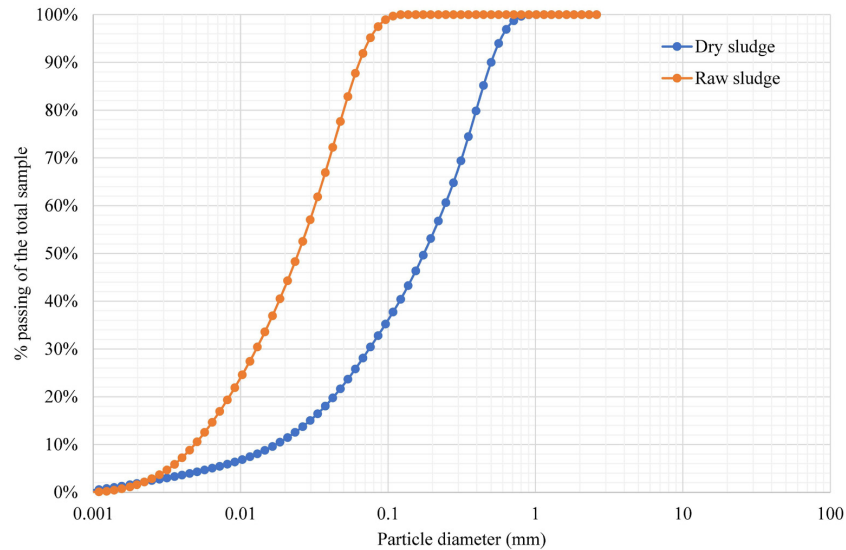


Figure 5. Particle size curves of dry and raw sludge.

The behavior of dry sludge differed significantly from other types. The liquidity limit was found to be 100%. Upon testing for plasticity limit, the dry sludge was classified as non-plastic.

Based on the granulometric analysis and Atterberg limits, it was determined that raw sludge is classified as an organic clay (OH) within the Unified Soil Classification System (SUCS) due to its high organic matter content and non-plasticity after drying (Roque et al., 2021). Dry sludge, however, has a different classification as it contains less than 50% fines. Based on the SUCS classification, dry sludge is classified as silty sand (SM).

The EDS analyses revealed that the sludge contains an average of 22.05% Carbon (C), 33.59% Oxygen (O), 21.54% Aluminium (Al), 6.82% Silicon (Si), 1.09% Sulphur (S), and 14.91% Iron (Fe). The micrographs show that, even at the highest magnification, the sludge components still occur in an agglomerated form, with fragments in the colloidal range, confirming the material's extremely fine nature.

Compaction tests were conducted for all variations of soil-sludge mixtures (0%, 2.5%, and 5%) using raw sludge. A compaction test was not performed on the pure sludge sample due to its semi-solid state and high moisture content, which made it impossible to compact.

Soil and sludge mixtures were created using the aforementioned soil. Compaction tests were conducted following the Normal Proctor energy method as per NBR 7182 (ABNT, 2016e). Different combinations of sludge were incorporated to replace the dry mass portion of the soil, including fresh sludge at two levels (2.5% and 5% of solids content) and dried and crumbled sludge (5% of solids content). Figure 6 displays the compaction curves of the amended materials in comparison to the compaction curve of the pure soil (soil-sludge 0%) and includes the saturation curves.

It can be observed that the inclusion of raw sludge has modified the position of the compaction curve, resulting in an increase in the optimum moisture content and a decrease in the dry apparent specific weight. This behavior is expected for soils with higher clay content (Pinto, 2006). When sludge was used in its dry and crumbled state, the curve changed very little, remaining close to the curve of the original soil. This behavior is consistent with the differences observed in the consistency tests carried out on raw samples and dry and crumbled samples. The former shows high plasticity indexes, while the latter shows non-plastic behavior, indicating the influence of the sludge preparation processes on the mixture behavior.

The results of the soil compaction tests indicate that the maximum dry apparent specific weight value of the soil under Normal Proctor conditions, governed by NBR 7182 (ABNT, 2016e), was 14.9 kN/m³, associated with an optimum moisture content of 24.6%. These values were crucial in defining the three compaction variations studied. The Table 2 shows the optimum moisture content and maximum apparent dry weight for each case.

For this study, tests were conducted on the application of sludge in low-permeability layers. To incorporate raw sludge into the soil mass, quantities of 2.5% and 5% were evaluated in the compaction tests. This option was deemed optimal because it allows for the sludge to be delivered directly to the WTP without requiring pre-treatment. Additionally, it leads to compaction of the soil, creating an impermeable layer suitable for use with clay materials.

The hydraulic conductivity test was conducted using a rigid-wall permeameter with a variable load, following the guidelines of NBR 14545 (ABNT, 2021), based on the results of previous tests.

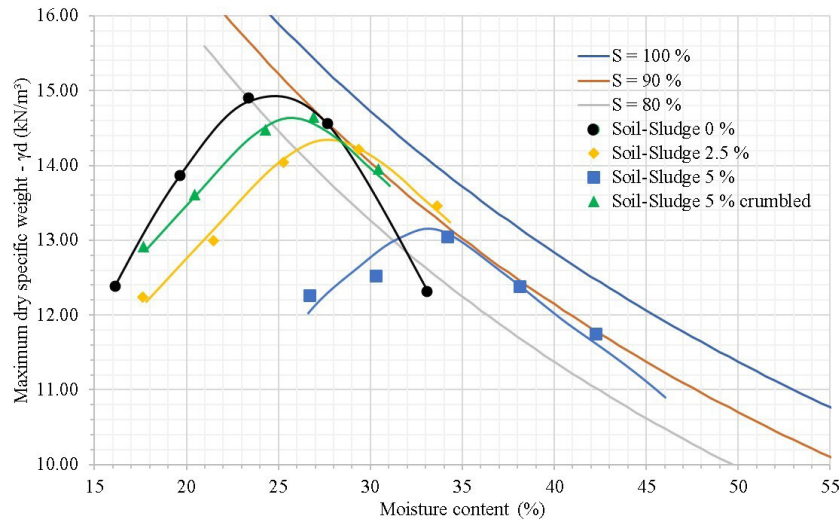


Figure 6. Compaction curves of soil-sludge mixtures.

Table 2. Optimum compaction values for pure soil and soil-sludge mixtures.

Variable	Sample evaluated		
	Pure Soil	Soil-Sludge 2.5%	Soil-Sludge 5%
Optimum moisture (%)	24.6	28.2	34.2
Maximum apparent dry unit weight (kN/m ³)	14.98	14.28	13.05

Table 3. Hydraulic conductivities obtained for pure soil and soil-sludge mixtures.

Variable	Sample evaluated		
	Pure Soil	Soil-Sludge 2.5%	Soil-Sludge 5%
Hydraulic conductivity (m/s)	8.5×10^{-8}	1.8×10^{-8}	4.9×10^{-9}

The samples for the hydraulic conductivity test were prepared with maximum compaction, meaning that compaction energy was applied to reach the γ_d and $\omega_{optimum}$ values. The hydraulic conductivity of the material was affected by the variation in the percentage of sludge applied to the mixtures. The raw sludge used in the tests has a high percentage of fines, as seen in the particle size curve. This portion of waste in the colloidal range helped to reduce the pore size between grains in the mixture, thus reducing the hydraulic conductivity coefficients. As a result, the mixtures were less permeable when compared to pure soil (Ferreira et al., 2011).

The results indicated that the addition of sludge maintained (2.5%) or improved (5%) the permeability characteristics of the residual soil under investigation. In the best condition, the hydraulic conductivity was approximately 10^{-9} m/s, making it ideal for waterproofing applications, such as mineral barriers in landfills. Table 3 displays the results of the tests conducted on the pure soil and with the raw sludge additions.

The deformability study of the soil, sludge, and soil-sludge mixtures was conducted through oedometer compression tests on compacted specimens, following the recommendations

of NBR 7182 (ABNT, 2016e). The samples were molded with the incorporation of raw sludge to verify the feasibility of using the material without prior treatment, apart from that already carried out at the WTP, as this would allow for the conservation of natural resources and add value to the waste. The oedometer compression tests were performed following the recommendations of NBR 16853 (ABNT, 2020), which included two preloading stages, six loading stages, and three unloading stages. The stresses applied at each stage are presented in Table 4. The stress level adopted as the limit for the tests (400 kPa) was determined based on the maximum expected surcharge values for the evaluated scenario of application in sanitary landfills.

For the execution of the test, displacement stabilizations between loading increments were observed. The acquisition time interval was defined so that the time intervals between each acquisition follow an exact square root sequence, as indicated by ASTM D 2435M-11 (ASTM, 2011). The samples were prepared within a circular mold with a diameter of 50.5 mm and a height of 20 mm, with mass control to achieve the maximum dry bulk density values for each sample. With the mold integrated into the consolidation cell and positioned at

the center of the equipment base, the material sample was saturated for the test execution.

For the calculation of the compression coefficients (C_c), related to soil compression under load during the primary consolidation phase, and recompression coefficients (C_r), related to the deformability of the soil when reloaded after being unloaded, the Equation 1 was used for both calculations.

$$C_{c,r} = \frac{e_2 - e_1}{\log_{10} p_2 - \log_{10} p_1} \quad (1)$$

where: e_1 e e_2 area the void ratios corresponding to two arbitrary points on the recompression or virgin compression lines; p_1 and p_2 are the pressures associated with the void ratios e_1 and e_2 .

Table 5 presents the physical properties of the specimens. The pure sludge was tested in its raw condition, while the soil-sludge mixtures were tested under optimum compaction conditions. Figures 7 and 8 display the consolidation curves of the soil, sludge, and soil-sludge mixtures obtained in this test.

The soil-sludge samples molded under the optimum moisture conditions have a higher initial void ratio, as indicated by the maximum dry specific weight (γ_d) obtained in the compaction curve. This leads to greater variations in the void ratio under the same loading stages compared to the sample without the addition of soil sludge. Although compressibility is higher in mixtures with a higher sludge content, hydraulic conductivity is lower in these cases.

Table 4. Loads applied in the consolidation test stages.

Step	Stage	Applied load (kPa)
Preload	1°	2
	2°	5
Load	1°	12
	2°	25
	3°	50
	4°	100
	5°	200
	6°	400
Unloading	1°	200
	2°	50
	3°	12

This indicates that, after consolidation by densification, the hydraulic conductivity of the mixture tends to decrease over time. This is a desirable characteristic for waterproofing layer materials in landfills.

For soil compacted under optimum conditions, the compaction index (C_c) was 0.046 and the recompression index (C_r) was 0.028. For the 2.5% soil-sludge mixture, the C_c increased to 0.051 and the C_r to 0.039. The 5% soil-sludge mixture showed the highest values, with C_c of 0.080 and C_r of 0.060. The compression and recompression results for both soil-sludge compositions were found to be higher than for pure soil, indicating that the mixtures are more compressible than pure soil compacted to the optimum moisture level. This is supported by the high compressibility of the pure sludge, which is unable to be compacted in the oedometric ring due to its consistency. This is evidenced by the C_c value of 1.399 and the C_r value of 0.691 (Table 6).

When analyzing the results from Table 6 and Figures 7 and 8, it is observed that the incorporation of WTP sludge influenced the C_c values of the soil-sludge mixtures. This trend was also noted by Boscov et al. (2021), where the authors concluded that sludge could increase the compressibility of the sample. Additionally, the sludge sample itself presented a C_c value approximately 30 times greater than that of the soil. This characteristic can be considered undesirable for applications in landfill layers that need to maintain their initial thicknesses. Preserving the geometry is important for maintaining other geotechnical characteristics, such as impermeability and strength, when subjected to progressive overloads caused by landfill height increases.

In Figure 7, both C_c and C_r increased progressively as the sludge percentage increased, with the difference between C_c and C_r values growing closer to that of the raw sludge sample. Observing this behavior corroborates the interpretation that higher sludge additions lead to greater compressibility and a higher tendency for compression after stress relief. Furthermore, the sludge exhibited a C_r value approximately 24 times greater than the residual soil. This trend is consistent when compared to C_c , as recompression shows a smaller void ratio variation, considering the elastic nature of the compressibility curve.

The high compressibility of the raw sludge sample (Figure 8) is justified by the inability to compact it in the consolidation ring due to its consistency. The C_c value of

Table 5. Initial physical properties of the samples for the consolidation tests.

Sample	Maximum dry specific weight	Optimum moisture	Void ratio
	γ_d (kN/m ³)	$w_{optimum}$ (%)	e_i
Sludge	2.39*	342.87*	7.828*
Soil-Sludge 0%	15.15	24.55	0.761
Soil-Sludge 2.5%	14.22	28.20	0.871
Soil-Sludge 5%	12.90	34.20	1.058

*Data from the raw sludge used to carry out the test. It was not possible to compact it due to its consistency.

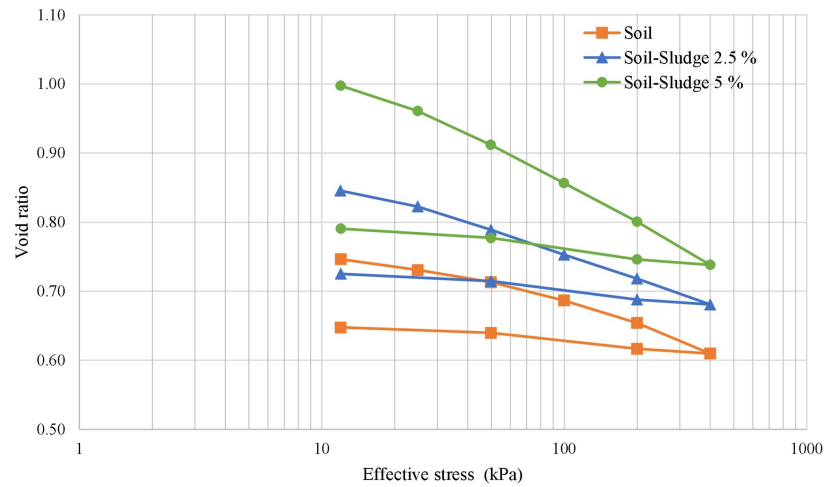


Figure 7. Consolidation curves - Soil and soil-sludge mixtures.

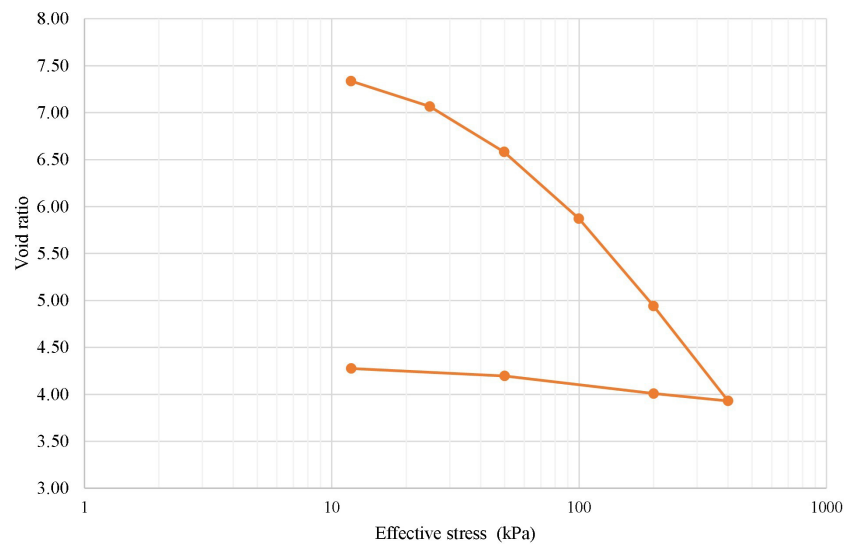


Figure 8. Consolidation curve - Sludge.

Table 6. Evolution and recompression index of soil, sludge and soil-sludge sample.

Sample	Compression index (C_c)	Recompression index (C_r)
Sludge	1.399	0.691
Soil-Sludge 0%	0.046	0.028
Soil-Sludge 2.5%	0.051	0.039
Soil-Sludge 5%	0.080	0.060

1.399 is considered compatible with those found by O'Kelly (2016). The soil-sludge mixtures exhibited C_c and C_r values closer to those of the soil, despite showing the previously mentioned trend of increasing coefficients as the sludge percentage in the mixture increased.

The compression index results were lower than those obtained by Montalvan (2016) in his study incorporating 16.7%, 20%, and 25% sludge into sandy soil, where compression index values ranged from 0.13 to 0.19, and recompression index values ranged from 0.02 to 0.03. This result indicates

that, in mixtures with a low percentage of WTP sludge, the soil characteristics are more dominant.

4. Conclusion

The study seeks to contribute to environmental sustainability concerning the proper disposal of waste generated in WTPs, using it as an addition to soil for waterproofing layers in landfills. The inclusion of WTP sludge in a semi-solid state, in mixtures with soil, demonstrated the technical feasibility of using the waste in civil engineering applications, such as landfill sites. The findings indicated that it is viable from the point of view of hydraulic conductivity, with values suitable for applications in waterproofing layers, and from the point of view of consolidation, indicating variations in void ratio that will further reduce permeability, with a compression coefficient in soil-sludge mixtures much lower than that of pure sludge.

The use of sludge as a substitute for soil (sludge in a semi-solid state) is a beneficial practice as it not only conserves natural resources such as water and soil, but also adds value to the waste, which has the potential to be used as a construction material in waterproofing layers in landfills where it would be disposed of, or even in the manufacture of products with a high production volume, such as road embankments, because it has acceptable hydraulic conductivity for waterproofing layer material.

The maximum dry specific weight values decreased, and the optimum moisture content increased as the percentage of sludge in the mix increased. Additionally, the hydraulic conductivity coefficient values decreased, indicating greater impermeability of the system and the possibility of applying the mixture in landfill layers from a hydraulic point of view. Although the values of the geotechnical parameters of the mixtures with 2.5 and 5 percent sludge incorporated into the soil are acceptable, it is necessary to consider their greater deformability than when compared to the use of soil alone.

The anticipated advantages include the provision of a viable solution for the disposal of WTP waste, a reduction in the consumption of natural raw materials and the sustainable use of waste that commonly occupies valuable space in landfills. This waste can be disposed of in an environmentally appropriate, socially responsible, and economically viable way.

The obtained data also indicate that the variability in the properties of WTP sludge, determined by factors such as the origin and quality of the raw water, requires specific geotechnical characterization for each application to ensure that soil-sludge mixtures meet the desired quality and performance standards.

In general, waste management policies and practices can benefit from the adoption of soil-sludge mixtures reinforced with geosynthetics, providing a practical solution for utilizing WTP sludge, adding value to the waste without the need for prior processing, and promoting environmental sustainability in infrastructure projects.

To further research on using WTP sludge in landfill impermeable layers, it is suggested to study the behavior of soil-sludge mixtures with the addition of geosynthetic barriers, analyze the economic and financial feasibility of this alternative, and monitor landfill cells to understand the long-term behavior of compressibility and settlement.

Acknowledgements

To Sanepar for providing samples of WTP sludge for this study. To the professionals at the Soil and Rock Mechanics Laboratory, C-LABMU, and the Water Resources Laboratory at the State University of Ponta Grossa for their help in obtaining the test results. To CAPES for granting us a one-year scholarship for my Master's studies. To the Academic Writing Center at the State University of Ponta Grossa for assistance with English language translation and developmental editing.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Daiana Tatiele Fiedler: conceptualization, data curation, visualization, investigation, writing – original draft. Maitê Milléo Almeida: conceptualization, data curation, visualization, investigation, writing – original draft. Carlos Emmanuel Ribeiro Lautenschläger: formal analysis, funding acquisition, investigation, methodology, project administration, resources, software, supervision, validation, writing – review & editing.

Data availability

Research data is only available upon request.

Declaration of use of generative artificial intelligence

This work wasn't prepared with the assistance of generative artificial intelligence. The authors assume full responsibility for the content of the publication, which did not have the help of GenAI.

List of symbols and abbreviations

e_i	Void ratio
$w_{ótimo}$	Optimum moisture
C_c	Compaction index
C_r	Recompression index
γ_d	Maximum dry specific weight
NP	Non-plastic soil

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Long-term geomechanical behavior of geogrid-reinforced ballast in dynamic triaxial tests

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Article

Keywords

Cubic particle
Cyclic triaxial tests
Geogrid stiffness
Geosynthetics reinforcement
Plastic deformation

Abstract

During operation, the ballast is subjected to a gradual breakage process that causes clogging of the substructure. Intervention periods on the ballast layer generate high costs that impact the long-term competitiveness of railways. In this context, this work aims to evaluate the geomechanical behavior of reinforced ballast, assessing the effect of reinforcement and stiffness provided by the addition of geogrids. Using 3D printing, the two reinforcement elements were manufactured with PETG and PLA polymers to represent the commercial geogrids Basetrac Grid PET40 and PET65, respectively. Long-term cyclic triaxial tests were used to evaluate the geomechanical behavior of the ballast. The results show that the PLA polymer was more efficient in reducing the deformability of the ballast. Long-term tests also demonstrated that granular materials tend to stabilize plastic deformations, which were not achieved under the stress level and loading frequency adopted in this research, even with the inclusion of geogrids. The low degree of grain breakage shows that the use of cubic particles, together with a well-graded distribution, helps to reduce aggregate degradation in the ballast layer. However, the high confinement levels used in the triaxial tests may have reduced the effect achieved by the reinforcement element. Factors associated with the model geogrids, such as high stiffness, low flexibility, and reduced aperture, also influenced the interaction with the aggregates.

1. Introduction

The railway track is one of the most important transportation systems worldwide, providing a surface that allows trains to transport passengers and loads by a set of two parallel rows of rails, which are located above the substructure, composed of ballast, subballast and subgrade. Ballast particles should resist vertical and longitudinal forces, hold track position, provide water drainage, prevent vegetation growth, provide energy absorption for the track and reduce permanent settlement.

During life span, ballast layer, submitted to cyclic loads, shows an elastoplastic response under traffic loading characterized by a recoverable deformation, due to the compaction process, and a permanent deformation, due to breakage of particles (Lekarp et al., 2000; Anbazhagan et al., 2012). Unlike elastic or residual deformation, the accumulation of plastic deformations is important to predict long-term behavior and understanding the failure mechanisms of granular materials.

Under cyclic loading, ballast particles gradually change in shape, size and compressibility because of attrition and breakage

over time, resulting in the formation of fines and contributing to track settlement (Sun et al., 2016). Werkmeister et al. (2001) improved the concept of shakedown and divided the geomechanical behavior of granular materials into three phases: elastic shakedown, plastic shakedown or creep and failure. On elastic shakedown stage, the permanent strain would eventually stabilize in response to cyclic loading, achieving a state of shakedown after the compaction period, which is acceptable to railroad. On creep stage, the permanent strain doesn't stabilize, remaining constant after the compaction period. On failure stage, poor track performance caused by excessive settlement leads the railroad to collapse.

The degradation of railways requires maintenance operations based on the track geometry to renew structure and improve the bearing capacity, because track performance is predominantly related to the ballast layer. Fine particles derived from degradation and other sources accumulate in the ballast layer in a process known as fouling. Under these conditions, the ballast loses its functional capacity over time, decreasing shear strength and drainage capability, leading to increase maintenance costs (Anderson & Fair, 2008; Huang et al., 2009; Indraratna et al., 2014; Koohmishi &

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Submitted on December 8, 2024; Final Acceptance on April 14, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.008924>



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Palassi, 2018). Over the years, railway projects have focused on the superstructure, which includes rails, fasteners and sleepers, due to the complexity of the parameters that influence the geomechanical behavior of granular materials. However, the costs associated with interventions on the ballast layer represent more than 70% of the total cost of the corrective maintenance process (Sgavioli et al., 2015). In addition, the demand for high-speed transport has grown in the last years, increasing densification on existing tracks.

Substructure reinforcement with geosynthetics presents an alternative method to reduce lateral spread and to enhance track performance, compared to standard corrective maintenance solutions such as tamping, stoneblowing, and ballast cleaning. Although tamping has been widely used for railway maintenance, being the most effective way of correcting geometry faults, this process causes a reversion to its previous track geometry with further repeated traffic loading by a phenomenon known as ballast memory. Eventually, the time between maintenance cycles becomes shorter and the grains must be replaced or cleaned again. Created to prevent ballast memory, stoneblowing is a technique that blows smaller stones below the sleeper without disturbing particles. However, this technique doesn't correct irregularities in the track and is considered a complementary solution to tamping (Guo et al., 2021).

In recent years, several authors have conducted studies about the inclusion of geogrids in ballast to improve its lateral stability and reduced track settlement (Tutumluer et al., 2009; Chen et al., 2013; Indraratna et al., 2013; Liu et al., 2016; Sweta & Hussaini, 2020). Geogrids are flat synthetic structures composed of longitudinal and transverse filaments, characterized by high tensile strength. The reinforcement element increases grain confinement and reduces lateral movement through a mechanism known as interlocking (Fernandes et al., 2008; Shukla et al., 2009; Abu-Farsakh et al., 2012; Palmeira et al., 2021; Desbrousses et al., 2023). However, despite the great improvement from this solution, the interaction between granular materials and geogrid is highly complex and challenging.

In consequence, understanding the reinforcement influence on geomechanical behavior of ballast settlement is vital to optimizing maintenance costs. Researchers around the world have been conducting studies under laboratory conditions to meet future requirements of heavy traffic and optimize the railroad life span (Suiker et al., 2005; Lackenby et al., 2007; Merheb et al., 2014; Delgado et al., 2019). Dynamic triaxial tests or Repeated Load Tests (RLT) may simulate in laboratory a stress state with confinement and deviator stress over granular particles.

On RLT, the axial load is programmed as a sinusoidal pulse, which is composed of a load period followed by a short rest period. The constant confining pressure simulates the mean pressure due to geostatic stress in the field and each vertical load applies a pulse to simulate the effect of the train wheel. This procedure is used to evaluate elastic

behavior by resilient tests and plastic behavior by permanent deformation or long-term tests.

However, despite the previous studies about the geogrid reinforcement of the ballast layer using RLT, the long-term geomechanical behavior of the aggregates is not fully known, because it is influenced by intrinsic and extrinsic factors which commit stress state representation and boundary conditions (Sun et al., 2016; Sweta & Hussaini, 2020; Alabbasi & Hussein, 2021). In this respect, the evaluation of the geomechanical behavior of the ballast involves field or laboratory procedures which are focused on the characteristics of interest, eliminating second-order effects associated with the remaining characteristics. Only in this way it is possible to determine the effect of each characteristic individually. For this purpose, this paper evaluated ballast reinforcement on small-scale samples using cubic cementitious particles to emphasize the effect of geogrid.

2. Materials and methods

2.1 Granular materials

Characterized by enhanced packing density, UHPC (Ultra-high Performance Concrete) presents superior durability compared to conventional concrete. In fact, UHPC is composed of fine materials, with optimized grading curves, microfibers, and a low water-cement ratio. Following these characteristics, all particles were produced using a type V Portland cement and rice husk ash as cementitious materials. Following the mix proportions established by Conceição et al. (2022) (Table 1), ground quartz, two quartz fine sands and wollastonite fiber were used as filler, fine, and micro-reinforcement, respectively.

Gneiss particles (medium-grained rock) were used as the natural railway ballast aggregate for comparison with UHPC. This metamorphic rock has a compressive strength of 124 MPa obtained from 50 mm cubic specimens and it was collected in the city of Itaperuna in Rio de Janeiro state, Brazil. Every particle has a cubic shape previously cut on a table saw.

The gradation No. 24, proposed by American Railway Engineering and Maintenance-of-Way Association (AREMA,

Table 1. Proportions of UHPC for 1 m³.

Material	Mass, <i>m</i> (kg)	Density, <i>ρ</i> (g/cm ³)
Cement	1013.0	3.10
Rice husk ash	53.3	2.09
Ultrafine quartz	81.6	2.65
Fine sand (150 μm)	61.0	2.65
Fine sand (425 μm)	832.0	2.65
Wollastonite	76.7	2.90
Superplasticizer	51.3	1.21
Water	196.1	1.00

2015), which is used on heavy haul tracks around the globe under a long-term loading, with a reduction factor of 3.5, was adopted as a reference for grain size distribution of the ballast (Figure 1) due to widespread use on heavy haul railways. Delgado et al. (2019) also explain that this gradation also provides good interlocking between grains.

All particles (19.0 to 4.76 mm) were produced in a cubic shape with roughly rounded corners (Figure 2). The cubic shape reduces the breakage during loading and centralizes the influence of reinforcement element on ballast, eliminating second-order factors on geomechanical behavior like strength, lithological and morphological characteristics of rock.

Ballast production was divided into three phases: mixture, curing and polishing. During the first phase, a paint mixer attached to a column drill was used to achieve the required workability of the concrete. The aggregates in silicone molds were demolded after 48 hours and remained in curing on water for 28 days. Then, all aggregates stayed at the Micro-Deval for 20 minutes to complete the polishing of edges. Along with the small particles, three 50 mm specimens were also produced following the same process on a concrete vibrating table to evaluate compressive strength.

In accordance with ASTM D6928 (ASTM, 2017) and ASTM C127 (ASTM, 2015), UHPC aggregates were characterized for compressive strength, Micro-Deval

abrasion, relative density and water absorption, respectively. For compressive strength, the three specimens of 50 mm were tested in a servo-hydraulic universal machine with a displacement rate of 0.5 mm/min. Using the caliper, the edges of the three samples were measured to the calculation of the cross-sectional area. For Micro-Deval abrasion resistance, a 1.5 kg sample (from 6.2 to 12.0 mm) was placed in a steel jar mill and rotated for 2 hours with 2 L of water and 5 kg of steel balls with 9.5 mm of diameter. Subsequently, the particles were washed and separated from the abrasive charges. All material retained on sieve #16 (1.18 mm) was dried in an oven at a temperature of $(110 \pm 5)^\circ\text{C}$. As presented in Equation 1, the abrasion loss (M_{de}) was determined from the relationship between the dry mass before (m_a) and after the test (m_b). For relative density and water absorption, six 19.0 mm particles were soaking in water to obtain submerged mass, dried in air to obtain saturated mass and dried on stove for 48 hours to obtain dried mass.

$$M_{de} (\%) = \frac{m_a - m_b (\text{kg})}{m_a (\text{kg})} \times 100 \quad (1)$$

2.2 Geogrids

Manufactured with polymers PET (Polyethylene Terephthalate) or PP (Polypropylene), Basetrac Grid is a group of geogrids of high flexibility and strength utilized for roadways and railway reinforcement. The biaxial geogrids PET40 (40 kN/m tensile strength) and PET65 (65 tensile strength) (Figure 3) were chosen as reference for the confection of geogrids model X (more flexible) and model Y (more rigid) to evaluate the effect of reinforcement on the long-term geomechanical behavior and stiffness on the confinement level, respectively. Both geogrids have a 25 mm mesh opening and 1.0 mm filament thickness.

Tensile strength tests were conducted in longitudinal direction by a servo-hydraulic universal machine with a rate of 20 mm/min to determine the tensile strength (T), obtained from the force as a function of the sample width, and deformation in the break, by the ratio between the displacement and the

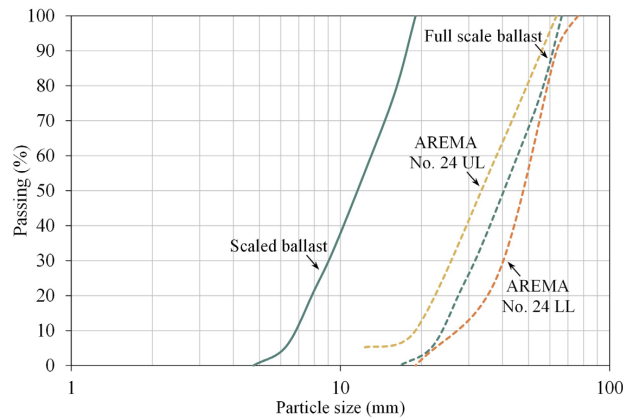


Figure 1. Particle size distribution of scaled ballast (UL: Upper Limit; LL: Lower Limit).



Figure 2. UHPC particles (19.0 to 4.76 mm).

distance between claws. Furthermore, the secant modulus (J) was determined by the ratio between resistance (T) and deformation (ε) (Equation 2).

$$J(\text{kN/m}) = \frac{T(\text{kN/m})}{\varepsilon} \quad (2)$$

According to Brown et al. (2007) observations, grid opening, rib width and secant modulus are the most influential properties of geogrids on ballast geomechanical behavior. These three characteristics were considered as reference to create the model geogrids. Like UHPC aggregates, a reduction factor (λ) of 3.5 was initially applied to model geogrids. The scale factors followed the physical modeling studies established by Viswanadham & König (2004) for 1g models.

Developed on SketchUp (3D design software), the model geogrids have 7.0 mm of aperture, 1.7 mm of filament width and 0.3 mm of thickness. Considering scale effect of 3.5, both geogrids were manufactured by 3D impression with the polymers PETG (Polyethylene Terephthalate Glycol), a more flexible material used to create geogrid model X, and PLA (Polylactic Acid), a more rigid material used to create geogrid model Y. The polymer TPU (Thermoplastic Polyurethane) was initially tested, but it presented lower resistance and higher deformability, characteristics unsuitable for use as a reinforcing element in granular layers.

In addition to tensile tests, static puncture tests were conducted at a deformation rate of 50 mm/min. The correlation between tensile force (F) and piston radius (r_p), proposed by Cazzuffi et al. (1986) (Equation 3), was also used to determine the tensile strength (T).

$$T(\text{kN/m}) = \frac{F(\text{kN})}{2\pi r_p(\text{m})} \quad (3)$$

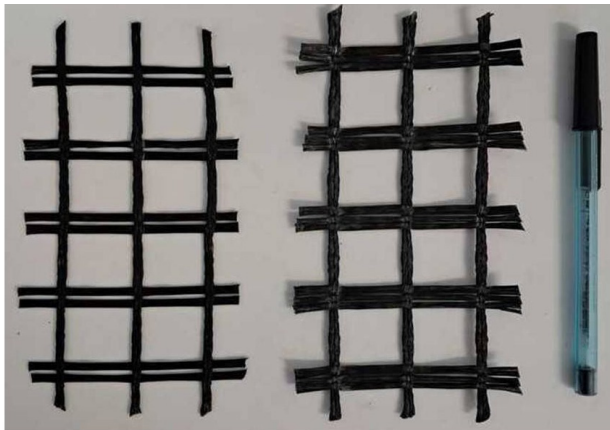


Figure 3. Geogrids Basetrac Grid PET40 and PET65.

2.3 Sample preparation and testing

Three cyclic loading tests were conducted on cylindrical specimens (20 cm in height and 10 cm in diameter), with a density of 1 g/cm³, to evaluate the contribution of geogrid in the permanent deformation. The first test (PD-I) was performed without reinforcement inclusion. The second (PD-II) and third tests (PD-III) were conducted with geogrids PETG and PLA, respectively. Both geogrids were positioned at 1/3 of bottom of the specimen. Several authors recommend a ratio between the maximum particle size and diameter of the specimen for scale effects to become negligible. Skoglund (2002) suggests values between the range 1/5 and 1/7 while the standard ASTM D5311 (ASTM, 2011) specifies a maximum ratio of 1/6. Following the recommendations, the ratio adopted was equal to 1/5.2.

During the preparation of the sample, PVC (Polyvinyl Chloride) flexible strips (220 mm in height, 15 mm in width, and 2 mm in thickness) were fixed at the lower cap. A latex membrane (0.4 mm thickness) was attached to the specimen, filled with cubic aggregates. At bottom of triaxial cell, a drain was connected to a vacuum pump to eventually check the presence of holes in membrane and prevent the occurrence of deformations on the sample before starting the tests. Next, a top cap was placed, and rubber O-rings were used to fix the membrane and seal the sample. Finally, a second membrane was used to cover the specimen (Figure 4). The values of displacement were obtained by two axial LVDT (Linear Variable Differential Transducer) installed around the sample. The use of three transducers to measure axial deformations, as recommended in literature, was not possible to implement considering the limitations of the data acquisition system.

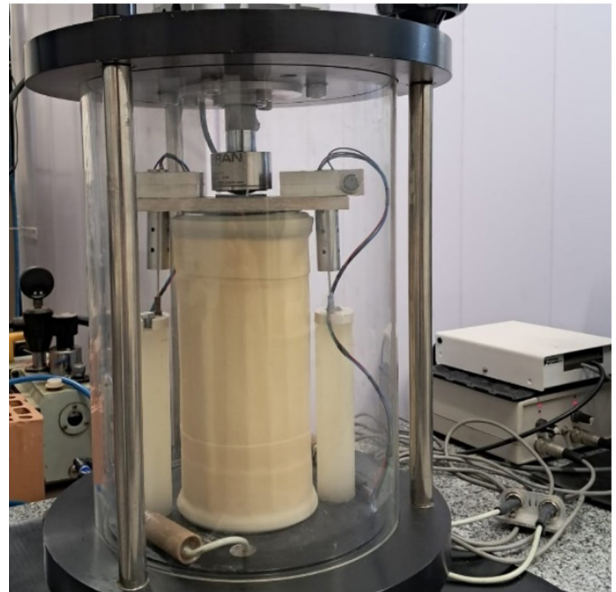


Figure 4. Specimen on triaxial chamber test.

Inside the triaxial cell, constant confinement stress was applied by compressed air. Deviatoric stress was repeated and applied using a vertical load provided by a 5 kN load cell connected hydraulically to an actuated piston and pneumatic system. The use of this load cell was appropriate for reduced-scale testing proposed in this research, considering its stainless-steel body, high resistance to vibrations and compact size.

Isolated tests conducted on the sample to analyze the membrane effect showed a correction of 12% for a confinement stress of 1 kPa. Indraratna et al. (1998) observed a correction of 8% for the same stress state and membrane. Although, Shi (2009) explains that membrane effect correction is negligible on tests conducted in dry conditions.

All single-stage permanent deformation tests were conducted in a single stage, corresponding to a heavy haul railway with 32.5 tons per axle, maximum speed of 80 km/h and 4 m distance between bogies. The deviator stress was estimated with empirical equations used by Delgado et al. (2019). Considering freight trains, the stress state adopted was 70 kPa for confinement stress and 280 kPa for cyclic deviator stress, with a frequency of 2 Hz and one million loading cycles. Lower frequencies, such as 1 to 3 Hz, commonly used on repeated load tests, do not affect significantly the permanent strain (Zhang et al., 2019).

A sinusoidal pulse, composed of a 100 ms (0.1 s) loading period followed by a 400 ms (0.4 s) rest period, was applied to the specimen. Figure 5 shows the load pulse applied during the repeated load test conducted in this current study. Due to data acquisition system limitations, readings were made at each 50,000 cycles for the first 250,000 cycles, 350,000 cycles, 450,000 cycles, 550,000 cycles, 700,000 cycles, and one million cycles. Therefore, the readings didn't follow standard EN 13286-7 (EN, 2004) recommendations.

This loading pattern pulse is a simplification designed for laboratory studies. In fact, three axles, corresponding to the wheelset rail, should be considered to represent correctly the railway cyclic loading, providing a more realistic and accurate prediction of the long-term geomechanical behavior of ballast (Jideani & Gräbe, 2020).

Despite the limitations of the triaxial system software used in this research, it was possible to observe the test configurations, such as the stress state, transducer readings, total number of cycles, real-time cycle and the load pulse.

After each test, the fines were sieved to evaluate the grain breakage, which was analyzed by index B_g (Grain Breakage) (Marsal, 1967). The index B_g (Equation 4) was

calculated as a summation of positive values of the difference in percentage by mass of particles retained on each sieve before (W_{ki}) and after triaxial test (W_{kf}). The index BBI (Ballast Breakage Index) (Indraratna et al., 2005) was calculated with the correlation established by Hussaini (2013) presented in Equation 5.

$$B_g (\%) = W_{ki} - W_{kf} (\%) \quad (4)$$

$$B_g (\%) = 0.25BBI (\%) + 0.32 \quad (5)$$

3. Results and discussions

3.1 Characterization of aggregates

The properties of UHPC and gneiss aggregates are available on Table 2. Compared to the physical and mechanical properties of the UHPC aggregates from Conceição et al. (2022), the particles from this research were significantly more porous (58.7% higher), even though the mix proportions were identical. These differences in physical properties are associated with the adverse reaction caused by the additive used in this study during the curing process. The effect can also be observed in the loss by abrasion in the Micro-Deval test (62.3% higher). This superficial loss was not observed in the particles produced for the characterization tests, although the layer was formed. However, it should be noted that, exceptionally, during the molding of the particles, for the physical and mechanical characterization tests, a concrete vibrating table was used. The other particles were molded only using a spatula over a bench.

Despite these differences, the compressive strength (σ_{cs}) achieved was very close to Gneiss (3.71% lower), attending to the minimum limit of 120 MPa requested by ASTM C1856 (ASTM, 2024) for a UHPC material. In addition, water absorption (α) and aggregate density (ρ) could be easily controlled by proportions and the use of other types of additives, which is a significant advantage of concrete over natural rock.

Table 2. Characteristics of particles.

Properties	Result of this study	UHPC*
Density, ρ (kg/m ³)	2330.03	2673.98
Porosity, η (%)	7.3	4.6
Water absorption, α (%)	3.1	1.9
Micro-Deval abrasion, M_{de} (%)	20.5	12.63
Compressive strength, σ_{cs} (MPa)	119.66	132.3

*According to Conceição et al. (2022).

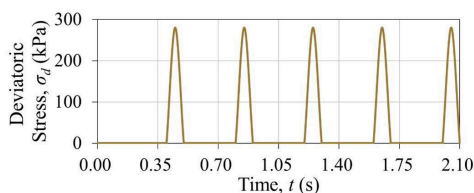


Figure 5. Load pulse in permanent deformation test.

3.2 Characterization of geogrids

During tensile tests on model geogrids, slippage was observed during loading due to the smooth texture of the printed material compared to rough surface of prototype geogrids. This slippage increased the displacements measured by the system, leading to errors in the stress-strain curves. To solve this issue, a low grit sandpaper was added to the front clamping jaws.

In addition, both PETG and PLA polymers showed a high stiffness (J) compared to PET used on Basetrac Grid geogrids. Consequently, ribs of the geogrid were breaking during the fitting into the grips, even without any application of pre-load. Therefore, the thickness (c) of the material was changed from 0.3 mm to 0.6 mm to continue the tests. All properties from model and prototype geogrids are available in Table 3.

Considering a scale factor of 3.5 squared for secant stiffness proposed by Viswanadham & König (2004) for 1g models, it was observed that PETG and PLA model geogrids exhibited higher stiffness compared to the Basetrac Grid PET40 and PET65 geogrids. Applying the scale factor on the prototype geogrids, it should be expected a secant stiffness of 33.5 kN/m for PETG and 50.5 kN/m for PLA. Thus, the results of 120.69 and 203.66 kN/m did not meet the scale factor. However, it is worth noting that the strength values of PET40 and PET65 geogrids obtained on tensile tests (29.23 and 40.10 kN/m respectively) were lower compared to the commercial specifications provided by the manufacturer (40.0 and 65.0 kN/m respectively). Figure 6 presents the final schematic representation of the model geogrids developed on SketchUp.

The static puncture tests didn't show good correlations on prototype geogrids showing 50% less strength compared to tensile tests, resulting in 14.13 kN/m for PET40 and 19.28 kN/m for PET65. On model geogrids, the correlations resulted in 1.52 kN/m for PETG, following the tendency on prototype geogrids, and 0.97 kN/m for PLA, being the worst correlation with a strength 76.4% lower.

3.3 Permanent deformation tests

During the first 350,000 cycles, increments of plastic deformation were higher due to the stabilization process

of axial deformations under the applied stress level. The ballast undergoes significant initial deformations due to grain movement, which promotes wear between the particles. In all tests, the material exhibited a tendency to stabilize, even without the inclusion of geogrid (Figure 7). This behavior was also observed by Indraratna & Salim (2003), Abu-Farsakh et al. (2007), Indraratna et al. (2010), and Qian et al. (2018). However, unlike the cited studies, plastic deformations did not reach the shakedown point. Even after reaching one million cycles, the deformations kept a tendency of growth.

All tests were executed with pauses every 50,000 cycles during the first 350,000 cycles. It should be noted that conditioning of the sample between readings may have influenced these results, considering that particles have shown a tendency of stabilization at the first 100,000 cycles based on other studies. This procedure was limited by the data acquisition system and software employed in this research, allowing the measurement of deformations only at the end

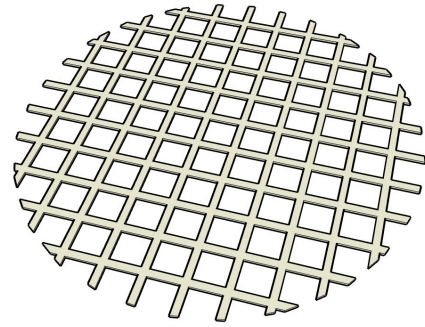


Figure 6. Perspective of model geogrid

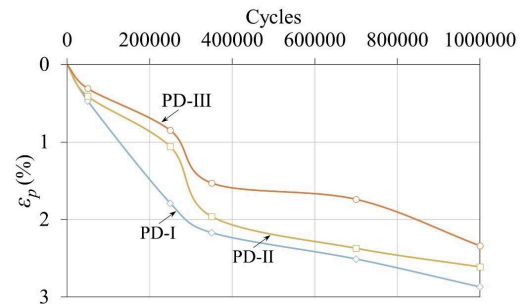


Figure 7. Permanent deformation vs. cycles.

Table 3. Characteristics of prototype and model geogrids from tensile tests.

Properties	PET40	PET65	PETG	PLA
Geogrid aperture, a (mm)	25.0	25.0	7.0	7.0
Rib width, b (mm)	6.0	6.0	1.7	1.7
Rib thickness, c (mm)	1.0	1.0	0.6	0.6
Tensile strength, T (kN/m)*	29.23	40.10	2.86	4.11
Deformation, ε (%)	7.12	6.47	2.37	2.02
Secant stiffness, J (kN/m)*	410.35	619.80	120.69	203.66

*On machine direction (MD).

cycle count, despite the procedure of 20 continuous readings for the first 20 cycles from standard EN 13286-7 (EN, 2004). Then, it was not possible to estimate the effect of deviatoric stress on plastic behavior because the long-term tests were limited by one stress state. Furthermore, the RLT doesn't allow to simulate the principal stress rotation, therefore, it was not viable to simulate the reversal of shear stress.

Geogrid PETG reduced a maximum deformation of 40.8% while geogrid PLA reduced to 52.5% (Figure 8). These results showed an influence of reinforcement effect and stiffness on the deformation of granular materials. Nevertheless, the ratio between the geogrid aperture size and the average particle diameter (a/d_{50}) used in this study was 0.6, which is considered a feeble interlocking zone (McDowell et al., 2006; Indraratna et al., 2013; Han et al., 2018). This observation corroborates the studies of Brown et al. (2007), proving that geometric parameters of the geogrid, such as aperture size and secant stiffness, may influence the deformability of the ballast layer. Abu-Farsakh et al. (2007) also recognized the influence of geogrid arrangement. However, this effect cannot be analyzed considering that the tests only used one geogrid for each specimen.

Commercial geogrids such as Basetrac Grid are rigid, but they exhibit a high level of flexibility in interaction with the ballast particles. This behavior was not observed in the

model geogrids PETG and PLA during characterization and triaxial tests. The alignment of the fibers in these polymers combined with the smooth texture contrasts with the rough surface of the prototype geogrids PET40 and PET65. Nonetheless, it should be noted that the high level of confinement (70 kPa) has influenced the interlocking mobilized by the reinforcement element.

As observed in Figure 9, the polymers exhibit high stiffness and low flexibility between filaments and junctions. The deformability of the components is more noticeable in PETG, due to its lower rigidity. It should be noted that the deformation presented in one of the PLA filaments may be associated with the ballast compaction process on PD-III test.

Compared to the granite aggregates submitted to one million cycles evaluated by Delgado et al. (2019), the degradation of UHPC particles without geogrid inclusion was 36.46% lower considering B_g index (2.30% against 3.62%). However, the triaxial tests conducted in this research were performed using a controlled form of ballast. Therefore, the shape control of the aggregate, together with the use of a uniformity coefficient (C_u) close to 2.0 (1.91), as Indraratna et al. (2006) and Shi et al. (2023) recommend, contributed to reducing material breakage, even without addition of the reinforcement element (Table 4).

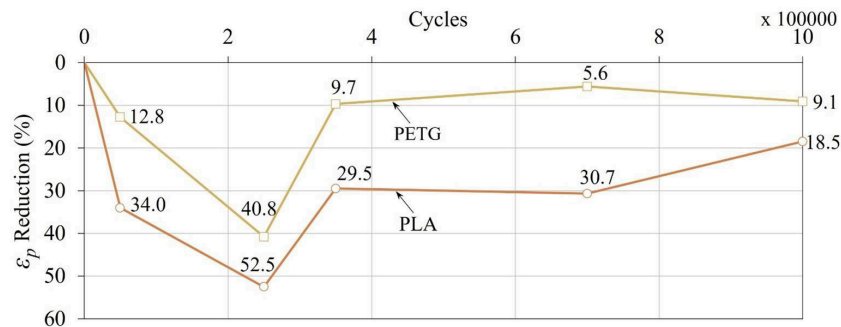


Figure 8. Geogrid action on long term behavior.

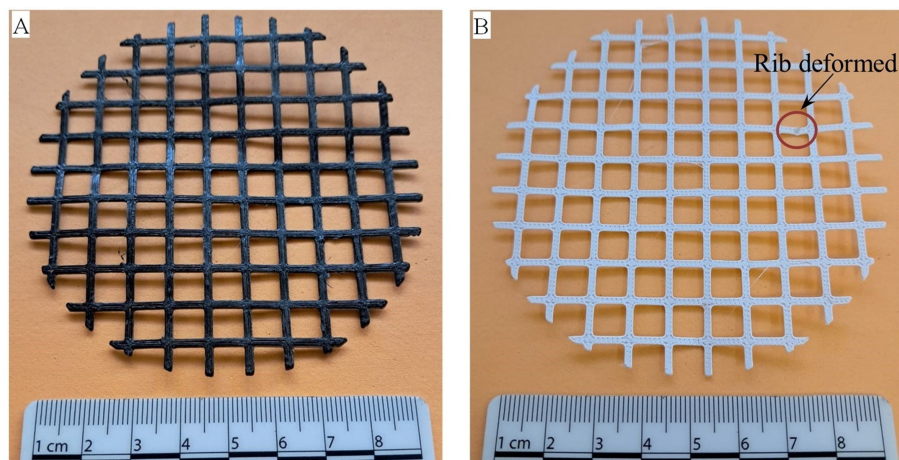


Figure 9. Model geogrids condition after long-term tests: (A) Model X (PETG) and (B) Model Y (PLA).

Table 4. Particle breakage.

Index	PD-I	PD-II	PD-III
B_g (%)	2.30	2.25	2.18
BBI (%)	7.92	7.72	7.44

Based on the previous results, it is highlighted that the long-term effects of the reinforcement elements were influenced by scale factor, stiffness of the PETG and PLA polymers, and stress state. Therefore, further studies with a lower confinement level should be conducted on cubic ballast particles to observe the recurrence of this behavior. Furthermore, geogrids with larger apertures should be used to meet the spacing requirements of the interface factor recommended in the literature, with a ratio of a/d_{50} situated between 1.2 and 1.4, providing an optimum interlocking zone instead of a feeble interlocking zone. Moreover, it should be tested other model geogrids manufactured with more flexible materials to achieve better interaction with the ballast aggregates, simulating an interlocking effect closest to commercial geogrids. In addition, future studies to evaluate the effect of interlocking on radial deformations and the geomechanical behavior on multiple-stage tests may be also executed to extend the conclusions observed in this research.

4. Conclusion

Dynamic triaxial tests were conducted to evaluate the long-term geomechanical behavior of ballast. This paper investigated and evaluated the effect of reinforcement on settlement and the influence of stiffness at grain confinement provided by the addition of geogrids. For this purpose, UHPC cubic particles with slightly rounded corners, produced with a scale factor of 3.5 applied on graduation No. 24 from AREMA, were used in repeated load tests, with and without the addition of reinforcement. The methodology used on RLT aimed to simulate dynamic vertical loading from a simplified pulse. Based on experimental program developed in this research, the following conclusions can be drawn:

1. High plastic deformations were observed on aggregates during initial cycles. This behavior shows that granular materials undergo a stabilization process of permanent deformations when they are close to reaching shakedown condition;
2. The interlocking effect, promoted by the geogrid, is more noticeable during initial cycles when grains rearrange due to reinforcement elements until reaching an equilibrium state, where deformations stabilize. Besides, higher confinement levels used on repeated load tests have reduced the effect promoted by geogrids;
3. Despite the tendency to reduce plastic deformations, the shakedown condition was not reached by the ballast, even when subject to a single stress state;
4. High stiffness and smooth texture present in PETG and PLA polymers inhibited the degree of adaptability and contact with aggregates, while mesh aperture

influenced the mobilization of interlocking. Therefore, geogrids exhibited partial interaction flexibility with granular material composing the ballast.

Acknowledgements

The authors would like to express their gratitude to Fundação Carlos Chagas Filho de Amparo à Pesquisa do Estado do Rio de Janeiro (FAPERJ) for financial support for this research. They would also like to thank HUESKER for providing geosynthetic samples, BLITZAR for providing 3D impression geogrids, SOLOTEST for providing triaxial software system, and Universidade Estadual do Norte Fluminense (UENF) for laboratory resources.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Lucas Machado de Souza: conceptualization, data curation, methodology, visualization, validation, writing – original draft & editing. Paulo César de Almeida Maia: funding acquisition, project administration, methodology, supervision, validation, resources, writing – review & editing. Maria Cecília Martins Gomes Rangel: data curation, investigation, writing – editing.

Data availability

Research data is only available upon request.

Declaration of use of Generative Artificial Intelligence

This work was not prepared with the assistance of Generative Artificial Intelligence (GenAI).

List of symbols and abbreviations

a	Geogrid aperture
b	Geogrid rib width
c	Geogrid rib thickness
d_{50}	Average particle diameter
m	Particle mass
m_a	Particle dry mass before Micro-Deval abrasion test
m_b	Particle dry mass after Micro-Deval abrasion test
r_p	Piston radius
t	Time
vs.	Versus

AREMA	American Railway Engineering and Maintenance-of-Way Association
ASTM	American Society for Testing and Materials
<i>BBI</i>	Ballast Breakage Index
B_g	Grain Breakage
C_u	Uniformity coefficient
EN	European Standard
F	Tensile force
FAPERJ	Fundação Carlos Chagas Filho de Amparo à Pesquisa do Estado do Rio de Janeiro
J	Secant stiffness
LL	Lower Limit
LVDT	Linear Variable Differential Transducer
MD	Machine Direction
M_{de}	Micro-Deval abrasion resistance
PD	Permanent Deformation
PET	Polyethylene Terephthalate
PETG	Polyethylene Terephthalate Glycol
PLA	Polylactic Acid
PP	Polypropylene
PVC	Polyvinyl Chloride
RLT	Repeated Load Test
T	Tensile strength
TPU	Thermoplastic Polyurethane
UENF	Universidade Estadual do Norte Fluminense
UHPC	Ultra-high Performance Concrete
UL	Upper Limit
W_{kf}	Summation of positive values of the difference in percentage by mass retained on each sieve after triaxial test
W_{ki}	Summation of positive values of the difference in percentage by mass retained on each sieve before triaxial test
α	Water absorption
ε	Deformation
ε_p	Permanent deformation
η	Porosity
λ	Scale factor
π	Pi
ρ	Density
σ_{cs}	Compressive strength
σ_d	Deviatoric stress

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Influence of loading mode on the response of a helical pile subjected to cyclic tensile loads in sand

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Article

Keywords

Helical pile
Tensile load test
Cyclic loading
Aeolian sand

Abstract

Helical piles are steel elements consisting of one or more helices placed along a central shaft and installed in the ground through rotation. This study describes a field investigation of the behavior of a helical pile prototype subjected to cyclic tensile loads in a site composed of pure sand layers. During the investigation, the helical pile prototype featured three helices and was equipped with electrical resistance strain gauges at three different sections along the shaft. Two cyclic tensile load tests were conducted during the investigation using quasi-static (low-frequency) cyclic loadings. In each test, cyclic loads were applied in four distinct stages. The maximum load reached in each stage was the same in both load tests. However, the cyclic amplitude between stages increased in one load test and kept constant in the other. A static load test was performed immediately after unloading one of the cyclic tests to evaluate the influence of the cycles on the static load capacity. The results revealed that the helix closest to the pile tip contributed the most to the total axial load capacity. Proportionally, the tip helix had the highest contribution among all the helices from the beginning of the test. The mode of load increment application influenced the pile response. Increasing cyclic loads between stages led to an increase in the proportion of the load mobilized at the tip helix and a decrease in the proportion mobilized at the upper helices.

1. Introduction

Helical piles are metallic elements of helices arranged along a central shaft, installed into the ground by torque to resist compressive, tensile, and lateral loads. The number and diameter of the helices, defined according to soil parameters, directly influence the load-bearing capacity.

Helical piles are a versatile solution, offering benefits for both new constructions and the rehabilitation of existing structures. When correctly installed, they not only meet but often surpass design capacity (Stephenson, 2003). They can be installed at any angle, generating minimal noise and vibration, and can be loaded immediately after installation. Furthermore, they are relatively lightweight and easy to transport, providing a cost-effective solution and positioning them competitively in the market (Perko, 2000; Stephenson, 2003).

The first records of helical pile usage date back more than 150 years. At that time, they were used to anchor vessels and improve construction in soils with low bearing capacity.

The piles had cylindrical or, in some cases, octagonal sections and were typically made of wood or cast iron. In Brazil, the application of helical piles commenced in 1998 with the purpose of resisting tensile forces in various structures. These structures include transmission line towers for electricity and telecommunications, as documented by Tsuha (2007). Helical piles are particularly advantageous in these applications due to their capacity to anchor in diverse soil types.

Helical piles, a common feature in foundation engineering, play a crucial role in resisting tensile forces in various structures such as communication towers, wind turbines, buried pipelines, and offshore structures. However, the loads these piles bear are often cyclic, which can lead to system failure due to the degradation of load capacity, making the investigation of their tensile behavior under cyclic loading a matter of utmost importance.

Cyclic loading can be applied unidirectionally, either in compression or tension, or bi-directionally, with alternating tension and compression loads. Cyclic loads can be considered static, quasi-static, or dynamic. Quasi-static load cycles are

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Submitted on August 12, 2024; Final Acceptance on May 6, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.006224>



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applied at low frequency, making inertial forces negligible, whereas dynamic load cycles have significant inertial forces. A frequency of 5 Hz is considered the threshold between quasi-static and dynamic behavior (Wichtmann, 2005).

Techniques to explore the effects of cyclic loading on load capacity include imposing uniform loading and unloading cycles, using fixed frequencies and amplitudes in each stage. The response of a pile to cyclic loading will depend on the combination of average load (Q_{avg}) and cyclic amplitude (Q_{cyc}) (Andersen et al., 2013), which can cause the pile to operate in a stable, metastable, or unstable zone. If maintained in a stable zone, low-frequency cycling can benefit load capacity, allowing piles to self-recover over time after modest losses in load capacity. Jardine & Standing (2012) reported gains of up to 20% in load capacity after applying stable cyclic loads to piles.

Resistance forces concentrate at the interface between intact and disturbed soil when a helical pile is subjected to cyclic loads. Under unidirectional tensile loading, after a certain number of load cycles (depending on cyclic amplitude), the disturbed soil within the cylinder formed by the helices and the ground surface is densified. Additionally, the repetitive uplift movement of a pile after applying cyclic tensile loads causes the formation of voids, which are more significant at higher cyclic amplitudes and low-density zones below the helix (Schiavon, 2016). Costa (2017) found that increasing load amplitude and mean loads reduce pile stability and system stiffness.

Cerato & Victor (2009) concluded that cyclic loads can increase tensile load capacity and minimize long-term creep, even with a reduction in shaft contribution, due to increased mobilization of resistance from the helices over cycles. They suggest that the increase in load capacity results from soil compaction above the helix during repetitive loading.

Changes in static tensile load capacity can occur after applying cyclic loads. Schiavon (2016) conducted four cyclic load tests interspersed with static tensile load tests (before and after each cyclic test). The results showed that the cyclic performance of the helical pile was affected by the static load tests. Applying static load improved the soil above the plate, eliminating accumulated cyclic displacements. However, cyclic loading did not improve static response; it only modified load distribution between the helix and the shaft. On the other hand, Costa (2017) observed a 6% increase in static load capacity after applying quasi-static cyclic loads.

Despite their increasing use in foundation engineering, the behavior of helical piles, particularly their performance under cyclic loading, remains under-researched and poorly detailed in the technical literature, especially in Brazil. This study, therefore, makes a significant contribution to the field by providing new insights into the tensile behavior of helical piles under cyclic loading in sandy soil, thereby filling a crucial gap in the existing knowledge.

Given this, the present study aims to investigate the tensile behavior of helical piles subjected to cyclic loading in sandy

soil, based on field test data and instrumentation. Specifically, the study focuses on analyzing the force distribution along the helical pile using instrumentation during tensile load tests and evaluating changes in static load capacity resulting from the application of quasi-static load cycles. Researchers conducted tests using a prototype helical pile equipped with three helices to measure axial loads at three different sections along the shaft. They performed two quasi-static cyclic tensile load tests at a site composed of pure sand layers. In one test, they applied increasing cyclic load throughout the loading stages, while in the other, they maintained a constant cyclic load. Both tests had equal maximum loads in the stages. Additionally, a static tensile load test was conducted in the same installation as one of the cyclic load tests (retest) to assess the influence of cyclic loading on static load capacity.

2. Study area

2.1 Location of load tests

The researchers conducted tests in an experimental field at the central campus of the Federal University of Rio Grande do Norte in Natal-RN (UTM coordinates: 255829.00 m E; 93544649.48 m S). They made two installations, one at position A and another at position B (Figure 1), ensuring a distance between them more significant than five times the diameter of the most giant helix to prevent interference between the installations (Livneh & El Naggar, 2008).

2.2 Subsoil characteristics

The subsoil investigation program included thirteen standard penetration tests (SPT) according to ASTM D1586 (ASTM, 2022). Figure 2 presents the results obtained from borehole SPT, which is representative of the soil at the experimental field. SPT-N values are for an assumed efficiency of 72%. The ground is composed of pure aeolian sand layers. The boreholes did not detect the water table.

Significantly, the soil, classified as poorly graded sand (SP) according to the Unified Soil Classification System (USCS), is a crucial factor in understanding the soil's behavior. It contains 27% fine sand, 72% medium sand, and 1% coarse sand. The sand's uniformity coefficient of 1.96, curvature coefficient of 0.99, and specific gravity of solids of 26.3 kN/m³ further contribute to our understanding. Direct shear tests on two samples collected from the ground at a depth of 1 m provided effective internal friction angles of 32.3° and 35.9° for relative densities of 34% and 71%, respectively, which are key geotechnical parameters for construction considerations.

3. Prototype description

A prototype, being a full-scale structure, plays a crucial role in research testing. Unlike scaled-down models, which

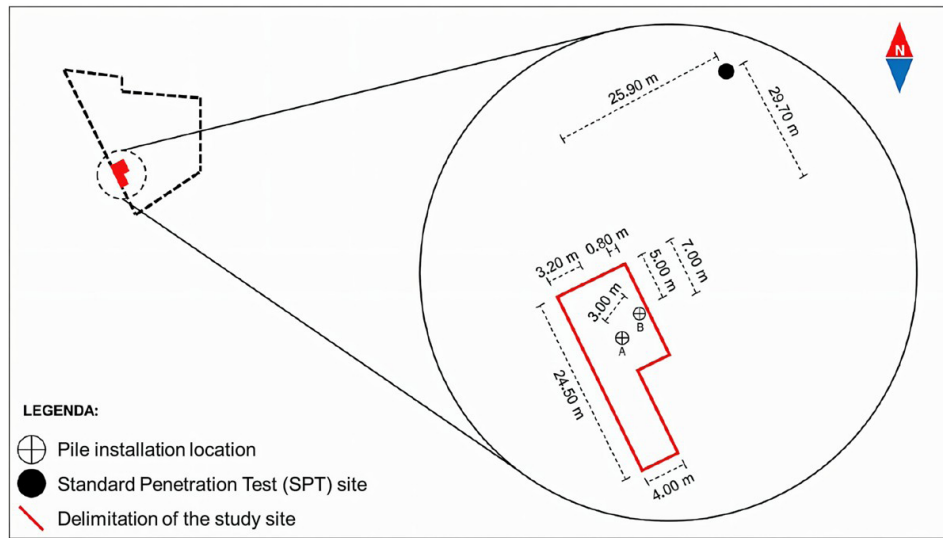


Figure 1. Study area.

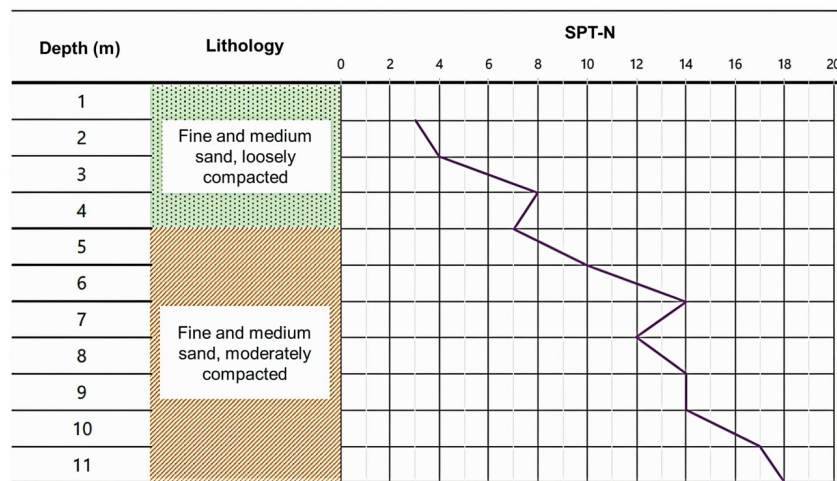


Figure 2. Typical soil profile.

frequently face challenges of similitude such as inadequate stress distribution and altered soil behavior, the full-scale prototype allows for the acquisition of more accurate and relevant data. The helical pile prototype used in this research was fabricated from steel and consisted of three distinct segments. The prototype features three helices with diameters of 350 mm, 300 mm, and 250 mm. The structural design was developed in compliance with the provisions of NBR 8800 (ABNT, 2008). All helices were manufactured with 12.7 mm thick plates and had a pitch of 75 mm. Figure 3a schematically illustrates the dimensions of the helical pile.

For precise data collection, three sections along the pile were meticulously instrumented with electrical resistance strain gauges, designated as S1, S2, and S3. Figure 3b showcases the fabricated prototype, clearly indicating the location of

these instrumented sections. Dual-rossette electrical strain gauges of 120 Ω were used to measure the normal force in the sections. The strain gauges were connected in a complete Wheatstone bridge circuit, a setup that effectively eliminates bending effects and thermal variations. Unfortunately, issues with the sensor during the installation of the prototype for Test B hindered data collection from section S2.

Costa (2017) and Queiroz (2018) provide all details regarding the design, fabrication, and instrumentation of the prototype used in this study.

3.1 Prototype installation in the ground

The prototype was inserted into the soil by applying torque at the top of the pile with the assistance of a hydraulic

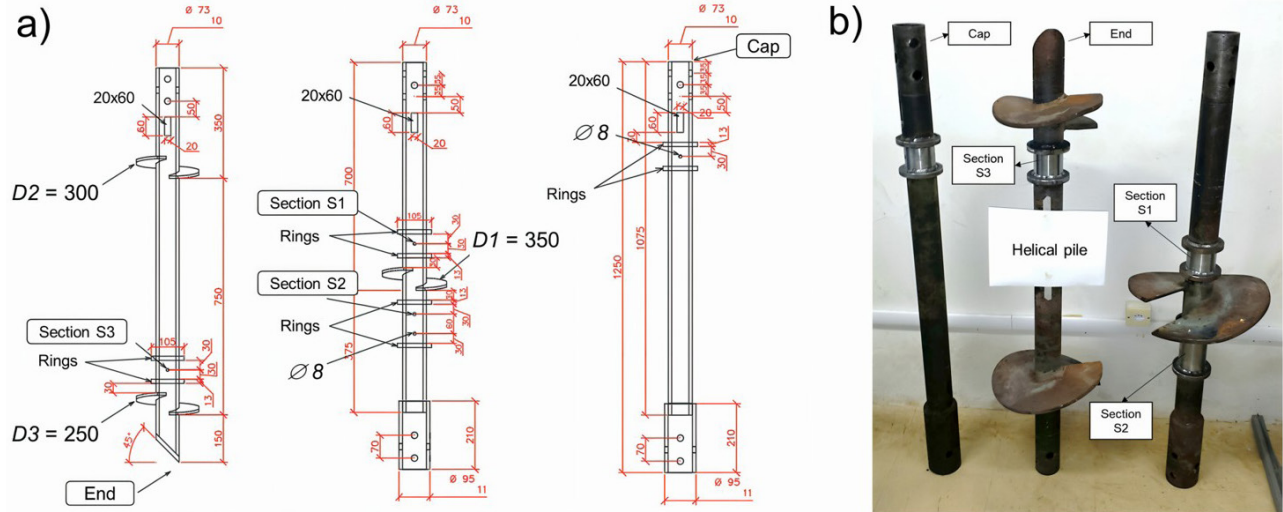


Figure 3. a) Schematic drawing of the prototype, indicating the dimensions of the constituent elements. Dimensions in mm, not to scale. b) Constructed prototype, indicating the instrumented sections.

drill. After each test, it was removed by performing reverse rotation with the drill. The prototype reached maximum depths of 2.57 m and 2.77 m at locations A and B, respectively.

The depths reached by the helices H1, H2, and H3, which are respectively the top, intermediate, and tip helices, are presented in Table 1. Table 2 shows the depth of each instrumented section after installation.

During the installation of each prototype segment, the instrumentation cables were meticulously routed through the pile's interior and emerged through an opening made at the top of the segment, always taking utmost care to avoid entanglement. Following the installation of the prototype in both tests, soil displacement around the shaft was noted to a minimum depth of 0.5 m, ensuring the safety of the process.

3.2 Load testing execution

Two single-way quasi-static cyclic tensile load tests, Tests A and B, were conducted. Each test was divided into four stages, each receiving 60 loading and unloading cycles. The duration of each cycle was 60 seconds, corresponding to a frequency of 1 Hz. The applied load oscillated between fixed minimum (Q_{min}) and maximum (Q_{max}) values within each stage. The loading plan for the tests is presented in Table 3. In Test A, the cyclic loads (Q_{cyc}) were programmed to increase from 5 kN to 20 kN, while in Test B, they were kept constant at 10 kN. The maximum load was uniformly applied across all stages in both tests.

Immediately after the meticulously executed quasi-static cyclic load tests, a post-cyclic static load test, named Test C, was conducted at the same location as Test B. The test was initiated immediately after unloading Test B. The procedures followed for this test were meticulously based on the universally recognized standard D3689 (ASTM, 2013).

Table 1. Depth of helices after installation.

Test	H1 (m)	H2 (m)	H3 (m)
A	0.84	1.60	2.35
B	1.04	1.80	2.55

Table 2. Depth of instrumented sections after installation.

Test	S1 (m)	S2 (m)	S3 (m)
A	0.77	0.98	2.28
B	0.97	1.18	2.48

In this test, the load was meticulously kept constant for 10 minutes during each loading and unloading interval. The test was concluded after five meticulously executed cycles of loading and unloading, with increments of 5 kN until failure.

4. Results and discussions

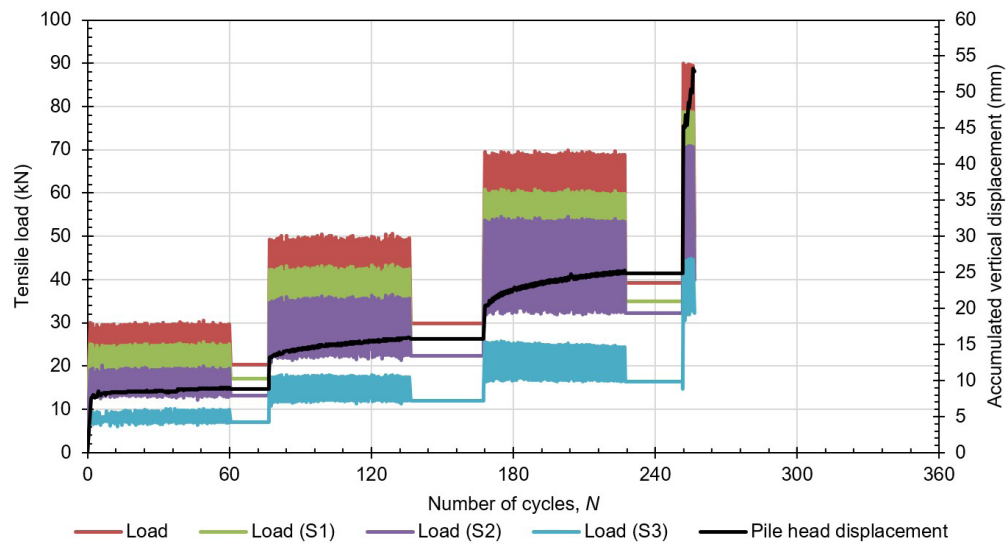
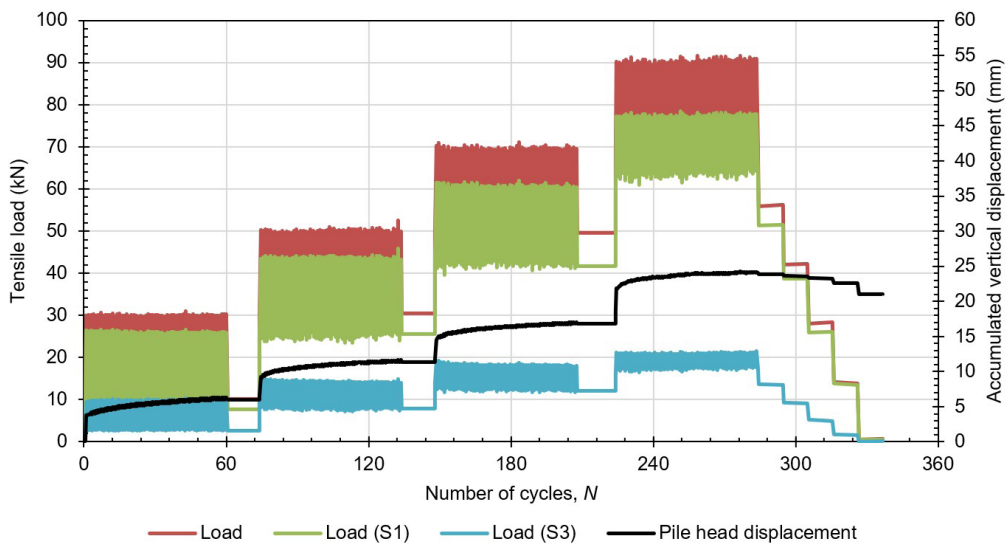
4.1 Cyclic load tests

Figures 4 and 5 depict the axial tensile load behavior over time measured at the top and instrumented sections of the pile in Tests A and B, respectively. The exact figures also show the vertical displacement variation over time measured at the top of the pile.

The significance of the observed pile behavior in Test A and Test B cannot be overstated. In stage A-04 of Test A (Figure 4), notable instability was witnessed, marked by a pronounced increase in vertical displacement after five cycles of loading-unloading. However, Test B (Figure 5) remained stable throughout. In this case, the permanent

Table 3. Planning of loadings for Tests A and B.

Test	Stage	Minimum load, Q_{min} (kN)	Maximum load, Q_{max} (kN)	Average load, Q_{avg} (kN)	Cyclic load, Q_{cyc} (kN)	Number of cycles, N	Period, T (s)
A	A-01	20	30	25	5	60	60
	A-02	30	50	40	10	60	60
	A-03	40	70	55	15	60	60
	A-04	50	90	70	20	5	60
B	B-01	10	30	20	10	60	60
	B-02	30	50	40	10	60	60
	B-03	50	70	60	10	60	60
	B-04	70	90	80	10	60	60

**Figure 4.** Tensile load and vertical displacement of the pile head over time in Test A.**Figure 5.** Tensile load and vertical displacement of the pile head over time in Test B.

vertical displacement reached after unloading was a mere 21 mm, a testament to its stability.

Table 4 presents the axial loads measured at the instrumented sections. It also shows the interaction load for each instrumented section, which occurs when the minimum load of a stage is lower than the maximum load of the previous stage (i.e. when there is load overlap). In this situation, part of the load applied in a new stage consists of a reload, which can increase the soil stiffness and reduce the corresponding displacements. It was noted that the interaction load increased with the stages in Test A, while in Test B, it remained at a low and approximately constant level.

The diagrams of axial load transfer, constructed from the average load of each stage (Q_{avg}), measured at the top and

instrumented sections, are presented in Figures 6a and 6b. In both tests, the variation of axial load with depth was greater between sections S1 and S3 than between the top and section S1. However, the discrepancies in load transfer mechanisms between the geotechnical tests were pronounced. Test B showed a greater reduction in axial load with depth, and the loading stages had a more pronounced effect on the load distribution with depth in Test B, highlighting the contrast between the two tests.

Table 5, a key component of our findings, illustrates the ratio between the load reaching the instrumented section and the load applied at the top of the pile (Q_{SI}/Q_{Cap}). This data is crucial as it reveals how the load is distributed along the pile, a fundamental aspect in the design and analysis of pile

Table 4. Axial loads measured in the instrumented sections of the pile.

Test	Section	Stage	Minimum load, Q_{min} (kN)	Maximum load, Q_{max} (kN)	Average load, Q_{avg} (kN)	Cyclic load, Q_{cyc} (kN)	Interaction load, Q_{int} (kN)
A	S1	A-01	15.76	25.63	20.69	4.93	-
		A-02	24.73	43.49	34.11	9.38	0.90
		A-03	33.74	60.93	47.33	13.59	9.75
		A-04	41.46	78.87	60.17	18.70	19.46
	S2	A-01	12.13	20.13	16.13	4.00	-
		A-02	19.18	36.50	27.84	8.66	0.95
		A-03	28.61	54.48	41.54	12.93	7.89
		A-04	34.16	70.85	52.51	18.34	20.32
	S3	A-01	5.69	10.17	7.93	2.24	-
		A-02	9.80	18.01	13.90	4.11	0.38
		A-03	14.24	25.73	19.99	5.75	3.77
		A-04	14.66	44.76	29.71	15.05	11.08
B	S1	B-01	7.26	26.72	16.99	9.73	-
		B-02	23.56	45.83	34.70	11.14	3.16
		B-03	39.67	62.10	50.88	11.21	6.16
		B-04	57.85	78.44	68.14	10.30	4.25
	S3	B-01	2.59	10.28	6.43	3.84	-
		B-02	7.15	14.98	11.07	3.92	3.13
		B-03	11.71	19.36	15.53	3.82	3.28
		B-04	15.63	21.46	18.55	2.92	3.73

Table 5. Ratio between the load measured in the instrumented sections and the load at the pile head (Q_{Cap}).

Stage	Test A			Test B	
	Q_{SI}/Q_{Cap}	Q_{SZ}/Q_{Cap}	Q_{S3}/Q_{Cap}	Q_{SI}/Q_{Cap}	Q_{S3}/Q_{Cap}
1	0.84	0.65	0.33	0.84	0.31
2	0.86	0.73	0.37	0.86	0.28
3	0.88	0.79	0.38	0.86	0.26
4	0.88	0.79	0.53	0.88	0.24

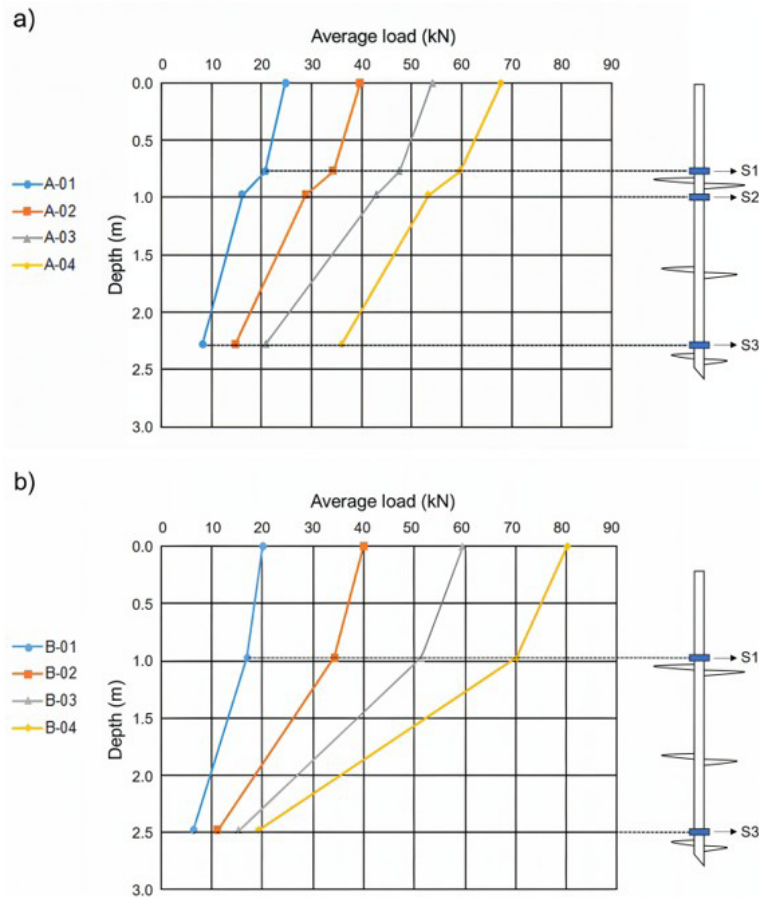


Figure 6. Load transfer diagram: a) Test A; b) Test B.

foundations. Notably, the load at the shallowest section (S1) showed a slight increase with the stages in both Test A and Test B. However, a contrasting trend was observed in the deepest section of the pile (S3): while in Test A there was a significant increase in load with the stages, in Test B it decreased slightly.

In addition to measuring the distribution of loads in the sections, it was also possible, through instrumentation, to analyze the contributions of resistances from the shaft and helices. Initially, the evolution of the traction loads acting on the helices with the cycles was analyzed. The mobilized loads on helices H1 and H3 in Test A and H3 in Test B were determined. The analysis excluded the effects of helices H2 in Test A and H1 and H2 in Test B because they could not be isolated from the shafts. Figures 7a and 7b present the results for Tests A and B, respectively. For each cycle, the average load is presented (indicated by the darker line) along with the maximum and minimum loads (represented by the lighter lines). The results of Test A show that the load mobilized on the top helix (H1) remained at an approximately constant level with the change of stages, below 10 kN. On the other hand, the load on helix H3 substantially increased with the change of stages. In Test B, there was also a significant increase in the load on H3 with the stages.

The occurrence of interaction loads between stages is a significant finding. In Test A, the interaction loads were significant on helix H1, while they were almost nonexistent on helix H3. In Test B, helix H3 experienced interaction loads of relevant magnitudes between all stages. This observation is crucial as it indicates the behavior of the soil in contact with the helix, contributing to the increase in its stiffness for the subsequent stage. Understanding these implications is key to optimizing the design and analysis of pile foundations.

Tables 6 and 7 present the resistance contributions from the shaft and helices of the pile in Tests A and B, respectively. Due to the instrumentation configuration, only the shaft segment F1, whose length extends from the ground surface to the upper helix, could have its contribution isolated. In Test A, the instrumentation configuration did not allow for separating the contribution of helix H2 from shaft segment F2. In Test B, a problem occurred with the sensors of section S2 during the pile installation, which made it impossible to separate the contributions between helices H1, H2, and shaft segment F2. The values in the tables represent the average readings of the cycles of each stage. The corresponding ratio between the average measured load and the load applied at the top (Q_{avg}/Q_{Cap}) is indicated in parentheses.

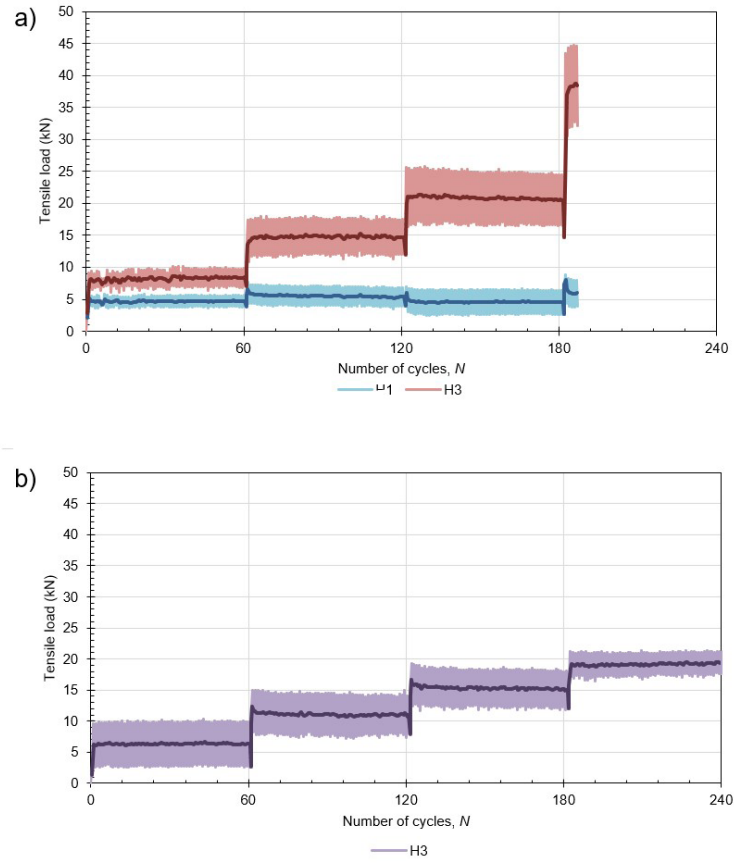


Figure 7. Loads mobilized in the helices: a) Test A; b) Test B.

Table 6. Loads mobilized in the pile in Test A.

Stage	Q_{cap} (kN) ⁽¹⁾	F1 (kN) ⁽²⁾	H1 (kN) ⁽³⁾	H2+F2 (kN) ⁽⁴⁾	H3 (kN) ⁽⁵⁾
A-01	24.8	4.0 (0.16)	4.7 (0.19)	7.9 (0.32)	8.2 (0.33)
A-02	39.6	5.3 (0.13)	5.5 (0.14)	14.1 (0.36)	14.7 (0.37)
A-03	54.1	6.5 (0.12)	4.6 (0.09)	22.2 (0.41)	20.8 (0.38)
A-04	67.9	8.4 (0.12)	6.2 (0.09)	17.4 (0.26)	36.0 (0.53)

⁽¹⁾Total load applied at the pile head, measured by the load cell. ⁽²⁾Lateral friction resistance in the shaft above the top helix, obtained by subtracting the total load from the load at section S1. ⁽³⁾Load carried by helix H1, determined by subtracting the load at section S1 from the load at section S2 (Only for Test A). ⁽⁴⁾Load resisted by helix H2 and the shaft between sections S2 and S3, determined by subtracting the load at section S2 from the load at section S3 (Only for Test A). ⁽⁵⁾Load on the tip helix, H3, obtained at section S3.

Table 7. Loads mobilized in the pile in Test B.

Stage	Q_{cap} (kN)	F1 (kN)	H1+H2+F3 (kN)	H3 (kN) ⁽¹⁾
B-01	20.0	3.1 (0.15)	10.6 (0.53)	6.3 (0.32)
B-02	39.9	5.7 (0.14)	23.1 (0.58)	11.1 (0.28)
B-03	59.5	8.2 (0.14)	36.0 (0.60)	15.3 (0.26)
B-04	80.1	9.9 (0.12)	51.1 (0.64)	19.1 (0.24)

⁽¹⁾Load resisted by helices H1, H2, and the shaft between sections S1 and S3, determined by subtracting the load at sections S1 and S3 (Only for Test B).

In both tests, our findings revealed a significant proportional contribution to the pile resistance from the shaft segment F1, starting at around 15-16% of the total load applied in the first stage and gradually decreasing with the progress of stages. This is a crucial discovery, particularly when considering the observed soil detachment near the ground surface, which has the potential to reduce lateral resistance. Li et al. (2018) argue that the contribution of the shaft to the load-bearing capacity is low due to tension planes or cracks that form as the pile moves upwards. For sandy soils, the authors estimate that the upper shaft is affected to a depth from the soil surface of approximately four times the average diameter of the helix. This depth was also investigated by Li & Deng (2018) through numerical modeling, further emphasizing the importance of our findings.

The proportional contribution from the tip helix (H3) in all loading stages of Test A was always more significant than that of the top (H1) and intermediate (H2) helices. In Test B, despite the problem with the sensor during pile installation and the impossibility of separating the load from helices H1 and H2, it is still possible to notice that, similarly to Test A, the tip helix (H3) proportionally mobilized more load than the two upper helices throughout the stages. The soil surrounding the tip helix undergoes less disturbance than the soil in the helices closer to the surface because the ground typically exhibits increasing resistance with depth (as shown in the profile in Figure 2). Sakr (2009) and Tsuha et al. (2012) observed in static load tensile tests that the tip helix H3 is responsible for most of the load-bearing capacity the helical pile provides. From the beginning of the cyclic tests, it was observed that the contribution of the tip helix was superior to the others. In static load tensile tests, Carvalho (2007) found that the first helix and the shaft resist the initially applied loads. As the loading stages progress,

the contribution of the second helix initiates, and in the final stages, nearing failure, the lower helix gains increased significance in mobilizing resistance.

Our analysis of tests A and B revealed distinct behaviors regarding the evolution of the proportional contribution of the helices with the loading stages. While in Test A there was a gradual increase in the proportion of contribution from the tip helix and a reduction in the contribution from the top and intermediate helices, in Test B the opposite behavior was observed. These differences in behavior have significant implications for the design and implementation of helical piles. The difference between the results of Tests A and B can be attributed to the loading method employed in each test. Although both tests reached the same maximum loads in each loading stage, the amplitudes of the stages were different. In Test A, increasing cyclic amplitudes were used, while in Test B, constant and comparatively smaller amplitudes were used, which may have caused less densification of the surrounding soil. Additionally, in the last stage of Test A, pile instability occurred, causing more transfer of load from the upper helices to the tip helix (H3) towards the end of the test. These insights highlight the need for careful consideration of loading methods and soil conditions in helical pile design.

4.2 Post-cyclic static load test

Following Test B, a static load test was conducted on the same installation, designated as Test C. The purpose was to assess how the previous cyclic loads impacted the system's static load-bearing capacity.

Figure 8 presents the result of Test C, comparing it with results from two pre-cyclic tests conducted by Queiroz (2018) at a nearby location on the same ground. For comparisons with Queiroz's (2018) data, the tensile load-bearing capacity

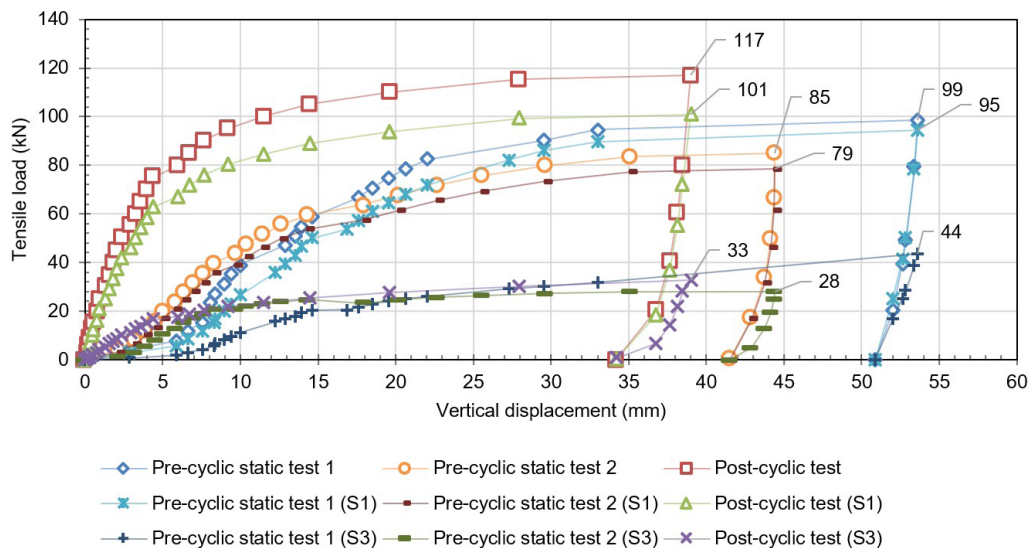


Figure 8. Comparison between the results of pre-cyclic load tests from Queiroz (2018) and post-cyclic load tests from the current study.

was obtained using the method proposed in specification AC358 (ICC-ES, 2007). This specification defines the load-bearing capacity as the maximum load on the axial versus displacement curve. In Test C, failure occurred distinctly, with a measured load at the top of the pile of 117 kN. The maximum loads reached in sections S1 and S3 were 101 kN and 33 kN, respectively. The tests indicate the failure loads, as shown in Figure 8.

The results demonstrated that the loading-unloading cycles on the pile promoted an increase in the load-bearing capacity of the upper segments of the pile (near the top) and a reduction in the load-bearing capacity of the lower segments (near the tip). The load-bearing capacities obtained in the tests conducted by Queiroz (2018) were 92 kN, 87 kN, and 36 kN for the top, section S1, and section S3, respectively. Test C produced a load-bearing capacity for the system 27% higher at the top of the pile, 16% higher at section S1, and 8% lower at section S3, compared to the average results of Queiroz's (2018) pre-cyclic tests.

5. Conclusions

This study presented an experimental analysis of the behavior of a prototype helical pile subjected to cyclic loads and installed in a pure sand deposit. With three helices, the pile was instrumented at three sections along the shaft and subjected to two quasi-static cyclic tensile load tests and one static tensile load test. Cyclic loading was applied in four stages during the quasi-static load tests. The maximum load reached in each stage was the same in both tests. However, in one of the tests, the cyclic amplitude increased between stages, while in the other, it remained constant. The static load test was conducted in five stages of constant load. From this investigation, the following conclusions can be highlighted:

- The contribution of the shaft segment between the ground surface and the upper helix was approximately 15% of the total load applied at the top of the pile in the quasi-static cyclic load tests. Despite the observed soil detachment around the shaft near the ground surface, this portion is significant.
- The helix closest to the pile tip contributed significantly to the total axial load-bearing capacity. Reduced soil disturbance during pile installation in this region led to an increase in soil stiffness with depth. The contribution of this helix remained the highest among all from the beginning of the test.
- The mode of application of load increments influenced the pile response. Increasing cyclic loads between stages led to an increase in the proportion of load mobilized in the tip helix and a reduction in the proportion mobilized in the upper helices. Additionally, instability of the pile occurred in the fourth stage, after five cycles. On the other hand, applying constant cyclic loads resulted in a decrease in the proportion mobilized in the tip helix and an

increase in the proportion mobilized in the upper helices without causing instability to the pile.

- The loading-unloading cycles applied to the helical pile promoted an increase in the load-bearing capacity of the upper segments (near the top) and a reduction in the load-bearing capacity of the lower segments (near the tip).

Acknowledgements

We would like to thank CNPq for the financial support provided for this research.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Manoel Leandro Araújo e Farias: conceptualization, data curation, formal analysis, funding acquisition, investigation, methodology, project administration, resources, software, supervision, validation, visualization, writing – original draft, writing – review & editing. Yuri Daniel Jatobá Costa: conceptualization, data curation, formal analysis, funding acquisition, investigation, methodology, project administration, resources, software, supervision, validation, visualization, writing – original draft, writing – review & editing. Felipy Lima de Queiroz: conceptualization, data curation, formal analysis, funding acquisition, investigation, methodology, project administration, resources, software, validation. João Paulo da Silva Costa: conceptualization, data curation, formal analysis, funding acquisition, investigation, methodology, project administration, resources, software, validation. Ana Leticia Feitosa de Macêdo: formal analysis, investigation, methodology, resources, software, validation, visualization, writing – original draft, writing – review & editing.

Data availability

All data produced or examined in the course of the current study are included in this article.

Declaration of use of generative artificial intelligence

This work was prepared with the assistance of Generative Artificial Intelligence (GenAI) ChatGPT with the aim of assisting in translation and improving the quality of the text. The entire process of using this tool was supervised, reviewed and when necessary edited by the authors. The

authors assume full responsibility for the content of the publication that involved the aid of GenAI.

List of symbols and abbreviations

ABNT	Associação Brasileira de Normas Técnicas
AC	Acceptance Criteria
ASTM	American Society for Testing and Materials
D	Helix diameter
H	Helix
ICC-ES	International Code Council-Evaluation Service
N	Number of cycles applied in a cyclic-loading test
NBR	Norma Brasileira Regulamentadora
Q_{avg}	Average load
Q_{cap}	Load at the pile head
Q_{cyc}	Cyclic load
Q_{int}	Interaction load
Q_{max}	Maximum load
Q_{min}	Minimum load
Q_{Si}	Load measured at the instrumented section of the pile
S	Instrumented section
SP	Poorly graded sand
SPT	Standard Penetration Test
$SPT-N$	Standard Penetration Test blow-count
T	Period
USCS	Unified Soil Classification System
UTM	Universal Transverse Mercator
$\varnothing$	Opening diameter in the shaft

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Strength of silty soil treated with cement and reinforced with plastic bottle waste under conventional and wetting-drying conditions

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Article

Keywords

Silty soil
Cement treatment
Water bottle waste
Durability
Compressive and tensile strength

Abstract

This research aims to study the engineering properties of silty soil reinforced with plastic waste fiber (PWF) extruded from water plastic bottles. Other reinforced soil samples have been treated with cement, except for control samples (untreated soil samples). Curing time, length and percentage of fiber are the variables studied in this research. Lab testing results show that adding plastic waste fiber to the silty soil helps to increase the unconfined compressive strength, indirect tensile strength and punching-tensile strength. The results of soil samples treated with 4% cement show that all strength values have increased significantly. These properties also increase with the increase in the curing time. The increase in strength values of natural soil samples compared with cement-treated samples was (91.7 kPa to 1800 kPa), (6.1 kPa to 312 kPa) and (12 kPa to 371 kPa) for the unconfined compressive, indirect tensile and punching-tensile strength for 28 days of curing time. Regarding the effect of wetting-drying cycles on the strength properties, the results show that the natural soil samples fall apart immediately after being soaked in water. However, the results for samples reinforced with plastic waste fiber, treated with 4% cement and cured for 28 days show that the unconfined compressive, indirect tensile and punching-tensile strength decrease with the wetting-drying cycle from (2500 kPa to 1000 kPa), (435 kPa to 189 kPa) and (580 kPa to 448 kPa), respectively.

1. Introduction

The integrity of the roads and pavement structures constructed on weak soil (like silty soil) can easily be affected, due to the low strength and bearing capacity, poor durability, and large deformation (Sagidullina et al., 2022). The problems accompanied by this type of soil are more exacerbated upon wetting, causing greater challenges to geotechnical engineers (Wang et al., 2016; Soldo & Miletic, 2022). The high costs of using good-quality soils in construction have necessitated to use of the local soil in its original location after improvement. However, silty soils with high water content and low strength pose difficulties for construction projects. Various construction engineers are currently forced to build foundations on such soil. The negative effects of weak soil offer no choice other than rejection, replacement excavation, or dumping soil (Alkiki et al., 2021).

Using natural ground with silty or sandy soil for engineering construction purposes requires ground enhancement techniques like soil reinforcement and soil stabilization (Consoli et al., 2017a;

Chenari et al., 2018; Almajed et al., 2021; Yusof et al., 2023). Most of the soils have an acceptable shear strength, good compressive strength, and poor tensile strength (Aldaood et al., 2020). To overcome the weakness in the tensile strength, fiber could be used. The reinforcement within the soil is closely related to the interfacial shear resistance between the fiber and the soil at the interface, which is influenced by four main factors as indicated in (Gelder & Fowmes, 2016). These factors are friction, bond strength, matrix suction, and interface morphologies. These factors, in turn, depend on the moisture content and dry density, as well as the shape and size of the fibers. These factors can act individually or collectively depending on the characteristics of both the fibers and the soil. The presence of fibers among the soil particles forms bridges that transfer the applied forces to the soil particles. Additionally, the fibers help achieve a balance of forces within the soil structure by transferring loads from weaker levels to stronger levels, thereby increasing the shear and tensile resistance of the soil, which is accompanied by an increase in the corresponding strain values of those forces.

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Submitted on November 2, 2023; Final Acceptance on May 20, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.010723>



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Fiber inclusion in soils brings numerous technical, economic and environmental benefits (Menezes et al., 2019; Hou et al., 2020; Abu Taiyab et al., 2022). In many regions worldwide, PWM use has generated environmental and disposal problems. The use of PWM in engineering construction can resolve disposal problems in an environmentally friendly and cost-effective manner. Peddaiah et al. (2018) mentioned that there is a significant increase in the California bearing ratio and the shear strength parameters of soil samples reinforced with the plastic waste of water bottles. The percentages of enhancement in the soil strengths are ruled by the quantity of these waste materials and their dimensions. Hassan et al. (2021) mentioned that the presence of plastic waste fibers extruded from water bottles improved the compressive strength and CBR results. Further, the greatest values were obtained at 4% fiber with a length of 20 mm. The size of fibers related to soil particle size plays a main role in governing how well the fibers can reinforce the soil. Michalowski (1997) mentioned in his study that to achieve the best interaction between soil and fibers, the fiber length should be no less than one order of magnitude of the sand grains in the soil model used. Therefore, as the length of the fiber increases, resistance also increases up to a certain point, after which it decreases.

Although the PWM addition enhanced the strength properties of soils, their properties are still weak against some of the environmental conditions, like wetting-drying cycles. Thus, chemical stabilization methods using chemical agents (like cement, lime, fly ash, etc.) represent the most important methods to enhance durability and increase soil stability under wetting-drying cycles (Aldaoood et al., 2014; Consoli et al., 2017b; Qureshi et al., 2017; Alkiki et al., 2023). Cement stabilization is widely used for engineering construction purposes, resulting in bearing capacity increase for the soil and enabling a reduction in the pavement layers' thickness. Sariosseiri & Muhunthan (2009) examined the impact of cement percentages (from 2.5% to 10%) on soil engineering properties. The results showed that the engineering properties of cement-treated soils depend not only on the amount of cement but also on the physical and chemical soil properties. Raftari et al. (2014) found that the increment of the cement content led to an increase in the UCS of the improved soil, and it also caused the water content to decrease after curing. Other studies related to the combined effect of fibers and cement illustrate their positive influence on the geotechnical properties of soils (Jassim, 2017; Aldaoood et al., 2021).

It is worth noting that, depending on the nature of the chemical agent used in the treatment, the geotechnical properties of treated soil are mainly affected by the environmental conditions due to cyclic wetting-drying (W-D) (Ashraf et al., 2018; Consoli et al., 2018; Aryal & Kolay, 2020). The variations in soil moisture during seasonal changes cause many geotechnical problems like settlement, cracking and other failures in the soil. According to Stoltz et al. (2014), variations in the engineering properties of soils are attributed to the progressive accretion of permanent strains upon sequential

W-D cycles. The extreme weather in Iraq makes the durability performance of soils under W-D cycles a serious concern. Thus, studying the behavior of cemented reinforced soils in Mosul City under W-D cycles is important. This study aims to evaluate the cement treatment on some engineering properties of silty soil reinforced with plastic waste fiber content (PWF). The durability of cemented reinforced silty soil under W-D cycles was also assessed. The influences of PWF length and percent on the strength properties of natural soil samples were considered. Further, the variations in strength properties of reinforced soil samples before and after W-D cycles were adopted. Loss in weight, pH and electrical conductivity during wetting-drying cycles were also determined.

2. Materials

Three types of materials were used in experimental tests. These materials were silty soil, cement, and plastic waste fiber (PWF). The silty soil samples were collected from a site near Khawaja Khalil regain in Mosul city. The natural soil was a non-plastic soil, with a specific gravity of 2.67% and an organic matter content of 1.37%. The soil consists of 10% gravel, 29% sand, 49% silt, and 12% clay. Based on the grain size analysis and the unified soil classification system, the silty soil was classified as low plasticity silt (ML). The ordinary Portland cement used as a cementing agent in this study was mainly composed of 53.64% tricalcium silicate (C_3S), 18.59% dicalcium silicate (C_2S), 8.58% tricalcium aluminate (C_3A) and 7.92% tetra-calcium aluminoferrite (C_4AF). PWF extracted from water bottles was used as a reinforcement element. These fibers were prepared by shredding the water bottle into sheets and then cutting them into fibers (Figure 1). The fibers were prepared with different



Figure 1. Preparation of PWF from waste plastic water bottles.

length values varying between (10-40 mm) and width values ranging between (2-4 mm). The lengths of the fibers were determined with these values to achieve good interaction and bonding with soil particles compared to the size of the soil samples (especially the unconfined compression test samples). Additionally, these fiber dimensions were chosen to fulfill Michalowski's (1997) assertion that the minimum dimension of the fibers should be at least one order of magnitude larger than the size of the sand particles. The specific gravity of the prepared fiber was 1.26 (ASTM, 2020) and the tensile strength was 345 MPa (ASTM, 2018).

3. Experimental program

3.1 Sample preparation

Silty soil, cement, PWF, and their combination were examined at various laboratory tests. Firstly, the soil samples were oven-dried for 48 hours at 60 °C, then grinded to pass sieve #4. To prepare the soil sample with PWF, a desired quantity of soil was mixed thoroughly with various percentages of PWF (0-2.0% of the dry weight of soil) in a dry state. The PWF was randomly added to the soil to reduce the weak planes that could be expected to develop in the soil-fiber matrix (Aldaood et al., 2021). A fixed quantity of water (equivalent to the optimum moisture content of natural soil) was added to the soil-fiber mixture and mixed thoroughly. The wet mixture was left for about 24 hours in plastic bags to get a uniform moisture distribution in the soil-fiber matrix. At the end of this stage, the soil-fiber mixture was compacted in the desired stainless-steel mold (related to the desired test) to achieve the maximum dry unit weight of natural soil using standard compaction effort. For cemented reinforced soil samples, a desired cement content of 4% was added to the soil sample with varying percentages of PWF ranging from 0.0% to 2.0%. This percentage was chosen (i.e., 4% cement) to satisfy the lower strength value for using soil as a sub-base layer for little traffic (Biswal et al., 2020). Firstly, the cement was mixed with soil in a dry state until a uniform color of the mixture was obtained. Then, the same procedure was adopted to prepare the cemented fiber-reinforced soil, except that the mixtures were left for about 10 minutes for moisture homogeneity before compaction. After compaction, the cemented fiber-reinforced soil samples were enclosed with cling film and covered with many nylon sheets to avoid water loss. Then, these samples were left for curing at 25 °C for 28 days before testing to achieve the minimum strength for use in sub-base or base layers in the case of light traffic. It is worth noting that all soil samples were prepared at an optimum moisture content of about (16%) and a maximum dry density of about 17 kN/m³ of natural soil. Further, the word "samples" mentioned in this paper means composite samples that contain soil, cement, and PWF.

3.2. Laboratory tests

Different laboratory tests were adopted in this study to examine the behavior of uncemented and cemented silty soil reinforced with PWF and exposed to many wetting-drying cycles. These tests include unconfined compression, indirect tensile, punching tensile, pH, electrical conductivity, and wetting-drying tests. The unconfined and indirect tensile tests were carried out on cylindrical samples (50 mm × 100 mm) following the ASTM D2166 (ASTM, 2010). In these tests, the rate of loading was 1.27 mm/min.

For the punching tensile test, the procedure suggested by Kim et al. (2012) and Hasan & Rashid (2017) was used, which employs small disks to determine soil mechanical and tensile properties. The punching tensile test was used in this study because it represents an intermediate case between both the unconfined compression and the indirect tensile tests, which fixes the laboratory results that will be obtained. This test was conducted on uncemented and cemented reinforced samples prepared in a Proctor compaction mold (101.6 mm in diameter and 116.8 mm in height). Two loading discs with a diameter of 25.4 mm were used at the top and bottom of the sample (see Figure 2). The loading rate was also 1.27 mm/min and the load increased till failure occurred. The punching tensile strength (PTS) was determined using the equation below (Equation 1):

$$\sigma_t = \frac{P_{max}}{\pi kbH - a^2} \times 100 \quad (1)$$

where: σ_t = punching tensile strength; P_{max} = maximum applied compressive load; b = radius of soil sample; H = height of soil sample; a = radius of loading disc; and k = factor governed by the angle of friction, sample dimensions, loading discs, cone failure angle and the compressive to tensile strength ratio. The k -value in the above equation is equal to 1.0.

The pH and electrical conductivity (EC) values were determined for the cemented soil before and after the wetting-drying cycle, following the procedure mentioned by Aldaood et al. (2014). After a curing period of 28 days, the cement-treated soil samples were exposed to numerous W-D cycles following the procedure mentioned by Aldaood et al. (2014). In this study, eight W-D cycles were used. Every cycle involved 48 hours of wetting by submerging the samples in water at room temperature (25 °C) and then drying the samples at 60 °C for another 48 hours. The variation in *weight*, *pH* and *EC* values was measured during W-D cycles. Also, the compressive, indirect tensile and punching tensile strength values of the soil samples were measured in the drying state of the 2nd, 4th, 6th, and 8th cycles.



Figure 2. Punching test: (A) sample with metal rings to align loading discs; (B) fixing the sample for test; (C) sample under testing.

4. Results and discussion

4.1 Characteristics of PWF-reinforced natural soil

The variations of unconfined compressive strength (UCS), indirect tensile (ITS) and punching tensile strength (PTS) values are shown in Figures 3-5. As can be seen, all strength values increased with the increase in PWF up to a certain limit, then gradually decreased. Also, there was a difference in the optimum PWF content, which gives the maximum strength value of the adopted tests. In all strength tests, the optimal PWF content was noted to be dependent on the sample size. The maximum PTS values were observed with 1.5% of PWF, while the maximum strength values in other tests were recorded with 1.0% of PWF. This behavior could be attributed to the larger size of the PTS samples as compared with those in other tests. Moreover, in the PTS test, there is further PWF, so the tensile stress in the fiber can be induced due to its packaging around the soil particles. It is worth noting that all strength values of reinforced soil samples were greater than those of unreinforced ones. The increase in the strength values with PWF addition could be due to the: (1) the non-uniform distribution of PWF within the soil matrix can reduce the weak planes between PWF

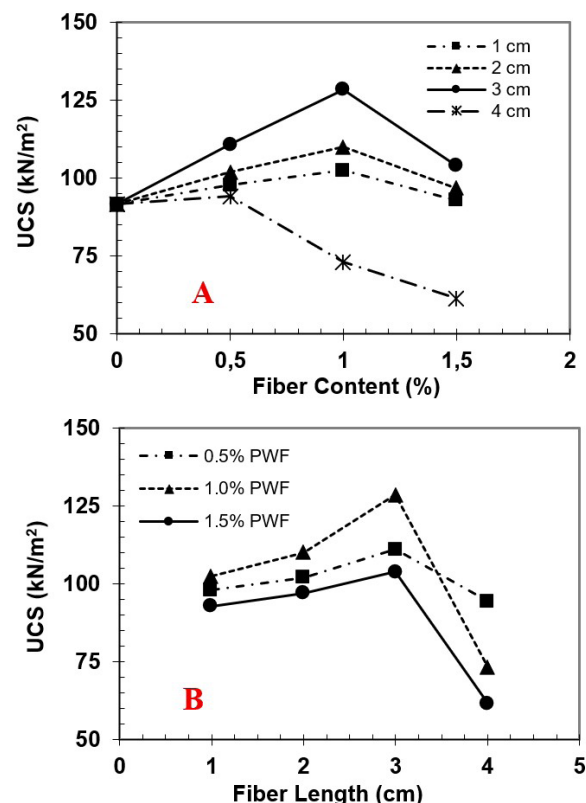


Figure 3. UCS variations with: (A) PWF content; and (B) PWF length.

themselves, and make these fibers act as filaments that intertwine with the soil particles leading to restricts deformation upon

loading; (2) the non-uniform distribution of PWF make these fibers to share the tensile stresses which were developed among soil particles due to the sliding restriction of PWF in the soil mixtures (Menezes et al., 2019); (3) increasing both cohesion and friction between soil particles and PWF, which resulted in the redistribution of the internal stresses through soil and PWF.

The reduction in strength values of reinforced samples with increasing PWF might be due to the poor mixing with higher PWF content, which leads to fewer contact points with soil particles. Also, the higher PWF content, more than the optimal value, caused fibers to clump over each other, leading to the development of a weak plane and forming voids between PWF and soil particles (Aldaood et al., 2021; Correia & Rocha, 2021).

To highlight the influence of fiber length on the strength properties of PWF-reinforced samples, various fiber lengths (ranging from 10 mm to 40 mm) were used. It is observed that fiber length plays an important role in soil strength improvement. All soil samples reinforced with 30 mm long PWF had the maximum values of UCS, ITS and PTS, respectively.

The increase in strength values with the increase in fiber length may be due to the increase in the tensile strength of the longer fibers. In addition to that, as the fiber length increases, the growing friction between fibers and soil increases and absorbs the extra tensile stress. Similar results have been mentioned by Sai & Srinivas (2019) and Louzada et al. (2019).

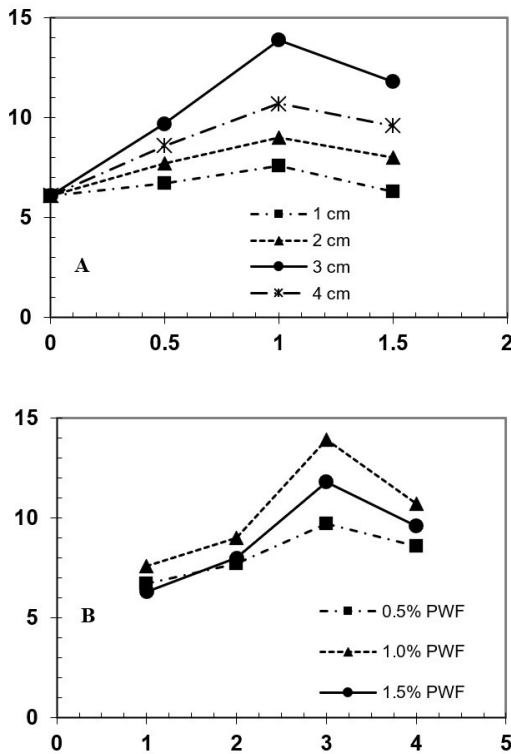


Figure 4. ITS variations with: (A) PWF content; and (B) PWF length.

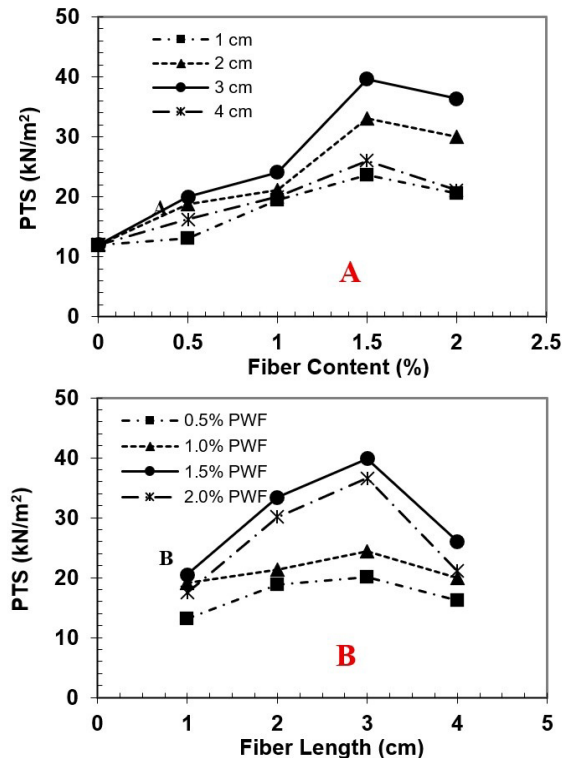


Figure 5. PTS variations with: (A) PWF content; and (B) PWF length.

4.2 Characteristics of PWF-reinforced cemented soil

To study the behavior of cemented reinforced soil samples, the soil samples were reinforced with various percentages (0.0-2.0%) of PWF having 30 mm long (this length value gave maximum strength values as mentioned previously).

4.2.1 Variation of pH, EC and residual moisture values

The pH, EC, and residual moisture content (RMC) values of reinforced soil samples treated with 4% cement and cured for 7, 14 and 28 days were determined. As is well known, PWF is a chemically inactive and hydrophobic material that does not react with cement and/or soil particles. Moreover, PWF materials do not absorb soil moisture. Thus, the pH, EC and RMC are not influenced by the addition of these materials. It is observed that both pH and EC values increased initially with cement addition. These values increased from 9.8 and 720 $\mu\text{S}/\text{cm}$ for natural soil to 12.9 and 1962 $\mu\text{S}/\text{cm}$ for cemented soil samples, respectively. The increase in these values with cement addition could be attributed to the increase in calcium ions (Ca^{2+}). The pH and EC values showed a continuous reduction with curing time. The pH value decreased from 12.9 to 11.78, 11.35 and 11.06, when the soil samples were cured for 7, 14 and 28

days, respectively. At this level of pH value, the pozzolanic reaction products (such as calcium silicate hydrate, CSH, and calcium aluminate hydrate, CAH) continue to form, leading to an increase in the soil strength. The EC values showed the same trend as pH values with curing time; these values decreased from 1962 $\mu\text{S}/\text{cm}$ to 1762 $\mu\text{S}/\text{cm}$, 1563 $\mu\text{S}/\text{cm}$ and 1378 $\mu\text{S}/\text{cm}$ when the soil samples were cured for 7, 14 and 28 days, respectively. The reduction in pH and EC values of cemented soil samples is connected to the reduction in the Ca^{2+} and OH^- ions consumed during pozzolanic reactions. The continuous decrease in moisture content through curing times might affect the pozzolanic reactions. Great attention has been given to estimating the RMC of the cemented soil samples. It was noticed that the RMC decreased with the increase in curing times. Most reduction in moisture content occurred during the first 7 days of curing times; thereafter the reduction in RMC continued slightly. The RMC values decreased from 16% to 12.7%, 12.3% and 11.8% when the soil samples were cured for 7, 14 and 28 days, respectively. The reduction in the RMC could be attributed to cement hydration and the completion of the pozzolanic reactions.

4.2.2 Variation of strength properties value

Figures 6-8 illustrate the variations of UCS, ITS and PTS values of cemented reinforced soil samples cured for 7, 14 and 28 days. As expected, cement addition enhanced the strength values of soil samples. The UCS increased from 91.7 kN/m^2 of natural soil to 1208 kN/m^2 , 1427 kN/m^2 and 1800 kN/m^2 for cemented soil samples cured for 7, 14 and 28 days, respectively. In the same sequences, the ITS increased from 6.1 kN/m^2 to 113 kN/m^2 , 245 kN/m^2 and 312 kN/m^2 respectively. While the PTS increased from 12 kN/m^2 to 125 kN/m^2 , 258 kN/m^2 and 371 kN/m^2 respectively. For cemented reinforced soil samples, it can be seen that all strength values of cemented reinforced samples were increased linearly with increasing curing times. It is also clear that cement addition did not affect the PWF percentage, which gave the maximum values of strength. This means that both maximum values of UCS and ITS remain at 1.0% of PWF, while 1.5% of PWF remains to give the maximum value of PTS (as mentioned previously). The increase in the strength values of reinforced soil samples with both cement addition and curing times was related to the two actions. The first action is cement hydration, which is necessary to provide calcium ions for ion exchange. The second one is the pozzolanic reaction products, such as calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) (Etim et al., 2021). Many studies associated with cement stabilization revealed that the strength characteristics of various types of soils improved with cement addition (Safdar et al., 2020).

The more interesting results observed in Figures 6-8 are that the strength values of cemented reinforced soil samples were larger than the strength of unreinforced cemented samples. This behavior was due to the contribution action of

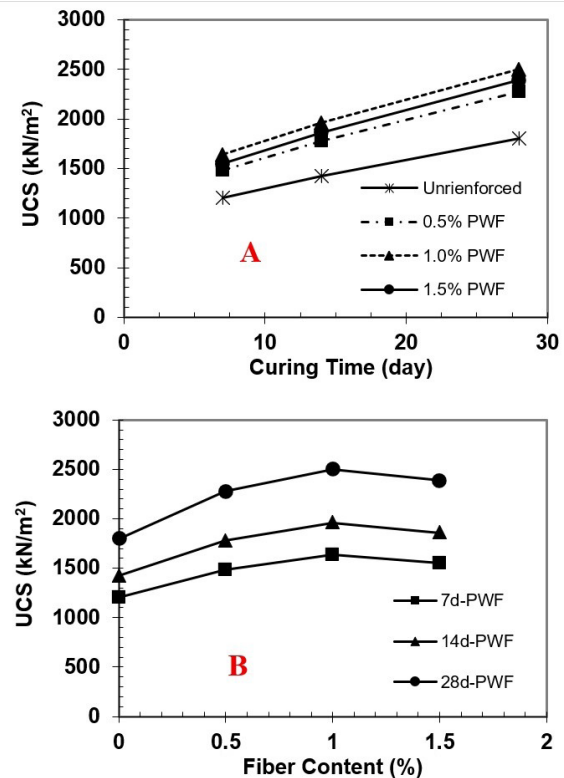


Figure 6. Variations of UCS: curing time (A); and fiber percent (B).

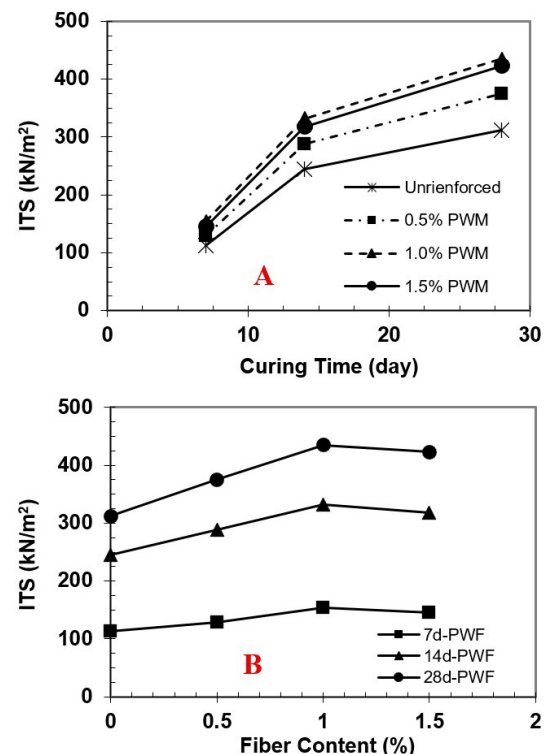


Figure 7. Variations of ITS: curing time (A); and fiber percent (B).

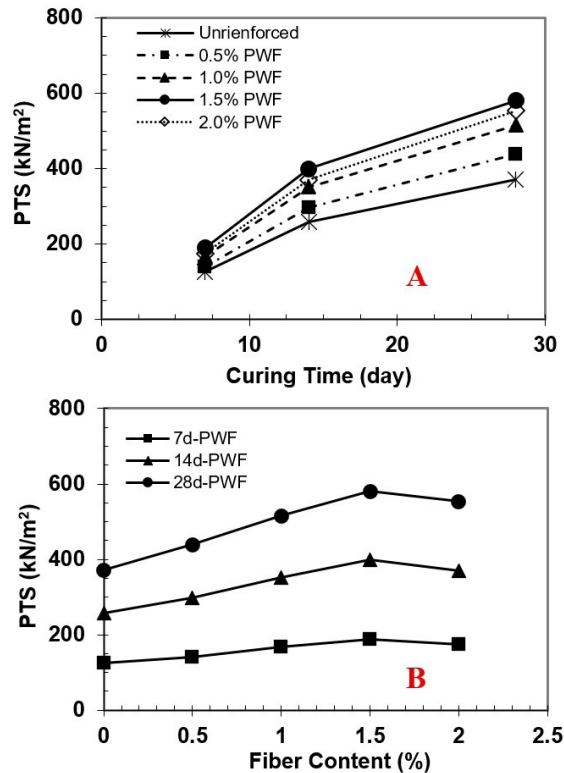


Figure 8. Variations of PTS: curing time (A); and fiber percent (B).



Figure 9. PWF along tension cracks (bridge action of PWF).

both cement and PWF to increase strength. During the curing period, some soil particles may be attached to PWF, resulting in to increase in friction and the bonding strength between them (the soil and the PWF). Furthermore, the sand particles could impact and abrade the PWF surface forming many grooves on the PWF surface and leading to the enhancement of the interlocking between them. Qu et al. (2013) mentioned that the increase in shear strength of the reinforced soil is related to the roughness of fiber and the interface between fiber and soil.

The UCS of cemented soil samples cured for 28 days increased from 1800 kN/m² to 2275 kN/m², 2500 kN/m² and 2390 kN/m² for cemented soil samples reinforced with 0.5%, 1.0% and 1.5% PWF, respectively. In the same manner and for similar samples, the ITS increased from 312 kN/m² to 375 kN/m², 435 kN/m², and 423 kN/m², respectively. Moreover, the PTS increased from 371 kN/m² to 439 kN/m², 515 kN/m², 580 kN/m², and 553 kN/m² for soil samples reinforced with 0.5%, 1.0%, 1.5%, and 2.0% of PWF, respectively. This behavior refers to the positive action of cement and PWF together in enhancing the strengths values, rather than the action of cement and/or PWF alone. The increase in strength value of cemented reinforced soil samples could be attributed to that: (1) the adhesion between PWF and cemented soil particles is further than that between PWF and un-cemented ones; (2) the bonding strength and the friction were also higher in cemented reinforced samples as compared with those in unreinforced cemented ones (Aldaood et al., 2021); (3) the small pores in cemented soil samples lead to more contact zones with PWF; (4) as soon as cracks propagate in soil samples, the PWF across these cracks will be responsible to resist the tension forces in the soil by bridge action of PWF (see Figure 9); (5) visual observation of cemented reinforced samples illustrated that, the surface of PWF became rougher than uncemented samples, thus the friction between PWF and cemented soil particles will be increased.

4.3 Characteristics of PWF-reinforced cemented soil with W-D cycles

The results of the durability test (i.e., wetting-drying test) are presented in terms of variations in pH, electrical conductivity (EC), and percent weight loss for cemented reinforced and unreinforced soil samples subjected to 8 cycles. Further, the UCS, ITS and PTS were also determined at the end of 2nd, 4th, 6th, and 8th cycles of wetting-drying. It is worth noting that natural soil samples (both unreinforced and reinforced samples) did not sustain the durability test. These soil samples failed after 5-15 minutes of soaking in water due to the nonexistent cohesion required to strengthen them against W-D cycles. Thus, all results discussed in the next sections were related to the cemented soil samples.

4.3.1 Variation of pH and EC values with W-D cycles

The variations in pH and EC values of cemented reinforced soil samples with W-D cycles are measured.

These values decreased in the same trend as W-D cycles increased. More reduction in the pH and EC values occurs during the 2nd cycle. At this cycle, the pH values decreased from 11.06 to 9.4, while the EC values decreased from 1378 $\mu\text{S}/\text{cm}$ to 680 $\mu\text{S}/\text{cm}$. After that, a slight reduction in pH and EC values occurs to reach 9 and 545 $\mu\text{S}/\text{cm}$, at the last W-D cycle, respectively.

The reduction in pH and EC values with W-D cycles is expected and was attributed to the reduction in the Ca^{2+} and OH^- ions, which are necessary to complete the pozzolanic reactions. Moreover, cement leaching during W-D cycles led to a reduction in the Ca^{2+} ions and resulted in a decrease in both pH and EC values.

4.3.2 Variation of loss in weight and strength properties with W-D cycles

The durability behavior of the soil could be detected from the weight loss value and the variation of strength properties during the W-D cyclic. Figure 10A shows the cumulative loss in weight of different soil samples for soil samples tested in UCS, ITS, and PTS, during W-D cycles. These values were presented for the dry state only. The results from Figure 10A show that the presence of PWF affects the

degradation process of soil samples. This figure also shows that the value of loss in weight of the soil samples during W-D cycles depends on the size of the soil samples. Different soil sample sizes illustrate different values of weight loss, which affects the strength values of soil samples when exposed to W-D cycles. With the increase in W-D cycle numbers, it was noticed that the loss in weight of soil samples increased at a fairly constant rate, especially in the first cycles. After the fourth cycle of W-D, the rate of loss in weight was increased. This behavior is attributed to the crack propagation, which increased with increasing W-D cycles.

Interestingly, the soil samples tested for PTS showed smaller values for loss in weight, as compared with other soil samples. In other words, PTS samples illustrated a good resistance for W-D cycles. This behavior could be attributed to the larger size of PTS samples, which contained more PWF. Thus, the reason that leads to more interaction between soil and PWF, which gave more resistance under W-D cycles. The loss in weight for UCS and ITS samples was more noticeable and approximately equal. Cement and PWF addition significantly reduced the values of weight losses as compared with natural soil samples (having fragile particle bonds, which was the cause of their failure after 5-15 minutes of soaking in water). The cementing materials (CSH and CAH) and the bonding between PWF and soil particles made the soil samples more durable than the natural soil against W-D cycles. Finally, a visual observation of soil samples, during W-D cycles, illustrated that there were a few cracks that propagated in these samples. Although these cracks affect the strength properties of soil samples (as will be discussed later), the samples kept their shapes till the last cycle of W-D (i.e., all soil samples could be tested in the planned experiment).

Figure 10B illustrates the variations in strength values of cemented reinforced soil samples against W-D cycles. It is observed that all strength values decreased linearly with increasing W-D cycles. The UCS of soil samples decreased from 2500 kN/m^2 to 1000 kN/m^2 , when subjected to eight cycles of W-D. At the same time, the ITS and PTS decreased from 434 kN/m^2 and 580 kN/m^2 to 189 kN/m^2 and 448 kN/m^2 , respectively. The reduction in strength values with W-D cycles could be attributed to the degradation of cementing bonds and a change in the soil microstructure, resulting in a greater loss of strength. Another possible reason for decreasing strength was the formation of cracks that could be created during the W-D cyclic. These cracks weakened the interaction matrix and therefore reduced the strength values. Similar observations were noticed by (Sun & Cui, 2018; Cuisinier et al., 2020; Nabil et al., 2020). Also, it shows that the resistance of the soil to cyclic W-D depends on the size of the soil samples. The highest loss percentages in strength values occurred for unconfined compression samples and indirect tensile samples. The punching-tensile samples illustrate the lowest reduction in strength. It is worth noting that, even after three cycles of W-D, the UCS and ITS

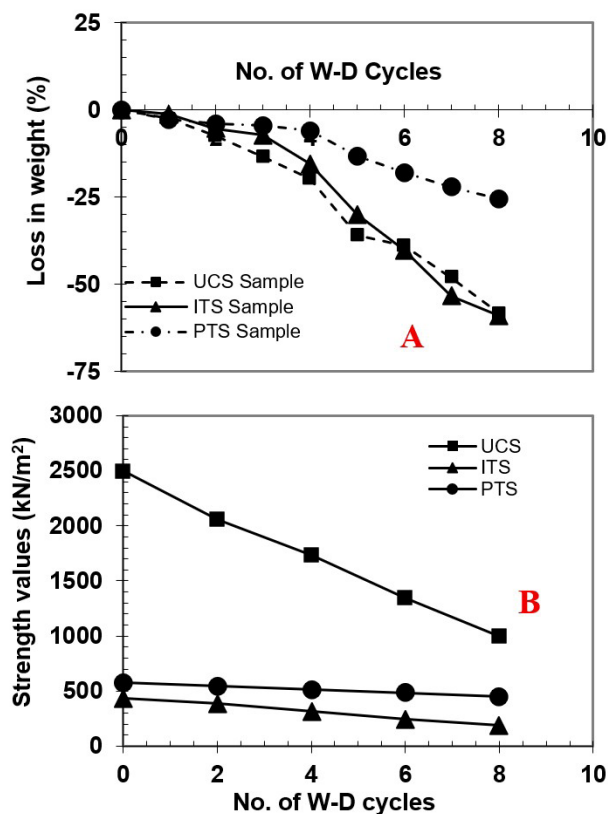


Figure 10. (A) loss of weight of soil samples; and (B) strength variations, with W-D cycles.

of cemented soil samples have not lowered below that of the cemented unreinforced one. While the PTS of the same samples remained higher than the cemented unreinforced even at the end of W-D cycles. Generally, this behavior shows good resistance to W-D due to the reinforcement of the soil samples with PWF. An additional benefit of PWF is the restraint of crack propagation in soil samples during W-D cycles and after initial formation. Before cracking, the PWF looked to have no noticeable effect on the soil behavior. While during W-D cycles, the existence of PWF modifies the mechanism of failure by preventing crack propagation. Further, the results show that the existence of PWF slowed the degradation process for reinforced soil samples during W-D cycles. This means that all soil samples survived for all W-D cycles.

5. Conclusions

Silty soil represents one of the problematic soil types due to its significant susceptibility to water. Therefore, improving the properties of this soil is necessary for its use in engineering projects. Reinforcing the soil using recycled materials, such as waste plastic water bottles, is important in geotechnical engineering applications. Using such materials has considerable economic benefits. Incorporating binding materials like cement with these waste materials greatly enhances the silty soil's properties, particularly under the influence of climatic conditions. The following conclusions can be deduced:

- o Using recycled PWF as reinforcement fibers for silty soil increases compressive and punching tensile strength. The strength values increase with increasing PWF percentage and fiber length up to a certain limit.
- o The combined effects of cement and fiber were better than their effects alone. Meaning that the strength of the soil samples treated with cement and reinforced with PWF was better than those treated only with cement and/or only with reinforcement.
- o The results of the UCS for all study variables illustrated the same trend as the ITS and PTS tests.
- o For all lengths of PWF used, there is an increase in strength values. The maximum increase in strength was noted in soil samples reinforced with PWF with 30 mm long.
- o Irrespective of combinations, all tested samples decreased in strength after W-D cycling, which confirms the negative effect of water during these cycles.
- o The PWF reinforcement of cement-treated soil resulted in good resistance to W-D cycles even up to three cycles. Thus, the choice of increasing cement content can be certain according to the expected severity of the cyclic W-D scenario.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper, and there is no financial interest in reporting.

Authors' contributions

Farah Zaal Mishaal: methodology, investigation. Abdulrahman Aldaood: writing, original draft preparation, reviewing, and editing.

Data availability

Enquiries about data availability should be directed to the authors.

Declaration of use of generative artificial intelligence

This work was done with minimal assistance from GenAI (ChatGPT), less than 20%, to improve readability and language. other aspects of the paper were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

pH	Potential of hydrogen
CSH	Calcium silicate hydrate
CAH	Calcium aluminate hydrate
EC	Electrical conductivity
ITS	Indirect tensile strength
ML	Low plasticity silt
NP	non-plastic
OH ⁻	Hydroxyl ion
OMC	Optimum moisture content
PTS	Punching-tensile strength
RMC	Residual moisture content
UCS	Unconfined compressive strength
USCS	Unified Soil Classification System
W-D	Wetting-drying

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TECHNICAL NOTES

Soils and Rocks
v. 48, n. 2

Mechanical parameters of soil-rubber mixtures from oedometer and direct shear tests

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Claudio Fernando Mahler² 

Technical Note

Keywords

Soil-rubber mixtures
Oedometer tests
Direct shear tests
Geotechnical parameters

Abstract

In Brazil alone, 12.23 million tires were manufactured in the first quarter of 2024. The inadequate disposal of rubber waste from scrap tires is a globally recognized problem. The incorporation of this solid waste into geotechnical projects can serve to mitigate the environmental impact of its disposal. This paper presents complementary results previously published by Abrantes et al. (2025). This first publication dealt with the following procedures applied to the soil and soil-rubber mixtures: specimen preparation, characterization, standard compaction tests and compaction tests with damping ratio measurement, CIU Triaxial tests and Dynamic Triaxial tests. The current publication (Part 2) presents results and discussions about the following tests performed in soil and soil-rubber mixtures: oedometer tests (consolidation and elastic parameters), oedometer tests (permeability) and direct shear tests (strength parameters). The tests were conducted with pure soil and with soil-rubber mixtures in proportions of 2.5%, 5.0%, 7.5%, 10.0%, 12.5%, and 15.0% of rubber weight in relation to the weight of the pure soil. Based on present study, the rubber from shredded scrap tires can be used in applications such as: a) compacted embankments; b) soil walls reinforced with geosynthetics. Nonetheless the use of rubber mixed with soil is not recommended in clay liners for sanitary landfills.

1. Introduction

This paper consists of a complementation of the study conducted by Abrantes et al. (2025) regarding soil-rubber mixtures mechanical behavior. This study has been previously published in the Journal Soils and Rocks under the title “Mechanical parameters of soil-rubber mixtures for sustainable geotechnical projects”. This current publication presents results and discussions about the following tests performed in soil and soil-rubber mixtures: oedometer tests and direct shear tests.

The permeability coefficient remains largely unchanged when rubber is added to the soil, as demonstrated by Özkul & Baykal (2007). Conversely, the compressibility of the soil tends to increase with the addition of rubber to the soil, as reported by Edil & Bosscher (1994) and Sellaf et al. (2014).

Venkatappa & Dutta (2006) performed oedometric tests on rubber-sand mixtures and observed an increase in compressibility with increasing rubber content. Lee et al. (2010) performed oedometric tests on rubber-sand mixtures and observed an increase in vertical deformation with increasing rubber content.

Ghazavi (2004) performed direct shear tests on pure sand and sand-rubber mixtures and reported an increase in friction angle of up to 13.0% with the addition of up to 15.0% rubber to the loose sand. The author also reported a slight increase of about 2.7% in the mixtures with compacted sand. As for the friction angle, the results presented in the study for clay soil are more similar to those obtained for compacted sand, according to the results presented by Ghazavi (2004).

Cetin et al. (2006), when performing direct shear tests under unsaturated conditions on predominantly pure clay soils and soil-rubber mixtures, observed a significant increase in the shear strength of the soil as rubber was added. Cohesion tended to increase to 40.0% rubber content in the soil. On the other hand, the friction angle tended to decrease up to that content.

2. Materials and methods

The rubber content used in the studies were 2.5%, 5.0%, 7.5%, 10.0%, 12.5%, and 15.0% of the rubber weight in relation to the weight of pure soil. Laboratory tests were conducted on pure soil and soil-rubber mixtures. The

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Submitted on April 16, 2025; Final Acceptance on April 17, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.006325>



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methodology and standards employed in the tests performed, both on pure soil and soil-rubber mixtures, are explained in detail by Abrantes et al. (2025).

2.1 Oedometer consolidation and permeability tests

Seven compression oedometer tests were performed on the pure soil and the soil-rubber mixtures. The tests were conducted in accordance with the recommendations of Brazilian Standard NBR 16853 (ABNT, 2020). The objective of the tests was to determine the following geotechnical parameters: E_{oed} = oedometer modulus; E = elastic modulus; K_v = vertical permeability coefficient; C_c = compression index; C_s = swelling index and c_k = rate of variation of the void index with permeability (a parameter from the Plaxis software for numerical modeling).

The values of E were calculated for each loading stage using Equation 1, and the values of E_{oed} were calculated directly from the specific deformation (ε) versus vertical effective stress where the specific deformation is a function of the void ratio (e). The Poisson's ratio (ν) was assumed to be equal to the average Poisson's ratio obtained from triaxial tests with the soil and the rubber content mixtures of 10.0% and 15.0%. Therefore, the value of 0.234 was used to represent ν in Equation 1.

$$E = E_{oed} \frac{(1+\nu)(1-2\nu)}{1-\nu} \quad (1)$$

where E = elastic modulus; E_{oed} = oedometer modulus; ν is the Poisson's ratio; $\nu = K_o/(1+K_o)$ and $K_o = 1 - \sin \phi'$. K_o is the coefficient of earth pressure at rest and ϕ' is the effective friction angle of the soil.

The specimens were molded by driving a steel ring onto the previously compacted soil in the compaction cylinder.

2.2 Direct shear tests

A series of direct shear tests were conducted on pure soil and soil-rubber mixtures. The tests were conducted under both saturated and unsaturated conditions, applying normal stresses of 12.5, 25, 50, 100, 200 and 400 kPa. One test was also performed on pure rubber under saturated conditions. The tests were carried out according to the recommendations of ASTM D 3080-98 (ASTM, 2003) and the applied strain rate was 0.055 mm/min. The main objective was to determine the shear strength parameters (cohesion and angle of friction) of the pure soil, the mixtures, and the rubber. The specimens were molded using a procedure similar to that used in the oedometer tests. A total of 84 direct shear tests were performed.

3. Results and discussion

3.1 Oedometer tests – consolidation and elastic parameters

Figure 1 shows the void ratio (e) versus vertical effective stress (σ'_v) curves obtained for the seven specimens tested.

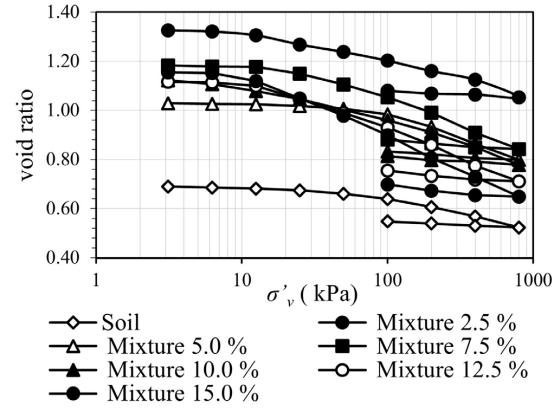


Figure 1. Vertical effective stress versus void ratio curves for pure soil and soil-rubber mixtures.

Figure 1 also shows that, in general, the compressibility of the material increases as the rubber content increases. This behavior is observed during both loading and unloading of the specimens. It should be noted that the C_c values range from 0.13 to 0.26 for mixtures with rubber content between 0.0% and 15.0%. In this instance the incorporation of 15.0% of rubber into the soil resulted in a 100% increase in compressibility. The C_s values remained between 0.029 and 0.080 for rubber contents ranging from 0.0% to 15.0%. The behavior of the compressibility coefficient (C_c) is also consistent, with the incorporation of 15.0% rubber resulting in an increase of 176%.

Figure 2a shows the variation of the compression index (C_c) with rubber content, while Figure 2b shows the variation of the swelling index (C_s), also in relation to the rubber content (0.0% to 15.0%). Both parameters were calculated for a vertical effective stress interval (σ'_v) of 200 to 800 kPa. This stress interval was chosen because the curves exhibit linearity within this stress range. In addition, consideration of higher stress levels provides values that can be considered conservative and on the side of safety.

Figure 2a shows that the compression index (C_c) value typically increases with increasing rubber content. Consequently, when an external load is applied, the deformations will be greater in the mixtures with the highest rubber content. Figure 2b shows that the swelling index (C_s) increases in direct proportion to the rubber content, exhibiting a linear growth pattern. Therefore, the elastic response of the material, in terms of strain, increases with increasing rubber content following the removal of the external load.

Figure 3a shows the variation of the oedometer modulus (E_{oed}) with vertical effective stress (σ'_v) for the loading stages applied to the specimens. Figure 3b shows the elastic modulus (E) values versus effective vertical stresses (σ'_v).

Figure 3 shows that as the rubber content increases, the oedometer (E_{oed}) and elastic modulus (E) decrease. The decrease in E_{oed} and E with increasing rubber content is less pronounced from $\sigma'_v \approx 200$ kPa, indicating that the addition of rubber increases the deformability of the material, making it more ductile.

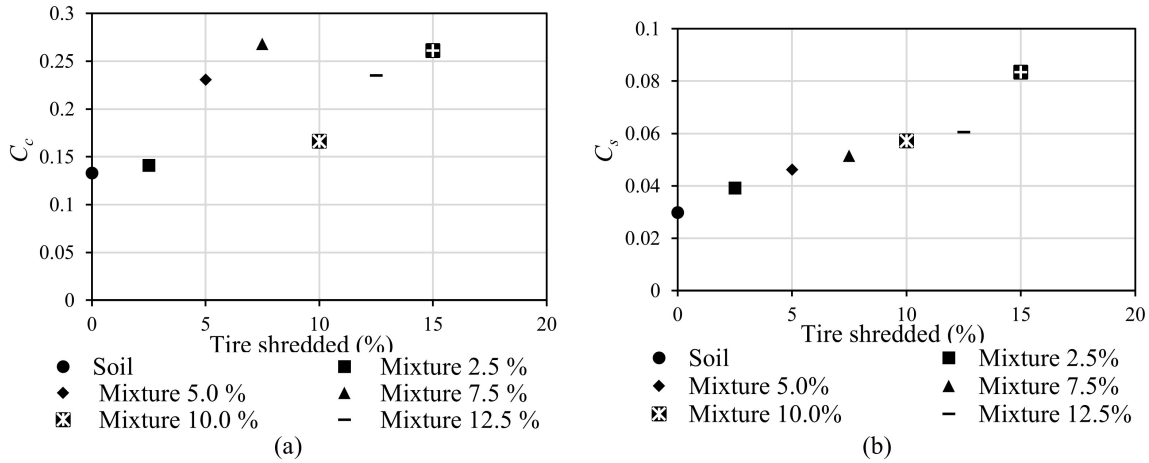


Figure 2. Compressibility parameters: (a) variation of C_c with σ'_v ; (b) variation of C_s with σ'_v .

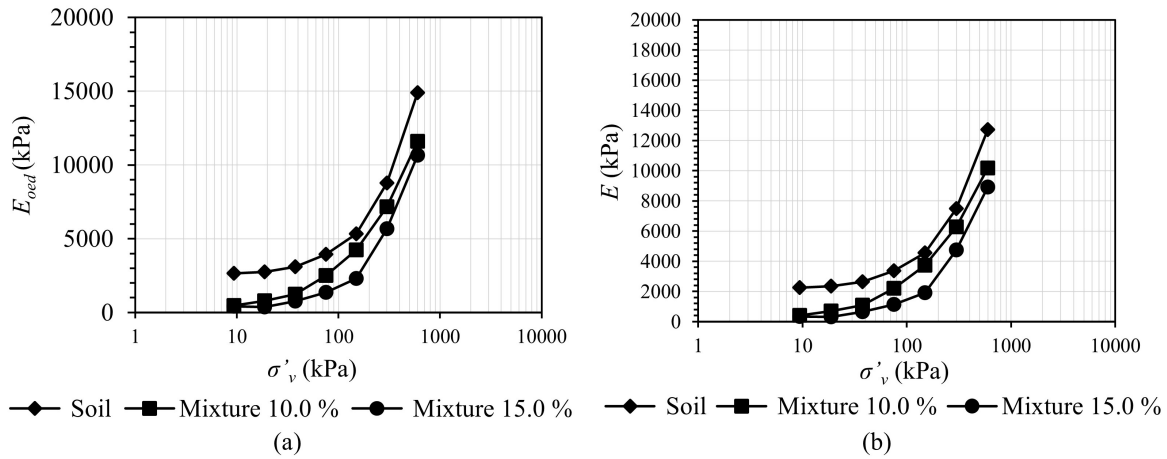


Figure 3. Elastic modules: (a) variation of E_{oed} with σ'_v ; (b) variation of E with σ'_v .

3.2 Oedometer tests – permeability

The variation of the permeability coefficient (K_v) as a function of the variation of the effective vertical stress (σ'_v) of pure soil and soil-rubber mixtures is shown in Figure 4.

As expected, Figure 4 shows a decrease in the permeability coefficient with increasing effective vertical stress. This behavior occurs for both pure soil and mixtures, regardless of rubber content. With respect to the permeability coefficient (K_v), the highest values are generally associated with the highest rubber content used in the mixtures. Lower rubber content (up to 7.5%) slightly changes the K_v value. The observed behavior shows that rubber particles can create a preferential flow path that increases the K_v value. For example, at an effective vertical stress of 100 kPa, the pure soil has a permeability of $1 \cdot 10^{-4}$ cm/s, while at a rubber content of 15.0%, the permeability increases by 700% to a value of $1 \cdot 10^{-4}$ cm/s.

Equation 2 translates the variation of the coefficient of permeability as a function of the void ratio. This relationship is essential if the intent is to model the material with a K_v update related to the deformations that have occurred in that material. Table 1 shows the c_k values (rate of variation

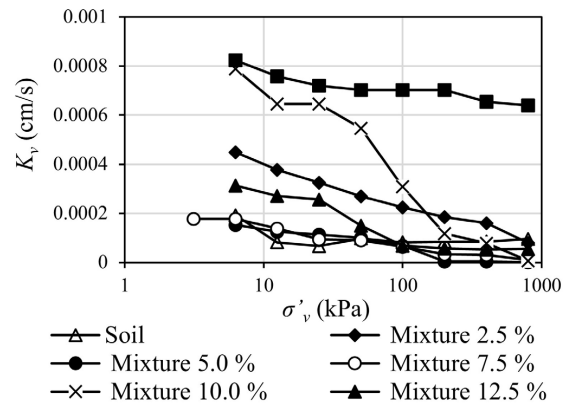


Figure 4. Coefficient of permeability versus effective vertical stress curve.

Table 1. Values of c_k as a function of rubber content.

Rubber content	0.0 (%)	2.5 (%)	5.0 (%)	7.5 (%)	10.0 (%)	12.5 (%)	15.0 (%)
c_k [-]	0.55	0.36	0.12	0.27	0.16	0.53	0.46

of the void index with permeability) for pure soil and soil-rubber mixtures.

$$c_k = \frac{\Delta e}{\log\left(\frac{k_v}{k_{v0}}\right)} \quad (2)$$

where: Δe = void ratio variation, K_{v0} = initial coefficient of permeability (first loading stage), and K_v = coefficient of vertical permeability (subsequent loading stages). Loading stages refer to the oedometer compression tests.

Table 1 shows that c_k ranges from 0.12 to 0.55 for the mixtures with rubber content between 0.0% and 15.0%. No

discernible trend of increase or decrease in c_k was observed with each increase in rubber content.

3.3 Direct shear tests

This section presents the results of the investigation of the shear strength parameters of the pure soil, rubber, and soil-rubber mixtures. The percentages of rubber content were 2.5%, 5%, 7.5%, 10%, 12.5%, and 15.0%. These parameters were obtained from direct shear tests conducted under both saturated and unsaturated conditions. Figures 5a to 5h show the strength envelopes obtained from the tests. Figure 6a shows

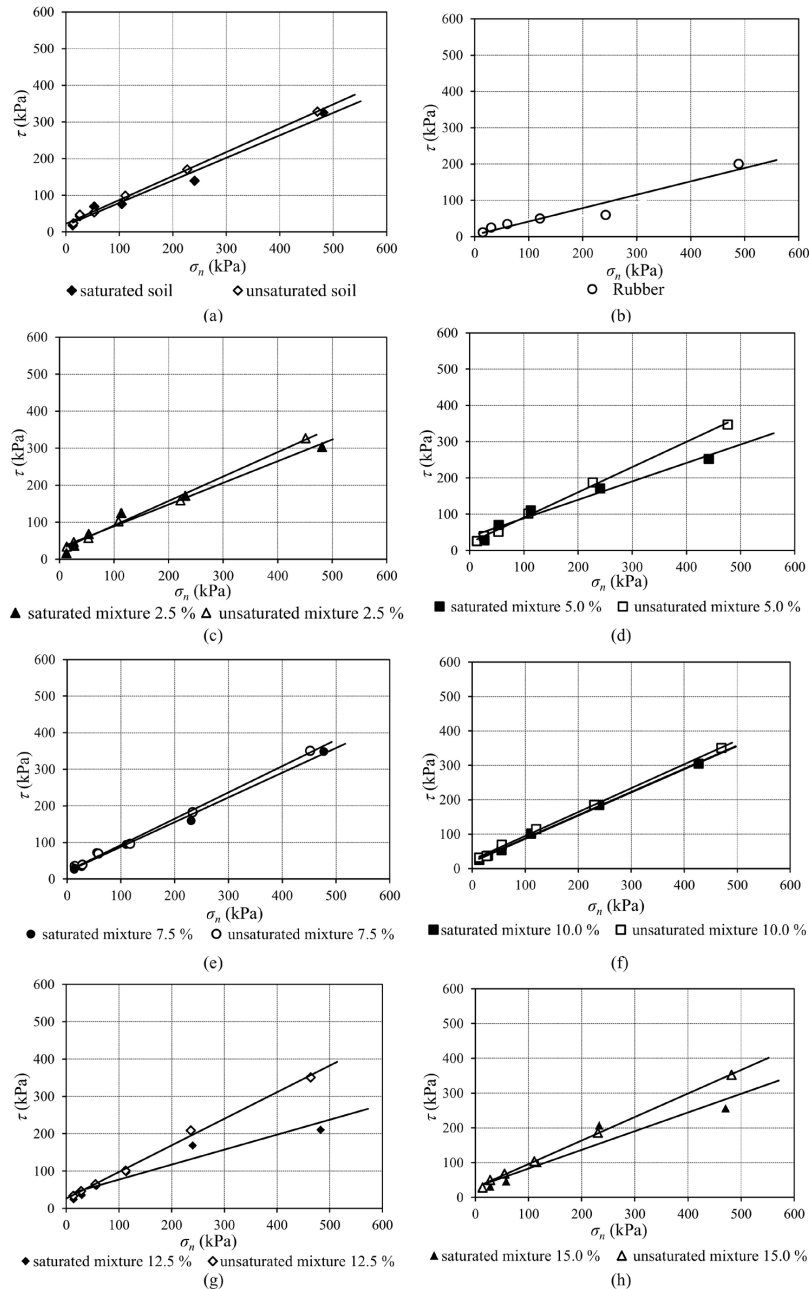


Figure 5. Strength envelope under saturated and unsaturated conditions: (a) pure soil; (b) pure rubber; (c) mixture 2.5%; (d) mixture 5.0%; (e) mixture 7.5%; (f) mixture 10.0%; (g) mixture 12.5%; (h) mixture 15.0%.

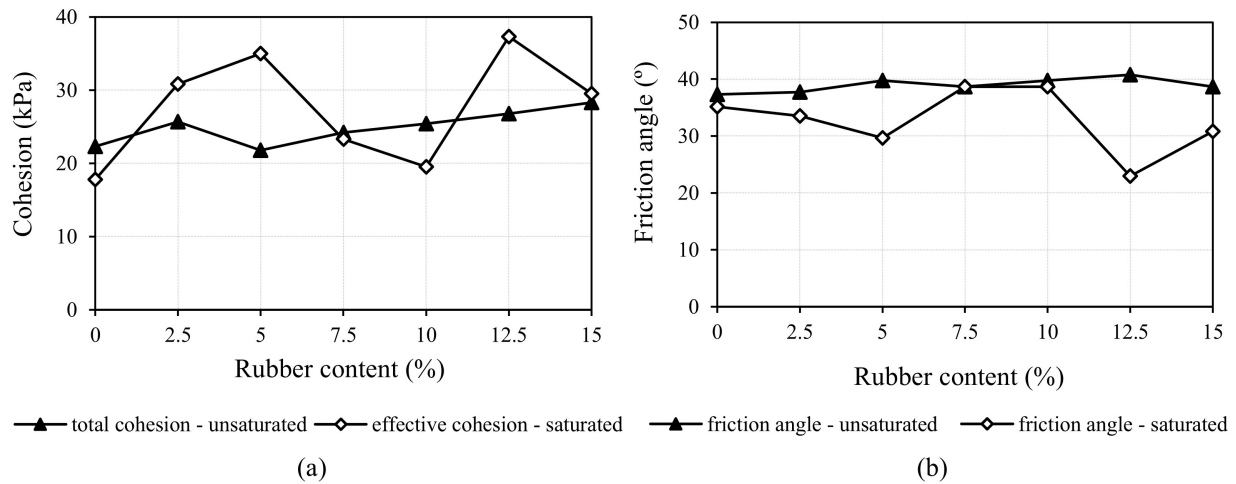


Figure 6. Variation of shear parameters (a) variation of cohesion; (b) variation of the friction angle.

the variation of effective cohesion (c') and total cohesion (C) with rubber content under saturated and unsaturated conditions. The results shown in Figure 6b illustrate the variation of the effective friction angle as a function of the rubber content applied, considering the envelopes for both saturated and unsaturated conditions.

Total cohesion (C) is defined by the following Equation 3, which is based on the classic statement by 0 regarding the shear strength of unsaturated soil.

$$C = c' + (u_a - u_w) \cdot \operatorname{tg} \phi^b \quad (3)$$

where C = total cohesion; c' = effective cohesion; $(u_a - u_w)$ = matric suction; $\operatorname{tg} \phi^b$ = relationship between shear stress and matric suction; u_a = pore air pressure; u_w = pore water pressure.

According to Figure 6, for the unsaturated condition, there was a slight tendency for the total cohesion to increase with increasing rubber content. In unsaturated conditions, the friction angle remained largely unchanged, regardless of the rubber content. Therefore, for the unsaturated condition, as found in compacted soil-rubber mixtures in the field, the addition of rubber does not cause a decrease in the shear strength parameters.

In the saturated condition, there was an increase in the cohesion intercept for the mixtures with up to 5.0% rubber content. However, a decrease in cohesion was observed for the mixtures between 5.0% and 10%. For mixtures with more than 10% rubber content, the effective cohesion increased again, followed by a subsequent decrease. The effective cohesion and friction angle exhibited oscillatory behavior in the saturated condition. In the saturated condition, which is critical for earthworks, there was generally no decrease cohesion, except for the 10.0% rubber content mixture.

Conversely, the friction angle decreased for all the rubber content mixtures, except 7.5% and 10.0% due to saturation.

4. Conclusions

- From the oedometer tests it can be concluded that: a) the highest permeability values are associated with the highest rubber content used in the mixtures. Lower rubber content (up to 7.5%) slightly changes the K_v value. The observed behavior shows that rubber particles can create a preferential flow path, increasing the K_v value; b) the compression index, C_c , and the swelling index, C_s , decrease as the amount of rubber in the soil matrix increases; c) the oedometer modulus (E_{oed}) and the elastic modulus (E) decrease as the rubber content increases;
- The results of the direct shear tests indicate that for the unsaturated condition there is a slight tendency for the cohesion to increase as the rubber content increases. For the saturated condition, an oscillation in the cohesion value was observed as the rubber content increase. The friction angle related to the unsaturated condition remains the same regardless of the amount rubber. The friction angle related to the saturated condition presents oscillations as the rubber content increases;
- Based on this study, the rubber from shredded scrap tires can be used in applications such as: a) compacted embankments; b) soil walls reinforced with geosynthetics. The first application is highly useful in terms of sustainability because the construction of embankments involves a very amount of material. The use of the soil-rubber mixture is not recommended in clay liners for sanitary landfills, because the rubber produces an increase in soil permeability.

Acknowledgements

The authors would like to thank CNEN Brazil, the National Nuclear Energy Commission, for the financial support provided for this.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have reviewed and affirmed the content of the paper and there is no financial interest to report.

Authors' contributions

Mario Vicente Riccio Filho: supervision, validation, writing – review & editing. Lorena Gomes Abrantes: conceptualization, data curation, formal analysis, investigation, methodology, software, writing – original draft. Claudio Fernando Mahler: validation, writing – review & editing.

Data availability

The datasets generated analyzed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

c'	Effective cohesion
c_k	Coefficient of variation of the permeability with void ratio
e	Void ratio
u_a	Pore air pressure
u_w	Pore water pressure
$(u_a - u_w)$	Matric suction
C	Total cohesion
C_c	Compression index
CIU	Consolidated, isotropic, and undrained triaxial test
C_s	Swelling index
E	Elastic modulus
E_{oed}	Oedometer modulus
K_o	Coefficient of earth pressure at rest

K_v	Coefficient of permeability
K_{vo}	Initial coefficient of permeability (first loading stage)
W_o	Optimum moisture content
γ	Unit weight
ε	Specific deformation
ν	Poisson's ratio
σ'_v	Vertical effective stress
ϕ'	Effective friction angle of the soil
$\text{tg } \phi^b$	Relationship between shear stress and matric suction
Δe	Change in void ratio

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Experimental analysis on the effect of overburden stress on the smear zone characteristics of clayey soil

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Technical Note

Keywords

Clay
Consolidation
Vertical drain
Smear effect
Overburden stress

Abstract

Preloading with vertical drains is a well-proven ground improvement technique suitable for soft clay deposits. This technique involves installing vertical drains that cause soil remolding in the immediate vicinity of its surface, creating a disturbed zone known as the smear zone. This study aims to investigate the impact of overburden stress on the characteristics of the smear zone. Very often, the vertical drains penetrate to depths up to 20 m. In such situations, the soil experiences varying stresses, leading to different responses to the drain-induced soil disturbances. Very few studies have investigated how overburden stress affects the smear zone. Through a series of experiments encompassing vertical and radial consolidation tests, along with vane shear tests, this paper evaluates the variation in shear strength and consolidation properties within the smear zone, simulating different depths. Employing three overburden pressure intensities (25 kPa, 50 kPa, 100 kPa, equivalent to depths of about 4 m, 8 m and 16 m), the study establishes that the remolding effect intensifies with depth. Furthermore, it demonstrates the influential role of the smear zone on the undrained shear strength properties of the improved ground, highlighting its variability based on overburden stress.

1. Introduction

Numerous regions across the world, particularly along the coastal area, feature very soft clay characterized by unfavourable geotechnical engineering properties. To improve the properties within the construction time available, it is essential to expedite the consolidation of such compressible soils. The use of preloading with prefabricated vertical drains (PVDs) is a viable option (Chakraborty et al., 2023; Baral et al., 2018; Sengul et al., 2017; Chu et al., 2004). In the course of PVD installation, an anchor plate is securely attached to the end of the drain and subsequently embedded into the ground using a hollow mandrel, ensuring penetration to the designated depth. PVD design entails specifying the spacing and length of PVDs based on the results of soil consolidation analysis (Wang & Shi, 2023). Vertical drains are commonly installed to significant depths, leading to varying overburden stresses along the soil depth. Consequently, the extent of remoulding the soil due to drain installation can differ across these depths.

Only a handful of studies (Indraratna et al., 2015; Juneja et al., 2013; Sathananthan et al., 2008; Hird & Moseley, 2000; Bergado et al., 1993; Akagi, 1981; Casagrande & Poulos, 1969) have attempted to capture the influence of

overburden stress on the induced smear zone. Their studies have mentioned that the smear thickness and the properties of soil within the zone of disturbance vary with changes in the overburden stress. Akagi (1981) and Bergado et al. (1993) suggested from the field study that the disturbance increases with depth. Akagi (1981) conducted a field study at the Asian Institute of Technology campus in Bangkok, Thailand, to examine the impact of sand drains on the behavior of soft clay. The average ratio of the remolded to undisturbed coefficient of consolidation ranged from 0.58 to 0.24 for vertical drainage and from 0.63 to 0.12 for radial drainage as the depth increased from 2.5 m to 7 m. However, a contrasting conclusion was drawn by Indraratna et al. (2015) and Sathananthan & Indraratna (2006), asserting that the diameter of the disturbed zone decreases at greater depths. Indraratna et al. (2015) collected soil samples from a vertical drain installed site, at Ballina, to investigate the effect of smearing on the consolidation characteristics of soil. They observed that smear extent decreases with an increase in depth. This discrepancy was attributed to the stiffening of clay at deeper layers, resulting in reduced disturbance. However, the limitation of the study is that they have considered only up to a depth of 2 m. From the numerical study by Sathananthan et al. (2008), it was noted that the

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Submitted on July 8, 2024; Final Acceptance on April 14, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.005524>



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radius of the smear zone varies from $6.6 r_w$ to $4.9 r_w$ when the depth increases from 0 to 11 m, where r_w is the drain radius.

The zone of smear exerts a noticeable influence on the undrained shear strength characteristics of the improved ground, and this influence varies with overburden stress. A previous investigation (Juneja et al., 2013) aimed at studying the undrained shear strength (s_u) of Kaolinite clay reinforced with SCP (sand compaction piles) and induced smear zones revealed that the presence of the smear zone impacts the ratio of s_u value to mean effective stress in the composite sample. Further insight into the smear zone characteristics under varying overburden stresses is imperative, pertaining to the lack of studies in this area. A comprehensive investigation into the variation of the shear strength and consolidation properties of the clay in the smear zone with increasing overburden stress is necessary. The current study developed a methodology to experimentally investigate the influence of overburden pressure on the characteristics of the smear zone in soil.

2. Methodology

In the present study, one dimensional and radial consolidation tests were performed on the soil to examine the influence of overburden stress on the smearing effect. The sample used was IIT Lake clay, collected from the lake in the IIT Madras Campus, Chennai, India. The properties of the sample used in the present study are given in Table 1. Preconsolidated (PC) through reconstitutions and Remolded (RM) samples were tested under different overburden stress intensities.

One-dimensional consolidation tests were performed using a standard consolidation cell measuring 40 mm in height and 100 mm in diameter, meeting the specifications outlined in IS 2720-Part 15 (IS, 1986) and ASTM D2435 (ASTM, 2011). The schematic diagram and photograph of the consolidation cell used in the study are shown in Figure 1. A radial consolidation test was carried out on the same cell by the insertion of rubber sheets with holes at their centres, positioned on both the bottom and top surfaces of the specimen, thereby preventing vertical drainage. Sridhar & Robinson (2011) and Aparna et al. (2023) used a similar apparatus setup.

Soil slurry was prepared with an initial water content of 1.5 times the liquid limit water content. For simulating the stress state close to the in-situ field conditions, the soil was given a stress history through preconsolidation. The reconstituted soil samples were prepared by consolidating the soil slurry under an effective normal stress of 12.5 kPa. A clamping ring, which also acts as the collar, was placed above the consolidation cell and the sample was filled up to the top of collar (Figure 2a). The frame used for preconsolidation and the specimen in the consolidation cell with frame is shown in Figure 2b and Figure 2c.

Preconsolidation continued till the primary consolidation was complete. After preconsolidating the sample, it was unloaded and trimmed to the required sample height of 40 mm. Shear strength of the specimen was determined using the vane shear apparatus and the water content was measured. This study's preconsolidated (PC) sample simulates the undisturbed in-situ soil. The same PC sample was remolded (RM) to simulate the smear zone. Three overburden pressure intensities of 25 kPa, 50 kPa and 100 kPa, were employed. This corresponds to depths of about 4 m, 8 m and 16 m, respectively, in the field. The effective pressure intensities used in the tests range from 12.5 kPa to 200 kPa. Both radial inflow consolidation tests with central sand drain and 1-D consolidation tests with double drainage were conducted. Figure 3 illustrates the methodology adopted for PC and RM samples under vertical and radial drainage conditions with an overburden pressure of 25 kPa. A similar methodology was adopted for 50 kPa and 100 kPa pressure intensity.

One-dimensional consolidation experiment was carried out on PC samples, subjecting them to a particular overburden pressure intensity. Subsequently, the specimen was unloaded, and its shear strength was assessed by performing a vane shear test (VST). The entire sample was then carefully removed and manually remolded. This RM sample was backfilled into the consolidation ring, and the pressure intensity was applied, ranging from 12.5 kPa to 200 kPa, using an incremental loading ratio of one. The sample underwent consolidation in its undisturbed condition for comparative analysis of the strength and consolidation properties of both preconsolidated (PC) and remolded (RM) samples. Notably, remoulding was not performed in the case of the preconsolidated sample.

Radial consolidation test was conducted by adopting the same procedure as 1-D consolidation, with the exception of the inclusion of sand drains. The loading process involved applying pressure to the preconsolidated (PC) samples until the intended pressure intensity was reached. Subsequently, the samples were unloaded and the shear strength was assessed using a vane shear device. The sample was then properly remolded and reinstated in the consolidation ring with central drainage facilitated by sand drain. After the preparation of the sample with radial drainage, the loading was progressively increased from 12.5 kPa to 200 kPa. In an undisturbed radial consolidation test, the sample was not remolded, and after applying a

Table 1. Basic characteristics of the clay utilized in the study.

Properties	Value
Plastic Limit (%)	25
Liquid Limit (%)	45
Plasticity Index (%)	20
Specific gravity	2.60
Sand (%)	57
Clay size (%)	33
Silt size (%)	10

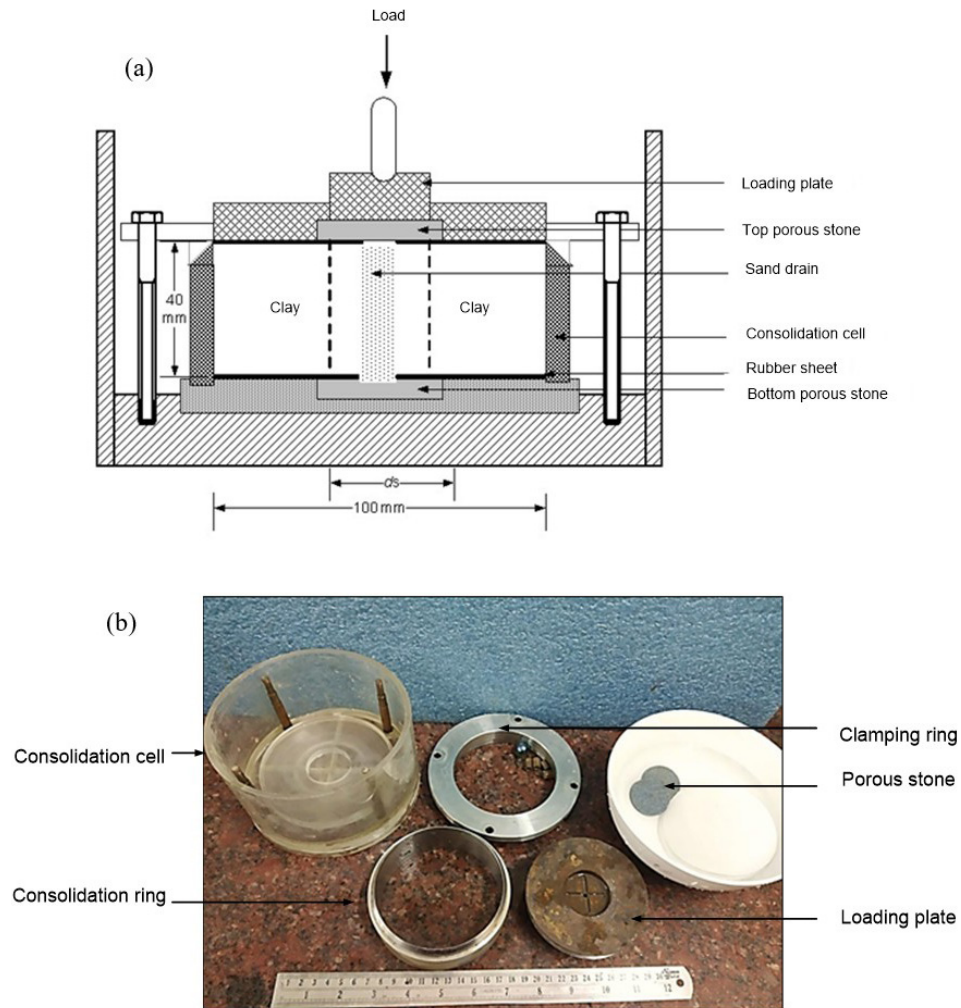


Figure 1. (a) schematic (Aparna et al., 2023); (b) photograph of the 100 mm consolidation cell utilized to perform the consolidation test.

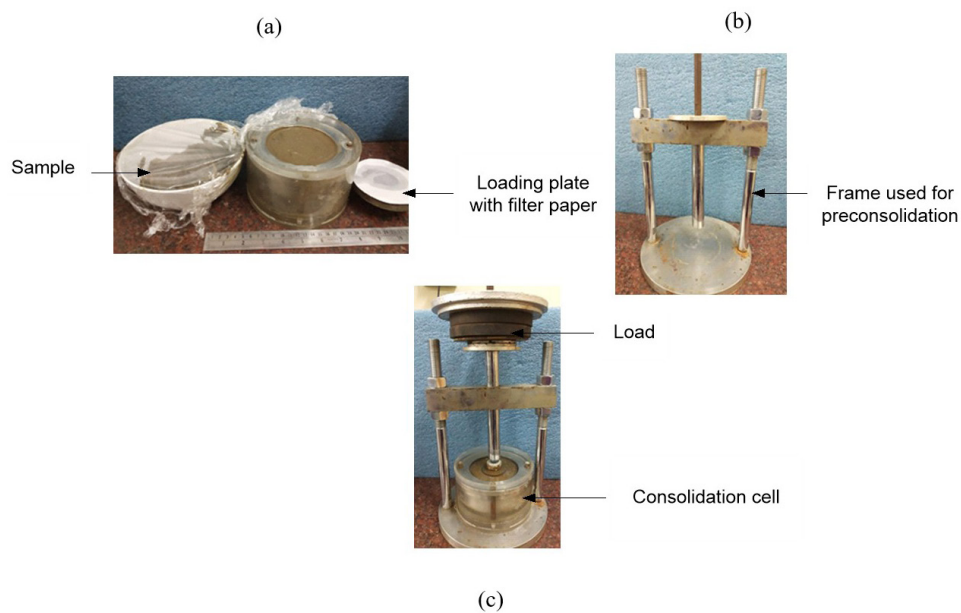


Figure 2. Photograph for preconsolidating the specimen: (a) cell with slurry sample; (b) frame used; (c) final set up.

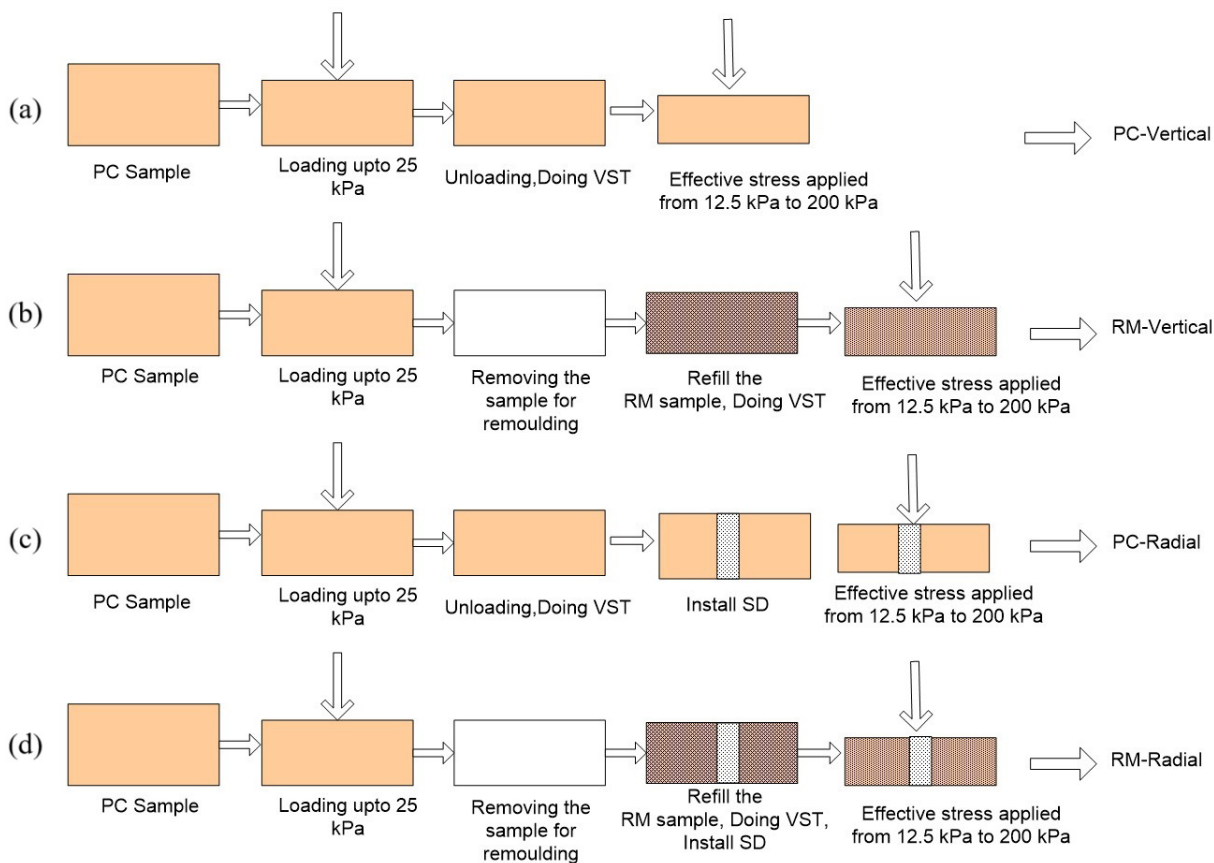


Figure 3. Methodology implemented for: (a) PC-vertical; (b) RM-vertical; (c) PC-radial; (d) RM-radial samples with an overburden pressure of 25 kPa.

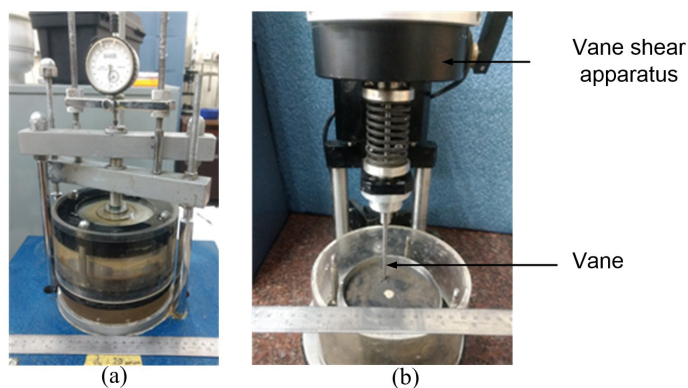


Figure 4. Photograph of: (a) consolidation setup; (b) vane shear test after consolidation of soil.

specific pressure intensity, unloading was performed, followed by the installation of a sand drain. Subsequent loading was conducted, ranging from 12.5 kPa. The VST was performed at the start and end of the test. A photograph of the consolidation set-up and the vane shear test setup is shown in Figure 4. Thus, the initial and final shear strengths can be found.

From this study, the consolidation coefficient for preconsolidated samples under vertical drainage ($c_{v(PC)}$) and radial drainage ($c_{h(PC)}$) conditions can be found. Additionally, the consolidation coefficient for the remoulded sample under vertical drainage ($c_{v(RM)}$) and radial drainage ($c_{h(RM)}$) conditions can be determined. The shear strength of both preconsolidated and remoulded samples can be

Table 2. Reduction percentage of shear strength with PC and RM samples.

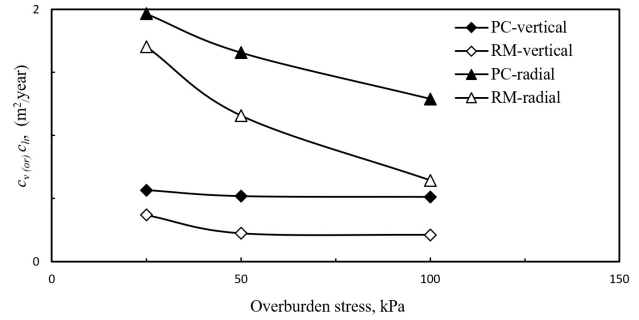
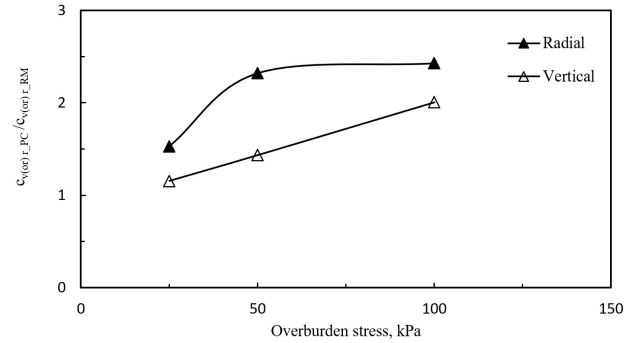
Overburden stress (kPa)	s_u of PC sample (kPa)	s_u of RM sample (kPa)	$\Delta s_u = (s_{u-PC} - s_{u-RM})$ (kPa)	Reduction percentage $(s_{u-PC} - s_{u-RM})/s_{u-PC}$ (%)	α
25	8.4	5.3	3.1	36.9	0.25
50	13.9	9.5	4.4	31.7	0.2
100	24.6	16.4	8.2	25.3	0.2

observed at various overburden pressure intensities. Through systematic analysis, the variation of the PC/RM ratio with respect to the coefficient of consolidation and shear strength for different overburden pressure intensities was determined and compared.

3. Results and discussion

The impact of smearing under specific overburden stress conditions was assessed by averaging the coefficient of consolidation at various effective stresses beyond the designated overburden stress. For instance, to determine the $c_{v_{RM}}$ value at overburden stress of 50 kPa, the sample was subjected to staged loading until reaching the targeted overburden stress of 50 kPa. Following consolidation, the specimen was unloaded, carefully extracted, remolded and then carefully backfilled into the consolidation cell. Despite the initial loading of 50 kPa, during the subsequent reloading phase, effective stress was applied in the range of 12.5 kPa to 200 kPa. The primary focus in this case is on the coefficient of consolidation beyond the effective stress value of 50 kPa, and its average value was subsequently derived. Figure 5 illustrates the plot of c_v and c_h values with overburden stress intensities for both PC and RM specimens. It can be observed from the figure that the c_h values of the PC and RM specimens are higher than the c_v values for the entire range of overburden stresses considered. The figure also shows that the c_h values of both PC and RM specimens exhibit a declining pattern as overburden stress intensifies. The reduction in c_h values of RM specimens in response to the increase in overburden stress is more intense compared to the PC specimens. The plot also indicates that the c_v values of both PC and RM specimens are relatively insensitive to the change in the applied overburden stress. This insensitivity implies that the vertical consolidation characteristics of the samples exhibit a degree of stability across the considered range of overburden stresses.

Figure 6 shows the variation of $c_{v_{PC}}/c_{v_{RM}}$ and $c_{h_{PC}}/c_{h_{RM}}$ with overburden stress. From the graph, it can be observed that both the $c_{v_{PC}}/c_{v_{RM}}$ and $c_{h_{PC}}/c_{h_{RM}}$ ratios increase with the increase in overburden stress. Hence, it can be inferred that the remolding increases with depth. While conflicting findings exist in the literature, the current experimental study confirms the observation reported by Akagi (1981) and Bergado et al. (1993) that the smearing increases with depth. The $c_{v_{PC}}/c_{v_{RM}}$ and $c_{h_{PC}}/c_{h_{RM}}$ at 2.5 m, 4.5 m and 7 m depths were reported by Akagi (1981), who observed that the smearing effect rises with depth.

**Figure 5.** c_v and c_h values at different overburden stress.**Figure 6.** $c_{v_{PC}}/c_{v_{RM}}$ and $c_{h_{PC}}/c_{h_{RM}}$ with the overburden stress.

The undrained shear strength of the PC and RM samples at various overburden stresses are reported in Table 2. The shear strength of the PC sample is higher than the shear strength of the RM sample. Upon remolding, the s_u value of the RM samples was reduced due to the damage of the soil structure caused by mechanical disturbance. It can be noted that the reduction in the percentage of shear strength due to remolding decreases with an increase in overburden stress. The ' α ' value ($\Delta s_u/\Delta \sigma'_v$) of the soil for different overburden stress remains almost constant, and it ranges from 0.22 to 0.25. A similar observation was reported by Remy et al. (2011).

4. Conclusion

In the present study, an experimental analysis of the smearing effect of soil under varying overburden stress was conducted. The observed trends in the $c_{v_{PC}}/c_{v_{RM}}$ and $c_{h_{PC}}/c_{h_{RM}}$ which exhibited an increase with higher overburden stress, strongly suggest a corresponding rise in the remolding

effect with depth. As anticipated, the shear strength of the PC sample is higher than that of the RM sample, underscoring the impact of mechanical disturbance on the undrained shear strength of the latter. This study contributes to a deeper understanding of the consolidation characteristics of soil under different stress conditions and highlights the importance of considering overburden stress in evaluating soil stability and strength characteristics.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Radhakrishna P. Aparna: conceptualization, data curation, visualization, writing – original draft. Retnamony G. Robinson: conceptualization, methodology, supervision, writing – review & editing. Shailesh R. Gandhi: formal analysis, methodology, supervision, writing – review & editing.

Data availability

All data produced or examined in the course of the current study are included in this article.

Declaration of use of generative artificial intelligence

No assistance of Generative Artificial Intelligence (GenAI) has been used for the preparation of this paper.

List of symbols and abbreviations

1-D	One – dimensional
c_h	Coefficient of consolidation under radial drainage
$c_{h(PC)}$	Coefficient of consolidation for preconsolidated samples under radial drainage
$c_{h(RM)}$	Coefficient of consolidation for remolded sample under radial drainage
c_v	Coefficient of consolidation under vertical drainage
$c_{v(PC)}$	Coefficient of consolidation for preconsolidated samples under vertical drainage
$c_{v(RM)}$	Coefficient of consolidation for remolded sample under vertical drainage
r_w	Radius of the drain
s_u	Undrained shear strength
$s_{u_{PC}}$	Undrained shear strength of preconsolidated sample
$s_{u_{RM}}$	Undrained shear strength of remolded sample
ASTM	American society of testing materials
IIT	Indian Institute of Technology
IS	Indian Standard

PC	Preconsolidated
PC/RM	Preconsolidated/Remolded
PVD	Prefabricated vertical drain
RM	Remolded
SCP	Sand compaction pile
VST	Vane shear test
α	Change in undrained shear strength to the change in overburden stress
Δs_u	Change in undrained shear strength
$\Delta \sigma'_v$	Change in overburden stress

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Comparative assessment of analytical methods for calcium carbonate determination in sands

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Technical Note

Keywords

CO₂ capture
Calcium carbonate content
Loss on ignition
Total inorganic carbon
X-Ray diffraction

Abstract

Accurate quantification of calcium carbonate (CaCO₃) in soil is critical for geoenvironmental engineering, influencing soil stabilization, carbon sequestration, and climate-resilient design. This study compares four commonly used analytical methods—Loss on Ignition (LoI), Total Inorganic Carbon (TIC), X-Ray Diffraction (XRD), and a Calcium Carbonate Analyzer (CCA)—over CaCO₃ concentrations from 0.3% to 50%. Controlled sand samples with known carbonate contents were tested to assess each method's accuracy, precision, and reproducibility. Although LoI proves the most practical due to its simplicity and throughput, all methods face marked difficulties at low concentrations. Root Mean Square Error (RMSE) analysis shows that small discrepancies at low CaCO₃ levels can produce disproportionately large relative errors, obscuring actual measurement uncertainties. Factors such as sample representativeness, operator handling, and instrument resolution further amplify inconsistencies, emphasizing the need for more rigorous protocols. Crucially, none of the evaluated methods consistently captures very low CaCO₃ concentrations, posing significant challenges for soil carbon capture initiatives, regulatory compliance, and soil management practices. These findings highlight the urgent need to refine existing procedures or use alternative approaches to more accurately quantify CaCO₃ content at low levels, thereby enhancing confidence in research outcomes and applied engineering solutions.

1. Introduction

Calcium carbonate (CaCO₃) is widely recognized as a cornerstone in geoenvironmental engineering, influencing a range of soil properties including stiffness, strength, and carbon sequestration potential (Li et al., 2023; Washbourne et al., 2012). Over the past decades, CaCO₃ has been central to the design of soil stabilization projects (Dejong et al., 2013; El Mountassir et al., 2018) and to the development of innovative biogeochemical pathways for CO₂ capture (Manning & Renforth, 2013), underscoring its importance in both environmental and geotechnical applications. Despite considerable advancements in understanding CaCO₃ behaviour in soils, accurately quantifying low concentrations remains challenging (Clarkson et al., 2024), potentially introducing errors in carbon accounting, impeding the verification of geoengineering interventions, and complicating regulatory compliance efforts.

Numerous analytical methods have been developed for CaCO₃ determination, including Loss on Ignition (LoI), Total Inorganic Carbon (TIC) via dry combustion, X-Ray Diffraction (XRD), and pressure calcimetry (commonly referred to as the calcimetric method – here referred as Calcium Carbonate Analyser - CCA). Each method offers unique strengths but also faces specific limitations, particularly when CaCO₃ concentrations are low. For instance, CCA is valued for its simplicity and cost-effectiveness, yet it often exhibits reduced precision below certain CaCO₃ thresholds due to organic matter interference and limited instrumental sensitivity (Cantera & Ozán, 2022). XRD enables mineral-specific quantification, but it can produce inaccuracies when overlapping diffraction peaks or water loss in clay minerals obscure the carbonate signature (Xiao et al., 2023). LoI, which is straightforward and relatively inexpensive, may overestimate CaCO₃ in low-carbonate sediments owing to the partial decomposition of non-carbonate minerals at elevated temperatures (Fu et al., 2020).

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Submitted on January 10, 2025; Final Acceptance on May 20, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.000425>



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Although modifications such as calibrating methods with soils spiked at known CaCO_3 levels have been proposed to enhance reproducibility (Heiri et al., 2001; Hoogsteen et al., 2015), these adjustments do not universally resolve the intrinsic shortcomings associated with each technique.

Comprehensive comparisons of these quantification methods, particularly at low CaCO_3 concentrations, remain scarce. Most published investigations concentrate on soils with high carbonate content or assess only a limited subset of approaches (Leogrande et al., 2021; Wang et al., 2012), limiting the broader applicability of their conclusions. This gap in the literature poses challenges for accurately assessing carbon sequestration, designing climate-resilient geoengineering strategies, and advancing sustainable soil management practices. Moreover, most comparative studies rely on real soils (Fu et al., 2020; Leogrande et al., 2021; Wang et al., 2012), where it is impossible to confirm the exact CaCO_3 concentration because each technique introduces an inherent measurement error. This core limitation complicates result validation and highlights the need for standardized methodologies to mitigate inaccuracies. Given that CO_2 capture estimates are highly sensitive to even minor errors, refining analytical procedures for low- CaCO_3 soils remains a pressing priority.

The present study addresses this gap by evaluating the performance of four commonly employed techniques—LoI, TIC, XRD, and CCA - across a range of CaCO_3 concentrations (0.3% to 50%) under controlled conditions. By systematically comparing these methods against known benchmarks, this work examines the respective accuracies, precisions, and limitations of current practices. The findings highlight the impacts of measurement uncertainty on carbon accounting and geoenvironmental applications, with particular emphasis on low- CaCO_3 soils that are prevalent in many engineered and natural environments. The results are intended to guide researchers and practitioners in selecting and refining analytical methodologies that align with contemporary needs in geoenvironmental engineering, including improved soil stabilization designs, robust carbon sequestration initiatives, and refined climate change mitigation strategies.

2. Materials and methods

Samples of pure quartz sand were prepared with varying CaCO_3 contents of 50%, 9.2%, 4.8%, 2%, 1%, and 0.3% by mass. These percentages were specifically chosen to cover a wide practical range, emphasizing lower concentrations due to their analytical challenges and critical knowledge gap identified in the literature. For each concentration, 3 replicates were prepared, except for the XRD analysis, where duplicates were prepared for the 50% and 0.3% samples only. To ensure accurate preparation, 2g of sand were used for most samples, with the appropriate mass of CaCO_3 (ACS reagent, $\geq 99.0\%$, powder) added. For the 0.3% samples, 5g of sand were used to avoid measurement errors due to the small quantity of CaCO_3 required. Before

mixing sand and CaCO_3 , both materials were dried at 40°C (for Total Inorganic Carbon – TIC test) and 105°C (all other tests), then the mix was ground using a pestle and mortar. The prepared samples were subsequently used for testing: approximately 2g for LoI, except for the 0.3% sample where 5g were used; around 0.15g for TIC; around 2g for XRD; and 1g for CCA, with an additional 2g sample (5g for 0.3%) for CCA to evaluate the influence of sample size.

For the LoI method, dried samples were placed in a furnace at 550°C for 5 hours, and then returned to the furnace at 950°C for an additional 3 hours before being weighed, following the procedure outlined by Heiri et al. (2001). To calculate the percentage of CaCO_3 , it was assumed that the sample contained no organic matter and that all carbonates present were calcium carbonate, which is reasonable given the sample composition. The CaCO_3 percentage was calculated by taking the mass difference between the 105°C and 950°C measurements, dividing it by the mass at 105°C . This result was then divided by 0.44, which corresponds to the CO_2 fraction released during the thermal reaction (Dean, 1974).

For TIC, specimens were analysed using a Leco CS-244 Carbon Determinator, calibrated prior to the analysis of samples. The difference between the total carbon and organic carbon was divided by 12 to obtain the CaCO_3 content (Bernard et al., 2022).

In the XRD analysis, samples were backloaded into 16 mm sample holders, and a preliminary scan over a 2θ range of $5\text{--}100^\circ$ was conducted to determine the crystallinity and appropriate scan settings. Phase identification was carried out using HighScore Plus software, with matching sequences performed using the PDF-2 database from the International Centre for Diffraction Data (ICDD) and the Crystallography Open Database (COD). Following phase identification, quantification was conducted via whole-pattern fitting using Rietveld refinement, with manual adjustments as necessary.

For CCA testing, procedures followed ASTM D4373-21 (ASTM, 2021). Two of the triplicated samples were analysed by placing 1g of specimen in a reaction cylinder with 20 mL of 1N HCl. The sealed reactor was shaken to ensure contact between the specimen and the acid, and after a 10-minute reaction, the pressure generated by CO_2 release was recorded. The pressure was then converted to CaCO_3 content using a previously established calibration curve. For the third sample in each set, a larger specimen (2g, or 5g for the 0.3% sample) was placed in the reaction cylinder, and the pressure reading was normalized by dividing the specimen mass before converting to CaCO_3 using the calibration curve.

3. Results and discussions

Figure 1 illustrates the correlation between the real CaCO_3 content and the average measured values obtained from the different analytical methods. As shown in Figure 1a, all methods underestimate the CaCO_3 content for the sample with 50% CaCO_3 . Figure 1b zooms in on the lower concentration

range (0–10% CaCO_3), where greater variability among the methods becomes more apparent. In this range, LoI tends to overestimate the CaCO_3 content, while TIC consistently underestimates it, except for the 0.3% sample. XRD shows overestimation for samples with 9.2%, 4.8%, and 0.3% CaCO_3 , but underestimates the others. CCA, on the other hand, overestimates the CaCO_3 content for the 9.2% and 1% samples, while underestimating the remaining concentrations. Additionally, the standard deviations for the replicated samples highlight that TIC generally have higher variability, while LoI and CCA produce more consistent results, although discrepancies still occur at lower concentrations.

The underestimation and overestimation observed across the methods can be attributed to several key factors. One issue is the representativeness of the specimens relative to the bulk sample. In methods like TIC, where only 0.15g of specimen is used, even slight variations in CaCO_3 distribution can lead to significant measurement discrepancies, especially at lower concentrations. In contrast, sample size might also influence the results, particularly for LoI, as highlighted by Heiri et al. (2001). To test this hypothesis, different amounts of pure CaCO_3 (2, 4, 6, 8, and 10g) were tested using the LoI method, with 100% CaCO_3 expected. However, the results averaged at 93.7%, with values ranging from 92.9% for the 10g sample to 96.6% for the 2g sample (Figure 2a), indicating that sample size might indeed impact the accuracy of the measurements. Conversely, for CCA, larger sample sizes yielded better results, with larger specimens producing more accurate measurements of CaCO_3 content (Figure 2b), suggesting improved representativeness in the CCA method when larger quantities are tested.

Instrument accuracy also contributes to variability, particularly in methods like CCA, where the manometer's coarse resolution (1 kPa per division) makes it challenging to detect small amounts of CO_2 release, leading to less precise results, especially at low CaCO_3 concentrations. Operator error is another significant factor, especially in CCA and LoI, where precision is required in both manometer readings and manual weighing. Even slight discrepancies, such as a 0.5 kPa difference in manometer readings or variations in the third decimal place of the sample weight, can result in notable errors when calculating CaCO_3 percentages.

Additionally, incomplete reactions may contribute to discrepancies, particularly in LoI. To investigate this, a 2g sample of pure CaCO_3 was sent for XRD analysis after LoI (LoI result = 96.6%). XRD confirmed that the sample contained 96.6% CaO and 3.4% CaCO_3 , indicating incomplete reaction. Similarly, a 50% sand and 50% CaCO_3 sample analysed by XRD after LoI (LoI result = 48.5%) was found to contain 38.5% quartz (which should have been 50%), 0.8% CaCO_3 , and 60.7% CaO, further suggesting incomplete decomposition of CaCO_3 and highlighting the limitations of XRD to quantify phases.

Despite these factors, the Root Mean Square Error (RMSE) values for the four methods—LoI, TIC, XRD, and CCA—are 0.778, 2.673, 0.989, and 1.229, respectively. These values indicate the average magnitude of error for each method when compared to the actual CaCO_3 content in the samples. At first glance, LoI and XRD exhibit lower RMSE values, suggesting better performance, while TIC shows the largest RMSE, indicating consistently higher deviations from the true values.

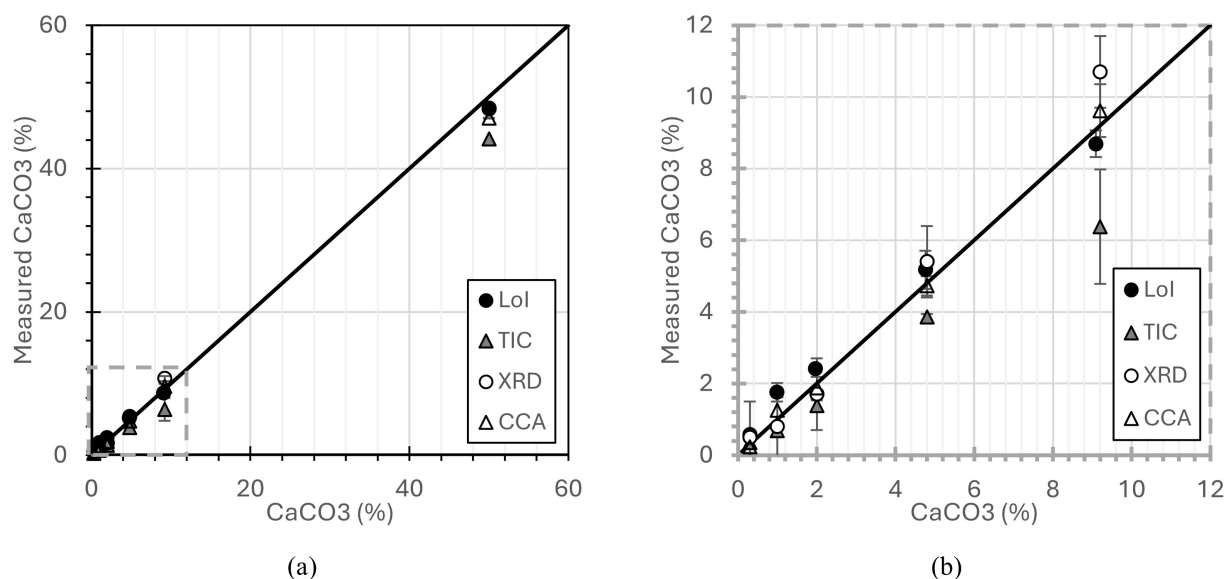


Figure 1. (a) Correlations between calcium carbonate content and average measured calcium carbonate content for different testing methods, (b) zoom on biaxial 10% calcium carbonate content, including standard deviation for replicated samples, where LoI: Loss on Ignition, TIC: Total Inorganic Carbon, XRD: X-Ray Diffraction, CCA: Calcium Carbonate Analyser.

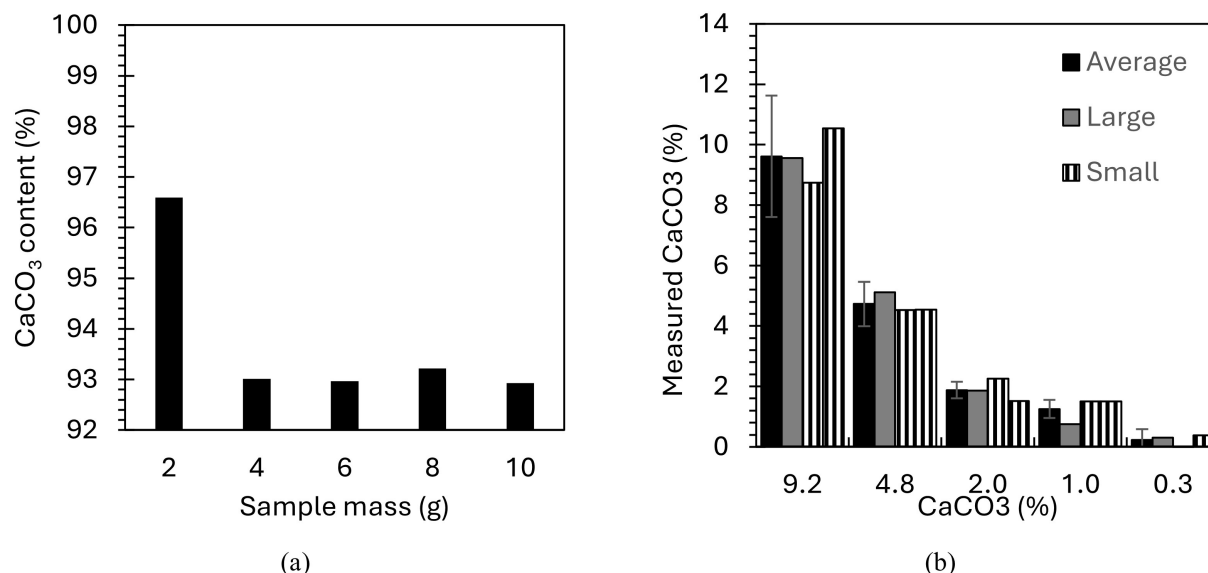


Figure 2. Sample size effect: (a) LoI and (b) CCA, where large refers to samples of 5g for 0.3% of CaCO₃ and 2g for all other concentrations and small for samples of 1g, as recommended by ASTM D4373-21 (ASTM, 2021).

However, interpreting these RMSE values presents certain challenges. For example, when examining the results for LoI at 0.3% and 50% CaCO₃ concentrations, significant differences become apparent. At 50% CaCO₃, the measured values are close to the actual content, with a maximum deviation of only 0.6%, resulting in accurate representation and contributing to the relatively low overall RMSE for LoI. In contrast, for the sample containing 0.3% CaCO₃, the measured values vary significantly, ranging from 0.5% to 0.7%. This creates large relative errors, with the highest measurement being more than double the actual content and even the lowest value reflecting an error of about 70%.

This example highlights the difficulty of interpreting RMSE in isolation. While LoI's overall RMSE of 0.778 appears small and suggests strong performance, this figure is largely influenced by the accurate measurements at higher concentrations, such as the 50% sample. Meanwhile, the large relative errors at lower concentrations, like the 0.3% sample, are masked by the averaging effect inherent in RMSE. Thus, although RMSE is a useful overall measure of error, it may not fully capture variability across different concentration ranges, particularly when small concentrations are critical.

4. Conclusions

This paper compared four commonly used analytical techniques for quantifying calcium carbonate (CaCO₃) in sand samples—Loss on Ignition (LoI), Total Inorganic Carbon (TIC), X-Ray Diffraction (XRD), and a Calcium Carbonate Analyzer (CCA)—with the overarching objective of assessing their accuracy and reliability across a broad range of concentrations. Through a series of controlled experiments spanning CaCO₃ contents from low (0.3%) to

high (50%) levels, the investigation aimed to highlight the strengths and limitations of each method.

The results indicate that while LoI, TIC, XRD, and CCA can reliably estimate moderate-to-high CaCO₃ concentrations, all four methods exhibit notable performance challenges at low concentration levels. Although Root Mean Square Error (RMSE) analyses can reveal overall accuracy trends, small absolute differences at these lower concentrations tend to produce disproportionately large relative errors. Method selection therefore requires careful consideration of factors such as specimen size, operator handling, and instrument precision. Among the techniques tested, LoI emerged as the most advantageous due to its simplicity, minimal equipment requirements, and rapid sample throughput. However, findings confirm that none of the four methods can provide meaningful CaCO₃ measurements in very low concentration regimes, underscoring a persistent challenge for both researchers and practitioners. These insights reinforce the need for continued refinement of existing analytical protocols, particularly in the context of soil carbon sequestration assessments and the development of climate-resilient engineering solutions.

Acknowledgements

This research was supported by the 2023–2024 Fellowship of the Coalition for Disaster Resilient Infrastructure (CDRI – Application ID: 2301061780), ISPF ODA Newcastle University Institutional Fund, and the Brazilian National Council for Scientific and Technological Development (CNPq – Call CNPq/MCTI/FNDCT No. 22/2024, Application ID: 445166/2024-0). The first author would like to thank the PIBIC-UFRR 2023–2024 program for the financial support received through the scientific initiation scholarship.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Isabelle Joanne Viana Coelho Cesar: conceptualization, data curation, formal analysis, investigation, methodology, visualization, writing – original draft. Victor Carlos Vital Cavalcanti: investigation, methodology. Mariana Ramos Chrusciak: funding acquisition, project administration, supervision, validation. Bruna de Carvalho Faria Lima Lopes: conceptualization, formal analysis, funding acquisition, methodology, supervision, validation, writing – original draft.

Data availability

The datasets generated analysed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

ACS	American Chemical Society
ASTM	American Society for Testing and Materials
CaCO ₃	Calcium Carbonate
CCA	Calcium Carbonate Analyzer
CDRI	Coalition for Disaster Resilient Infrastructure
CNPq	Brazilian National Council for Scientific and Technological Development
COD	Crystallography Open Database
FNDCT	Brazilian National Fund for Scientific and Technological Development
HCl	Hydrochloric acid
ICDD	International Centre for Diffraction Data
ISPF ODA	International Science Partnerships Fund - Official Development Assistance
LoI	Loss on Ignition
MCTI	Brazilian Ministry of Science, Technology and Innovation
PIBIC-UFRR	Institutional program for scientific initiation scholarships, Universidade Federal de Roraima
RMSE	Root Mean Square Error
TIC	Total Inorganic Carbon
XRD	X-Ray Diffraction

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CASE STUDIES

Soils and Rocks
v. 48, n. 2

Finite element prediction of long-term behaviour of a test embankment based on back-analysis of early stages

Fernando da Silva Oliveira^{1#} , Francisco de Rezende Lopes² 

Case Study

Keywords

Embankment on soft clay
Settlements
Horizontal displacements
Soft clay
Consolidation
Numerical modelling

Abstract

The paper presents the finite element modeling of a test embankment built at one of the construction sites of the 2016 Olympic Games in Rio de Janeiro, Brazil. The embankment was raised over 6 weeks to a height of 6.7 m, and was monitored during construction and in the following month. The experiment had to end after a month of measurements due to the need to develop the test area. Instrumentation consisted of electric piezometers, settlement plates and inclinometers. The subsoil featured a pack of sedimentary soils, including soft clays. Firstly, field and laboratory tests were evaluated for numerical modelling parameters, and a sensitivity study of the various parameters was carried out. Program PLAXIS was used with the most suitable soil models for the different soil layers. The time-dependent behaviour of the clayey layers followed Terzaghi's consolidation theory for pore pressure dissipation coupled with a Buisman (1936)-type solution for viscous behaviour. The final analysis used parameters adjusted by back-analysis of measurements in the monitoring period (2.5 months) in a 3D model. This analysis was extended to 50 years, aiming to predict the long-term behaviour of the embankment. Results of the 50-year analysis indicated that after 20 years the excess pore pressures would have practically dissipated, but that the long-term settlement would be 2.5 times greater than that at the end of construction. Furthermore, the ground horizontal displacement would be about 4 times greater than that measured at the end of construction.

1. Introduction

An embankment was needed to raise the grade of the construction site of the Athletes' Village of the 2016 Olympic Games in Rio de Janeiro, Brazil. There were concerns about both the long-term settlements of the embankment and the horizontal displacements it could induce on nearby foundations. A test embankment was raised over 6 weeks to a height of 6.7 m and was monitored during construction and in the subsequent month. Due to the construction schedule of the entire project, monitoring could not be extended further. Instrumentation consisted of electric piezometers, settlement plates and inclinometers. The local ground conditions featured a pack of sedimentary soils, including soft clays. Firstly, field and laboratory tests were evaluated for numerical modelling parameters; secondly, a sensitivity study of the various parameters was carried out with a 2D model with program PLAXIS. Third, a 3D model was run with the average parameters, and the most sensitive of them were further adjusted to match measurements in the monitoring period (2.5 months). With these parameters, a

long-term (3D) analysis was performed in order to predict the long-term behaviour of the embankment.

The time-dependent behaviour of clayey soils is related to two phenomena: consolidation (of hydrodynamic nature) and creep (of viscous nature). The first rational approach to the consolidation phenomenon was proposed by Terzaghi (1923), for 1D strains and constant excess pore pressure with depth. In the following years, different authors proposed extensions of this theory, such as 3D conditions, excess pore pressures not constant with depth, coupling pore-pressure dissipation with stresses, large strains etc., but respecting the effective stress principle – i.e., not considering creep. However, a few years after the publication of Terzaghi's theory, experimental results began to reveal that an additional phenomenon existed, the most evident indication being a straight slope of the ε vs. $\log t$ curve – instead of a horizontal asymptote – at the end of the load stages in oedometer tests. This additional phenomenon was termed as “secular effect” (Buisman, 1936) or “secondary time effect” (Gray, 1936) and, in fact, is a manifestation of the viscous nature of soils. The most widespread theory for the prediction of viscous

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Submitted on May 24, 2024; Final Acceptance on January 13, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.003824>



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strains is based on Buisman's formulation, which requires the secondary compression index C_{α} . Other insights and formulations were put forward by Taylor (1942) and Leonards & Girault (1961), to mention just a few, and, more recently, in the works of Martins and colleagues (e.g., Martins, 2023; Oliveira et al., 2023; Santa Maria & Santa Maria, 2018).

PLAXIS solution for time-dependent deformation of clayey layers – Soft Soil Creep model – couples strain due to consolidation (strains associated with pore pressure dissipation) and due to creep (viscous strains). Consolidation strains follow Terzaghi's consolidation theory for pore pressure dissipation and creep strains are derived from a Buisman (1936)-type solution extended to three-dimensional conditions (Stolle et al., 1999). This is a model based on the isotache concept of Suklje (1957), and can be categorized under Hypothesis B of Ladd et al. (1977).

Although some numerical analyses of test embankments have been published (e.g., Huang et al., 2006; Mirjalili et al., 2012; Chai et al., 2013; Muthing et al., 2018), the present paper presents a 3D analysis of an embankment on heterogeneous ground, designed to induce large horizontal displacements, and includes a sensitivity study of the overall behaviour due to the parameters of the clayey layers.

In a case study, it is not possible to predict which contribution, between pore pressure dissipation (consolidation) and viscosity, is the most important, especially with layers at different depths. A sensitivity study such as the one presented here is necessary.

2. The test embankment

2.1 Location and geology

The test embankment was built in the Athletes' Village of the 2016 Olympic Games, in the Jacarepaguá Lowland, Rio de Janeiro. The local subsoil is characterized by a package of Holocene and Pleistocene sediments, originating from continental, sea, and lagoon deposits, consistent with the history of sea level variations. These deposits led to the formation of spits in this region, composed of low-density sands, peat, and soft plastic clays (Maia et al., 1984).

2.2 Geometry and instrumentation

Figures 1 and 2 show the embankment, geotechnical investigations and instrumentation in both plan and cross-section.

Before constructing the embankment, a geogrid was placed over the ground. The embankment had different slopes, with a steeper side reinforced with woven geotextile, in 0.5m thick layers. The purpose of this steeper slope was to induce high horizontal deformations in the ground, as part of a study of the effect of the embankment on nearby

foundations. The material used was a sandy silt, with a natural unit weight of 19.5 kN/m³ after compaction.

The instrumentation installed for monitoring the test embankment, shown in Figures 1 and 2, consisted of: (a) 3 settlement plates at the base of the embankment; (b) 4 electric vibrating wire piezometers installed in 4 verticals (totaling 16 piezometers) under the embankment and around it; (c) 2 inclinometer tubes.

3. Geotechnical investigation and geomechanical model of the ground

Field tests (SPT, CPTu, vane) and laboratory tests (oedometer, UU, CIU, CAU triaxials) were performed. Deformed samples extracted from the SPT sampler tip were subjected to lab tests for natural water content, fines content (passing through the No. 200 sieve), LL and PL . The location of the investigations can be seen in Figure 1.

The definition of stratigraphy – or geomechanical model of the ground – followed: (1) analysis of field and laboratory tests; (2) division of the subsoil into layers based on the results of these tests; (3) evaluation of the quality of the test results, where the good quality ones were selected to estimate the necessary parameters for the chosen soil behaviour models.

Based on the results of SPTs and CPTus, the subsoil was divided into 9 layers, up to the bedrock. Figure 3 shows the division of the profile based on SPT-03, while Figure 4 indicates the division based on Robertson's (1990) normalized parameters Q_p , F_r and B_q from CPTu-01, and also S_u values from vane test VT-01, corrected for field conditions according to Bjerrum (1973).

The average points corresponding to the pairs (Q_p, F_r) and (Q_p, B_q) of each layer, obtained in CPTu-01, were taken to Robertson's (1990) charts for soil behaviour classification. Overall, this classification was in good agreement with the tactile-visual descriptions of the SPT samples. However, Layer 3 of peat could not be distinguished from Layer 4 of silty clay by CPTu-01. Its delimitation and classification as "peat" came from the tactile-visual descriptions of the SPT samples. Although the water content of Layer 3, around 100%, is low when compared to that of superficial peat in the Jacarepaguá Lowland, which can exceed 600% (Baroni, 2010), this difference can be explained by the compression of the peat caused by the 4m thick layer of sand overlying it, existing in the location for centuries according to the geological history of the region (Roncarati & Neves, 1976).

Figure 5 presents two sections, one across and the other along the axis of the embankment, showing the adopted division into layers.

As seen in Figure 1, the available geotechnical investigations are concentrated in the central region of the embankment and in front of its steepest slope, not covering its entire plan projection. As shown in the sections of Figure 5, in the space between the geotechnical investigations an interpolation of the geometry of the layers was carried

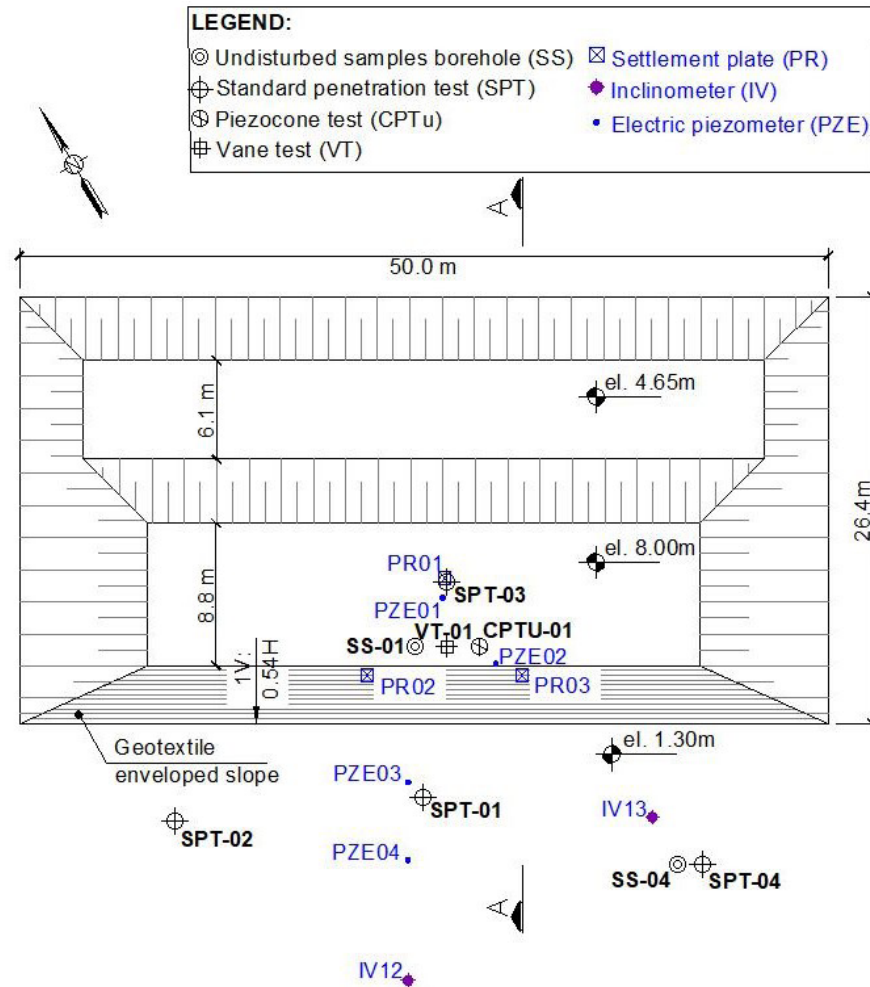


Figure 1. Plan view of the test embankment with investigations and instrumentation.

out, and externally a horizontal extrapolation was made as detected in the extreme investigation holes. On the one hand, the area of the natural ground subjected to greatest stress increases and where the instrumentation was installed is well defined by the geotechnical investigations. On the other hand, 4 CPTu profiles distant between 60m and 240m from the embankment indicate some heterogeneity of the ground, although in most of them soft clay layers (4, 5 and 7) and sand layer 6 were detected. Therefore, some heterogeneities where geotechnical investigations were extrapolated may occur, mainly variations in the location of free-draining layers, influencing the rate of pore pressure dissipation and settlements; the final settlements would be less affected, given the very similar thicknesses of the compressible layers. Horizontal displacements tend to be more affected than settlements by these possible inaccuracies.

To generate the volumes that model the soil layers in PLAXIS 3D, the option of providing the program with separation surfaces between the layers was used. Each surface was generated in the AutoCad Civil 3D software,

with interpolation between points corresponding to the layer divisions of the cross and longitudinal sections shown in Figure 5. Some adjustments were necessary, through the addition of more interpolation points, in order to obtain a more realistic surface from a geotechnical viewpoint.

4. Soil models and parameters

For each soil layer, the most appropriate constitutive model was chosen, and a range of most likely values was defined for the necessary parameters. These ranges were defined after interpretation of field and laboratory tests and, in their absence, from correlations in the literature. Sensitivity studies were carried out, varying the parameters to the bounds of these ranges.

4.1 Soft clay layers

For the soft clay layers, the Soft Soil Creep model – SSC (described, for instance, in Stolle et al., 1999) was chosen

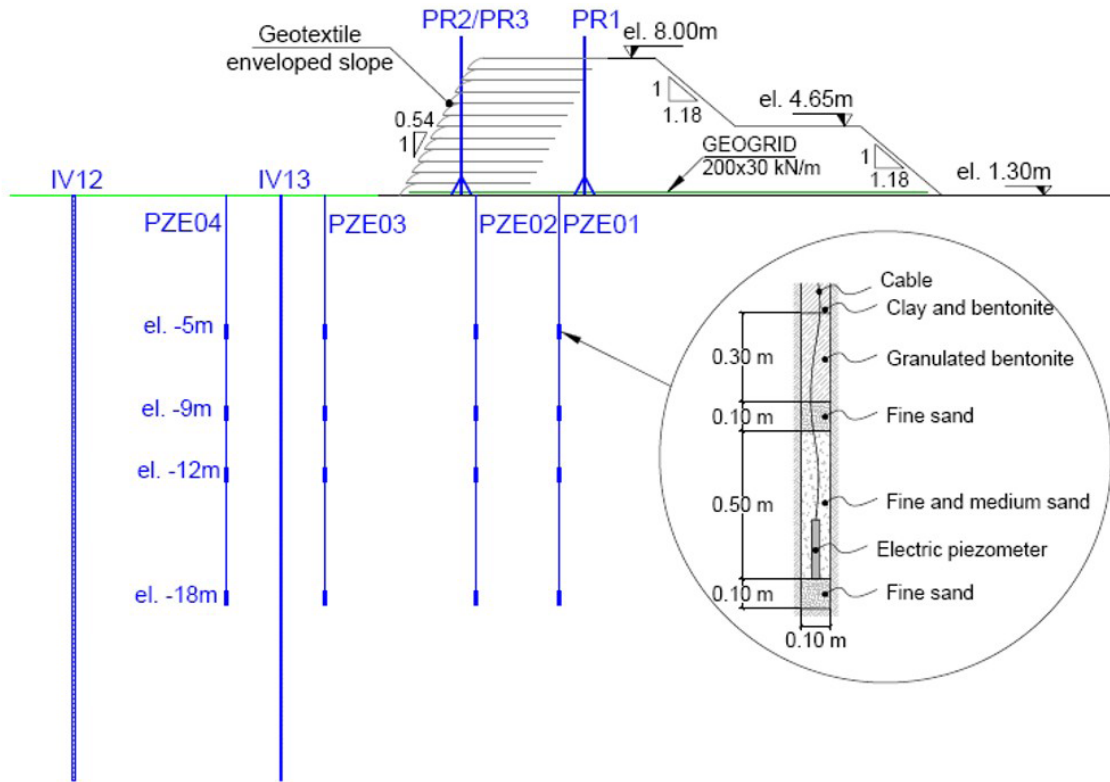


Figure 2. Cross-section AA indicated in Figure 1.

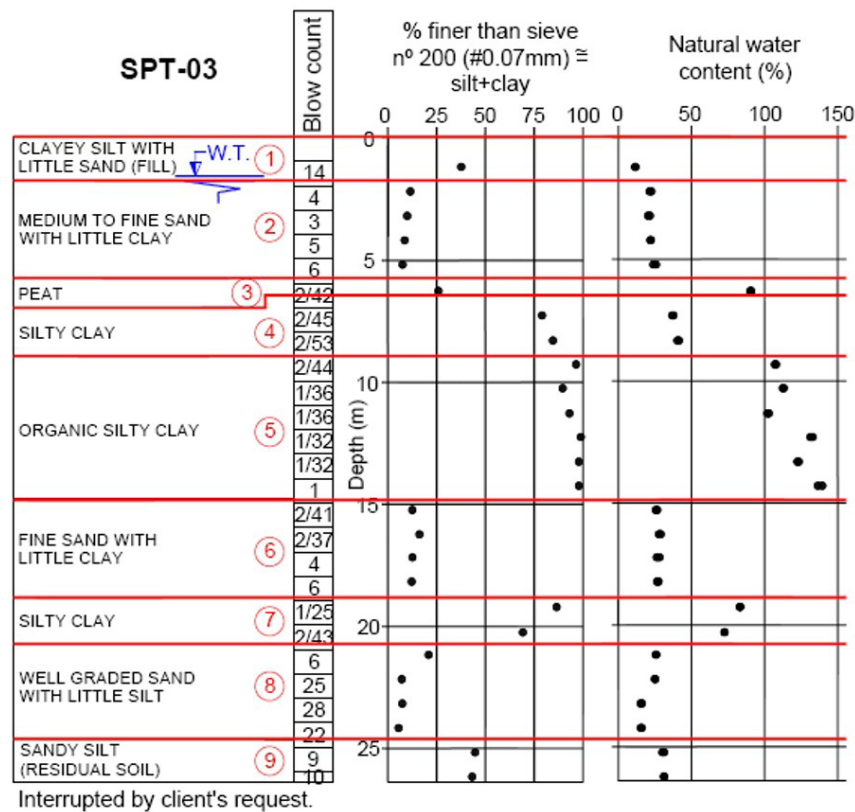


Figure 3. SPT-03 profile with indication of main soil layers.

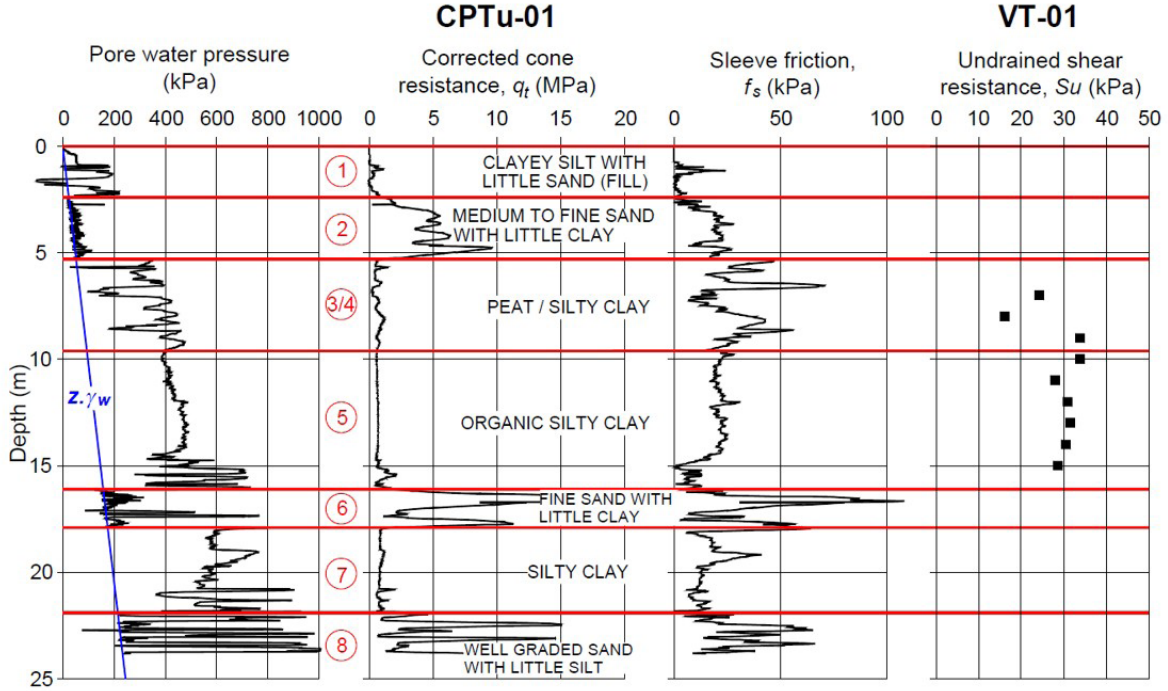


Figure 4. CPTu-01 and VT-01 results with indication of main soil layers.

as it allows the analysis of both primary and secondary consolidation (creep), important aspects in the present case.

Initially, the available oedometer tests were evaluated regarding the quality of the samples, based on the proposals of Lunne et al. (1997) and Coutinho (2007). Of the 16 available tests, 6 were classified as produced by good quality samples, all from Layer 5, with their curves e vs. σ'_v shown in Figure 6. The curves are similar, indicating very close values for C_c and C_s , which were assigned to Layer 5.

For the other clay layers, C_c was estimated from the correlation with the natural water content, w_{nat} , for soft clays of the Brazilian coastal region presented by Silva (2013), shown in Figure 7. In this figure, points from the 6 tests of the Athletes' Village fit well with local data by Baroni & Almeida (2017) and Almeida et al. (2013). For the definition of C_s , the same average value for C_s/C_c of 0.08, obtained in the Layer 5 tests, was considered.

The oedometer tests and the geological history of the region indicate that the soft clay layers present overconsolidation due to aging, and an OCR of 1.6, typical of the local soft clays (e.g., Baroni & Almeida, 2017), was adopted.

For the value of ν_{ur} , related to the elastic behaviour in the unloading/reloading domain, there is little information in the literature (Kulhawy & Mayne, 1990), and the default value of PLAXIS equal to 0.15 was adopted.

The SSC model requires the use of the critical state friction angle ϕ'_{cv} . Triaxial test results on a normally-consolidated specimen that reached the critical state indicated $\phi'_{cv} = 32^\circ$, which was adopted for all clayey layers. As the clays are only slightly overconsolidated, $c' = 0$ was adopted.

As for the dilatancy angle ψ , for slightly overconsolidated soils, it can be considered equal to 0° .

For the earth pressure coefficient at rest, Jaky's (1944) expression for normally-consolidated soils was used, which, for the friction angle of 32° , led to $K_0^{nc} = 0.47$.

The initial values of the permeability coefficients in the three coordinated directions are necessary, in addition to the coefficient c_k , which defines their variation with the void ratio. For Layer 5, the values of $k_{z,init} = 1.0 \times 10^{-9}$ m/s and $c_k = 1.80$ ($e_{init} = 3.20$) were defined in order to have a reasonable adjustment to the c_v values obtained in the oedometer tests shown in Figure 8 for the expected σ'_v range (~ 50 to 220 kPa).

To estimate k in the horizontal direction, CPTu dissipation tests were used. In Layer 5, the average c_h value estimated using the method of Houlsby & Teh (1988) was 8.8×10^{-7} m²/s. Danziger & Schnaid (2000) suggest that the c_h value obtained through this methodology corresponds to the material in the recompression domain. When comparing this average value of c_h with the average value of c_v at recompression in oedometer tests, a ratio $c_h/c_v = 2.2$ is obtained, indicating $k_x = k_y = 2.2 k_z$.

For the other clayey materials, the k and c_k values were estimated based on the mean c_h values of the piezocone dissipation tests and on the following approximate relationships obtained for Layer 5:

$$c_v \cong c_h / 2.2 \quad (1)$$

$$c_{v, recompr} \cong 13 c_{v, virgin} \quad (2)$$

$$c_k \cong 0.6 C_c \quad (3)$$

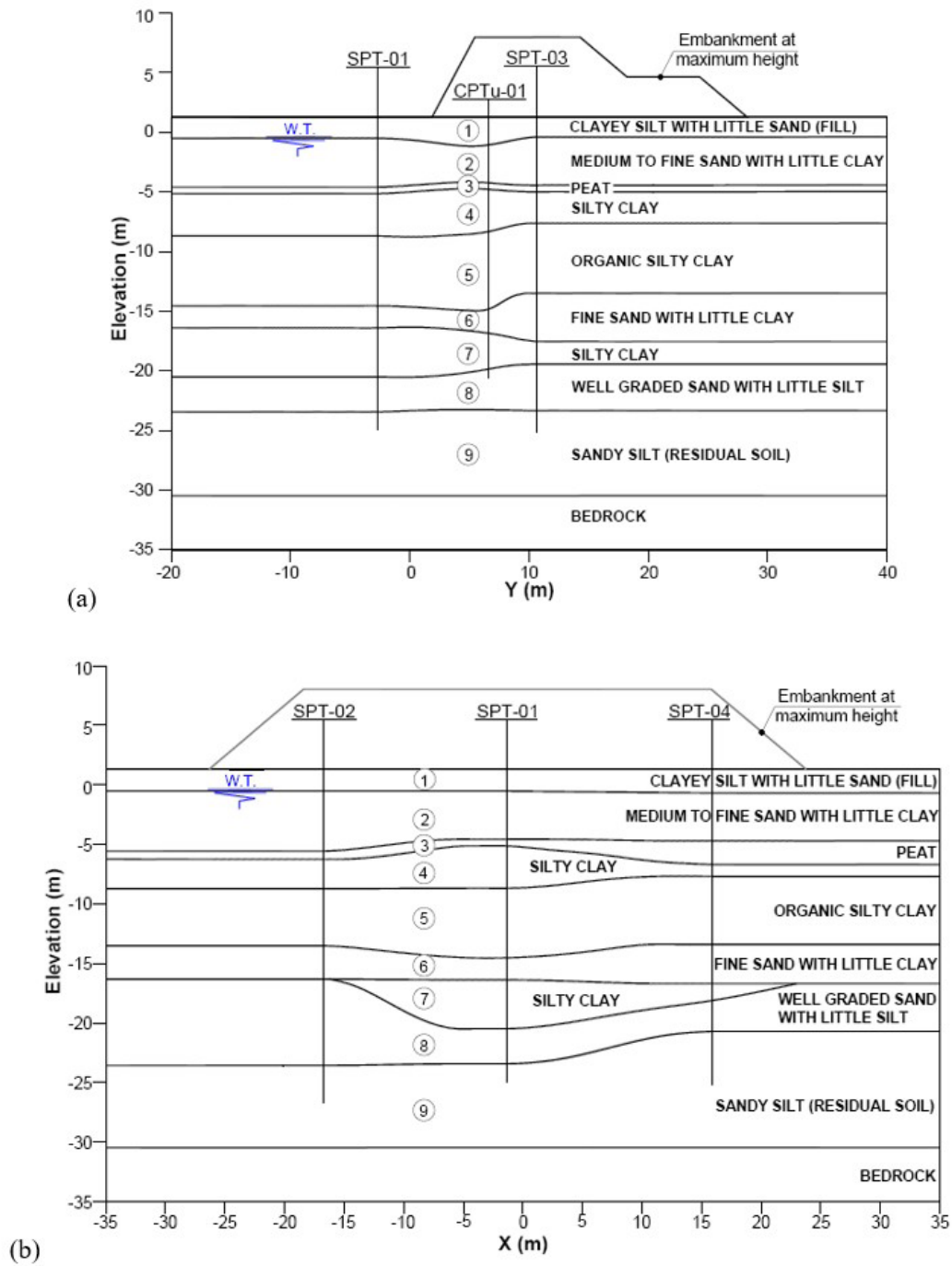


Figure 5. Ground profiles: (a) cross section and (b) longitudinal section.

The modified creep index μ^* can be obtained from the secondary consolidation (creep) index C_α with equation:

$$\mu^* = C_\alpha / \left[(1 + e_{init}) \ln 10 \right] \quad (4)$$

A C_α/C_c ratio equal to 0.05 was adopted for all clayey materials based on the results of oedometer tests and on typical values presented by Terzaghi et al. (1996).

Table 1 shows, for the clayey layers, the ranges of the parameters of the Soft Soil Creep model.

PLAXIS 3D has the SoilTest tool that allows the simulation of some laboratory tests with the specified parameters. Oedometer tests were simulated to check the e vs. $\log \sigma'_v$ curves, as exemplified in Figure 9a for Layer 5. It was observed that the simulated C_c values

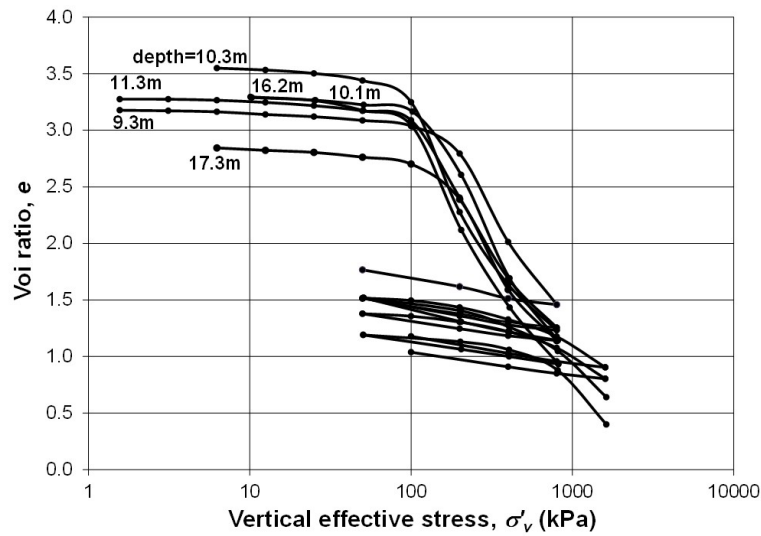


Figure 6. Oedometer compression curves of 6 good quality samples collected from Layer 5.

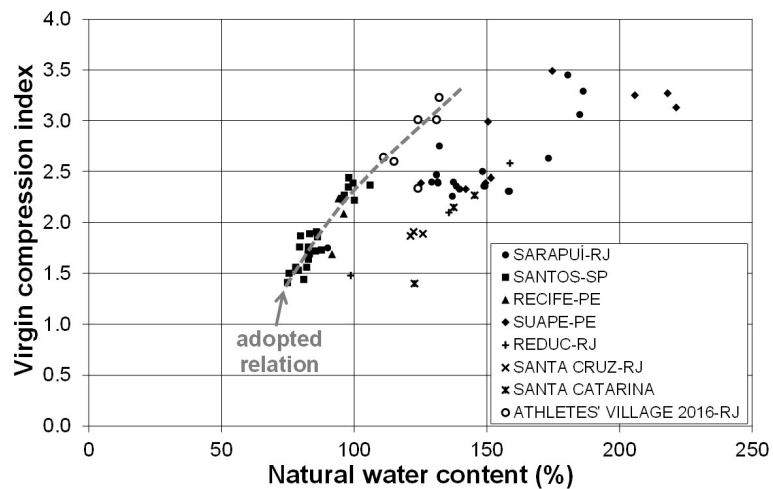


Figure 7. C_c vs. w_{nat} relation for high quality soft clay samples from the Brazilian coast (adapted from Silva, 2013).

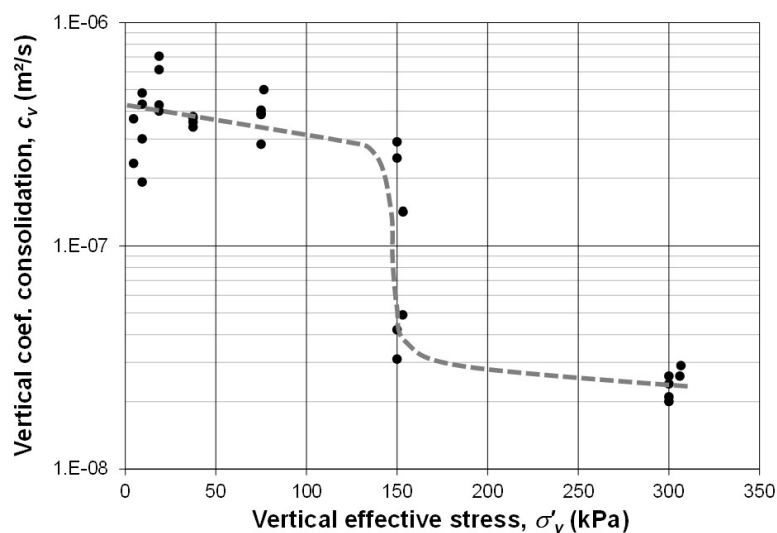


Figure 8. c_v vs. σ'_v data from oedometer tests on the best quality soft clay samples collected from Layer 5.

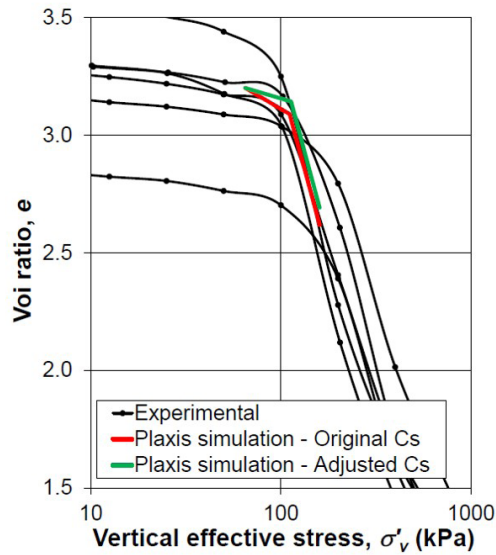
were consistent while simulated C_s values were double the specified. This can be explained by the fact that the conversion from C_s to κ^* , which is the parameter effectively used by the program, is not exact. For the program to consider the correct C_s value, it was necessary to specify half of the values in Table 1.

The results of the triaxial tests – in which the critical state was reached – were compared with those simulated with the program, as exemplified in Figure 9b, with the following results:

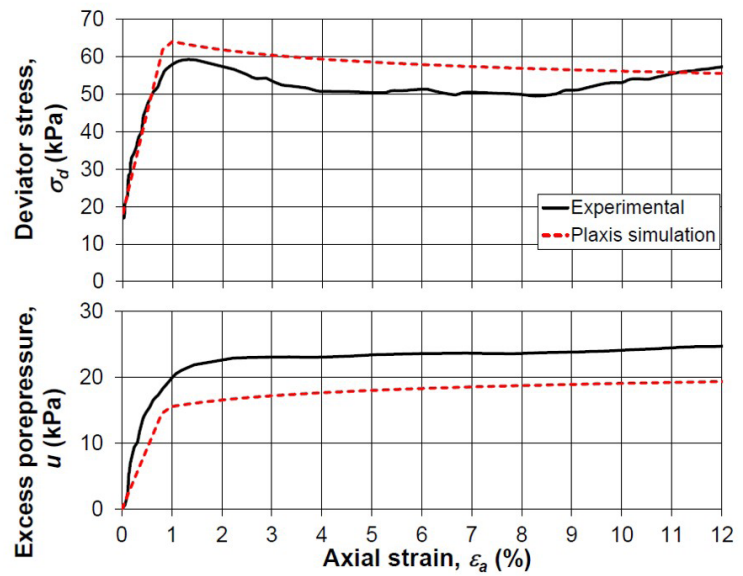
- The σ_d vs. ε_a and σ'_1/σ'_3 vs. ε_a simulated curves matched reasonably well with measurements; however, for

Table 1. Parameters of the Soft Soil Creep model for the clayey materials.

Parameter	Layer 3			Layer 4			Layer 5			Layer 7		
	min.	aver.	max.	min.	aver.	max.	min.	aver.	max.	min.	aver.	max.
γ (kN/m ³)		14.2			15.9			13.5			14.9	
e_{init}		2.6			1.6			3.2			2.1	
C_c	2.10	2.35	2.60	1.20	1.50	1.70	2.70	3.00	3.20	1.30	1.60	1.90
C_s	0.17	0.19	0.21	0.10	0.12	0.14	0.22	0.24	0.26	0.10	0.13	0.15
OCR	1.3	1.6	1.9	1.3	1.6	1.9	1.3	1.6	1.9	1.3	1.6	1.9
v_{ur}	0.05	0.15	0.25	0.05	0.15	0.25	0.05	0.15	0.25	0.05	0.15	0.25
c' (kPa)		0			0			0			0	
ϕ'_{cv}	24°	32°	40°	24°	32°	40°	24°	32°	40°	24°	32°	40°
ψ		0°			0°			0°			0°	
K_0^{nc}	0.36	0.47	0.59	0.36	0.47	0.59	0.36	0.47	0.59	0.36	0.47	0.59
$k_x = k_y$ (10 ⁻⁹ m/s)	9.3	37.4	112.2	6.6	26.4	79.2	0.5	2.2	6.6	0.3	1.2	3.6
k_z (10 ⁻⁹ m/s)	4.2	17.0	51.0	3.0	12.0	36.0	0.2	1.0	3.0	0.1	0.6	1.8
c_k	0.19	1.41	2.35	0.12	0.90	1.50	0.24	1.80	3.00	0.13	0.96	1.60
C_α	0.12	0.14	0.16	0.06	0.08	0.09	0.12	0.15	0.18	0.06	0.08	0.10



(a)



(b)

Figure 9. Comparison between experimental and simulated results using the PLAXIS 3D SoilTest tool for Layer 5: (a) oedometer test and (b) CAU triaxial test.

some tests, they did not produce the peak observed in the measurements;

- The excess pore pressures simulated were underestimated by around 20%.

4.2 Sandy and silty layers

For the sandy and silty layers, the *Hardening Soil* model - HS (described, for instance, in Schanz et al., 1999) was chosen. Although not all parameters could be obtained directly from the appropriate tests, the HS model was considered the most suitable due to its consideration of the change in soil stiffness with the stress level and modeling of the loading history and also to the availability of correlations and typical values in the literature.

Layer 1 is a silty clay, compacted fill, classified as SM-SC by the Unified Soil Classification System, with an average N_{SPT} value equal to 15, in which no field tests were carried out. Its parameters were estimated after the table of typical hyperbolic parameters for compacted Brazilian soils of Ehrlich & Becker (2010). For Layers 2, 6 and 8 the parameters were estimated mainly from correlations for granular soils with results of CPTu-01.

Layer 9 is a silty residual soil and was only reached by the SPT. After an estimate of q_c values based on the correlations between SPT and CPTu results by Danziger & Velloso (1986), the same correlations of Layers 2, 6 and 8 were used.

For the HS model, the peak ϕ' is specified. The correlation of ϕ' with q_c by Robertson & Campanella (1983) was used. A cohesion intercept $c' = 0$ was adopted for all materials. As for the dilation angle ψ , the following expression suggested by PLAXIS was adopted

$$\psi = \phi - 30^\circ \quad (5)$$

The proposals of Kulhawy & Mayne (1990) and Eslaamizaad & Robertson (1996) were used to estimate the tangent modulus E_{oed} for the initial vertical effective stress, σ'_{v0} , from the cone resistance q_c . Subsequently they were converted to the values of E_{oed}^{ref} relative to σ'_v equal $p_{ref} = 100$ kPa with an expression of the HS model.

Regarding the parameter R_p , which varies in a relatively narrow range, in the absence of CD triaxial tests, the typical value of 0.9 was adopted for all layers.

Although the original hyperbolic model uses the initial tangent modulus E_p for PLAXIS, the equivalent parameter is E_{50} . These two parameters are related by the following equation:

$$E_{50} = E_i \left(1 - \frac{R_f}{2} \right) \quad (6)$$

The relation of E_i to the confining stress (in the test) σ_3 follows (Janbu, 1963):

$$E_i = K P_a \left(\frac{\sigma_3}{P_a} \right)^n \quad (7)$$

The K values were estimated through correlation with the friction angle proposed by Kulhawy & Mayne (1990). It was observed that the values of E_{50}^{ref} thus estimated and E_{oed}^{ref} had approximately the following relationship between them: $E_{50}^{ref} = 1.1 E_{oed}^{ref}$, which agrees to the recommendation of PLAXIS to use $E_{50}^{ref} \sim E_{oed}^{ref}$.

As for the E_{ur}^{ref} modulus, PLAXIS manual suggests $E_{ur}^{ref} = 3 E_{50}^{ref}$. Duncan & Chang (1970) state that the K_{ur}/K ratio ranges from 1.2 to 3.0, which, for $R_f = 0.9$, corresponds to the ratio $E_{ur}^{ref}/E_{50}^{ref}$ ranging between 2.2 and 5.5. Based on these two indications, it was adopted for all layers: $E_{ur}^{ref} = 3.5 E_{50}^{ref}$.

Regarding the exponent m , which defines the dependence of stiffness (E_{oed} , E_{50} , E_{ur}) in relation to the confining stress level, the typical value for sands 0.5, was adopted for all layers in the absence of laboratory tests.

In the absence of laboratory tests, $\nu_{ur} = 0.2$ was adopted for all layers, following suggestion in the PLAXIS manual.

For the coefficient of earth pressure at rest in the normally consolidated condition, K_0^{nc} , the Jaky's expression (Jaky, 1944) was used. All silty and sandy materials were considered normally consolidated, except for Layer 1, which is a compacted material.

The mean values of k were estimated from the grain size distribution, according to Cedergren (1967) and Robertson (1990).

Table 2 shows, for the sandy and silty layers, the ranges of the parameters of the Hardening Soil model.

4.3 Embankment

The focus of the analyses is on the subsoil's response to the applied surcharge, with no interest in the behaviour of the embankment body itself. Therefore, the simplest option would be to model the embankment as a distributed load applied to the ground surface, with the load value proportional to the embankment height at each point. However, the more rigid and resistant the embankment is in relation to the subsoil, the more the load distribution deviates from this pattern, tending to concentrate stresses in some regions. A more realistic option would be to separately model the compacted soil as an elastoplastic material, and the reinforcement elements (geotextiles and geogrids) using a specific model provided by PLAXIS 3D. The disadvantage of this option would be the likely occurrence of localized soil failures, which would make numerical convergence of the entire model difficult. Considering that these localized failures would not represent a general failure of the embankment due to the presence of reinforcement elements, it was considered an adequate option to model the embankment, together with its geotextiles and geogrid reinforcements, as an equivalent

linear elastic material, with the parameters in Table 3. This approximation was considered satisfactory, bearing in mind the uncertainties of the stiffness and strength parameters of the compacted soil, on which no specific tests were available.

5. Parameter sensitivity studies with 2D model

PLAXIS 2D was used in the sensitivity studies due to its much shorter computation time compared to PLAXIS 3D (about 100 times less). In these studies, one parameter was varied at a time, the other parameters maintaining the average values of Tables 1 and 2. The parameter evaluated in an analysis was modified in all layers that had the same soil model. Two analyzes were performed for each parameter, one with the minimum, and the other with the maximum value of the range.

Figure 10 shows typical results of the sensitivity studies in the form of tornado-type graphs. They show the degree of relative influence that each parameter has on a given analysis result, ordered from top to bottom, from the most influential to the least influential, which is quantified by the length of the variation range of the specific result. The parameters in black correspond to layers with SSC model and the ones in

red, to layers with HS model. The empty circles correspond to the lower value of the parameter range.

It can be observed that the parameters of layers with the HS model have little influence on the results when compared to those of the layers with the SSC model. The SSC parameters that most influence the results are, in order of importance:

- settlement at the end of monitoring: k_{init}^{init} , c_k , OCR
- horizontal displacements at the end of monitoring: OCR , K_0^{nc} , k_{init} , c_k , v_{ur}
- excess pore pressures: OCR , k_{init} , C_a , c_k

In a similar sensitivity analysis, Muthing et al. (2018) verified the dominant influence of OCR on settlements. Here, the greatest influence of the permeability parameters is due to the fact that the settlements were compared at the end of the monitoring period, when not all excess pore pressures had dissipated, and not at the end of the consolidation process, when the influence of permeability would certainly be much less.

6. 3D modeling of the embankment

Figure 11 shows the 3D model created in PLAXIS 3D, with the embankment at its maximum height. The ground

Table 2. Parameters of the Hardening Soil model for the silty and sandy layers.

Parameter	Layer 1			Layer 2			Layer 6			Layer 8			Layer 9		
	min.	aver.	max.	min.	aver.	max.	min.	aver.	max.	min.	aver.	max.	min.	aver.	max.
γ (kN/m ³)		20.0			19.6			19.3			19.2			19.0	
e_{init}		0.50			0.60			0.60			0.68			0.71	
E_{oed}^{ref} (MPa)	30	40	50	35	50	60	30	45	55	20	30	40	30	40	55
E_{50}^{ref} (MPa)	33	44	55	39	55	66	33	50	61	22	33	44	33	44	61
E_{ur}^{ref} (MPa)	116	154	193	135	193	231	116	173	212	77	116	154	116	154	212
m	0.25	0.50	0.75	0.25	0.50	0.75	0.25	0.50	0.75	0.25	0.50	0.75	0.25	0.50	0.75
R_f	0.70	0.90	0.98	0.70	0.90	0.98	0.70	0.90	0.98	0.70	0.90	0.98	0.70	0.90	0.98
OCR		2.0			1.0			1.0			1.0			1.0	
v_{ur}	0.10	0.20	0.30	0.10	0.20	0.30	0.10	0.20	0.30	0.10	0.20	0.30	0.10	0.20	0.30
c' (kPa)	20	30	40		0			0			0			0	
ϕ'	33°	36°	39°	39°	42°	44°	32°	38°	42°	27°	32°	37°	32°	35°	40°
ψ	0°	6°	9°	0°	12°	14°	0°	8°	12°	0°	2°	7°	0°	5°	10°
K_0^{nc}	0.34	0.41	0.48	0.28	0.33	0.40	0.31	0.38	0.50	0.37	0.47	0.58	0.33	0.43	0.50
k (10 ⁻⁶ m/s)	0.2	2	20	5	50	500	1	10	100	50	500	5000	1	10	100
c_k		10 ¹⁵			10 ¹⁵			10 ¹⁵			10 ¹⁵			10 ¹⁵	

Table 3. Parameters for the embankment.

Parameter	Value	Source
γ (kN/m ³)	19.6	Average of in-situ tests
E' (MPa)	15	Estimated, considering little compaction control
v'	0.3	Estimated
k (cm/s)	1.0 x 10 ⁻³	Estimated, considering a sandy soil

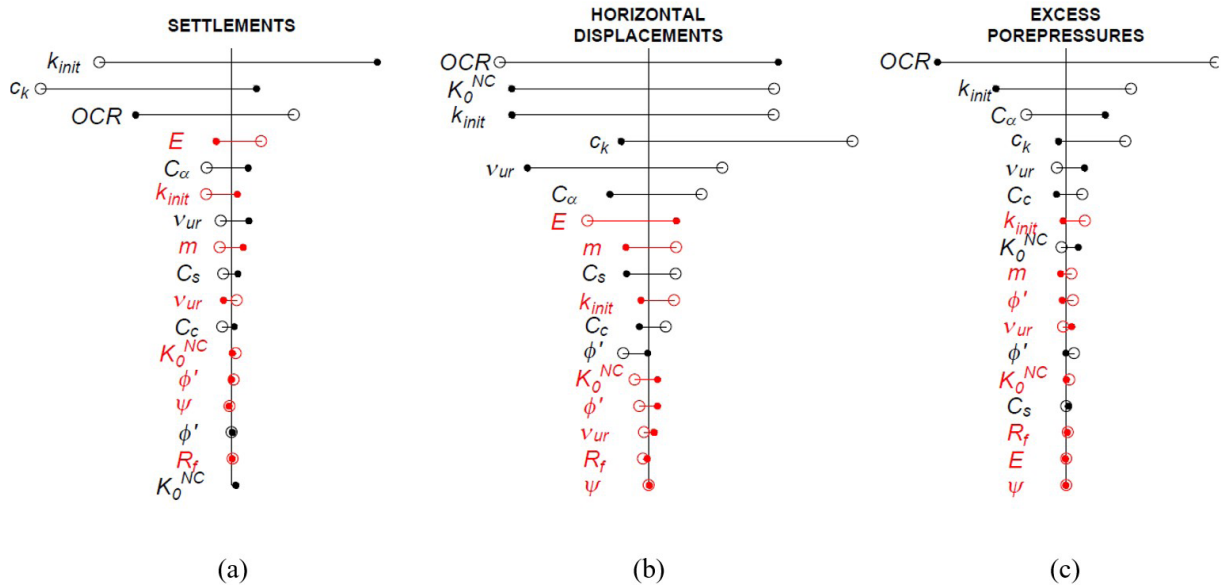


Figure 10. Sensitivity analysis: (a) settlements, (b) maximum horizontal displacements at the end of monitoring, and (c) excess pore pressures in Layer 5 at the end of embankment construction.

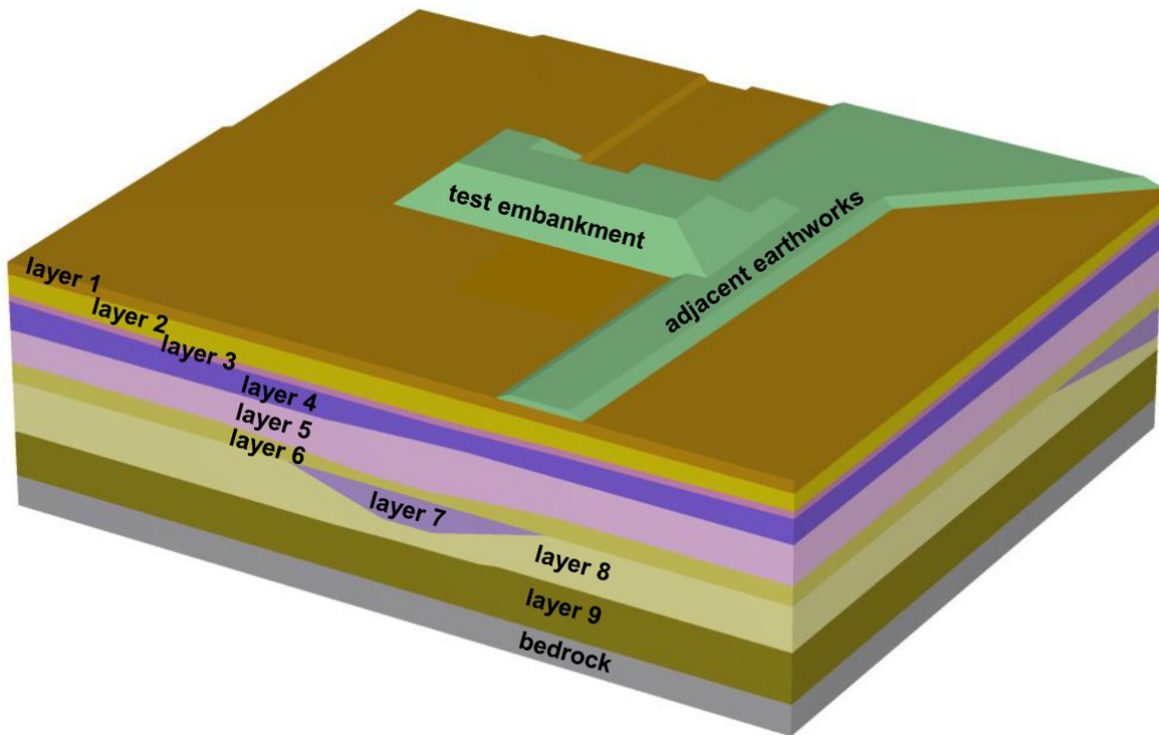


Figure 11. PLAXIS 3D model with the embankment at its maximum height.

was modeled with 120 x 100m plan dimensions, 36m deep. These dimensions were considered to be large enough to avoid the influence of prescribed boundary conditions (e.g., the maximum increase in vertical stress at boundaries is about 2% of the applied surface pressure).

The finite element mesh was generated by the software, resulting in the division of domain into 94630 ten-node tetrahedral elements, with average size around 2.3m. The prescribed boundary conditions were: on the four vertical faces, perpendicular displacements were prevented,

and pore water could flow across; for the bottom, the displacements in the three directions were prevented and water flow was null.

6.1 Analysis sequence

Initial stresses were calculated through the PLAXIS K0-Procedure, based on the prescribed values of K_0^{nc} , OCR , and unit weight of each layer, considering the configuration that the ground presented before the construction of the Athletes' Village, which is presumed to have been the condition in which the site was for centuries, according to the history of the region.

For a more realistic simulation of the ground effective stresses states at the beginning of the test embankment, the modeling also included past earthworks carried out at the site and surrounding areas, which began approximately 16 months before the first raising of the embankment. These past stages were simulated using Consolidation type analyses, since the time elapsed until the beginning of the test embankment was not sufficient for the complete dissipation of excess pore pressures in the clayey layers, making it not possible to use a Drained type analysis.

The placing of layers of the test embankment were spaced from 2 to 5 days. These time intervals were not small enough to consider an undrained behaviour of the clay layers, which was confirmed by piezometers readings. Therefore, the process of raising the embankment was modeled in a Consolidation type analysis.

Since the number of readings on the inclinometers is very large (53 readings), comparisons between measured and computed horizontal displacements were made only for the following four dates/events:

- 04.Jul.2013: 4.5 days after the embankment reached 3.1 m;
- 15.Jul.2013: 5.5 days after the embankment reached 4.7 m;
- 31.Jul.2013: immediately after the embankment reached 6.7 m;
- 02.Sep.2013: last measurements.

In addition to raising the test embankment itself, other earthworks were carried out in its vicinity during the monitoring period, which were also considered in the model for more realistic results.

6.2 Final adjustment of parameters

After a 3D analysis with the average soil parameters, analyzes were performed again varying the parameters of the clay layers (the more influential parameters by the sensitivity studies). The analyses sought to improve matching of settlements measured by the settlement plates, which were underestimated, as well the horizontal displacements measured by inclinometers, which were overestimated.

In soft clays, the relationship between horizontal displacements and settlements in a 2D or 3D problem is strongly influenced by drainage conditions during load application: predominantly undrained loadings tend to generate greater horizontal displacements for a given value of settlement (due to the incompressibility of the soil in this condition) than drained loadings. Undrained displacements are directly derived from loading but may be increased by undrained creep, which is more significant the lower the factor of safety (Foott & Ladd, 1981). This behaviour can be observed in the sensitivity analysis presented in Figure 10b, where the lowest hydraulic conductivity value tested is associated with the greatest horizontal displacement. Therefore, the hydraulic conductivity of the clayey layers was the parameter that underwent the greatest adjustment. Furthermore, the pore pressure measurements (by piezometers) served to adjust parameters related to the rate of excess pore pressure dissipation.

The parameter adjustment was carried out by trying different values, guided by the sensitivity analyses, not using more advanced back-analysis techniques, as performed, for example, by Muthing et al. (2018).

Figures 12 to 14 show comparisons between measurements and analysis results, the initial analysis with average parameters in gray lines and final adjusted analysis in black lines. Due to similar behaviours, only the results of one settlement plate and one piezometer are presented.

The following observations can be made about the adjusted analyses:

- 1) Field measurements showed that the 3 settlement plates, despite being in different positions under the embankment, presented quite similar settlements (46, 45 and 44 cm on the last day of monitoring). The results of the adjusted analysis showed less uniform values, with settlements of 49, 39 and 41 cm on the last day. Nevertheless, for PR-1, central plate, the adjustment was quite satisfactory, with the difference between the calculated and the measured settlement of a maximum of +1 cm until 13.Aug.2013 and of +3 cm on the last day of monitoring (difference of + 6.5%). For PR-2 and PR-3, settlements were underestimated by the analysis.
- 2) The adjustment of the ground horizontal displacement profile was reasonable, since most profiles were within the uncertainty range of inclinometer measurements. For ground above Layer 7, there was no pattern in the deviations of the analysis in relation to the measurements: for certain depths/dates, horizontal displacements were overestimated and, for others, underestimated. From Layer 7 downwards, there was a general tendency to underestimate horizontal displacements, especially in the last monitoring dates.
- 3) As for excess pore pressures, it was observed that in absolute terms the results of the analysis were greater than those measured. In relative terms, it was observed that the difference between the computed

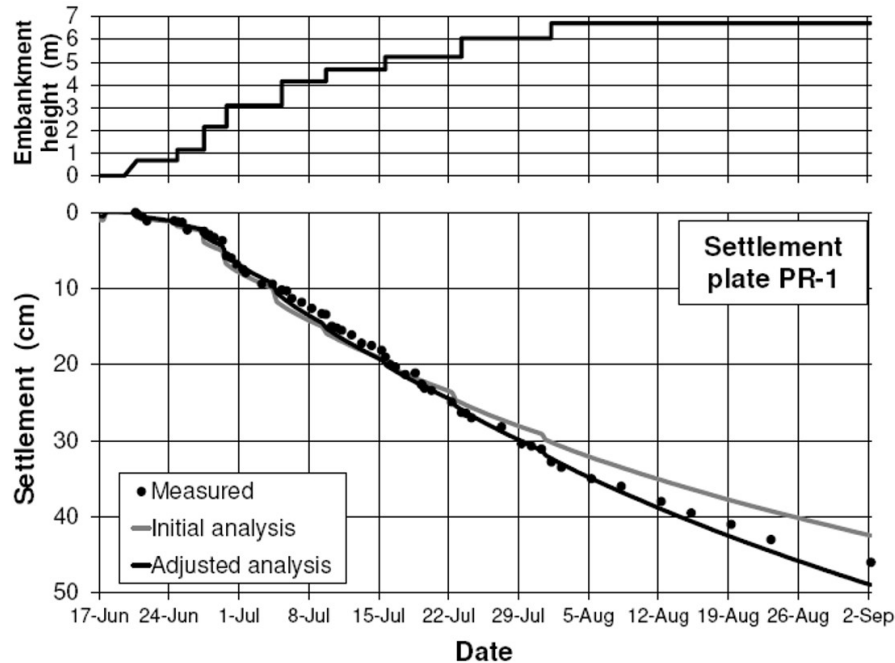


Figure 12. PR-1 settlement plate: measurements and results of both initial and adjusted analyzes.

and the measured values is approximately constant from 8.Jul.2013 onwards, which shows that the absolute differences observed are either due to the excess pore pressures values calculated before this date, or to the poor response of the piezometers due to a possible lack of saturation.

- 4) Despite the limited monitoring period, the model's ability to capture the field behaviour was on a par with other studies (e.g., Huang et al., 2006; Chai et al., 2013; Rezanian et al., 2018), where good agreement is seldom obtained at the same time for settlements, horizontal displacements, and pore pressure development.

Table 4 shows the adjusted parameters of the clay layers in bold, with others initially defined. For a better adjustment of the settlement curves over time, the hydraulic conductivity values of Layers 5 and 7 were increased (6 and 3 times greater than the initial value). To reduce the horizontal displacements obtained in the initial analysis, in addition to increasing permeability, the v_{ur} and C_s values were also reduced.

For the other layers, the average parameters of Table 2 were maintained, with the exception of the strength parameters of Layer 1, that had to be increased to $c' = 50$ kPa and $\phi' = 38^\circ$ due to local failures that interrupted the numerical solution.

7. Long-term prediction

The test embankment was monitored up to 32 days after reaching its maximum height. In order to predict its long-term behaviour, the analysis was continued for another

50 years. Neither the reduction in the drainage distance of the clay layers (the option of updating the finite element mesh was not used) nor the load relief by submersion of the embankment were taken into account.

7.1 Long-term pore pressures and settlements

In the authors' understanding, the deformation process during clay consolidation is a visco-elasto-plastic phenomenon, as indicated by the mechanism devised by Terzaghi (1941) to explain the secondary effects of time. This secondary effect has a viscous nature and is an additional mechanism to Terzaghi's (1923) classical theory for one-dimensional consolidation. The mechanism suggests that film bonds are present in the contacts between the grains, made of viscous absorbed water, delaying the particles approximation, and that the deformations cease only after all contacts have transformed into solid bonds, which occur between the solid portions of the absorbed water layers.

As discussed by Oliveira et al. (2023), considering valid this mechanism, the distinction between primary compression and secondary compression only makes sense in the case of thin layers, such as in the oedometer test, where high strain rates mobilize a significant amount of viscous resistance during the so-called primary consolidation. This viscous resistance delays the compression process, and its demobilization occurs mainly after the excess pore pressure is essentially dissipated (which occurs in a short time due to the small thickness), generating the strains corresponding to the so-called secondary consolidation. In the case of thicker

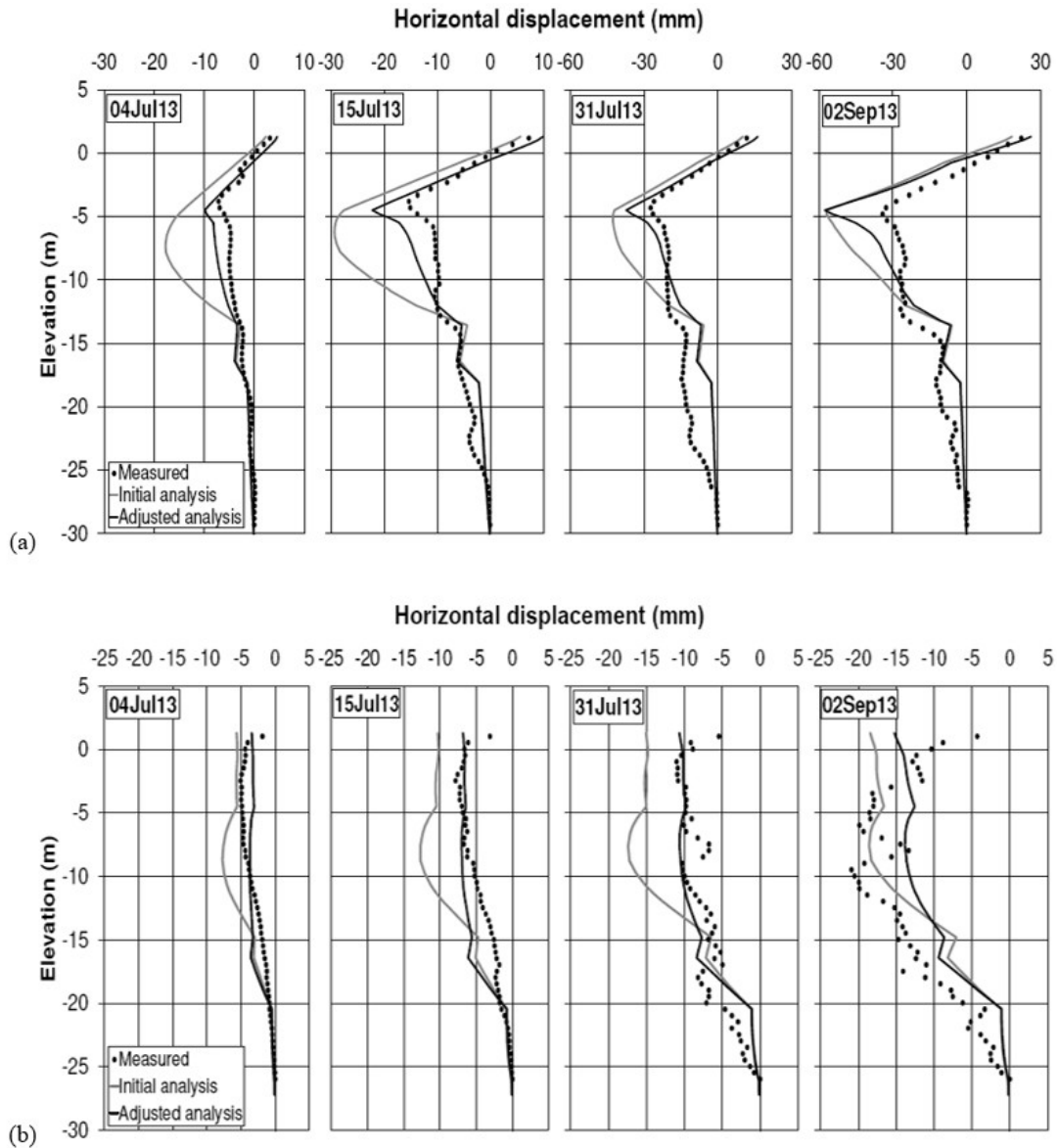


Figure 13. Measurements and results of both initial and adjusted analyzes: (a) Inclinator IV-13; (b) Inclinator IV-12.

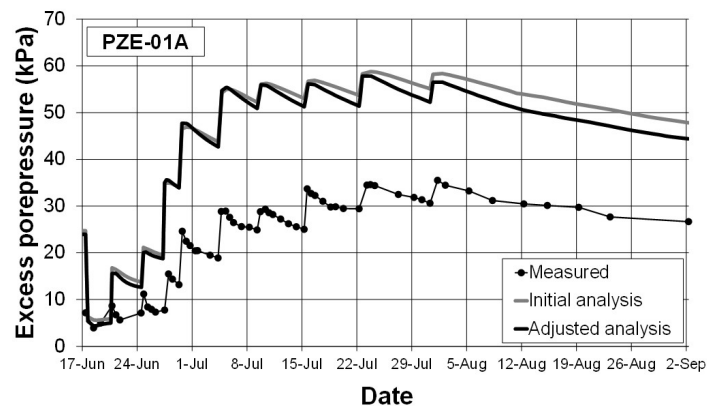
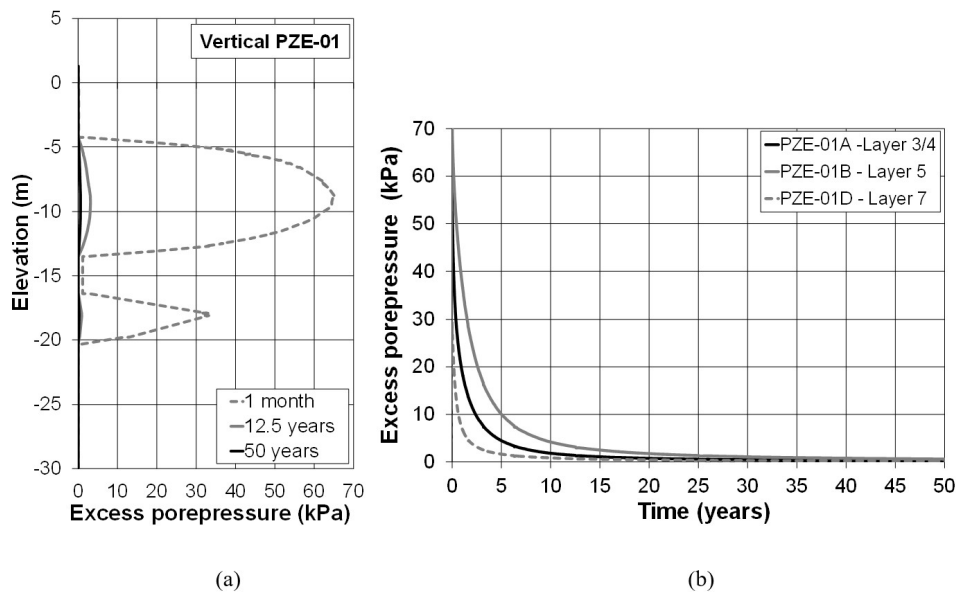


Figure 14. Piezometers at vertical PZE-01: measurements and results of both initial and adjusted analyzes.

Table 4. Adjusted parameters of the clay layers.

Parameter	Layer 3		Layer 4		Layer 5		Layer 7	
	initial	adjusted	initial	adjusted	initial	adjusted	initial	adjusted
γ (kN/m ³)	14.2	14.2	15.9	15.9	13.5	13.5	14.9	14.9
e_{init}	2.6	2.6	1.6	1.6	3.2	3.2	2.1	2.1
C_C	2.35	2.35	1.50	1.50	3.00	3.00	1.60	1.60
C_S	0.19	0.22	0.12	0.05	0.24	0.07	0.13	0.13
C_α	0.14	0.14	0.08	0.060	0.15	0.165	0.08	0.096
OCR	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6
v_{ur}	0.15	0.05	0.15	0.05	0.15	0.05	0.15	0.15
c' (kPa)	0	0	0	0	0	0	0	0
ϕ'_{cv}	32°	32°	32°	32°	32°	32°	32°	32°
ψ	0°	0°	0°	0°	0°	0°	0°	0°
K_0^{nc}	0.47	0.47	0.47	0.47	0.47	0.47	0.47	0.47
$k_x = k_y$ (10 ⁻⁹ m/s)	37.4	37.4	26.4	26.4	2.2	13.3	1.2	4.0
k_z (10 ⁻⁹ m/s)	17.0	17.0	12.0	12.0	1.0	6.1	0.6	1.8
c_k	1.41	1.41	0.90	0.90	1.80	1.80	0.96	0.96

**Figure 15.** Excess pore pressures: (a) profiles at the vertical of PZE-01 and (b) development at piezometers' positions over time.

layers, such as those found in the field, the strain rates are very low, mobilizing little viscous resistance, so that there is practically no delay in the compression, which will occur during the pore pressure dissipation process (which develops slowly due to the great thickness). Thus, it is not divisible a period of primary compression and another of secondary compression, i.e., in field conditions, secondary consolidation occurs together with the primary consolidation.

In PLAXIS 3D, the soil model that comes closest to the behaviour described above, and that was adopted for

clayey layers, is the Soft Soil Creep, which considers the occurrence of secondary consolidation (or creep) together with the primary consolidation, although it does not consider a final strain value for a given load (i.e., creep strains reduce logarithmically with time, but never cease).

Bearing in mind these considerations, which are important for simulations of long-term events, the model results follow.

Figure 15a presents the excess pore pressure profiles computed at the vertical of PZE-01 piezometers at different

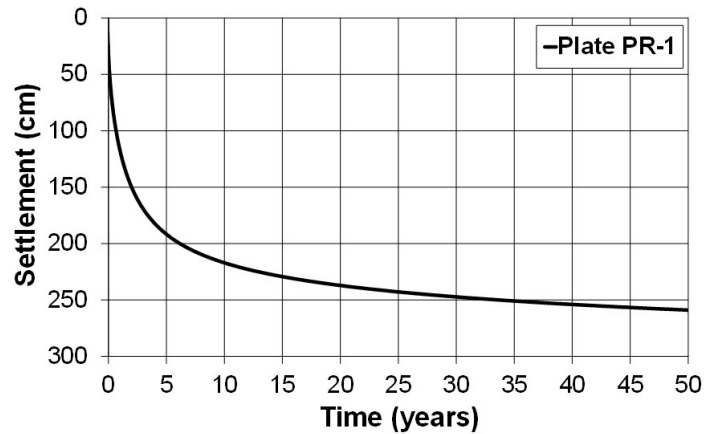


Figure 16. Settlement at the position of plate PR-1.

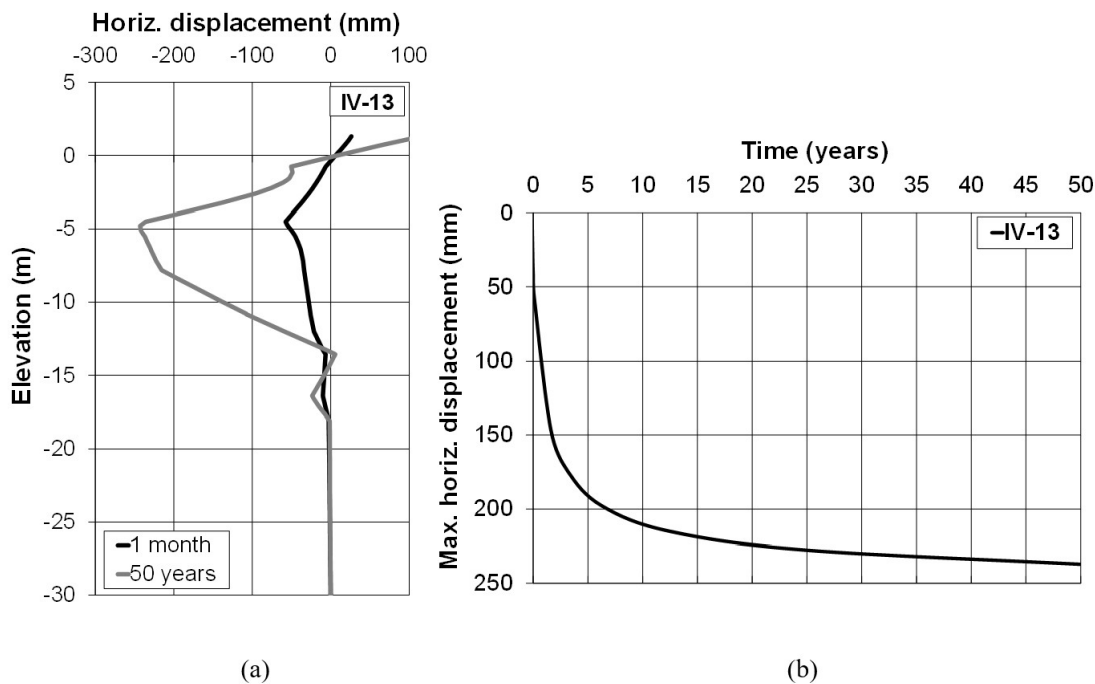


Figure 17. Horizontal displacements in the IV-13 position: (a) profiles at different times and (b) development of the maximum horizontal displacement.

times after the end of the construction. Figure 15b shows the decrease of excess pore pressures over time in 3 piezometers. It is observed that, in the region of these piezometers, from 4 to 7 years after the last rise, 90% of excess pore pressures would have dissipated, and that 20 years later they would be practically null.

Figure 16 shows settlements of plate PR-1 over time, where it can be observed that, unlike excess pore pressures, there is no stabilization even after 50 years. This is due to the consideration of secondary consolidation in the Soft Soil Creep model adopted in the clay layers. Settlements after

50 years are about 2.5 times greater than those at 1 month after the final embankment rise.

7.2 Long-term horizontal displacements

Figure 17a shows horizontal displacement profiles at the inclinometer IV-13 position, on the last monitoring date (02.Sep.2013) and at the end of the 50-year analysis. Figure 17b shows the development of the maximum horizontal displacement, where it can be noted that it would quadruple in relation to those after 1 month of completing

the embankment. And, as is the case of settlements, the horizontal ground displacements after 50 years would not be fully stabilized, due to creep.

8. Conclusions

1. The geotechnical investigations carried out at the test site allowed a good definition of the geometry of the subsoil layers, at least in the area subjected to the greatest stresses increase, consistent with the local geological history.
2. The sandy and silty layers were modeled in PLAXIS as Hardening Soil and the clayey layers as Soft Soil Creep. The former had their parameters estimated mainly from SPT, CPTu and correlations, while the latter from vane, oedometer and triaxial tests.
3. Sensitivity studies indicated that the parameters of the clayey layers were the ones that most influenced the analysis results, mainly the *OCR* and the permeability parameters.
4. The results of the initial 3D analysis of the embankment with the values for the parameters taken as the average of the probable range, agreed to some extent with the measured ground displacements. However, adjustments in the parameters of the clayey layers produced better predictions to both horizontal displacements, which had been overestimated, and settlements, which had been underestimated. Quantitatively, the average error in the settlements of PR-1 plate, for example, went from 12% in the initial analysis to 6% in the adjusted analysis, and the average error in the horizontal displacements in INC-13 inclinometer went from 214% to 169%.
Regarding the excess pore pressures, although a good match with the piezometer measurements in absolute values was not achieved (analysis results being always higher than those measured), after a certain time, the incremental values came close, suggesting that the differences are due to the values calculated at the beginning of the monitoring, perhaps due to model inaccuracies, or possibly due to a lack of saturation of the piezometers.
5. Regarding the long-term behaviour, the analysis, extended to 50 years, indicated that:
 - (i) after 20 years, excess pore pressures would be practically dissipated;
 - (ii) even after 50 years, settlements and horizontal displacements would not be fully stabilized due to creep;
 - (iii) at 50 years, settlements would be about 2.5 times greater than those at 1 month after the end of construction, and maximum horizontal displacements of the ground would be about 4 times greater.

Acknowledgements

The embankment test was designed by Uberescilas F. Polido, Geoconsult. Hugo F. França performed/supervised most of the in situ and laboratory tests as part of his MSc research. The use of software PLAXIS 3D was made possible by Jean Pierre P. Rémy, Mecasolo Engineering. Prof. Marcio S.S. Almeida kindly reviewed the manuscript. CAPES - Coordenação de Aperfeiçoamento de Pessoal de Nível Superior, Brazil, partly financed this research under Finance Code 001.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Fernando da Silva Oliveira: conceptualization, methodology, formal analysis, visualization, writing – original draft. Francisco de Rezende Lopes: conceptualization, data curation, methodology, supervision, validation, writing – review & editing.

Data availability

Data generated and analyzed in the course of the current study are available in the Civil Engineering Program - COPPE/UFRJ repository, <http://www.coc.ufrj.br/en/master-dissertations/582-2015/5242-fernando-da-silva-oliveira>.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

c'	Cohesion intercept
c_h	Horizontal coefficient of consolidation
c_k	Coefficient of change of permeability
c_v	Vertical coefficient of consolidation
e_{init}	Initial void ratio
f_s	Unit sleeve friction resistance
k	Hydraulic conductivity
m	Power for confining stress dependency of stiffness
n	Modulus exponent
p_a	Atmospheric pressure

p^{ref}	Reference stress for stiffness
q_c	Measured cone resistance
q_t	Corrected cone resistance
w_{nat}	Natural water content
B_q	Pore pressure parameter, $(u_2 - u_0)/(q_t - \sigma_{v0})$
CAU	Consolidated anisotropically, undrained triaxial test
C_c	Compression index
CIU	Consolidated isotropically, undrained triaxial test
CPTu	Cone Penetration Test with pore water pressure measurement
C_s	Swelling/recompression index
C_α	Secondary compression index
E'	Effective Young's modulus
E_{s0}	Secant stiffness in standard drained triaxial test
E_i	Initial Young's modulus
E_{oed}	Tangent stiffness for primary oedometer loading
E_{ur}	Unloading/reloading stiffness
F_r	Normalized friction ratio $= f_s/(q_t - \sigma_{v0})$
G_0	Initial shear modulus
G_s	Secant shear modulus
K	Dimensionless stiffness modulus
K_0^{nc}	Coefficient of earth pressure in normal consolidation
K_{ur}	Dimensionless unloading/reloading stiffness modulus
LL	Liquid limit
N_{SPT}	Blow count in the SPT
OCR	Overconsolidation ratio
PL	Plastic limit
Q_t	Normalized cone resistance $= (q_t - \sigma_{v0})/\sigma'_{v0}$
R_f	Failure ratio q_f/q_a
SPT	Standard Penetration Test
Su	Undrained shear resistance
UU	Unconsolidated, undrained triaxial test
γ	Unit weight
ε_a	Axial strain
κ^*	Modified swelling index
μ^*	Modified creep index
ν'	Drained Poisson's Ratio
ν_{ur}	Poisson's Ratio for unloading/reloading
σ'_1	Major principal effective stress
σ'_3	Minor principal effective stress
σ_d	Deviator stress
σ'_v	Vertical effective stress
ψ	Dilatancy angle
ϕ'	Peak effective friction angle
ϕ'_{cv}	Critical state effective friction angle

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Development of a low cost experimental trigger for dynamic compaction and dynamic replacement

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Case Study

Keywords

Dynamic compaction
Dynamic replacement
Experimental trigger
Finite element analysis

Abstract

Dynamic compaction and dynamic replacement represent techniques for improving soils with low bearing capacity or with a high degree of deformability. Those techniques are performed through the percussive process of large hammers or plates with metric tons of weight released from large heights on the soil. Currently, there are many companies worldwide using different types of equipment with distinct specifications regarding height and lifting capacity, potency and release form (cables or free fall). For experimental purposes, this article proposes the development of a trigger system composed of a release lever and a body, which is formed by the fixed body, the moving body and a lock, inspired by Giese (2019). The proposed trigger body was developed with SAE 4140 steel and SAE 1020 steel in connection with the low cost release body. After the tests, the objective of the study was achieved in the laboratory tests (80 kN) and field tests (55 kN of tamper weight plus friction and adherence between soil/tamper).

1. Introduction

Soil improvement or reinforcement involves the modification or manipulation of physical and chemical processes to enhance the mechanical properties and parameters of soil, such as shear strength, compressibility, and hydraulic conductivity, in order to meet specific requirements in civil construction projects. In recent years, various innovations in geotechnical engineering have explored commercial solutions for improving and reinforcing different soil types. These include consolidation methods (e.g., vertical drains and vacuum consolidation) and reinforcement techniques with driven aggregate piles, controlled modulus columns, dynamic replacement, jet grouting, and vibro-replacement (Kézdi, 1979); Geosynthetics (Koerner, 1997); Chemical methods with Portland cement, Lime and Bituminous (Ingles & Metcalf, 1973); as well as compaction methods with vibro-compaction and dynamic compaction (Slocombe, 2013; Hamidi, 2014; Giese, 2019). Concepts and other

soil improvement methods can be found in the works of Mitchell (1981), Terashi & Juran (2000), Han (2015) and Almeida et al. (2022).

Dynamic compaction can lead to significant soil improvement at substantial depth, often with considerable savings when compared to other geotechnical solutions (Hamidi, 2014). It has become a popular method for deep improvement of soils, especially in Middle Eastern, European and North American countries. According to Souza Júnior (2021), dynamic compaction has existed since Roman times in a rudimentary form, without standardized technical procedures. Formally, dynamic compaction was standardized through large-area improvement techniques developed by Menard, as presented in Menard (1974) and Menard & Broise (1975), using adapted machinery.

The method involves repeated application of high energy impacts on the soil surface using compactors with 20 t of weight, falling from 20 m high, for densification or consolidation the soil layer to a considerable depth (Slocombe,

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Submitted on November 27, 2024; Final Acceptance on February 27, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha <https://orcid.org/0000-0002-2264-9711>

<https://doi.org/10.28927/SR.2025.008324>



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2013; Pan & Selby, 2002). The specifications of the compaction weights, heights and distance between compaction points are defined as a function of the improvement to be achieved, based on the design needs. Therefore, these parameters vary according to the soil improvement depth. There are reports of different studies using varied compaction weights and fall heights, such as those presented by Slocombe (2013), Pan & Selby (2002), Gambin (1983), Mayne et al. (1984), Rollins & Rogers (1994), Feng et al. (2011), Hamidi (2014).

Currently, there are many companies worldwide that work with this type of technique on different types of soil. In these companies, there are several pieces of equipment with different configurations and specifications, such as shape, lifting height, lifting capacity and release form (free fall or through cables) (Hamidi, 2014). However, although there are commercial trigger systems available, there are few reports of this device in the literature. For experimental purposes, a method was released at Federal University of Rio Grande do Sul (UFRGS) to drop compaction hammers with the aid of cranes, simplified trigger, straps and bundles of steel cable in order to investigate the effect of compaction on Brazilian tropical soils (Giese, 2019; Rizzon, 2016). Based on the experiences of previous studies (Giese, 2019; Rizzon, 2016), this paper aims to present the development of an experimental manual release trigger designed for low-cost dynamic compaction and dynamic replacement coupled to a crane system.

2. Dynamic compaction and dynamic replacement method

2.1 Principle of technique

Dynamic compaction method involves the repeated application of high energy impacts on the soil's surface using either concrete or steel compactors weighing 100-250 kN, falling from heights of 10 m to 30 m high, compressing the soil strata to a considerable depth. These weights, also known as hammers, are released repeatedly on the same impact spot according to a predetermined standard (Porán & Rodriguez, 1992). The mass and the fall height of the compactor vary according to each researcher; however, the order of magnitude remains the same (e.g., Slocombe, 2013; Mayne et al., 1984; Feng et al., 2011; Hamidi, 2014; Menard & Broise, 1975; Craig, 2012; Feng et al., 2015). Additionally, the number of hits varies according to the type of soil and as a function of the dissipation of the water pressure (Menard & Broise, 1975).

The principle of the technique is based on the reduction of intergranular empties, specifically for treatment of natural, mixed and cohesive sands (Giese, 2019). However, the technique was extended to fine soils (silts and clays), where high energy impact effectively provides a concentrated load

that dissipates water pressure from silts and clays, known as dynamic consolidation (Menard & Broise, 1975). Dynamic replacement was then developed to build large-diameter columns of granular materials through soft grounds close to the surface, particularly for turf and Sabkha layers (Hamidi, 2014; Tarawneh et al., 2017; Tarawneh et al., 2019).

As dynamic replacement represents a specialized form of compaction, there are many similarities between both processes, such as hammer mass, fall height and equipment. In this method, the granular fill is dynamically compressed through a percussive process involving loads varying from 150 kN to 250 kN and heights from 10 m to 25 m (Tarawneh et al., 2017). It produces a large diameter reinforced soil column (known technically as pillar) with diameters from 2.0 m to 2.5 m and depths of up to 7 m. On average, this method might reach replacement rates of up to 25% and each column has a load capacity up to 1500 kN.

2.2 Items and equipment

Regarding equipment specifications, in Onshore constructions it is used wheeled or crawler cranes to lift hammers from 15 t to 25 t. The Menard Tripod can also lift hammers up to 40 t (Hamidi, 2014). Hammers are typically built from hardened steel sheets, box steel and concrete or concrete of suitably reinforced mass and distinct geometries, where durability is the main requirement. From a technical point of view, the geometry and the contact tension of the base are relevant aspects to the formation of craters (Giese, 2019; Mayne et al., 1984; Rizzon, 2016; Mehdi pour & Hamidi, 2017), to the influence depths (Mayne et al., 1984; Smits & De Quelerij, 1989; Lukas, 1995; Ghassemi et al., 2009; Hajjalilue-Bonab & Rezaei, 2009) and affecting the plasticizing bulbs (Porán & Rodriguez, 1992; Oshima & Takada, 1997). In terms of release methods, the literature reports that hammers can be released by free falls or cables.

In the free fall method, the hammer is released without cables from a specified height, reducing friction losses. In contrast, for the cables system, the hammer is released while fixed to steel cables in a similar way to that which occurs to pile drivers. Ménard has developed a trigger system that enables a pseudo free fall, on which the hammer is released in free fall with a cable fixed on the trigger in order to prevent the impacts between the hook and the boom, called MARS (Menard Accelerated Release System) (Hamidi, 2014).

Giese (2019) has developed a trigger with a manual aperture system, similar to the MARS one. The device was produced with SAE 1020 steel, which is simple carbon, with other negligible elements in percentage and manganese content of 1%, and flow resistance value of 351,6 MPa. However, the trigger created presented: reduced lever stroke; lock/lever connection limited to the cable and therefore low stiffness which gave fatigue of the connection; body steel with stiffness and strength not corresponding to lifting forces; and no steel protecting system.

The cost of acquisition and maintenance is extremely high, which makes the method often not economically feasible. The proposed trigger coupled to a crane can be an interesting alternative for soil improvement in works of low to medium complexity due to the proven efficiency of the method in soil treatment. Another important point to be exposed is the sustainability generated by the method of dynamic compaction and dynamic replacement because the method is free of the use of cementing agents and allows the use of demolition waste as fill material.

3. Trigger conception

3.1 Dispositive function

The operation of the trigger is based on the principle of moments. Once the hammer is lifted to the designed height, it is released by lifting the lock fixed on the release lever, which prevents the moving body from tipping prematurely. The tipping of the moving body allows the handle, attached to the hammer, to be released into free fall. This moment is produced by a force at the end of the lever, which is derived from the tensile force on the wire attached to its end (Figure 1). This wire is designed to disengage the hammer at the desired height. The effort to release the wire is carried out manually by the operator.

3.2 Trigger details

The software Autocad® (student Version) was used to detail the views of the device created for simulation and machining of the parts. It is shown in Figure 2 the dimensions of the complete body and of the moving body and in Figure 3 the detailed dimensions of the fixed body. Finally, Figure 4 shows the detailed dimensions of the lock and its opening system. Cuts and geometric details of the piece can be seen in more detail in Souza Júnior (2021).

4. Materials and methods

In terms of loading on the trigger, it was considered a load of 70 kN. This corresponds to the static effort to remove the hammer from the crater, which is the sum of the load of the weight of the compaction hammer, the base adhesion of the it and the friction of its base. For this, the trigger was modeled with the cylinder coupling supported in a delimited region of the moving body (as it simulates the handle supported in the part) based on Finite Elements Methods (FEM).

The trigger body was designed with SAE 4140 steel, which is a chromium-molybdenum alloy characterized by high mechanical strength, stiffness and fatigue, impact and abrasion resistance. The release lever of the hammer was

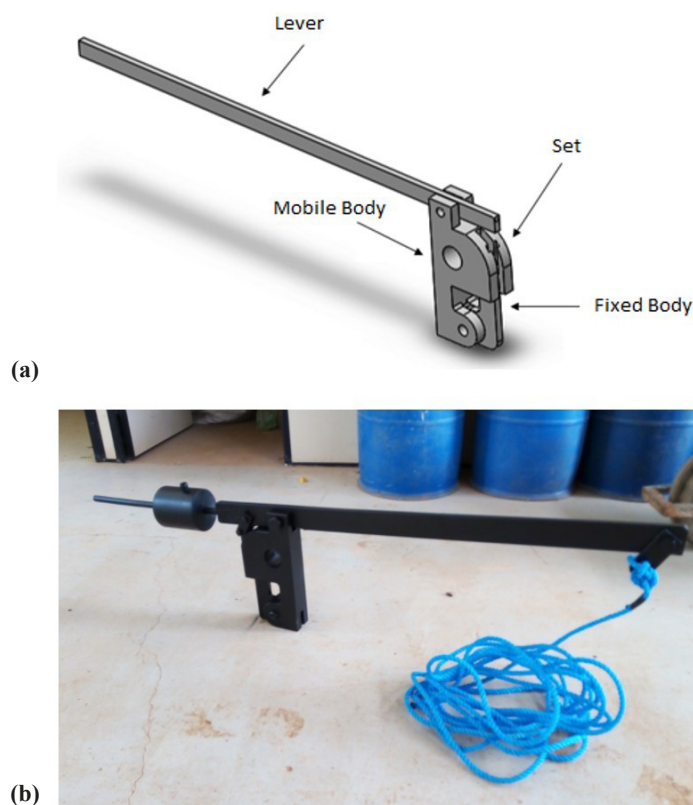


Figure 1. Experimental trigger: (a) 3D-Figure in Solidworks®; (b) built.

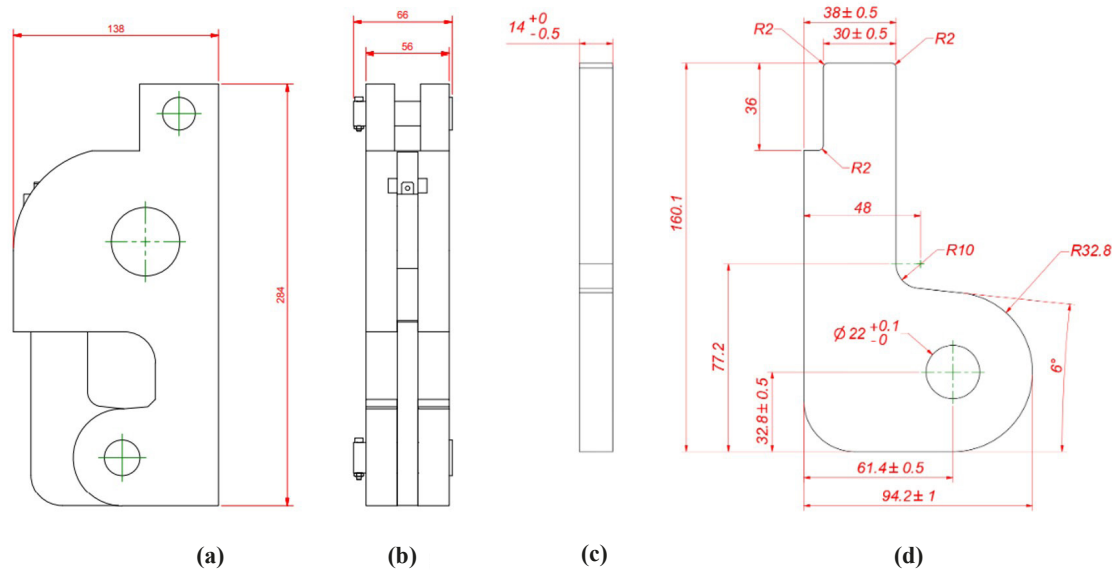


Figure 2. Details of the complete body and the moving body (in millimeters): (a) side view of the complete body; (b) front view of the complete body; (c) front view of the moving body; (d) side view of the moving body.

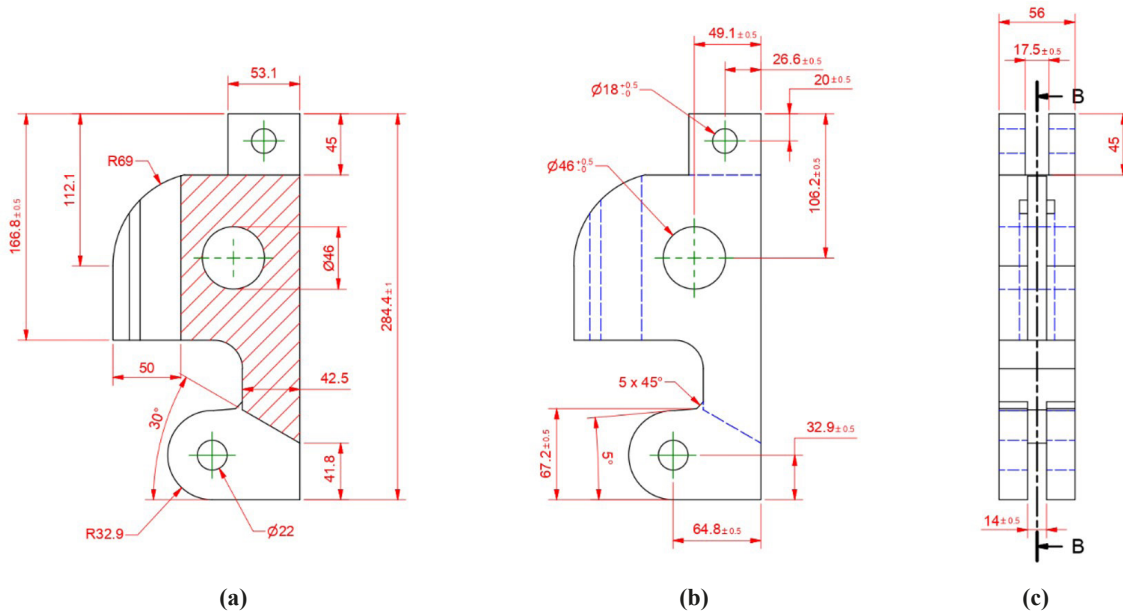


Figure 3. Details of fixed body (in millimeters): (a) side section of fixed body – section B-B; (b) side section of fixed body; (c) front view of fixed body.

designed with SAE 1220 steel, which is a carbon steel consisting basically of iron, carbon, silicon and manganese. It has lower mechanical properties compared to SAE 4140 steel, as it will not be subjected to high stress. The chemical compositions of both materials are described in Table 1, according to ABNT (2000), and the mechanical properties are described in Table 2.

The trigger was designed with Solidworks® Software, version 2016. It was based on FME using a mixed standard

mesh with 4 Jacobian points and it was also performed a mesh convergency test based on 5 evaluations. Von Mises' plastic constitutive model is suitable for the description of the behavior of ductile materials such as metals (Chen & Han, 1988). This model, according to Chen & Han (1988) and Chen & Zhang (1991) is based on the following premises:

- Von Mises' criteria states that the plasticization of a material would occur when the maximum shear strain energy at a point in the material reaches a

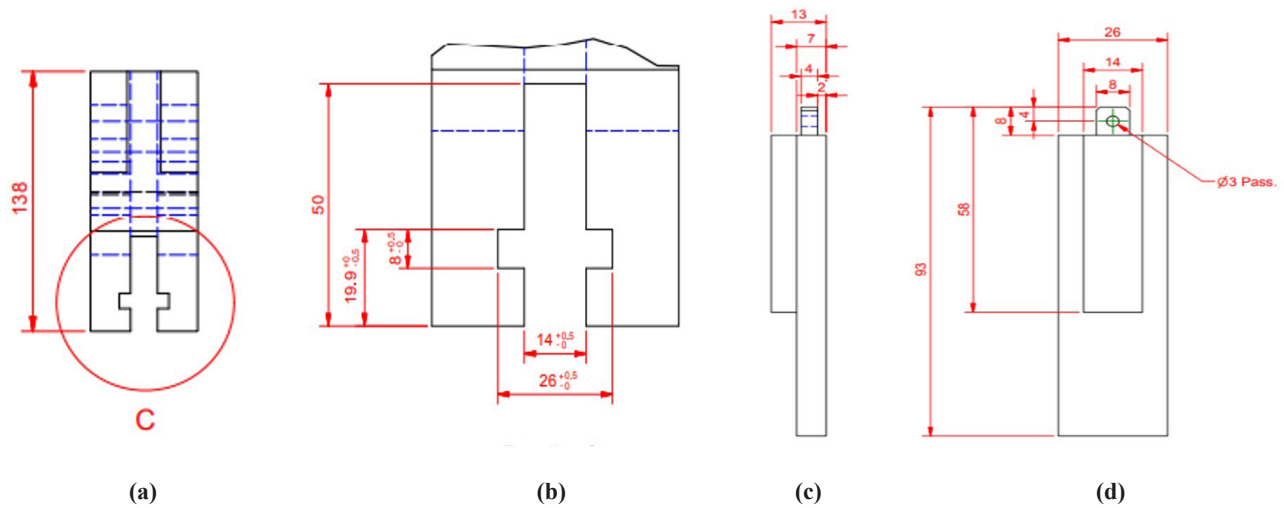


Figure 4. Details of the lock (in millimeters): (a) upper view; (b) dimensions of upper view; (c) side view of lock opening system; (d) lock front view.

Table 1. Chemical composition of steels used in the trigger (ABNT, 2000).

Steel	C (%)	Mn (%)	Si (%)	Cr (%)	Mo (%)	P (%)	S (%)
SAE 1020	0.18 – 0.23	0.30 – 0.60	---	---	---	0.040 (max)	0.050 (max)
SAE 4140	0.38 – 0.43	0.75 – 1.00	0.15-0.35	0.80-1.10	0.15-0.25	---	---

Table 2. Mechanical properties of the steel used on the trigger design.

Properties	Unity	Steel	
		SAE 4140	SAE 1020
Modulus of Elasticity	GPa	209.00	211.00
Poisson Coefficient	---	0.32	0.29
Shear Modulus	GPa	81.57	77.00
Specific Mass	kg/m ³	7850.00	7900.00
Tensile Strength	MPa	1132.00	420.00
Yield Value	MPa	724.00	351.57

critical value, this energy being proportional to the second variant of the stress tensor;

- An associated flow rule assumption is made, where the plastic flow is associated with the yield criterion;
- Isotropic and kinematic hardening rules are valid, as the associated flow rule for irreversible plastic materials is applicable, meaning plastic deformation cannot be recovered;
- In large plastic flow problems, elastic tension can be neglected, and the material can be idealized as rigid and perfectly plastic.

To certify the quality of SAE 4140 steel of the trigger body, 10 samples were tested, being 5 tensile tests, following the procedures of ABNT (2015), and 5 Charpy impact tests, following ASTM E-23 specifications (ASTM, 2018), as shown in Figure 5. In addition, 5 hardness tests were performed on an impact strength sample (see CP5 in Figure 5), based on ABNT (2009) and ASTM (2019).

Figure 6 shows the equipment used to certify the quality of the steel used in the composition of the trigger body. As can be seen in Figure 6a, the electronic press used in the direct tensile tests was the model AGX Plus, with capacity up to 250 kN, of the company Shimadzu®. Figure 6b shows the Charpy test equipment, with a hammer of 30 kpm (with a mass of 18,75 kg and a rotating arm length of 0.825 m), made by German company Wekstoffprüfmaschinen Leipzig®. Finally, Figure 6c presents the Digital Hardness Test, model Duravision 30, from the company EMCO TEST®, with Rockwell C test option of 150 kgf load.

A tensile test was performed in the laboratory to validate and verify that the trigger modeled in software Solidworks®, version 2017, was consistent with the mechanical forces expected in the field. In addition, a Strain Gage model PA 09 060TG 350L (see Figure 7) was used to monitor the behavior of the trigger fixed body.

In the automated press, illustrated in Figure 6a, the trigger was attached to two handles, each of which was held

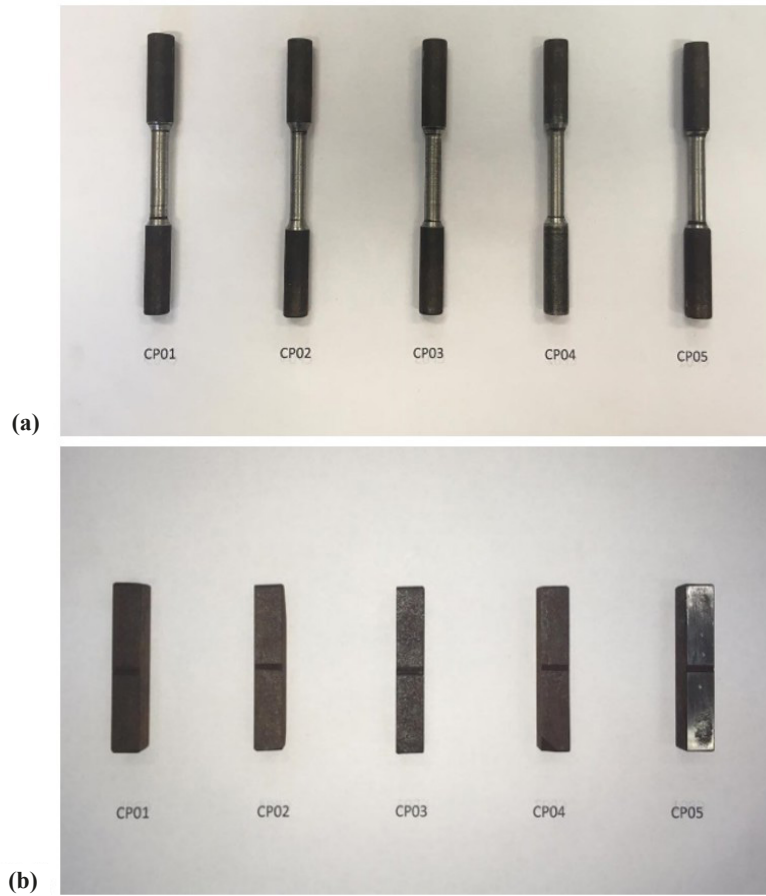


Figure 5. SAE 4140 steel sample: (a) direct tensile tests sample; (b) impact tests sample.

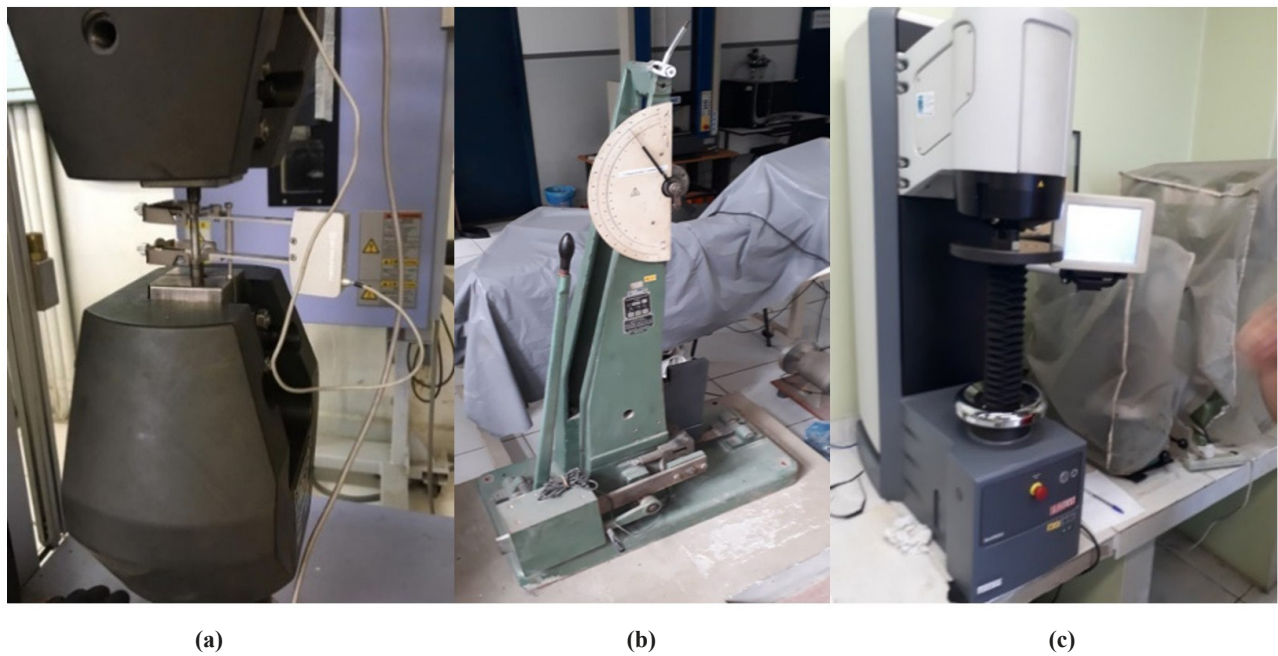


Figure 6. Equipment used for steel testing: (a) electronic press for direct tensile test; (b) impact test equipment; (c) hardness test equipment.

by a steel rope loop, which were fixed to the press claws, as shown in Figure 8. Deformation measurements were evaluated during the application of the monotonic load, through a strain gauge, as shown in Figure 9. It was attached to the face of the fixed body with highest stress, according to the numerical modelling.

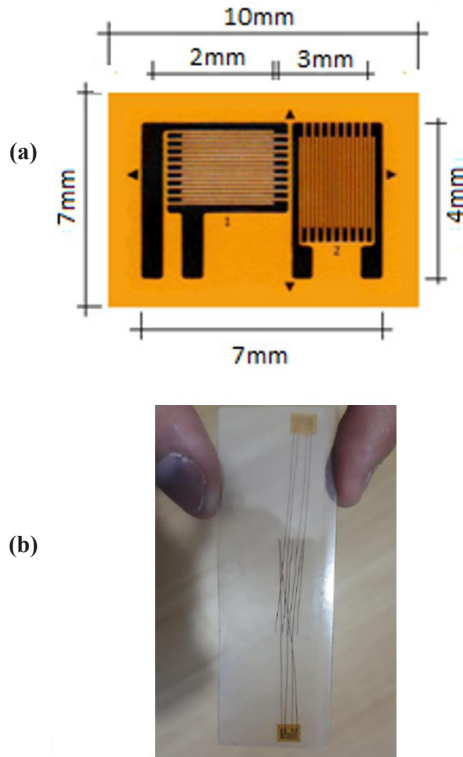


Figure 7. Strain gage model PA 09 060TG 350 L: (a) detail schematic; (b) real illustration.

The static equilibrium was used, with Ftool® Software, to determine the trigger lever length. For this, three analytical simulations were performed, with lengths of 0.4 m, 0.6 m and 0.8 m, and a fixed support was created at the end of the lever drive to release the lock. Hypothetically, the fixed support reaction would be achieved with the minimal effort to keep the system in balance. Therefore, if a higher load were applied to the fixed support, the static system would be transformed into a mechanism.

5. Results and discussions

The numerical convergence test of the mesh on the trigger showed that using a 0.007 m mesh on the body as well as refining the mesh in the support region with a 0.005 m characteristic size element, was sufficient for a consistent simulation, since the impact on its results was less than 1%, as shown in Table 3.

Numerical analysis was performed by adopting the scenario with characteristic size of the element with 0.005 m in the trigger body and 0.002 m in the region with the

Table 3. Numerical convergence test of the mesh on the trigger.

Characteristic Size of the Element (m)		$\delta_{\max}$ (mm)
Trigger Body	Mesh Support Region	
0.014	---	0.0461
0.014	0.007	0.0466
0.007	0.005	0.0473
0.005	0.003	0.0475
0.005	0.002	0.0476

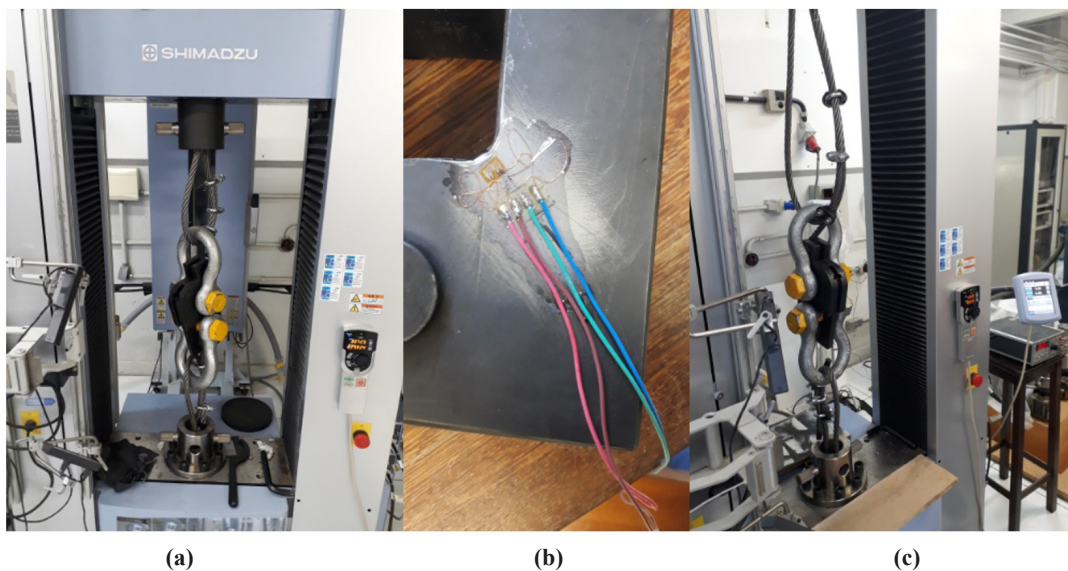


Figure 8. Direct tensile test: (a) trigger body fixed to the press; (b) strain gauge installation on trigger body; c) test set for trigger deflection reading.

highest stress concentration. Regarding the evaluation of tension and deformation in the elastic system of the trigger body (fixed body with the moving body locked), the numerical simulation showed that the fixed body presented small deformations (maximum of 0.154%) and a maximum stress of 370.11 MPa concentrated in the boundary region between the moving body and the fixed body.

It occurs due to the eccentricity of the lifting hole (top opening) and the coupling point of the weight (support region of the handle with the compaction hammer). The balanced position of the loaded trigger is represented by a slight rotation in the direction of opening the moving body, concentrating high stress in this region. Such eccentricity allows the weight to be released more easily when the device is opened.

Furthermore, in numerical simulation, a maximum displacement of 0.0476 mm was obtained, however, in the other parts of the body, displacements varied from 0.0198 mm to 0.357 mm, as shown in Figure 9a. The standard ASME BTH-1 (ASME, 2008) was used to determine the design factor. According to ASME BTH-1, situations in which the magnitude and load variations are not predicted, or the conditions are classified as severe or not precisely defined, the Category B is then used. In such category, the lifting devices shall not be less than 3.00, for yield or flash limit states, or less than 3.60, for fracture limit states or for connection design.

A value of 1.917 was obtained for the safety factor (FS) of the device. Since the concentrations that caused the reduction of the FS_{min} were punctual, it can be considered that the overall design safety factor of the piece was equal to 3.938, as shown in Figure 9b.

5.1 Trigger steel verification tests

After the tests on the SAE 4140 steel sample, it was perceived, through the results shown in Table 4, that the material is adequate when compared to the input data of the computational simulation, even reaching values higher than the specific one.

Furthermore, as it can be seen in Figure 10, the steel yield value has reached value from 1200 MPa to 1400 MPa, being approximately 65% a 93% bigger than the one used in the trigger modelling. This increase is due to the heat treatment

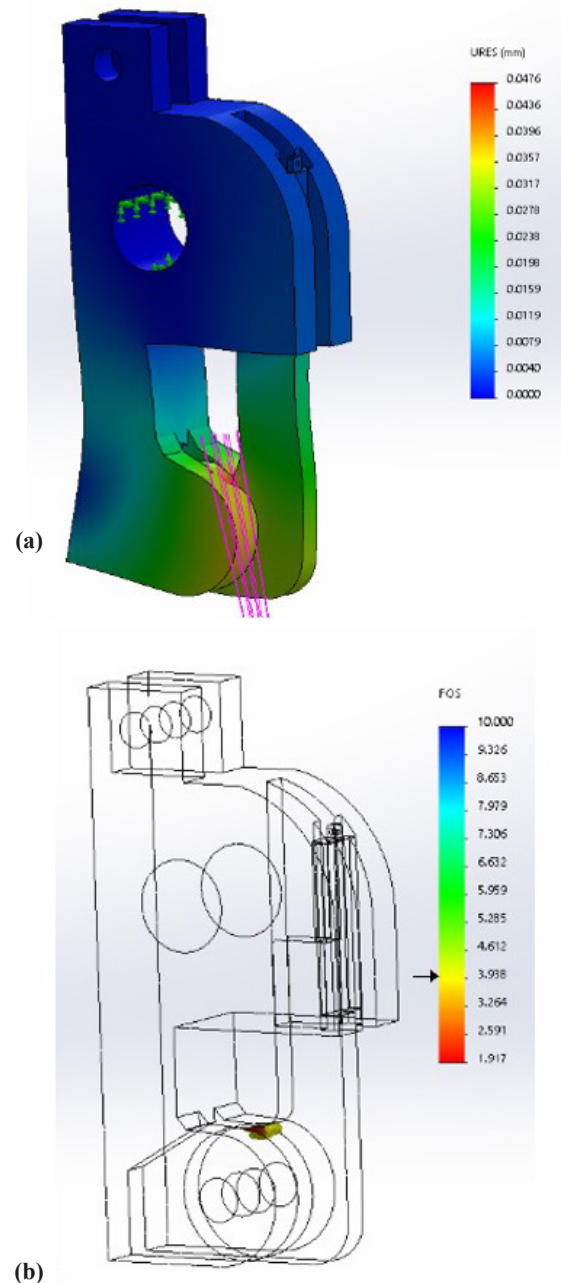


Figure 9. Trigger numerical simulation: (a) evaluation of displacements; (b) evaluation of safety factor.

Table 4. Results of the tests on the SAE 4140 steel sample.

Test	Values					Average
	1	2	3	4	5	
Hardness (HRC)	45.60	45.48	47.05	48.72	45.02	46.37
Impact Resistance (J/Rad)	29.42	29.42	33.34	23.54	32.36	29.62
Tensile Strength at Break (MPa)	1312.37	1337.98	1302.11	1319.34	1327.85	1319.93
Maximum Tensile Strength (MPa)	1402.87	1425.27	1372.69	1402.15	1414.93	1403.58
Yield Value (MPa)	1220.35	1249.99	1247.59	1189.49	1238.19	1225.92
Young's Modulus (GPa)	220.00	229.36	224.26	236.17	230.90	228.14

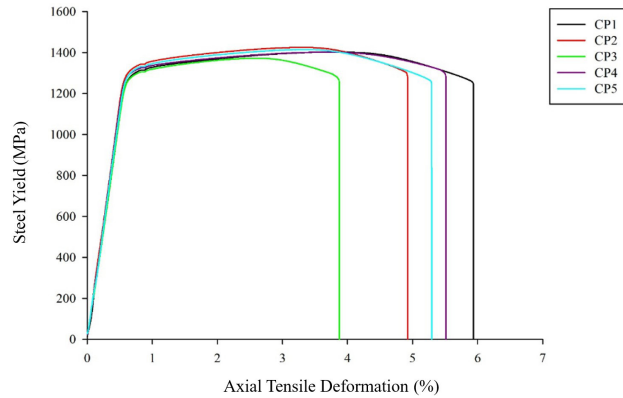


Figure 10. Direct tensile strength tests of SAE 4140 steel sample.

process applied to the steel called tempering. The heating and sudden cooling cycle allows better crystal arrangement, that raises the hardness and mechanical strength of the material.

5.2 Trigger verification tests

After the direct tensile tests on the trigger, it was found that the device body, in four tests, performed resisted tensile forces of up to 80 kN (8 t) without damage to the part, although the set has reached large displacements (between 0.020 m and 0.100 m), as shown in Figure 11, due to the stretching effect of the traction cables anchored in the press grip. Unfortunately, it was not possible to create a rigid connection at the time of testing, because there was no device that could easily connect to the trigger.

In terms of deformations suffered by the trigger after static press loading, two tests were performed. The first one was per phase, where every 1 t was stopped and the deformation value was taken, and the second one was per total load, where the deformation value was taken when the load was up to 5 t at the end of the test. By these means, the deformation of the piece was similar. In the first test, the final deformation was equal to 439 $\mu\text{m/m}$ and in the second test it was equal to 474 $\mu\text{m/m}$. This confirms that the component operates within the elastic regime under the stress/strain levels analyzed, as shown in Figure 12.

5.3. Estimated Selection Length

The verification of the trigger opening force was carried out through simulations using a hyperstatic portic with Ftool® Software. In this model, two loads were applied: one corresponding to the trigger counterbalance (70 N) and the other to the horizontal force component of the hammer's weight (8.437 kN). Variations in the device's lever length were studied at 0.40 m, 0.60 m, and 0.80 m.

As it can be seen in Figure 13, as the lever length increases, the support reaction reduces. This indicates that

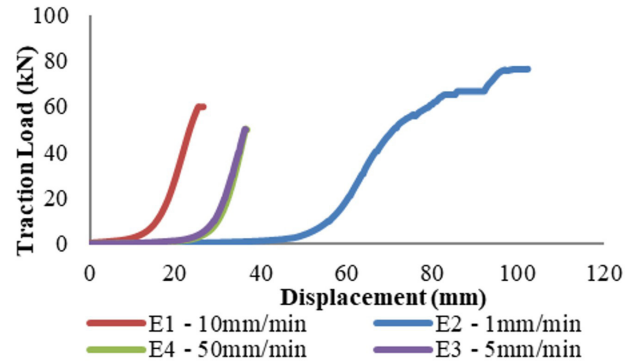


Figure 11. Direct tensile tests of the trigger with the handles attached.

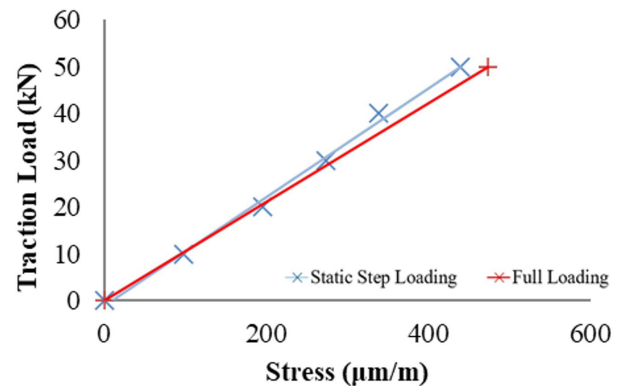


Figure 12. Elastic deformations suffered by the trigger in static step loading (blue) and full loading (red).

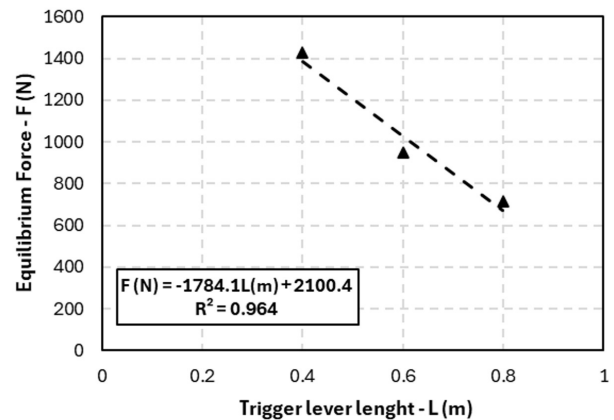


Figure 13. The relationship between lever balance effort and lever length.

increasing the lever length reduces the effort required by the operator for locking. While the support reaction value represents a balance effort, slight variations in the effort applied during release—ranging from approximately 10% to 20%—would be sufficient to unlock the trigger from an empirical standpoint. The relation can be considered normalized due to the excellent correlation obtained ($R^2=0.964$).

It is observed that, with a length of 0.8 m, equivalent efforts of 700 N (corresponding to the average weight of an operator) are achieved, thus reaching what could be considered the minimum length. However, due to the simplicity of the force equilibrium model, a 25% increase was applied, resulting in a total length of 1.0 m.

5.4 Trigger opening tests

The tests were carried out on a 55 kN hammer. In this test, the compaction hammer was hoisted up to 3 m, 5 m and 8 m and, thus, the release of the trigger was evaluated, as can be seen in Figure 14. This evaluation was conducted from a qualitative perspective, aiming to confirm whether the calculated strengths for device opening were consistent. A 70-t capacity crane was employed. To attach the trigger to the crane, a 10-t polyester strap was used coupled to the 20-t shackle. As for the hammer, steel ropes with a loop were used, which were attached to a washer and which, in turn, formed the compaction hammer assembly, according to item 3.1.

A priori, trigger counterbalancing was performed with the use of washers, totaling a mass of 7 kg, and the connection

between the lock and the lever with the use of a 0.004 m steel cable. After the trigger opening test, some modifications were made to the structure. First, the washers were replaced by a single weight in the section of the cylindrical axis, because they change their positions when triggered. In addition, the steel cable has been replaced by a 0.008 mm braided multi-stranded rope to facilitate the drive. Finally, the connection between the latch and the lever suffered continuous disruption during the trigger drive, and, therefore, it was replaced by an articulated plate system (latch-lever), according to Figure 15a. Figure 15b presents the details of the modification that occurred in the trigger structure.

6. Conclusion

The present article presented the development of a low-cost experimental trigger device for releasing hammers in dynamic compaction and dynamic replacement methods. The study employed numerical simulations using FEM, applying the Von Mises constitutive model through SolidWorks software. Additionally, laboratory tests were



Figure 14. Field opening verification test.

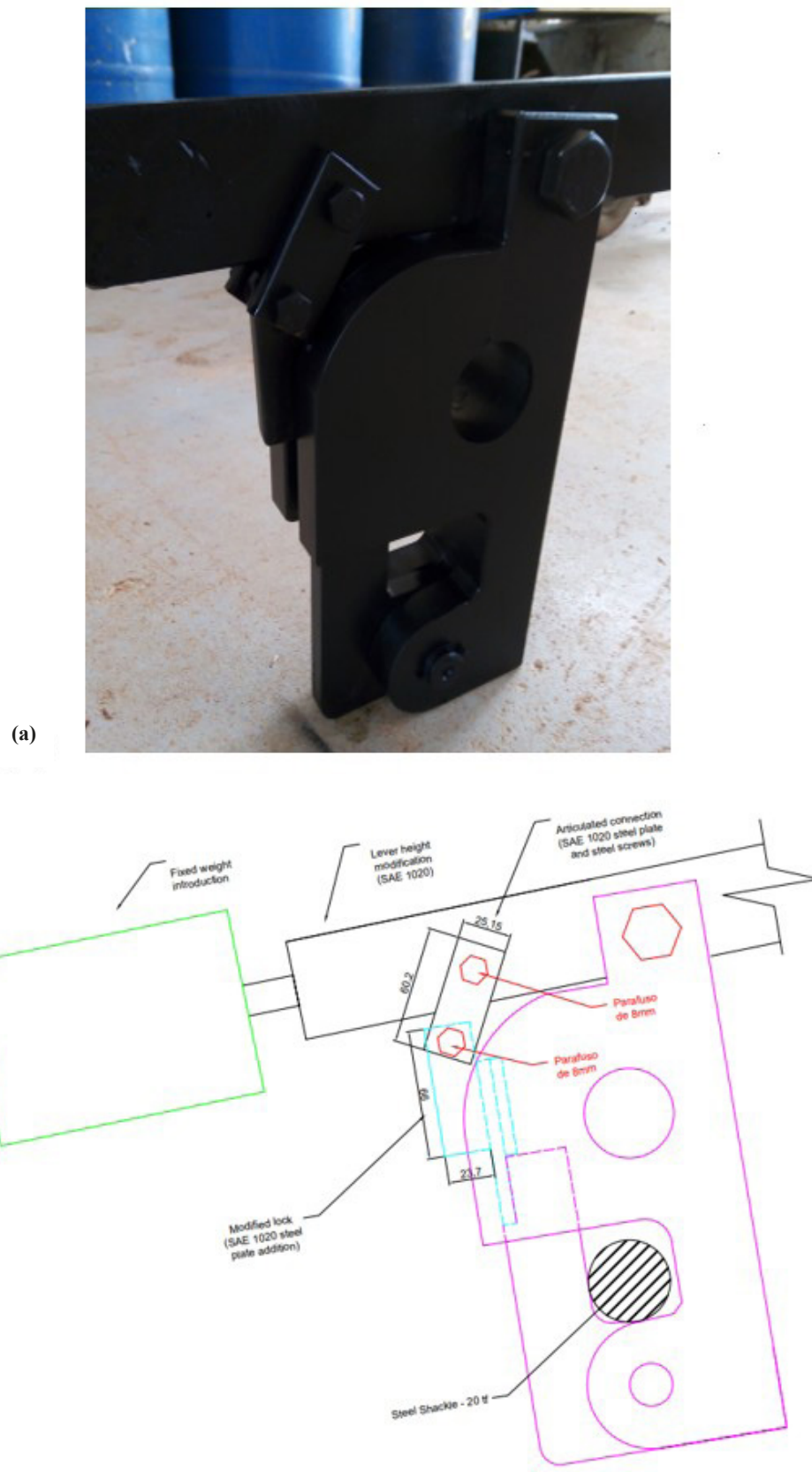


Figure 15. Modified release trigger: (a) trigger side view; (b) details of the modifications made.

conducted to characterize the material used in the trigger, as well as numerical simulations under monotonic static loading, analyzing the respective mechanical responses of the developed system. Based on the results obtained, the following considerations can be made:

- a. As perceived, a value of 1.1917 was obtained when performing the calculation of the safety factor of the piece (FS_{min}). Considering that the concentration that caused the reductions was punctual, it can be considered that, in a general way, the design safety factor of the part was equal to 3.938;
- b. As for the evaluation, after the numerical simulation, of the tension by the deformation, in elastic regime, of the trigger body (fixed body with the moving body locked), it was noticed that the fixed body presented small deformations (maximum of 0.154%) and maximum stress of 370.11 MPa concentrated in the boundary region between the moving and the fixed body.
- c. In the numerical simulation, a maximum displacement of 0.0476 mm was obtained, however, in the body, the displacements varied from 0.0198 mm to 0.357 mm;
- d. Although simplified, the use of isostatic and hyperstatic frames can be interesting alternatives for estimating the length of the trigger lever. Assuming that the support effort created (representing the trigger point) would be the minimum effort to maintain the static system, slightly higher efforts would unlock the mechanism to release the hammer;
- e. Field tests have proven that the lever length of 0.8 m is sufficient for the operator to drive and that the designed trigger supports the stresses by the dynamic compaction procedure;
- f. As a proposal for future studies, it is recommended to conduct comparative analyses between the proposed device and commercial devices, as well as studies on the dynamic unloading of the device in situations of rapid stress relief.

Acknowledgements

The authors thank CAPES (Coordination for the Improvement of Higher Education Personnel) for the research funding of the PPGE (Civil Engineering Postgraduate Program) of UFRGS (Federal University of Rio Grande do Sul). Writers would also like to thank anonymous reviewers for their insightful comments and suggestions that have improved the content of this article.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Tennison Freire de Souza Junior: Conceptualization, Investigation, Methodology, Formal Analysis. Tiago de Jesus Souza: Methodology, Data curation, Writing – original draft, review & editing. Gustavo Pizarro Meneghello: Writing – review. Tiago Augusto Pianezzer: Translation. Anderson Bonsanto: Writing – reviewing & editing.

Data availability

All data produced or examined in the course of the current study are included in this article.

Declaration of use of generative artificial intelligence

This work was not prepared with the assistance of Generative Artificial Intelligence (GenAI).

List of symbols and abbreviations

C	Carbon
Cr	Chrome
FS_{min}	Minimum safety factor
L	Trigger lever length
FEM	Finit Element Method
MARS	Menard Accelerated Release System
Mn	Manganese
Mo	Molybdenum
P	Phosphorus
S	Sulfur
Si	Silicon
UFRGS	Federal University of Rio Grande do Sul
δ_{max}	Maximum stress

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Method for predicting the shear mechanical behavior of rockfill material

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Alessander Christopher Morales Kormann¹ 

Case Study

Keywords

Rockfill
Large scale test
Friction angle

Abstract

Rock is a commonly utilized material in the construction of rockfill dams, protective ripraps for dam slopes, and stabilization of natural slopes. These materials demonstrate favorable performance under both static and dynamic loading conditions and are frequently more cost-effective compared to alternative stability solutions and construction methods for dams. The large dimensions and density of rock materials make it challenging to conduct representative mechanical tests and develop practical mathematical models that accurately describe their mechanical behavior at a real scale. This study proposes a hyperbolic prediction model for the shear mechanical behavior of earthfills, based on the outcomes of large-scale shear tests conducted on riprap samples with varying average dimensions (D_{50}). Analysis of the data revealed that the hyperbolic model exhibited good agreement with the graphs depicting shear box inclination mobilized friction angle θ ($^\circ$) versus shear displacement δ (mm), in relation to the riprap diameter D_{50} . Moreover, the parameters α (Stiffness index) and β (Asymptotic index), along with the hyperbolic model, displayed a promising interrelationship of high quality.

1. Introduction

In general, rockfill is a composition of natural rocks of various dimensions arranged in a layer to serve specific purposes in geotechnical projects. According to Cruz et al. (2014), rockfills consist of large rock blocks with diameters ranging from 0.50 m to 1.0 or 2.0 m, and their mechanical strength is influenced by factors like mineralogy, granulometry, void ratios, shape, moisture content, grain compressive strength, size, texture, and time. The characteristics of rocks can be categorized into two main aspects: physical properties (including density, porosity, permeability, water absorption, hardness, and thermal conductivity) and mechanical properties (such as compressive, direct or indirect tensile, shear and impact resistances, deformability, elasticity, and abrasiveness) (Fracá, 2019).

These rocky materials are commonly utilized in civil engineering projects, including as rockfill for natural slopes (Maynard et al., 1989), gabion walls (Fracassi, 2017), dam slope protection, and coastal margin protection (Vick, 1990; Cruz et al., 2014; Sandroni & Guidicini, 2021; Rawee et al., 2022). Due to their density and weight, they demonstrate

increased resistance to static loads and dynamic wave impacts, rendering them mechanically robust and long-lasting.

From a technical perspective, the slope inclination of a rock-filled embankment can serve as a contributing factor to failure. Hence, the configuration of the slope significantly influences the static stability of a dam or natural slopes. Moreover, the base of the slope is recognized as the most vulnerable component of the structure, highlighting the necessity for performing appropriate mechanical assessments.

The substantial geometric dimensions of these material particles render it unfeasible to conduct laboratory testing under controlled conditions. ASTM D4767-04 (ASTM, 2020a) and ASTM D7181 (ASTM, 2020b) stipulate that the grain diameter should be 1/6 of the specimen diameter to prevent scale effects in triaxial tests, whereas rockfill particle diameter typically exceeds several decimeters. Cruz et al. (2014) note that much of the information available in the literature has been derived from back-analysis in numerical models of rockfill dams with concrete facings. In dam structure analysis practice, the Leps graph (Leps, 1970) is commonly utilized to estimate rockfill strength parameters. Nevertheless, these estimates are not based on measured data but are instead approximations.

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Submitted on January 5, 2025; Final Acceptance on May 5, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.000225>



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One method for testing these materials involves conducting large-scale direct shear tests, also known as tilt tests, as proposed by Barton & Kjaernsli (1981). This testing procedure mimics a shear test on a scale appropriate for assessing rockfill, while keeping the normal stress on the sample constant. The test yields data on the angle of failure based on the inclination angle of the box. Nevertheless, there is currently no numerical model available that can replicate the response of this test to simulate various scenarios involving rocks with intermediate D_{50} values and their corresponding trends.

Hence, the objective of this article is to introduce a hyperbolic model for the mechanical characteristics of rockfills. This model enables the representation of the relationship between shear box inclination (θ in degrees) and displacement (δ in millimeters), facilitating the determination of the material's stiffness and its corresponding failure angle. This could be done since, similarly to stress-strain curves, displacement and mobilized friction angle shows a hyperbolic trend, especially due to the existence of an asymptotic limit value (the failure angle). At this stage, the rockfill reaches a constant shear stress as continuous displacement occurs, representing, from a mechanical standpoint, the full mobilization of friction and interlocking between particles.

2. Background

A recommendation suggests that the slope of the riprap should ideally have a ratio of 2:1 under standard conditions. However, a flatter slope may be deemed necessary based on the slope factor of safety, taking into account groundwater percolation or the potential occurrence of rapid drawdown. In situations where steeper slopes exceeding 2:1 are unavoidable, it is advisable to contemplate increasing the average size of the stones and the thickness of the riprap.

Additionally, the angularity of the rocks plays a crucial role; rocks with higher angularity exhibit a greater angle of repose. In cases where rounded stones are utilized, they should be positioned on relatively flat slopes (not surpassing 2.5:1), and the D_{50} size should be augmented by 25%, along with an increase in the riprap thickness (USDA, 1989).

The optimal suggested gradation for riprap involves a continuous gradation curve, where D_{max} equals 1.5 times D_{50} . The minimum diameter may range from 2.5 cm to 60% of D_{50} (Sandroni & Guidicini, 2021). In terms of the riprap layer thickness, it is advised that the thickness falls within the range of 1.2 to 1.7 times D_{50} , with the possibility of greater thickness depending on D_{50} (Sherard et al., 1963).

Geotechnical engineers often use Leps' chart (Leps, 1970) for estimating the friction angle of failure. However, as can be seen in Figure 1, this method is based on the estimation of the parameter using various data from studies on rocks with certain mineralogies, considering the quality of the rock and the confining stress. Nevertheless, it is very important to consider factors such as angularity, shape, degree of saturation, grain size, and even other mineralogies beyond those presented in the study. Besides that, as can be seen in Leps chart, the friction angle influenced by normal stresses. As normal stress increases, the friction angle decreases. Particles breakage at contact can be an explanation for this observation.

Cruz & Nieble (1970) and Midea (1973) conducted research on modified basaltic and gneissic rockfill materials. They determined the strength characteristics through direct shear tests (DS) using boxes of dimensions 20 cm × 20 cm × 20 cm and 100 cm × 100 cm × 40 cm, as well as triaxial tests (TR) on specimens with a height of 10 cm. Midea (1973) reported a decrease in the friction angle of gneiss samples with an increase in fine content. Cruz & Nieble (1970) found that

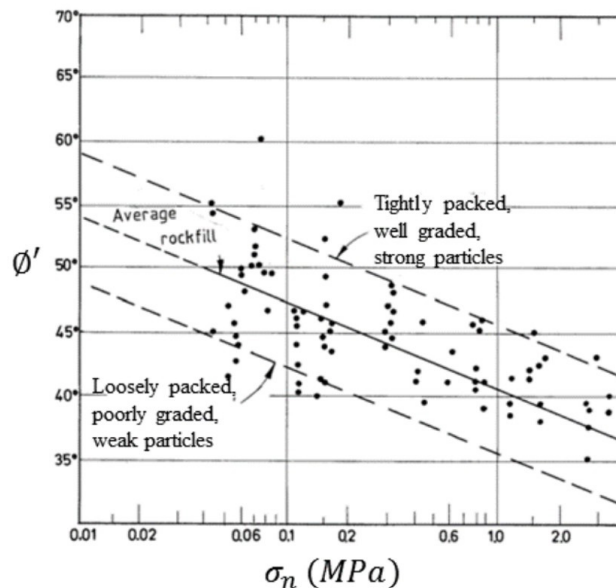


Figure 1. Empirical correlation between the angle of repose vs confining pressure (Leps, 1970).

cycled basalt samples exhibited a reduction of approximately 20%. These findings are summarized in Table 1 below.

Terzaghi (1960) conducted mechanical tests on dry rocks, wet rocks, and saturated rocks. The study concluded that water does not act as a lubricant in rock contacts, leading to similar friction angles in wet and saturated rock masses. However, the presence of water weakens the rock, increasing deformation due to particle breakage and crushing. Additionally, the strength of the blocks is influenced by stress levels, akin to soils in general, and by progressive fracturing, resulting in a reduction of the friction angle. Ultimately, the study demonstrated that rock masses containing angular blocks exhibit greater deformability compared to those with rounded blocks.

However, research on triaxial and direct shear tests carried out on basalt earth fill samples from Marimbondo Dam in Brazil has shown that the shear strength is positively correlated with relative density and negatively correlated with the degree of material weathering. Results from accelerated weathering tests have indicated that the rate of strength reduction due to weathering diminishes over time. Additionally, the strength envelope converges towards a lower limit, representing the fully weathered material (Sayão et al., 2005).

The four primary types of riprap failure identified in the literature are particle erosion, translational sliding, modified slumping, and slumping (Blodgett, 1986). Particle erosion is characterized by the displacement of rock blocks and is typically associated with the use of undersized rocks, impact from debris, or direct flow impingement. Translational sliding, which involves the failure of the revetment parallel to the side slope, is often caused by toe scour and a decrease in the support capacity along the base of the revetment. Modified slumping, where the blanket shifts without toe failure, is commonly triggered by excessively steep slopes. Slumping, a rotational failure, typically occurs on high banks due to the static instability of the protected slopes.

Rock riprap offers erosion resistance by utilizing a blend of stone size and weight, stone durability, and the gradation and thickness of the riprap blanket. The interlocking of angular rocks enhances resistance to the displacement of individual blocks within the revetment. In channel structures, the stability of riprap protections is notably influenced by stream characteristics, as the sizing is interconnected with flow velocity, rock dimensions, and channel dimensions (such as the slope of the banks, slope of the bed, and area) (Fracassi, 2017).

Table 1. Type of altered rocks and parameters (Cruz et al., 2014).

Material	Test	D_{max} (mm)	$\phi^1(^{\circ})^*$	Ref.
Compact crystalline basalt (A) in nature	DS	80	53	Cruz & Nieble (1970)
	DS	50		
	TR	25		
Vesicular and Amygdaloidal Basalt (sample 1)	DS	80	52	Cruz & Nieble (1970)
	TR	40		
		25		
Vesicular and Amygdaloidal Basalt in natura	DS	80	55	Cruz & Nieble (1970)
	TR	40		
		25		
Vesicular and amygdaloidal basalt (sample 2) cycled	DS	80	44	Cruz & Nieble (1970)
	TR	40		
		25		
Vesicular and amygdaloidal basalt (sample 3) cycled	DS	80	43	Cruz & Nieble (1970)
		40		
	TR	25		
Gneiss with 3% to 9% fines	DS	50	50	Midea (1973)
Gneiss with 26% to 49% fines	DS	5	47	Midea (1973)
Gneiss with 100% fines	DS	-	40	Midea (1973)

*Friction angle to $\sigma = 1 \text{ kgf/cm}^2$ ($^{\circ}$).

3. Methodology

The proposed methodology for obtaining the results is based on real-scale studies considering the granulometric variation of tinguaité riprap. The entire executive procedure of the experimental tests is described below.

3.1 Rockfill material

The rockfill material utilized underwent laboratory characterization based on physical and mechanical criteria. The rock was identified as Tinguaité, a type commonly found in the southeastern region of Brazil. Tinguaité is described as a medium- to coarse-grained, unsaturated igneous rock consisting of essential alkali feldspar, nepheline, and aegirine, with or without sodium amphibole or biotite. Tinguaité is considered the hypabyssal equivalent of phonolite (Allaby, 2008).

The rockfill blocks were divided into samples of various diameters. The sample consisting of small blocks ($D_{50} = 150$ mm) is indicative of the transition material located at the base of the dam (refer to Figure 2a). On the other hand, the samples containing medium and large blocks ($D_{50} = 400$ mm and

$D_{50} = 600$ mm, respectively) represent the material forming the upstream and downstream slopes of the dam (Figure 2b). This material is classified as grading E, which is characteristic of a densely graded zone (DNIT, 2012).

Unconfined compression tests were conducted on tinguaité samples in saturated and unsaturated conditions based on ASTM D7012 (ASTM, 2014). The available tests also show that the altered material exhibits greater sensitivity in terms of strength when in saturated conditions. Additionally, the variability in block rupture stress may be related to the variability in the degree of weathering of the rockfill and the mineralogy of the blocks.

As can be seen in Table 2, depending on the condition of alteration and saturation of the rock, the average compressive stresses can vary. The more natural and intact the rock, the higher the ($\sigma_{cm} = 72.82$ MPa). However the more saturated and altered the rock, the lower the value of the compressive stress ($\sigma_{cm} = 23.05$ MPa). It is noted that the material has mechanical strength with a heterogeneous distribution ($CV > 10\%$).

It was found that the studied rockfill blocks exhibited a water absorption capacity exceeding 0.5% (refer to Table 3), a factor that contributes to the acceleration of the material



Figure 2. Grain size distribution of the rip rap: (a) $D_{50} = 150$ mm; (b) $D_{50} = 600$ mm.

Table 2. Compressive strength of the rockfill material.

Type	Condition	Samples	σ_c	σ_{cm}	Δc	CV
			(MPa)	(MPa)	(MPa)	%
Tinguaité intact	Natural	1	53.9	72.83	17.43	24%
		2	88.2			
		3	76.4			
	Saturated	4	80.4	54.30	35.48	65%
		5*	13.9			
		6	68.6			

*With a pre-existing fracture.

Table 3. Characterization of rockfill material.

Samples	Shape index	Dry unit weight (kN/m ³)	Bulk density (kN/m ³)	Water absorption (%)	Wear (%)
BR-PI-05	2.24	22.624	25.076	4.24	20.1
BR-PI-06	2.24	22.536	24.928	4.18	21.6

Table 4. Los Angeles Abrasion tests on rockfill samples data.

Sieve (mm)		Mass (g)	BR-PI-05		BR-PI-06	
			Retained mass (g)	Mass loss (%)	Retained mass (g)	Mass loss (%)
Coarse range	60.3 - 50.0	1500	1463.80	2.41	1462.40	2.51
	50.0 - 38.0	1500	1492.20	0.52	1487.30	0.85
	38.0 - 25.0	1005	980.40	2.45	983.53	2.14
	25.0 - 19.0	495	442.97	10.51	440.20	11.07
	19.0 - 12.7	670	596.70	10.94	571.05	14.77
	12.7 - 9.5	330	251.07	23.92	268.99	18.49
Fine range	9.5 - 4.8	110	107.66	2.13	107.66	2.13
	4.8 - 2.4	110	108.00	1.82	108.00	1.82
	2.4 - 1.2	110	102.90	6.45	102.90	6.45
	1.2 - 0.6	110	106.20	3.45	106.20	3.45
	0.6 - 0.3	110	103.00	6.36	103.00	6.36

weathering process (Pichler, 1942). Regarding the material's resistance to degradation on Los Angeles Machina, only particles with a grain size between 12.7 mm and 9.5 mm showed a mass loss exceeding 18% (see Table 4), which is characterized as "desirable" for rockfill structure construction (Yoshida, 1972). The tests were performed by ASTM C535 (ASTM, 2016) and ASTM C131 (ASTM, 2006).

3.2 Large-scale shear test

The procedure for the large-scale shear test closely followed the methodology proposed by Barton & Kjaernsli (1981). Due to the large size of the rock blocks, the density of the rockfill was not measured directly. Instead, it was controlled by gently placing the blocks into the box without the use of vibrations. This test aimed to assess the roughness of rock joints, specifically the Joint Roughness Coefficient (JRC), without the necessity of parallel grading curves. The test procedure was structured into four main stages. Initially, the apparatus was assembled, comprising two semi-boxes measuring 1.00 m × 2.00 m × 5.00 m (height × length × width). The test was conducted at a lifting rate of approximately 10°/min.

The semi-boxes were overlapped with a 5 cm spacing between them, as illustrated in Figure 3a. This spacing was maintained by screws on the sides, which were subsequently removed before the test. Markings on the external walls of the semi-boxes were used to monitor the displacement between the upper and lower sections during lifting. An angle gauge

with a magnetic base was employed for angle measurements, while photographs taken orthogonally to the box's surface were utilized for displacement readings (refer to Figure 3b).

The second stage involves filling the container with the rockfill sample (refer to Figure 4). Backfilling was executed using a backhoe, where the material was directly deposited by the scoop of the equipment. Three different material samples were examined, differing in the average diameter of the blocks. The samples tested had D_{50} values of 150 mm, 400 mm, and 600 mm, categorized as small, medium, and large-sized blocks, respectively.

The third step involves removing the screws that secure the boxes and commencing the process of lifting the apparatus using a crane. During this step, measurements of the displacement between the boxes and their corresponding inclination angles are taken (refer to Figure 5a). Subsequently, the angle leading to the sample's rupture is identified (refer to Figure 5b).

3.3 Hyperbolic model proposed

The stress-strain response of soil is influenced by a multitude of factors, encompassing density, moisture content, composition, drainage characteristics, deformation conditions (e.g., plane strain or triaxial), loading duration, stress background, confining pressure, and shear stress (Duncan & Chang, 1970).

Initially, the hyperbolic model was developed to replicate the behavior of natural clays (Kondner & Horner, 1965;



Figure 3. Equipment used in the test: (a) crane and the assembled artifact; (b) displacement and angulation measuring.



Figure 4. Filling the shear box with the rockfill material: (a) filling in progress; (b) filling completed.

Kondner, 1963) and natural sands (Kondner & Zelasko, 1963). This model is applicable for analyzing both drained and undrained soils. Kondner's research suggested that the correlation between deviator stress and axial strain follows a hyperbolic curve that can be defined by specific parameters. By establishing an analogy with the approach proposed by Duncan & Chang (1970) for a direct shear condition, it is possible to describe it through Equation 1. The definition

of symbols used on equations can be found on the List of symbols and abbreviations.

$$\tau = \frac{\varepsilon_s}{\alpha + \beta \varepsilon_s} \quad (1)$$

The partial derivative of the shear stress concerning shear strain in the elastic stage ($\varepsilon_s = 0$) yields Equation 2:

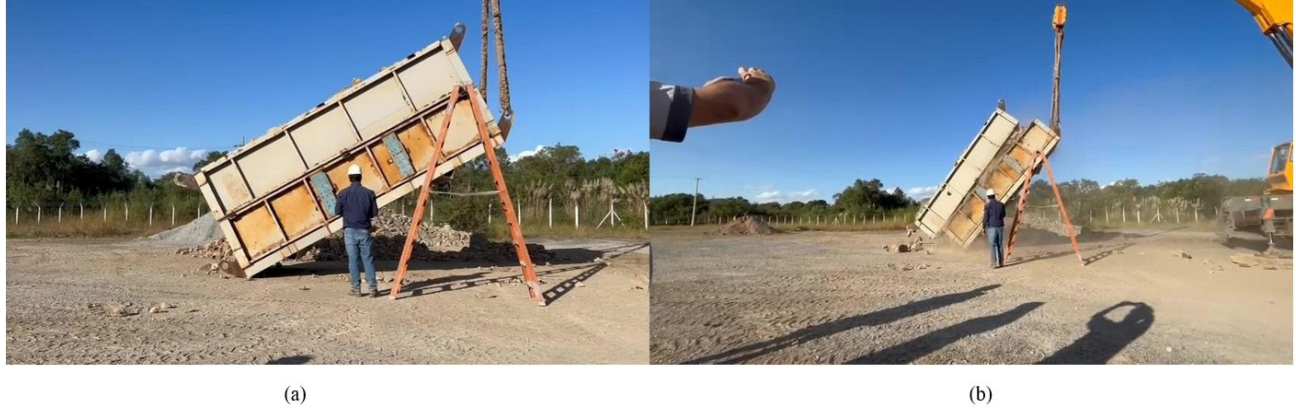


Figure 5. Large scale shear test: (a) during test; (b) final test.

$$\frac{\partial(\tau)}{\partial \varepsilon_s} = \frac{1}{\alpha + \beta \varepsilon_s} - \frac{\beta \varepsilon_s}{(a + \beta \varepsilon_s)^2} = \frac{1}{\alpha} \quad (2)$$

The limit of the shear stress for infinite axial deformation (plastic stage) is equivalently equal to $\frac{1}{\beta}$ (see Equation 3):

$$\lim_{\varepsilon_s \rightarrow \infty} (\tau) = \frac{\varepsilon_s}{\alpha + \beta \varepsilon_s} = \frac{1}{\beta} \quad (3)$$

In the direct shear test, the ultimate shear stress is related to the friction angle at failure (see Equation 4):

$$\varphi_f = \int(\tau) \quad (4)$$

During the test, the rotation of the box allows gravitational forces to mobilize the sliding of the rockfill. In this moment, the contact friction and interlocking of the rockfill create resisting forces. Under the limit equilibrium condition, where the rockfill ($c' = 0$) is in imminent failure ($FS=1$) based on Equation 5 and 6 (Coulomb, 1776). Therefore, when $\theta > \varphi_f$, the box reaches an unstable condition (failure), but when $\theta < \varphi_f$, the box is in a stable condition. At the same time, in the large-scale shear test, there is an approximate relationship between the final box inclination and the rock's friction angle at failure.

$$FS = \frac{\tan \theta_{\text{final}}}{\tan \varphi_f} \rightarrow 1 = \frac{\tan \theta_{\text{final}}}{\tan \varphi_f} \rightarrow \tan \theta_{\text{final}} = \tan \varphi_f \quad (5)$$

So,

$$\varphi_f = \theta_{\text{final}} \quad (6)$$

According to Duncan & Chang (1970), by plotting the stress-strain data using the hyperbolic model, it is simple to determine the values of the parameters α and β corresponding

to the best-fit hyperbolic curve and the test data. Generally, the asymptotic value, represented by the ultimate deviator stress, is slightly greater than the shear strength, since the hyperbola remains below the asymptote at all finite strain values. Therefore, the hyperbolic model can be adapted to a relationship with the shear box inclination angle based on Equation 7. The mobilized friction angle during the test can be considered equal to the shear box inclination.

$$\theta = \frac{\delta}{\alpha + \beta \delta} \quad (7)$$

The α parameter influences the relationship between the box inclinations and low shear displacements. The β parameter directly influences the failure inclination (high shear displacements) of the box and, therefore, the failure friction angle (φ_f).

Figure 6 illustrates the proposed hyperbolic model with its respective parameters α and β . Through this graph, it is possible to observe the non linear correlation between φ_f and δ and the best fit of the calibration parameters. Ultimately, the model allows the normalization of the behavior between the rupture angle and the displacement according to Equation 8:

$$\frac{\theta}{\varphi_f} = \frac{\frac{\delta}{\delta_{\text{max}}}}{a_n + \beta_n \frac{\delta}{\delta_{\text{max}}}} \quad (8)$$

4. Results and discussions

In Figure 7, it can be observed that the experimental data for the box inclination vs. displacement relationship showed a hyperbolic trend regardless of the D_{50} value, such that the proposed hyperbolic mathematical model accurately described the large-scale shear test results. The inclination angle of the box is equal to the friction

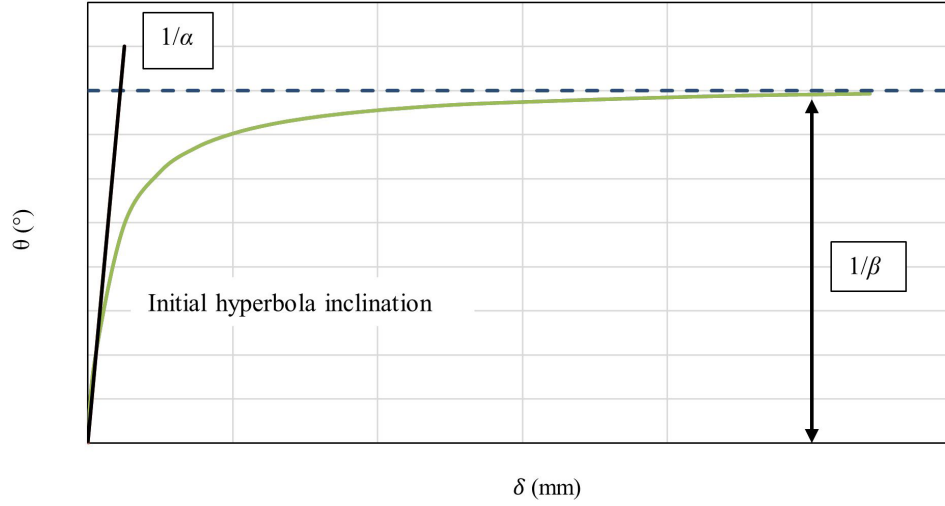


Figure 6. Hyperbolic model proposed.

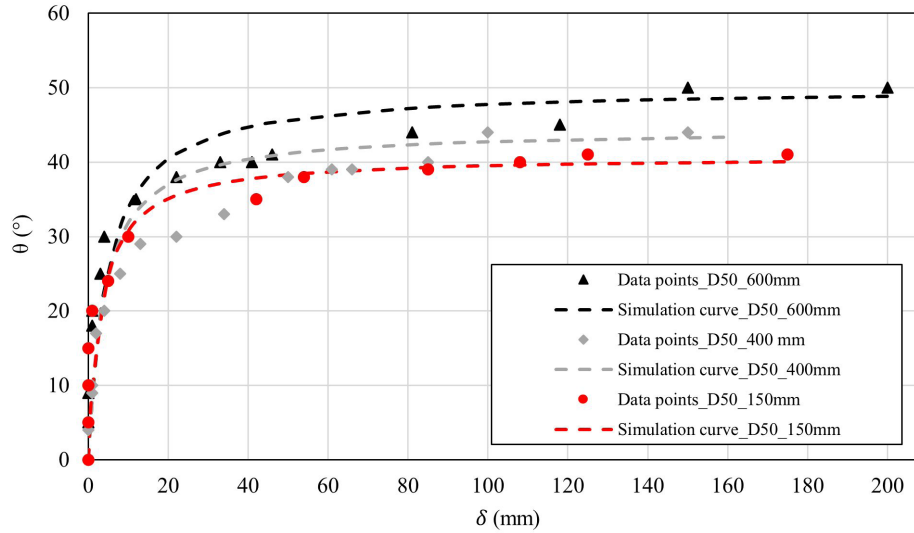


Figure 7. Relationship between the riprap's angle of repose and displacement in the shear plane.

angle mobilized during the test until reaching the failure condition, which is related to the constancy of the angle at large displacements.

It can be stated that as the D_{50} value increased, the angle of failure also increased. This behavior relates to the weight-to-density ratio of the rockfill blocks, such that a higher ratio tends to result in greater inertia caused by shear strength (friction between the rocks).

Figure 8 perfectly illustrates the relationship between D_{50} and φ_f . An excellent linear correlation between these parameters can be observed, characterized by $R^2 = 0.94$. Equation 9 mathematically reproduces this relationship:

$$\varphi_f (^\circ) = 0.0197D_{50} + 3.459 \quad (9)$$

Finally, when evaluating the values of parameters α (mm/°) and $1/\beta$ (°), it was observed that there is a linear correlation between them as a function of D_{50} (see Figure 9). Therefore, it is possible to simulate θ vs δ curves for intermediate values, and thus, it would be possible to estimate possible displacements as well as other failure angles. Equation 10 describes the linear correlation with excellent quality ($R^2 = 0.962$):

$$\alpha = 0.0101\left(\frac{1}{\beta}\right) + 0.0479 \quad (10)$$

As can be seen in Figure 10, it was possible to normalize the behavior of the curves using Equation 11. From the calibration of the $a_n = 0.03$ and $\beta_n = 1$ parameters, the test responses were normalized relative to the maximum rupture values. The curve was able to present the response of

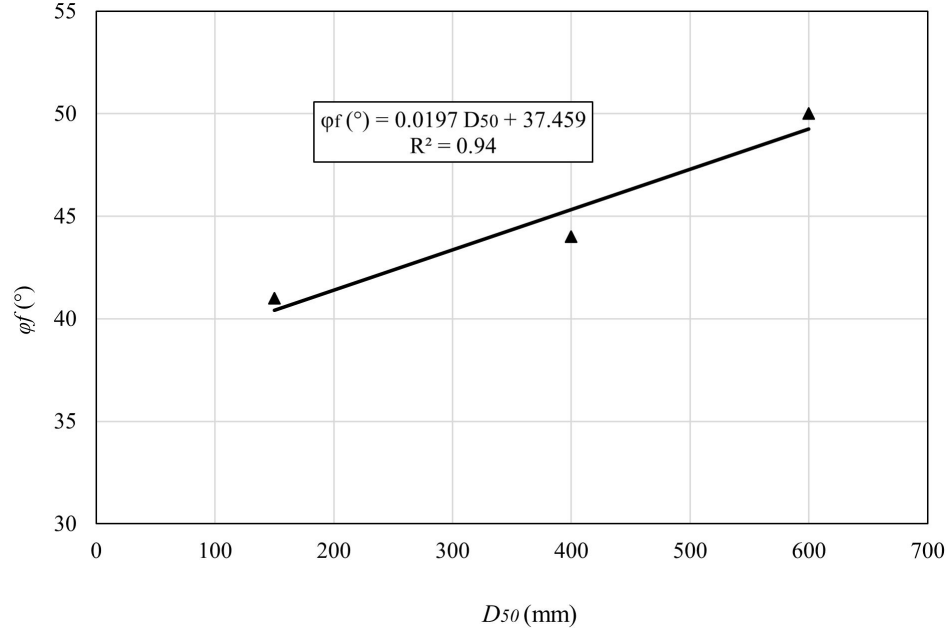


Figure 8. Relationship between the riprap's angle of repose and average diameter of the riprap.

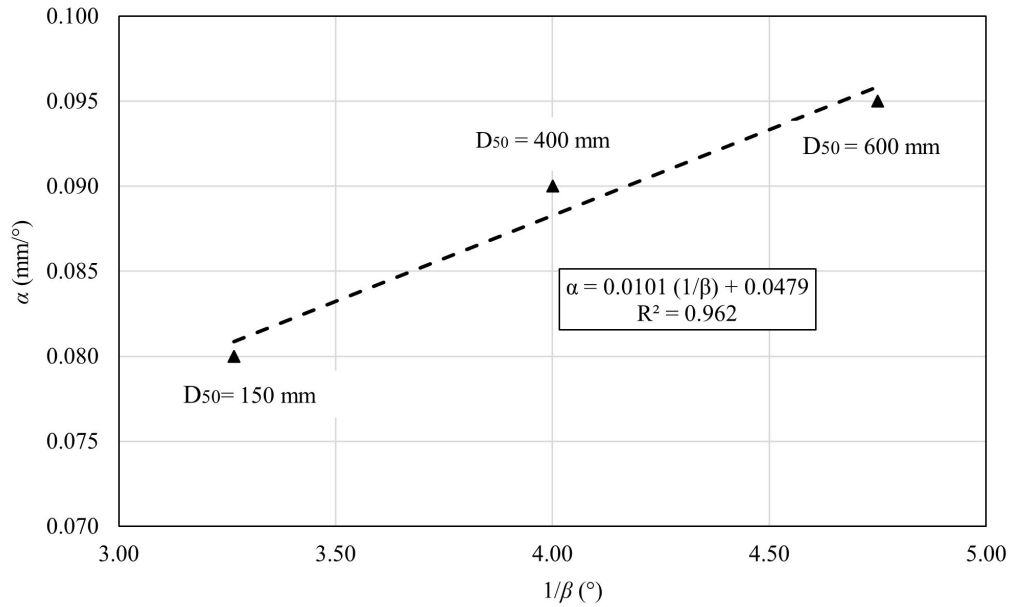


Figure 9. Relationship between calibration parameters of the hyperbolic model.

different rock samples with different D_{50} with low dispersion and simplicity, making it suitable for estimating rocks with intermediate dimensions but with similar characteristics and properties (density, shape factor, mineralogy, etc.).

$$\frac{\theta}{\varphi_f} = \frac{\frac{\delta}{\delta_{\max}}}{0.03 + 1 - \frac{\delta}{\delta_{\max}}} \quad (11)$$

The analysis of the curves presented, especially in Figures 7 to 10, provides a clearer understanding of the influence of the α and β parameters on the mechanical behavior of the rockfill. The α parameter is associated with the initial stiffness of the material, affecting the slope of the curve at small displacements, while the β parameter governs the behavior at large deformations and is directly related to the failure angle. By comparing Figure 8 and Figure 9, it can be seen that as the α parameter increases,

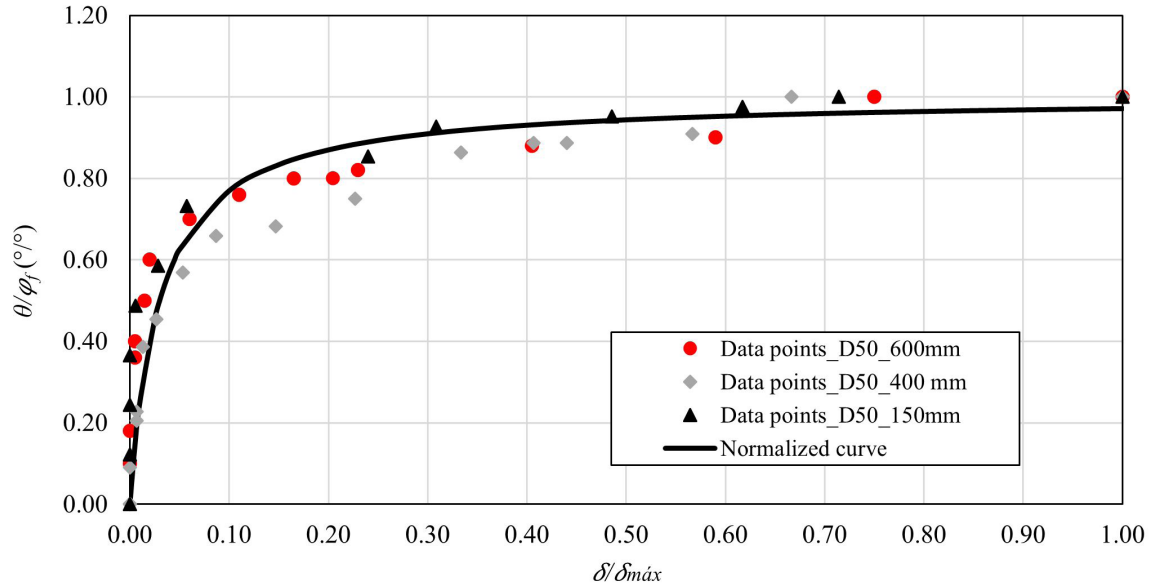


Figure 10. Normalization curve of the mechanical behavior of rockfills.

the value of $1/\beta$ (Equation 10) increases. This trend is also associated with D_{50} and friction angle.

The hyperbolic shape of the curves reflects the gradual transition from the initial elastic regime to the failure state, accurately characterizing the material's nonlinear behavior. Furthermore, the variation in D_{50} proved to be a determining factor in the overall performance of the rockfill: samples with higher D_{50} values exhibited greater failure angles, which can be attributed to the increased inertia and internal friction between the blocks due to their greater mass and interlocking. These observations have direct practical implications for the design of rockfills, suggesting that controlling D_{50} can be an effective strategy to optimize the stability of slopes and rockfill-based structures.

5. Conclusions

This article presents a method for estimating the angle of shear rupture through large-scale shear tests on tinguaita rockfill. During the research, it was observed that the hyperbolic model was able to accurately reproduce the variation in box rotation relative to relative displacement with excellent adherence, and it was also possible to normalize the behavior using a single hyperbole. Regarding the behavior of the proposed model, some aspects can be stated below:

- The hyperbolic model effectively and concisely replicated the mechanical response of the rockfill in tilt tests. The suggested mathematical model demonstrated a strong correlation with the experimental findings of the θ (°) vs δ (mm) relationship derived from the extensive shear tests introduced by Barton & Kjaernsli (1981);

- The relationship between the failure friction angle (φ_f) in degrees and mean stone size of the rockfill (D_{50}) was found to be linear. Specifically, as the value of D_{50} increased, the angle of failure also increased;
- The parameters α and β , when appropriately calibrated, exhibited a linear relationship among themselves and with the D_{50} value. A decrease in the D_{50} value corresponded to a increase in the α value and an increase in the value of $1/\beta$;
- Through the proposed hyperbolic model, it is feasible to replicate the mechanical response of rockfill particles of intermediate sizes, given that the parameters α and β are appropriately calibrated;
- Upon assessment of the tests, the test results were normalized according to the ratio $\delta/\delta_{\max}$;
- This research was conducted based on a limited number of tests and unique normal stress. Although relevant conclusions were found, they may be limited and could differ for other untested materials and normal stresses.

Acknowledgements

The authors acknowledge the support of CNPQ (National Council for Scientific and Technological Development) and CAPES (Coordination for the Improvement of Higher Education Personnel) for funding research in Brazil.

Declaration of interest

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.

Authors' contributions

Tennison Freire de Souza Junior: conceptualization, methodology and writing - original draft preparation. Sidnei Helder Cardoso Teixeira: investigation, data curation, reviewing and validation. Alessandro Christopher Morales Kormann: writing - reviewing and editing.

Data availability

The data supporting the analyses and results of this study are available from the corresponding author upon request and as needed.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

α	Stiffness index (mm/°);
β	Asymptotic index (σ^{-1});
α_n	Normalized Stiffness index;
β_n	Normalized Asymptotic index;
γ	Specific weight of the rock (kN/m ³);
Δc	Standard deviation;
δ	displacement (mm);
δ_{max}	Max. displacement (mm);
ε_s	Shear deformation (%);
θ	Rotation box angle (°);
θ_{final}	Final Rotation box angle (°);
φ_f	Failure friction angle (°);
φ^1	Friction angle to $\sigma = 1$ kgf/cm ² (°);
σ_c	Compressive strength (MPa);
σ_{cm}	Mean compressive strength (MPa);
σ_n	Normal stress (kPa);
τ	Shear stress (kPa);
CV	Variation coefficient;
c'	Cohesion intercept (kPa);
Z	Height of the half-box (m);
D_{50}	Mean stone size (mm);
$D_{m\acute{a}x}$	Maximum diameter (mm);
DS	Direct shear test;
TR	Triaxial compression test;
JRC	Joint Roughness Coefficient.

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REVIEW ARTICLES

Soils and Rocks
v. 48, n. 2

A literature review about the deployment of accelerometers as non-seismic soil landslide tilting sensors

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Review Article

Keywords

Geotechnical monitoring
Landslide early warning system
Time-of-failure prediction
Tilt angle

Abstract

Monitoring is an element of landslide early warning systems. Fukuzono's method, based on displacement and velocity monitoring, is one of the most popular methods for slope time of failure (TOF) prediction. This critical literature review aimed to systematically analyze the state-of-the-art accelerometers used for slope monitoring and TOF prediction. Two peer-review online article platforms were used to build a database of 50 articles. Time history analysis showed that from 2014 onwards the interest in accelerometers has increased and using them as tilt sensors has become a well-accepted approach for geotechnical monitoring. However, special attention must be paid to calibration and noise reducing of low-cost sensors. Also, the installation setup may influence measurements. Alert thresholds and TOF prediction methods based on tilting rates for sensors buried at shallow depths have been proposed and few real validated cases are reported. These predictions attempt to find a simple relation between TOF and slope tilting rate, similar as Fukuzono's method. Relations between tilting rate and surface displacement is an aspect that remains unclear. In order to clarify those aspects, more real slope cases should be reported. A more established use of accelerometers is as a chain of sensors buried in a borehole, similar to a permanent inclinometer. In these cases, accelerometer data are interpreted as displacement, in a more traditional way.

1. Introduction

Landslide forecasting is one of the key elements of Landslides Early Warning Systems (LEWS), which, in turn, are powerful tools for disaster risk reduction (United Nations Office for Disaster Risk Reduction, 2023). According to Intrieri et al. (2019), landslide forecasting comprises the prediction of the slope failure in spatial or temporal terms. The temporal prediction aims at determining the time-of-failure (TOF) and can be performed in global/regional or slope scales, depending on the monitored parameters. LEWS based on rainfall monitoring, as presented by Stähli et al. (2015), Piciullo et al. (2018), Pecoraro et al. (2019) and Guzzetti et al. (2020), work well to evaluate the likelihood of potential landslide in large areas (global/regional scales) but they are not effective to predict failure of an individual slope (Xie et al., 2020b). Slope-scale predictions are based on geotechnical monitoring related to displacement and its derivatives: velocity and acceleration (Intrieri et al., 2019).

One of the most used TOF prediction methods is the inverse velocity method, developed by Fukuzono (1985), which is a simple graphical approach to estimate TOF. It is based on the variation of the inverse velocity ($1/v$) along time. Extensometers, inclinometers, global positioning system (GPS) or robotic total station and interferometric techniques, such as ground-based interferometric synthetic aperture radar (GB-InSAR) and light detection and ranging (LiDAR), have been successfully used for displacement monitoring and landslide TOF forecasting with the inverse of velocity method (Federico et al., 2019; Loew et al., 2016; Ju et al., 2020; Zhang et al., 2020a).

However, the price of an entire monitoring system with most of those techniques/instruments is high and can be unacceptable for wide-scale application. Additionally, instruments such as extensometers require long cable connections, which constitute an inconvenient for installation and maintenance, increasing the cost even more (Uchimura et al., 2015; Qiao et al., 2020). A field of expertise where slope

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Submitted on March 26, 2023; Final Acceptance on March 6, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.003023>



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monitoring and landslide forecasting is needed is civil protection. However, available budgets in this sector are usually smaller than in the mining industry. Therefore, when it comes to risk-management strategies for civil protection, monitoring systems may represent an unaffordable cost and developing low-cost monitoring solutions is urgent.

The advances in Internet of Things (IoT) and wireless sensor network, data transmission and real-time monitoring enhanced cost-effective solutions for landslide monitoring, with reliable results (Ooi et al., 2014; Xie et al., 2019; Abraham et al., 2020a; Wang et al., 2022a). Moreover, the development of microelectronic technique made it possible to manufacture new low-cost sensors using micro electromechanical systems (MEMS) technology. MEMS sensors have the additional advantages of being smaller and lighter, as compared to traditional geotechnical monitoring systems. In this sense, using MEMS accelerometers stands out as a powerful solution for low-cost LEWS. According to Uchimura et al. (2010, 2015) and Sheikh et al. (2021) installation and maintenance of those sensors are easier than extensometers, for example, which results in lower costs. Additionally, the use of low-cost sensor networks allows installation of more sensors for effective slope monitoring (Abraham et al., 2020b).

Araújo et al. (2023) pointed out that originally digital instruments or the digitalization of traditional instruments have been the main tools for improvement of slope monitoring methods in recent years. MEMS Accelerometers show great potential to be used in LEWS. However, there is no recent work in the literature that systematically explains the application of accelerometers for slope monitoring as well as the advantages and limitations of the technique. This paper aims to fill this gap.

Accelerometers have been used for many purposes over the last 20 years: for structural health monitoring (Fukao et al., 2016; Xiao et al., 2016; Chen et al., 2021), for seismicity monitoring (Coccia et al., 2010; Wasowski et al., 2011; Tu et al., 2013; Del Gaudio et al., 2014, 2019) and for geotechnical monitoring, such as rockslides, rock falls and debris flows monitoring (Enet et al., 2003; Xu et al., 2011; Harding et al., 2014; Hu et al., 2018; Feng & Zhuang, 2021). In this paper we focused on the use of accelerometers for non-seismic soil landslide monitoring. In this type of use, measured acceleration is interpreted as tilt or inclination and these sensors are sometimes called “tilt sensors” or “tiltmeters”.

We present a critical literature review aiming at assessing the state of art of using accelerometers to detect soil landslide impending failure signals in non-seismic situations. The literature reviewed is limited to slides in soil, which involve clay and silt rotational and planar slides, gravel, sand and debris slides and clay and silt compound slides, as defined by Hungr et al. (2014). The term “landslide” used in this paper refers to these types of movement. The focus was placed on answering the following research questions: how

have accelerometers been recently deployed in the laboratory and in field for slope monitoring, how has the data acquired by these sensors been interpreted in terms of slope stability and where are these new techniques being used?

2. Construction of the literature database

Articles were searched in two peer-reviewed online article platforms to build the literature database: Web of Science (Clarivate Analytics) and Scopus. Seven keywords were chosen and divided in three categories to perform the search: sensor type (“Accelerometer”, “Tilt sensor”, “Tiltmeter”), object of study (“Landslide”, “Slope”) and final purpose (“Monitoring”, “Early warning”). No time period constraints were used in the search.

All possible combinations of one word of each category using Boolean criterion “AND” were applied to “title”, “abstract” and “keywords” of the publications in the search engines of the platforms and a preliminary database of 142 complete articles was obtained. The analytic procedure shown in Figure 1 was followed to filter the article database.

The first step was to analyze the article’s title and keywords in order to make a preliminary division of our database to define the stress condition in which sensors were used. Dynamic stress articles group gathers articles in which sensor devices were used to monitor high and sudden frequency accelerations, as in high seismicity environments, whereas static stress articles assemble articles in which those sensors were used in low frequency and long-term vibrations context.

By reading all article’s abstracts it was possible to classify them in three groups:

- Seismicity monitoring articles: in this group, sensors were used to monitor seismicity, in a similar way to seismometers. These 44 articles are typically related to landslides triggered by earthquakes. Sensors were not used to monitor soil movement but seismic activity or buildings structural integrity;
- Sensor network deployment and data processing: refers to the 27 articles that are focused on network architecture and processing procedure and mathematical procedures for data analysis;
- Geotechnical monitoring articles: this group gathers 75 articles which describe situations in which sensors are used as slope instrumentation to monitor soil kinematics.

The conclusions of all 75 geotechnical monitoring articles were read in order to analyze how accelerometers are being used for geotechnical monitoring purposes. After this step the 25 articles that described other landslide types monitoring cases, such as debris flows, earth flows, rock falls or submarine landslides, were excluded. The remaining 50 articles were classified as landslide monitoring articles and are the object of this paper. All cases in which sensors were used to monitor up to 1m soil depth were considered surface monitoring in this literature review. This includes experimental and real working conditions.

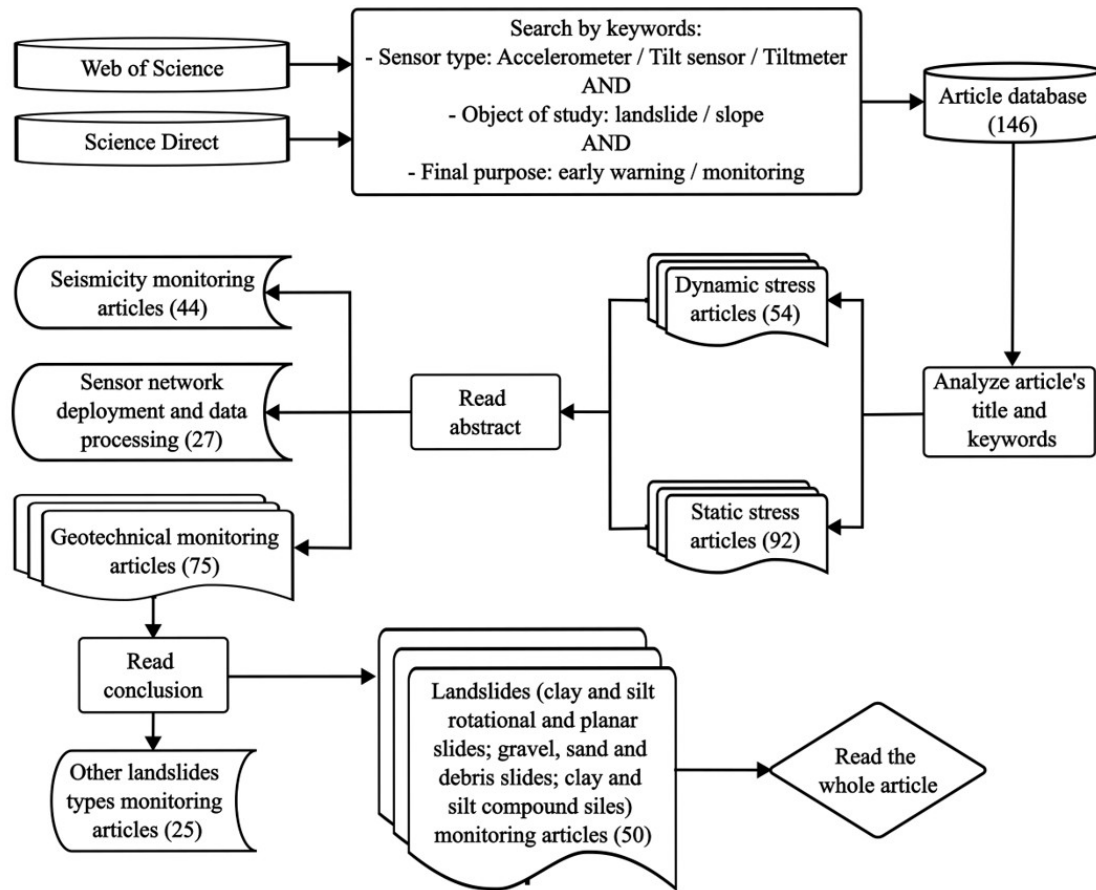


Figure 1. Flowchart of the methodology followed to build the database. Numbers in bracket correspond to the quantity of articles in each group.

3. Literature analysis

3.1 Temporal and geographical analysis

Details of the 50 articles analyzed in detail in this paper are presented in Otero et al. (2024). Of the 50 articles, 14 were published at conferences and 36 were published in peer-reviewed journals. They spread in a 17-year period: from 2006 to 2023, as presented in Figure 2.

Regarding the quantity of articles published per year, between 2006 and 2013 the literature production concerning accelerometers for landslide monitoring remained under two articles per year. In fact, in 2007, 2008, 2011 and 2012 no articles regarding this issue were published. From 2014 to 2022, the rate increased to more than three articles per year (second period). 2018 and 2020 are the years with the highest numbers of publications (7 and 9, respectively).

Between 2006 and 2020, around 30% of the articles published were related to sensor development whereas 70% to sensor deployment and landslide monitoring. After 2020, 20% of the articles were related to sensor development and 80% to sensor deployment and landslide monitoring.

Figure 3 shows the geographical distribution for each approach, based on the first author's affiliation. The research was done in four continents and 14 countries. Most of the articles (35) come from Asia followed by Europe (12), North America (3) and South America (1). Our research did not come up with articles from Africa and Oceania.

Considering the 14 countries where research was made, Japan has the largest number of articles (11) followed by China (9), Indonesia (5), India (4) and Taiwan (3). Those five countries make up almost 70% of the database (32 articles). This denotes that there is a stronger technical interest in Asian countries in using accelerometers for landslide monitoring than in Europe and in America.

According to their main objective regarding landslide monitoring, these 50 articles were divided in two categories: sensor deployment and slope monitoring and prototype development and calibration procedures. Both categories are discussed following.

3.2 Sensor deployment and slope monitoring

This category gathers 37 articles. They were thoroughly and completely analyzed in terms of working scale, installation

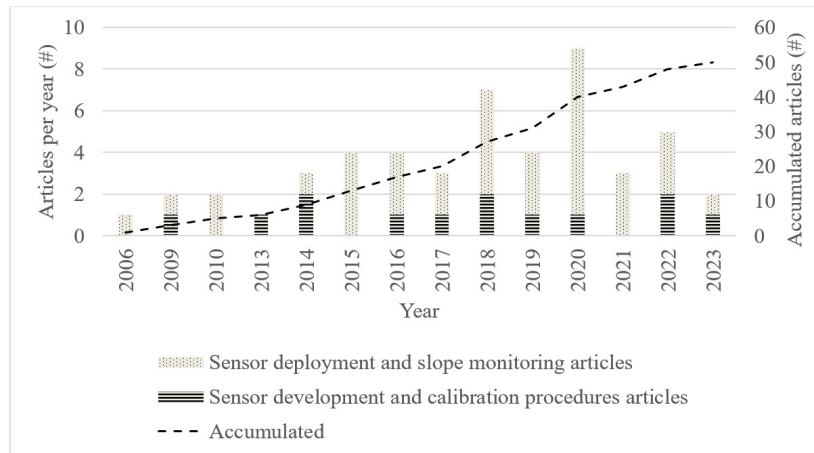


Figure 2. Temporal distribution of the articles in the final database. Vertical bars show number of articles published each year. Dashed line shows accumulated articles during time series.



Figure 3. Geographical distribution of 47 articles of database. Capital letter refers to the main article approach and numbers beside them, to articles quantity regarding each approach. (D) prototype development and calibration procedures and (M) sensor deployment and landslide monitoring.

setup, data acquisition and transmission, associated sensors and time of failure prediction or alert thresholds definition.

3.2.1 Working condition and scale

Articles were divided in terms of working conditions: real or experimental. In case of experimental conditions,

two situations were identified: reduced scale experiments in flumes and full scales experiments. Table 1 presents details of working conditions and scale.

In experimental conditions, Ooi et al. (2014), Habil et al. (2016), Atmajati et al. (2017), Giri et al. (2018), Krokidis et al. (2018), Xie et al. (2019), Chen & Zhang (2021), Giri et al. (2022) and Otero et al. (2022) worked with laboratory flumes

with dimensions ranging from $0.60 \times 0.20 \times 0.10$ m to $2.00 \times 1.49 \times 0.50$ m (length, width and height/depth, respectively). Uchimura et al. (2010), Feng et al. (2020a), Xie et al. (2020a, b), and Qiao et al. (2020) worked with real scale slope models in laboratory or in the field. In experimental conditions landslides were triggered by simulating natural conditions, such as rainfall infiltration or water table elevation.

In real working conditions, natural or built slopes were monitored under natural weather conditions, like in Chelli et al. (2006), Újvári et al. (2009), García et al. (2010), Wang et al. (2015, 2022a, b), Mendes (2015), Uhlemann et al. (2016), Jeng & Sue (2016), Dikshit et al. (2018), Artha & Julian (2018), Bednarczyk (2018), Dikshit & Satyam (2019a, b), Abraham et al. (2020a, b), Sheikh et al. (2020, 2021) and Putra et al. (2021),

In papers such as Uchimura et al. (2015), Towhata et al. (2015), Wang et al. (2017) and Xie et al. (2020a, b) the authors worked firstly in experimental conditions and then they validated their results in natural slopes that showed signs of impending failure.

3.2.2 Installation setup

Accelerometers were used to monitor surface or subsurface movements. Surface monitoring was more common than subsurface monitoring (29 articles against 5 articles of available data), as presented in Table 1. Some research groups attach the sensor to rods or install them into tubes, while other groups placed directly over the surface or embedded the sensor directly in the soil at shallow depths (less than 0.3 m), as presented in Table 2.

In reduced models, accelerometers were simply leaned over the surface soil, as in Giri et al. (2018, 2022), Feng et al. (2020a) and Chen et al. (2021), or embedded into the soil up to 30cm depth. Feng et al. (2020b), Xie et al. (2020a, b) and Otero et al. (2022) embedded sensors directly into the soil at shallow depth (from 3 cm to 20 cm). Habil et al. (2016) and Atmajati et al. (2017) embedded sensors at 20 cm and 30 cm respectively using Polyvinyl Chloride (PVC) pipes. Uchimura et al. (2010) and Qiao et al. (2020) attached sensors at the top of up to 30cm length rods.

In real scale models and in natural slopes the most common way to deploy the sensors is using steel rods. Some examples of this type of deployment can be found in Uchimura et al. (2015), Wang et al. (2017, 2022b), Dikshit & Satyam (2019b) and Xie et al. (2020a, b). Measuring devices are attached at the top of the rod that is inserted into the soil at 50 cm to 150 cm depth. Hence, if the soil experiences some movement, the rods move together, and this is measured by the devices installed at the top of the rod.

In the cases of subsurface monitoring, sensors are deployed in boreholes in real slope conditions. Újvári et al. (2009), García et al. (2010) and Mendes (2015) installed sensors at 3m depth with PVC pipes filled with granular soil, in order for the sensor to be strongly coupled to the ground.

Uhlemann et al. (2016) and Bednarczyk (2018) also deployed sensors in subsurface conditions, similarly to inclinometers. Uhlemann et al. (2016) deployed a commercial solution, developed by Abdoun et al. (2006, 2007), called “Shape acceleration array (SAA)” at 2.5 m depth and Bednarczyk (2018) used strings of rigid segments attached to each other, up to 16 m depths. Each segment contains one measuring device and the distance between them is 50 cm. Strings and SAA were installed inside PVC flexible pipes.

Segalini et al. (2014, 2015, 2019) deployed a system called Modular Underground Monitoring System (MUMS) that consists in an acceleration sensor chain up to 111 m. These papers were not retrieved in the literature search, because this group does not use the terms “Accelerometer”, “Tilt sensor” and “Tiltmeter” in their articles, but the MUMS are clearly an array of accelerometers deployed in subsurface conditions, similarly to inclinometers. For this reason, the papers produced by this group are discussed in this paper.

3.2.3 Data acquisition and transmission

Data acquisition rate was available in 23 articles (Table 1). The most common acquisition frequency used is 1 per ten minutes. This acquisition frequency was first reported by Uchimura et al. (2010) and the other works that used the same frequency are from the same research group (for example, in Towhata et al., 2015; Wang et al., 2015, 2022b; Xie et al., 2019) or are works based on Uchimura et al. (2010) tilting monitoring proposal, as in Dikshit & Satyam (2019a, b) and Putra et al. (2021).

In researches developed exclusively in experimental conditions frequency rates reported are: 1/60 Hz (every ten minutes) (Uchimura et al., 2010; Qiao et al., 2020), 1 Hz (Xie et al., 2020b), 30 Hz (Giri et al., 2022), 2500 Hz (Feng et al., 2020b), 4000 Hz (Otero et al., 2022) and 8000 Hz (Krokidis et al., 2018) whereas in real slope monitoring conditions frequency rates reported are 1 Hz, 1 per minute (García et al., 2010), 1 per ten minutes Hz (Uchimura et al., 2015; Towhata et al., 2015; Wang et al., 2015; Dikshit & Satyam, 2019a, b; Abraham et al., 2020a, b; Sheikh et al., 2020; Xie et al., 2020b; Putra et al., 2021), 1 per hour (Újvári et al., 2009; Mendes, 2015; Uhlemann et al., 2016), 1 per 6 hours (Bednarczyk, 2018) and 2 readings per day (Chelli et al., 2006). We hypothesize that higher acquisition rate is used in experimental conditions probably because in these conditions there are no constraints regarding power supply and data storage.

The highest acquisition rates were used in experiments that aimed to find a frequency signature for landslides and micro-cracks development (Krokidis et al., 2018; Feng et al., 2020a). In those papers data were analyzed in frequency domain, similar to Feng et al. (2020b), although they do not present acquisition rate information in this last work. For the other cases, data were analyzed in time domain, which means acceleration or tilting variation along time. Otero et al. (2022)

Table 1. Summary of working conditions and scale, installation setup, acquisition frequency, measured parameter and associated sensors in each reference.

	Reference	Scale	Dimensions - Height (m) x length (m) x width (m)	Installation setup	Acquisition frequency	Measured parameter	Associated sensors
Real conditions	Chelli et al. (2006)	Natural slope	NM	NM	2/day	Tilt	Jointmeters, inclinometers, incremental extensometers, piezometers, rain gauges
	Újvári et al. (2009)	Natural slope	58.97 x 222 x 30	Subsurface	1/hour	Tilt	GPS pillars. Rainfall and water level from stations at other locations
	Uchimura et al. (2010)	Model slope and natural slope	Model: 0.5 x 6 x 1.5	Surface	Every 10minutes (real slope)	Tilt (Rotation (mm/m))	Volumetric water content
	García et al. (2010)	Natural slope	NM	Subsurface	1 per minute	Tilt	Pressure transducer (groundwater and pore pressure), thermistor, rainfall from station 2km away
	Uchimura et al. (2015)	Model slope, real slope (failure was forced) and natural slope (real unstable case)	NM	Surface	Every 10minutes	Tilt	Volumetric water content; geomagnetic sensor (tilt direction); extensometer (for comparison)
	Towhata et al. (2015)	Real slope (failure forced) and natural slope (real unstable case)	NM	Surface	Every 10 minutes	Tilt	Volumetric water content (optional)
	Wang et al. (2015)	Natural slope (real unstable cases)	NM	Surface	Every 10 minutes	Tilt	Volumetric water content
	Mentes (2015)	Natural slope	NM	Subsurface	1/hour	Tilt	Temperature
	Uhlemann et al. (2016)	Natural slope (landslide complex that is reactivated from time to time)	NM	Subsurface	1/hour (SAA and tiltmeter)	Deformation/ Tilt	Real time kinematics GPS (RTK-GPS), inclinometers, active waveguides with AE, piezometers
	Jeng & Sue (2016)	Natural slope	NM	NM	NM	Tilt	Inclinometer, crack gages, groundwater level wells, settlement and displacement observation marks, rebar strain gages, concrete strain gages, rain gages
	Wang et al. (2017)	Model slope, real slope (failure was forced) and natural slope (real unstable case)		Surface	NM	Tilt	Volumetric water content, Inclinometer, extensometer
	Dikshit et al. (2018)	Natural slope	NM	Surface	NM	Tilt	Volumetric water content
	Artha & Julian (2018)	Natural slope	NM	Surface	NM	Tilt	Inclinometer, rainfall sensor
	Bednarczyk (2018)	Natural slope	NM	Subsurface	Conventional: every 1-2 months and then 3-6 months. MEMS tilt sensor every 6h	Tilt	Inclinometer, piezometer, weather station, pore-pressure transducers
	Dikshit & Satyam (2019a)	Natural slope	NM	Surface	Every 10 minutes	Tilt	Volumetric water content
	Dikshit & Satyam (2019b)	Natural slope	NM	Surface	Every 10 minutes	Tilt	Volumetric water content
	Abraham et al. (2020a)	Natural slope	NM	Surface	Every 10 minutes	Tilt	Volumetric water content
	Abraham et al. (2020b)	Natural slope	NM	Surface	Every 10 minutes	Tilt	Volumetric water content
	Sheikh et al. (2020)	Cut slope	30 x 60 x 100	Surface	Every 10 minutes	Tilt	Water level and rainfall gauges; pipe strain gauges
	Sheikh et al. (2021)	Natural slope	30 x 60 x 100	Surface	Every 10 minutes	Tilt	Water level and rainfall gauges; pipe strain gauges
	Putra et al. (2021)	Natural slope	NM	Surface	Every 10 minutes	Tilt	Water level, in situ investigations
	Wang et al. (2022a)	Natural slope	NM	Surface	NM	Tilt (surface)	NM
	Wang et al. (2022b)	Natural slope, cut slope, landfill	NM	Surface	Every 10 minutes	Tilt	Inclinometers, water level; volumetric soil water content

NM stands for “Not Mentioned”.

Table 1. Continued...

		Reference	Scale	Dimensions - Height (m) x length (m) x width (m)	Installation setup	Acquisition frequency	Measured parameter	Associated sensors
Experimental conditions	Laboratory flumes	Ooi et al. (2014)	Reduced model (Flume)	0.60 x 1 x 0.452	Surface	NM	Acceleration	Porewater pressure transducer
		Habil et al. (2016)	Reduced model (Flume)	0.40 x 0.80 x 0.80	Surface	NM	Acceleration	Moisture sensor (FC-28-C)
		Atmajati et al. (2017)	Reduced model (Flume)	0.40 x 2 x 0.45	Surface	NM	Tilt	Soil moisture sensors
		Giri et al. (2018)	Reduced model (Flume)	0.30 x 1.83 x 1.49	Surface	NM	Acceleration	Pi-cameras
		Krokidis et al. (2018)	Reduced model (Flume)	0.08 x 0.58 x 0.56	Surface	8000 Hz	Acceleration	NM
		Feng et al. (2020a)	Reduced model (Flume)	0.60 x 15 x 0.60	Surface	2,5 kHz	Applied acceleration response	Self potential; piezometer; soil water content sensors; Cameras
		Chen & Zhang (2021)	Reduced model (Flume)	0.20 x 0.70 x 0.30	Surface	NM	Tilt	Shear wave and moisture transducers
		Giri et al. (2022)	Reduced model (Flume)	0.30 x 1.83 x 1.49	Surface	30 Hz	Acceleration (linear)	Pi-cameras
		Otero et al. (2022)	Reduced model (Flume)	0.50 x 1.60 x 0.50	Subsurface	4000 Hz	Tilt	NM
	Different scales	Xie et al. (2019)	Reduced model (flume) and real slope (artificial rainfall)	Flume: 0.38 x 1.165 x 0.45 Real slope: NM	Surface	NM	Tilt	Camera (to measure displacements)
		Xie et al. (2020a)	Model 1: flume; Model 2: flume; Model 3: model slope; Model 4: real slope	Model 1: 0.38 x 1.165 x 0.45 Model 2: 0.40 x 0.60 x NM Model 3: 2 x 7.4 x 3.8 Model 4: NM	Surface	NM	Tilt	Camera (flume); Extensometers (field)
		Feng et al. (2020b)	Model slope (in field)	NM	Surface	NM	Applied acceleration response	Microphones
		Xie et al. (2020b)	Reduced model (flume) and model slope (artificial rainfall)	Flume: 0.38 x 1.165 x 0.45 Field: NM	Surface	1Hz (laboratory) and 1/60Hz (real slope)	Tilt	NM
		Qiao et al. (2020)	Small scale (flume, rainfall triggered and inclination variation triggered) models and field tests	Flume: 0.38 x 1.165 x 0.45 Field: NM	Surface	1/60Hz	Tilt	NM

NM stands for "Not Mentioned".

Table 2. Summary of article quantity regarding installation setup (when reported in the text).

	Installation setup			
	Directly over surface or shallow depth	Attached		Sensor chain (up to 16 meters depth)
		Up to 1.5m length	More than 1.5m length	
Article quantity	11	14	3	2

also used high acquisition rates in order to verify if there were any impulsive signals during failure (that were not found in their experiments with laboratory flumes).

Regarding data transmission, two ways are reported in the literature: wire connected directly to a data logger or via wireless transmission. In this last case, the sensor is connected to a microcontroller equipped with a communication

interface which sends data to a gateway via wireless connection. 28 articles showed information regarding this topic: in 10 cases data were transmitted with wires to a datalogger and in 18, via wireless transmission. Data transmission with wires is more common in experimental conditions in laboratory whilst in real conditions wireless transmission is more frequent.

3.2.4 Measured parameter

Most of the analyzed papers focused on measuring rotational movements (tilting angles): Chelli et al. (2006), Újvári et al. (2009), García et al. (2010), Uchimura et al. (2010, 2015), Ooi et al. (2014), Towhata et al. (2015), Mentés (2015), Uhlemann et al. (2016), Jeng & Sue (2016), Atmajati et al. (2017), Wang et al. (2015, 2017, 2022a, b), Dikshit et al. (2018), Artha & Julian (2018), Bednarczyk (2018), Xie et al. (2019, 2020a, b), Dikshit & Satyam (2019a, b), Abraham et al. (2020a, b), Sheikh et al. (2020, 2021), Qiao et al. (2020), Chen & Zhang (2021) and Putra et al. (2021).

One concerning issue regarding tilting monitoring is the relation between surface displacement and tilt variation. Uchimura et al. (2010) were the first to highlight that tilting angles may not necessarily imply in surface displacement, but it was only with Xie et al. (2019) that further studies were performed to investigate the relationship between displacement and tilting angle of slope surface in shallow landslides.

Xie et al. (2019) identified a linear relationship in reduced model laboratory experiments between tilting angle and surface displacement. Their findings indicated that tilting behavior could in fact be used in a similar way to slope surface displacement as an indication of shallow landslides. Similar results were found by Xie et al. (2020a) for rotational movements. However, those authors worked only in laboratory conditions.

Wang et al. (2022a) acknowledge that there are still few reported cases on the relationship between landslide behavior and tilting angle. Sheikh et al. (2021) suggest that there is a need for a newly developed time failure prediction model based on tilting behavior for global application. In this sense, future research should be focused on exploring the methodology under different natural slope types and weather conditions.

In the other 8 cases, the measured parameter is acceleration which can be analyzed in terms of time domain (Ooi et al., 2014; Habil et al., 2016; Giri et al., 2018, 2022; Otero et al., 2022) or frequency domain (Krokidis et al., 2018; Feng et al., 2020a, b).

The variation was analyzed in time domain in each sensor axis in order to investigate movement kinematics by Ooi et al. (2014), Habil et al. (2016) and Giri et al. (2018, 2022). Otero et al. (2022) show results in terms of linear acceleration and also tilting estimated from acceleration. Krokidis et al. (2018) attempted to find an acceleration frequency spectrum signature of micro cracks developed before failure and Feng et al. (2020a, b) analyzed the acceleration frequency spectrum to identify a landslide frequency signature.

Segalini et al. (2014, 2015, 2019), Uhlemann et al. (2016) and Bednarczyk (2018) interpreted accelerometers data in terms of displacement, as they used a sequence of sensors in a vertical array, and the result is similar to those acquired from inclinometers.

3.2.5 Aim of sensor deployment

As understanding landslide kinematics is essential to predict future landslides and to establish alert thresholds, in 17 articles, sensors were used to monitor the movement while it was happening in order to qualify the slope deformation: Chelli et al. (2006), Újvári et al. (2009), García et al. (2010), Ooi et al. (2014), Mentés (2015), Habil et al. (2016), Uhlemann et al. (2016), Jeng & Sue (2016), Giri et al. (2018, 2022), Bednarczyk (2018), Xie et al. (2019), Feng et al. (2020a, b), Qiao et al. (2020), Chen & Zhang (2021) and Putra et al. (2021). In some cases, the main objective of the research was to build a geological model or to perform back analysis of previous ruptures or to understand triggers that reactivated movements (Chelli et al., 2006; García et al., 2010; Mentés, 2015; Uhlemann et al., 2016; Bednarczyk, 2018; Putra et al., 2021).

In the other 20 articles, sensors were used for purposes related to LEWS and slope monitoring during impending failure in order to identify pre-failure signals: Uchimura et al. (2010), Uchimura et al. (2015), Towhata et al. (2015), Wang et al. (2015, 2017, 2022a, b), Atmajati et al. (2017) Dikshit et al. (2018), Krokidis et al. (2018), Artha & Julian (2018), Dikshit & Satyam (2019a, b), Abraham et al. (2020a, b), Xie et al. (2020a, b), Sheikh et al. (2020, 2021) and Otero et al. (2022). Hence, research was focused on studying sensors signals before landslides happened. Some articles defined alert thresholds and a few proposed mathematical solutions to forecast TOF based on tilting behavior, as discussed in section 3.2.6.

3.2.6 Time of failure prediction or thresholds definition based on tilting angles

These TOF prediction methods are based on the readings of individual sensors. Uchimura et al. (2010) noticed that 30 minutes before failure the surface showed abnormal tilting behavior that could be used to define thresholds. However, the authors did not propose alert thresholds for early landslide warning systems based on soil tilting monitoring.

Based on that first work, Uchimura et al. (2015) published another article with two tilting rates alert levels, which are precaution (0.01°/hour) and warning (0.1°/hour). Alert thresholds were tested in laboratory reduced model conditions and, after that, validated in real slope conditions. Towhata et al. (2015) used alert threshold value of 0.1°/hour for evacuation and they validated the accelerometers results with extensometers readings in real scale conditions. Alert thresholds proposed by Uchimura et al. (2015) were also reported in Wang et al. (2017, 2022b), Dikshit et al. (2018) and Putra et al. (2021). Dikshit & Satyam (2019a, b) suggested a third alert level at 1°/hour in their case study in the Darjeeling Himalayas. Abraham et al. (2020a) defined alert levels at 0.03°/hour and 0.1°/hour for their case study and they used tilt sensors together with rainfall thresholds in order to achieve a more robust LEWS. Table 3 summarizes

Table 3. Summary of LEWS thresholds based on tilting rates found in the literature.

Reference	Threshold	Conditions	Installation setup	Acquisition frequency	Depth
Uchimura et al. (2015)	Precaution: 0.01°/hour Warning: 0.1°/hour	Laboratory (flume) and real conditions	Surface: steel rod inserted into the soil and subsurface: multi-segment inclinometer	1 per ten minutes	Steel rod: 1 m Multi-segment inclinometer: 0.75 m
Towhata et al. (2015)	Evacuation: 0.1°/hour	Real slope conditions	Surface: steel rod	1 per ten minutes	0.5 – 1.0 m
Wang et al. (2017)	Same as Uchimura et al. (2015)	Real slope conditions	Surface: steel rod	Not mentioned	0.5 – 1.0 m
Dikshit et al. (2018)	Same as Uchimura et al. (2015)	Real slope conditions	Surface: steel rod	Not mentioned	1.0 m
Dikshit & Satyam (2019a, b)	Precaution: 0.01°/hour Warning: 0.1°/hour Alert: 1°/hour	Real slope conditions	Surface: steel rod	1 per ten minutes	1.0 – 1.5 m
Abraham et al. (2020a)	0.03°/hour 0.1°/hour	Real slope conditions	Surface: steel rod	1 per ten minutes	1.0 m
Putra et al. (2021)	Same as Uchimura et al. (2015)	Real slope conditions	Surface	1 per ten minutes	Not mentioned
Wang et al. (2022b)	Same as Uchimura et al. (2015)	Real slope conditions	Surface: steel rod	1 per ten minutes	Not mentioned

tilting rates thresholds suggested in the literature together with experimental conditions, installation setup and sensor depth.

Xie et al. (2020b) pointed out that although tilt measurements were successfully used to establish alert thresholds, as previously elucidated, there was a lack of knowledge regarding failure prediction methods. Therefore, they established a mathematical relation to estimate time of failure based on tilting rate. The method is based on a linear relation between reciprocal tilting rate and time during acceleration stage before failure, similar to the method proposed by Fukuzono (1985) and commonly used for TOF prediction. This method was optimized by Wang et al. (2022a), who validated it in a real case study in China.

In a similar way, Sheikh et al. (2020) used the alert thresholds defined by Uchimura et al. (2015) and additionally they proposed a method for TOF prediction based on tilting behavior. The authors proposed three regression formulas for time-prediction based on ground water table change, tilting rate, rainfall intensity and cumulative rainfall. No information about validation of the method in real conditions is presented.

3.2.7 Time of failure prediction or thresholds definition based on displacements in inclinometers

As stated before, some research groups report the use of a chain of accelerometers inside a borehole to act

as a permanent inclinometer. Most of them report the interpretation of the data collected in terms of failure surface depth definition, which is the most common use for inclinometers.

Segalini et al. (2019) report the use of Fukuzono's inverse of velocity method using displacements calculated from these inclinometers. Authors were able to successfully forecast the slope failure and to activate alert procedures. Another paper published by this group (Valletta et al 2023) proposes a statistical treatment of the observed displacement velocities in order to establish an alert threshold.

3.2.8 Associated sensors

According to Uhlemann et al. (2016) no single technique or monitoring device can provide complete landslide information. Hence, a combination of different techniques should always be employed. For non-seismic landslides, the primary parameters of interest are those related to deformation and pore-water pressure. Information related to those parameters makes it possible to understand movement rate and magnitude and changes in effective stress which, in the end, directly influence slope stability. Table 1 presents associated sensors used in each reference.

In laboratory experimental conditions where landslide simulation tests were performed with flumes, Ooi et al.

(2014) worked with pore pressure transducers, Habil et al. (2016), Atmajati et al. (2017), Feng et al. (2020a) and Chen & Zhang (2021) used soil moisture sensors and Uchimura et al. (2010) used volumetric water content sensors. In this type of experiment, cameras for digital image correlation are the most common technique used to monitor soil deformations (Giri et al., 2018, 2022; Xie et al., 2019, 2020a; Otero et al., 2022). No other sensors for soil deformation were reported in laboratory experimental conditions. Feng et al. (2020b) used microphones together with accelerometers to measure acoustic signals generated by water flow and landslides. Chen & Zhang (2021) used shear wave sensors to establish a relationship between shear wave velocity and tilt variation.

In real scale conditions and considering parameters associated with effective stress monitoring, volumetric water content is the most common parameter measured in association with tilting angles, as in Uchimura et al. (2015), Towhata et al. (2015), Wang et al. (2015, 2017, 2022b), Dikshit et al. (2018), Dikshit & Satyam (2019a, b) and Abraham et al. (2020a, b). Piezometers and water level wells are also commonly used (Chelli et al., 2006; García et al., 2010; Uhlemann et al., 2016; Jeng & Sue, 2016; Bednarczyk, 2018; Sheikh et al., 2020, 2021; Putra et al., 2021; Wang et al., 2022b).

Many authors deployed rainfall gauges in their study areas, such as Chelli et al. (2006), Jeng & Sue (2016), Artha & Julian (2018) and Sheikh et al. (2020, 2021). Other authors used rainfall data from nearby weather stations, as Újvári et al. (2009), García et al. (2010) and Bednarczyk (2018).

In fact, Abraham et al. (2020a) suggest that data from tilt sensors should always be correlated with rainfall and soil moisture data before arriving at any conclusion because they identified abrupt changes in tilting data time series data that were not always related to slope failure.

Regarding soil deformation monitoring, traditional inclinometers are the most used devices (Chelli et al., 2006; Uhlemann et al., 2016; Jeng & Sue, 2016; Wang et al., 2017, 2022b; Artha & Julian, 2018; Bednarczyk, 2018). Other devices used for this purpose are extensometers (Chelli et al., 2006; Uchimura et al., 2015; Xie et al., 2020a), pipe strain gauges (Sheikh et al., 2020, 2021), global position satellite (GPS) techniques (Újvári et al., 2009; Uhlemann et al., 2016), jointmeter (Chelli et al., 2006), active waveguides with acoustic emission (Uhlemann et al., 2016) and crack gauges (Jeng & Sue, 2016).

Magnetometers are reported in Segalini et al. (2011, 2014, 2015), Segalini & Carini (2013), Uchimura et al. (2015) and Giri et al. (2018, 2022). According to these authors, this sensor can be used to identify the direction towards which the slope is tilting. However, none of the articles presents data regarding the direction of the movement based on data provided by magnetometers.

3.3 Prototype development and calibration procedures

This category gathers 13 articles that do not deal directly with slope monitoring: 8 articles focus on developing

prototypes in laboratory conditions and 5 others are related to calibration procedures.

3.3.1 Prototype development

Eight articles are related to prototype development (de Dios et al., 2009; Marciano et al., 2014; Liu & Lei, 2014; Alimuddin et al., 2017; Zhang et al., 2018; Wielandt et al., 2022; Coppola et al., 2022; Freddi et al., 2023).

Five of them present a vertical solution to monitor landslides (de Dios et al., 2009; Marciano et al., 2014; Zhang et al., 2018; Wielandt et al., 2022; Freddi et al., 2023). In terms of deployment, the solution is very similar to inclinometers, with the advantage of working with wireless data transmission, allowing to increase temporal reading resolution when compared to traditional inclinometers. Columns proposed by de Dios et al. (2009), Marciano et al. (2014) and Zhang et al. (2018) consist of 50 to 100 cm sensor segments and each segment is connected to the other with flexible joints. de Dios et al. (2009) and Marciano et al. (2014) solutions measure acceleration, and they also attached soil-water content sensors to the device, whilst Zhang et al. (2018) solution measures displacements. Wielandt et al. (2022) developed a flexible probe with tri-axial accelerometers that measure deformation. Freddi et al. (2023) present a solution similar to in-place inclinometers based on MEMS accelerometer that can reach up to 2 m depth.

Another solution that stands out is the Modular Underground Monitoring System (MUMS) developed by Segalini et al. (2011) that measures underground displacements based on acceleration sensors. MUMS are quite similar to devices developed by de Dios et al. (2009), Marciano et al. (2014), Zhang et al. (2018), Wielandt et al. (2022) and Freddi et al. (2023). Other sensors, such as pore pressure cells, extensometer, load cell, among others, can be accommodated in the system. Sensor nodes are connected to each other along a single cable. Distance between nodes and chain length are defined according to the case. The system was validated in laboratory conditions (Segalini et al., 2011; Segalini & Carini, 2013) and successfully deployed in real slopes (Segalini et al., 2014, 2015, 2019) with lengths up to 111 m.

On the other hand, Zhang et al. (2020b) and Coppola et al. (2022) present different types of solutions. Zhang et al. (2020b) proposed a solution where sensors are placed into a pipe made of spiral steel wire hose. The pipe needs to be buried throughout the unstable slope body to deform together with it. The sensors measure acceleration and, after a mathematical procedure, displacement along the pipe is calculated. The solution was successfully tested in real unstable slope conditions.

Coppola et al. (2022) developed a device named “tension-inclinometer”. The device combines a conventional tensiometer with an accelerometer placed at the top of the tensiometer. The tensiometer measures pore-water pressure

range from -85 kPa to 100 kPa and reaches up to 2 m depth. The accelerometer measures inclination with an accuracy of 0.05°. The device was successfully tested in flume experiments.

3.3.2 Calibration procedures

When considering the use of low-cost sensors for landslide monitoring, Cina et al. (2019) highlight the need to perform calibration procedures to improve accelerometers accuracy and remove bias. According to the authors, systematic errors affect the sensors' performance characteristics. Additionally, Cmielewski et al. (2013) emphasize that it is also fundamental to check sensor stability and repeatability. Hence, Cmielewski et al. (2013) and Cina et al. (2019) present calibration procedures that were used in laboratory conditions. Cina et al. (2019) reported that their proposal improves sensor accuracy by one order of magnitude.

Concerning sensor readings verification, Salam et al. (2016) developed a tilt calibrator to test accelerometers readings. It consists of a tilt motor that provides inclination while accelerometers measure tilt variation.

Another important aspect regarding sensor calibration is related to signal noises. Weerasinghe et al. (2018) point out that even when sensors are at rest readings can be noisy and unstable in long term. The authors used Kalman filters to remove noise components and suggest that only after filtration data can be used for further calculation.

The other 43 articles of the database do not mention any concerns about accuracy, bias, sensor stability, readings repeatability, sensor readings verification or concern with signal noises. Although this issue has not been the main focus of the majority of our database, it is a relevant aspect to be considered when using accelerometers.

4. Discussion and perspectives

Our literature review shows that using accelerometers for landslide monitoring has become more frequent over the last two decades. Uchimura et al. (2010) can be considered the pioneering article on the subject and it is the basis of other works published after it, which in many cases involve definition of alert thresholds based on tilting rate with success (Uchimura et al., 2015; Towhata et al., 2015; Wang et al., 2017, 2022a, b; Dikshit et al., 2018; Dikshit & Satyam, 2019a, b; Abraham et al., 2020a; Xie et al., 2020a; Sheikh et al., 2020, 2021).

Accelerometers are millimetric devices and, as pointed out by Otero et al (2022), they are capable of detecting rotations at particle scale and adding value to traditional geotechnical monitoring, which is usually based on macroscopic measurements. However, one aspect that needs to be highlighted is that all the threshold values reported so far are based on the signals provided by accelerometers attached to a rod and then embedded in the soil. In other words, the values proposed so far are based on macroscopic

movements/behavior of the slope and there is still room for research concerning the use of these instruments to monitor microscopic scale movements.

One challenge related to using accelerometers for slope monitoring is that the relation between acceleration or tilt angle and displacement is not very clear. Some authors have already proposed mathematical relations with good results. However, users should know that this relation may need detailed analysis site by site. Tilt angle readings can also be influenced by the installation setup (surface, shallow rod or deep rod).

Regarding landslide early detection based on accelerometers, authors have made efforts to develop TOF predictions since 2020 (Sheikh et al., 2020; Xie et al., 2020b; Wang et al., 2022a). A few successful cases were reported in the literature in very recent years, that show the great potential of establish thresholds based on slope tilt rate.

These predictions attempt to find a simple relation between slope tilting rate and time and are similar to Fukuzono's method. The authors succeed with their proposals in their study cases. However, Zhang et al. (2020a) made a critical review of 50 soil or rock landslides cases and found out that in 30 of them the difference between TOF predicted with Fukuzono's method calculated from slope displacement and real TOF was more than one day, ranging from 1.26 day to 86.24 days. Factors such as measurement errors and environmental noise contributed to that difference, that may directly affect the reliability of the system. Unsuccessful cases of TOF predictions based on slope tilting are expected to be reported in technical literature in the future.

Another important aspect is the development of low-cost monitoring systems. Asia is the continent with the highest number of landslides reports and fatalities. India, the Philippines, China, Nepal and Indonesia have the highest numbers of reports. (Kirschbaum et al., 2015). In fact, most of the papers that highlight that accelerometers have lower costs than traditional geotechnical monitoring techniques come from Asian countries (de Dios et al., 2009; Uchimura et al., 2010; Ooi et al., 2014; Wang et al., 2017, 2022a; Dikshit et al., 2018; Xie et al., 2019; Abraham et al., 2020a; Sheikh et al., 2021). This explains the investments in the development of new technologies in these countries. Developing low-cost monitoring systems is also particularly interesting for low-income countries, where the cost of a traditional monitoring geotechnical system can be unaffordable.

Accelerometers also take advantage from wireless data transmission technologies that reduce the need of long wires. This results in easier and less expensive deployment and sensor maintenance, which in the end contributes to lower costs. Nevertheless, special attention must be paid to calibrate low-cost accelerometers and reduce noise and sensor bias.

Finally, it is highly recommended to associate other types of sensors with accelerometers in order to develop more robust systems, because changes in monitoring acceleration data may not be always related to slope failure. In non-seismic rainfall

triggered landslides, the most common associated sensors are soil moisture sensors and rainfall gauges. Those types of sensors are used because water directly influences slope stability.

5. Conclusions

Since 2006 efforts have been made to develop new geotechnical monitoring techniques based on accelerometers.

From 2015 onwards, tilting alert levels and TOF prediction methods based on accelerometers readings have been proposed and validated by many authors, in laboratories and in real conditions. Research groups from Asia are leading this effort. Nevertheless, the literature review revealed that, unlike existing proposals based on displacement monitoring with various sensors, alert levels and TOF prediction based on tilt angles are still immature. Aspects related to sensor installation and the relation between slope displacement and measured tilt still need to be investigated. More successful and unsuccessful real-scale cases need to be reported, so that these aspects can be clarified. Moreover, even though sensors were always placed at shallow depths. Different research groups use different installation procedures, and their results are not necessarily comparable.

As interpreting accelerometers measurements in terms of tilt angles is a growing, but still developing area of expertise for slope monitoring, accelerometers should always be associated to other sensors, such as soil moisture sensors and rainfall gauges. The use of less expensive sensors is appealing, but low-cost tilting sensors require individual calibration, and the installation mode should be carefully analyzed in order to acquire data effectively. Moreover, for low-cost sensors aspects such as calibration, stability and repeatability are of special concern and are seldom addressed in the literature.

A more established use of accelerometers is as a chain of sensors installed in boreholes. Using accelerometer chains is more advantageous than traditional inclinometers because they are remotely operated and read outs can be made in a few minutes. In this kind of deployment sensor data is interpreted in terms of displacement and the results are commonly used to detect the depth of the failure surface. The use of Fukuzono's method with displacement estimated from data collected in this way is reported in technical literature.

Acknowledgements

This work was supported by the Coordination of Higher Education Personnel Improvement – CAPES, the São Paulo Research Foundation – FAPESP (grants numbers: 2017/50343-2, 2018/15869-6 and 2019/16458-2) and the National Council for Scientific and Technological Development – CNPq (grant number 405565/2021-6). The authors would also like to thank the reviewers for their valuable comments.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Malena D'Elia Otero: conceptualization, methodology, investigation, writing – original draft and visualization. Ana Elisa Silva de Abreu: conceptualization, validation, writing – review & editing, supervision and project administration. Rynaldo Zanotele Hemerly de Almeida: writing – review & editing. Alessandra Cristina Corsi: writing – review & editing and funding acquisition. Eduardo Soares de Macedo: writing – review & editing and funding acquisition.

Data availability

Data generated and analyzed in the course of the current study are available in Otero et al. 2024

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of Generative Artificial Intelligence (GenAI).

List of symbols and abbreviations

1/v	Inverse velocity
CAPES	Coordination of Higher Education Personnel Improvement
CNPq	National Council for Scientific and Technological Development
D	Prototype development and calibration procedures
FAPESP	São Paulo Research Foundation
GB-InSar	Ground-based interferometric synthetic aperture radar
GDP	Gross domestic product
GPS	Global positioning system
IoT	Internet of things
LEWS	Landslide Early warning system
Li-Dar	Light detection and ranging
M	Sensor deployment and landslide monitoring
MEMS	Micro electro mechanical systems
MUMS	Modular Underground Monitoring System
NM	Not mentioned
PVC	Polyvinyl Chloride
SAA	Shape acceleration array
TOF	Time-of-failure
UNDRR	United Nations Office for Disaster Risk Reduction

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A literature review based on the potential of use of RAP (Reclaimed Asphalt Pavement) in the base and subbase layers of flexible pavements

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Review Article

Keywords

Portland cement
Road base and sub-base material
Paving
Reclaimed asphalt pavement
Pavement rehabilitation
Pavement recovery

Abstract

The Brazilian Road transport mode accounts for approximately 65% of the freight transport matrix, playing a significant role in the productivity of all economic sectors. Therefore, the maintenance of road infrastructure impacts resource optimization, safety, and user comfort. This article reviews the use of Reclaimed Asphalt Pavement (RAP) as an aggregate in the base and sub-base layers of pavement, employing cementitious materials as a binder/stabilizer. The ProKnow-C method was used to identify articles for discussions in the systematic literature review (SLR). The results show that the resilient modulus (MR) increases with the higher RAP content in the mixtures, improving the load-bearing capacity of granular layers, while the California Bearing Ratio (CBR) decreases as the RAP content increases. However, treated RAP exhibits high-strength properties. As the RAP content increases, unconfined compressive strength decreases. Future investigations are needed to address durability, life cycle analysis (LCA), and economic feasibility.

1. Introduction

Globally, the transportation sector is responsible for 21% of total greenhouse gas emissions, with road transport accounting for 15% of total CO₂ emissions. Most of these emissions come from passenger vehicles, which contribute 45.1%, while 29.4% comes from freight transport. Figure 1 illustrates the CO₂ emissions from 2018, totaling 8 billion tons of CO₂ (Ritchie, 2020).

In a global context, when evaluating road quality, Brazil ranks 116th, with an average score of 3.0, where 1 corresponds to extremely underdeveloped and 7 to efficient by international standards. Figure 2 presents road quality; the study was conducted in 141 countries, with an overall average road quality score of 4.1 (Schwab, 2009).

The Brazilian Road transport mode accounts for about 65% of the country's freight transport matrix, playing a significant role in the productivity of all economic sectors. Therefore, the maintenance and preservation of road infrastructure directly impact resource optimization, user safety, and comfort. The main critical points that hinder traffic flow and increase accident risks include landslides, road erosion, potholes on driving lanes, and narrow or closed bridges (CNT, 2022).

The aim of this study is to investigate the feasibility of using Reclaimed Asphalt Pavement (RAP) in the base and sub-base layers of pavement as a substitute for natural aggregates.

The current national production of crushed stone is approximately 256 million tons per year, with 85% used in concrete production and 15% in road and pavement construction. Road paving requires around 5,500 m³ of crushed stone and 2% of the total crushed stone as sand per kilometer, equating to about 9,500 tons per kilometer, while road maintenance requires 3,000 tons per kilometer (ABDI, 2012; IBRAM, 2022).

The crushed stone mining sector is characterized by pollution and environmental impact, contributing to the depletion of natural resources, deforestation, alteration and destruction of native vegetation, noise pollution, air pollution, and the decline of local fauna, among other issues. The production processes for crushed stone follow the steps outlined in Figure 3 (Monteiro & Silva, 2018; ABDI, 2012; IBRAM, 2022):

The previously mentioned environmental impacts can be measured through carbon dioxide emissions during the extraction, processing, and transportation of fine and coarse

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Submitted on December 7, 2024; Final Acceptance on April 14, 2025; Discussion open until August 31, 2025.

Editor: Renato P. Cunha 

<https://doi.org/10.28927/SR.2025.010224>



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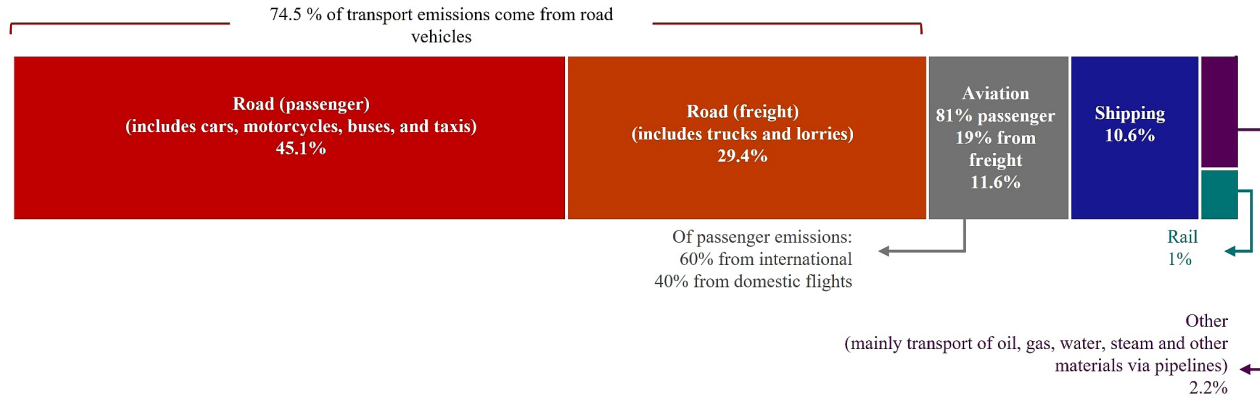


Figure 1. Global CO₂ Emissions by Transportation System. Adapted from Ritchie (2020).

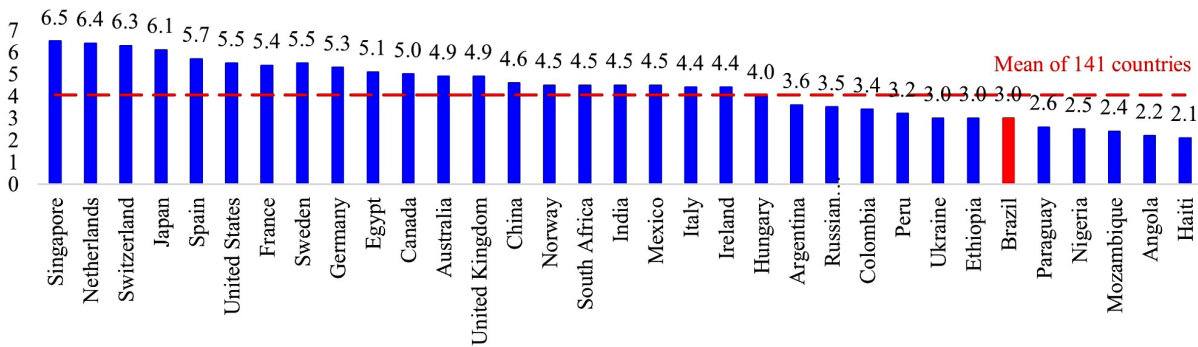


Figure 2. Road Quality – 2019. Adapted from Schwab (2009).

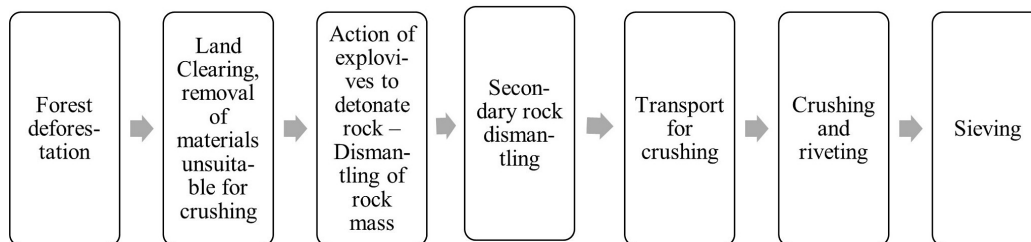


Figure 3. Crushed Stone Production (Monteiro & Silva, 2018; ABDI, 2012).

aggregates, Costa (2012) reports that producing one ton of fine aggregate emits 0.0861 tons of CO₂ and producing coarse aggregates emits 0.0858 tons of CO₂. The calculation of the data was conducted by considering the energy required for extraction and processing, as well as diesel fuel and electricity. For the transportation of materials, an average distance of 225 km for a round trip using a diesel-powered semi-truck was taken into account.

Alternative products can replace the use of natural aggregates in asphalt layers and the structural layers of pavements, such as RAP.

According to the National Asphalt Pavement Association (NAPA, 2020) 89.2 million tons of Reclaimed Asphalt Pavement (RAP) are used annually in the construction of new asphalt pavements in the United States. About 94% of RAP is utilized in new asphalt mixtures, while a small percentage is incorporated into granular layers. The net reduction in greenhouse gas emissions resulting from the use of RAP in asphalt mixtures from 2009 to 2019 was approximately 21.1 million tons of CO₂, which results in savings of approximately 44 million cubic meters in landfills.

The relevance of this study is linked to the significance and intensive use of the road transport mode, the exploitation of natural resources for the construction and maintenance of the road network, and the reduction of CO₂ emissions through the application of recycled materials in the base layer.

Reclaimed Asphalt Pavement (RAP) is a milled material generated from the removal of asphalt pavements for rehabilitation, resurfacing, or other maintenance activities. RAP is a graded aggregate coated with a layer of aged binder, which can be recycled and used in asphalt mixtures and other civil engineering applications (NAPA, 2020). RAP consists of 90% to 95% aggregate and 5% to 10% bitumen, with material recycling beginning in the 1970 (Plati & Cliatt, 2019).

Studies from different locations indicate the appropriate RAP contents for application in the pavement base layer. The permitted limits are shown in Table 1.

The Brazilian pavement recycling standard specifies that the base layer should contain a maximum of 50% Reclaimed Asphalt Pavement (RAP) with the addition of Portland cement. Additionally, the granulometric composition must comply with the ranges established in Table 2. The standard emphasizes that the aggregate used for the base must meet the following criteria: at least 95% of the material should pass through the 2" sieve; the percentage of fines passing

the N° 200 sieve (0.075 mm) should not exceed 15%; and it is desirable that at least 50% of the fraction passes through the N° 4 sieve (4.75 mm) (DNIT, 2013).

Specific conditions are presented in Table 3, which can assist in engineering projects.

Using only RAP material in granular layers is a more sustainable alternative; however, the pavement's functionality may be compromised due to permanent deformation, inadequate resistance characteristics, stiffness, and the variability in the composition of the material. The permanent deformation of RAP is 1.7 to 3 times greater compared to flexible base material due to its viscous properties (Khan & Puppala, 2023; George et al., 2021; Khan et al., 2023; Seferoğlu et al., 2018). However, reinforced or treated RAP exhibits high shear strength and stiffness properties, making it capable of withstanding traffic loads and consequently suitable for use in base and sub-base layers (Udomchai et al., 2021).

2. Methods

This research presents a systematic literature review (SLR) based on studies, research, and the analysis of various articles related to the use of RAP (Reclaimed Asphalt Pavement) in the base and sub-base layers of flexible pavement.

2.1 Systematic analysis

The identification of articles for discussion was followed by a selection approach using the systematic literature review method the ProKnow-C (Knowledge Development Process – Constructivist).

Three databases in the field of Civil Engineering were defined in the Capes Portal, as well as, Scopus, Science Direct e Web of Science, where the search terms aligned with the research theme are the following keywords: base, subbase, layers, foundation, pavement, recycled asphalt pavement, RAP e cement.

The search was carried out in the eighth month of the year 2023, and the selection only included scientific articles from the last 10 years. The articles were filtered in the database and then extracted and inserted into the reference manager. Duplicate articles indexed in more than one database were

Table 1. RAP content limit for base layer (Lee et al., 2015; DNIT, 2013).

Jurisdiction	RAP content limit for base layer
Victoria	30%
New South Wales	15%
Tasmania	15%
Queensland	15%
South Australia	20%
Western Australia	10%
New Zealand	15%
USA / Canada	30% or more
Brazil	50%

The content for the use of RAP depends on the type of base, the quality of the RAP, and it must meet the project specifications.

Table 2. Granulometric composition (DNIT, 2013).

Square mesh sieves		Percentage passing, by mass (%)		Design range tolerance (%)
ABNT	Aperture (mm)	I	II	
2"	50.800	100	100	-
1"	25.400	75 - 90	100	± 7
3/8"	9.500	40 - 75	50 - 85	± 7
N° 4	4.750	30 - 60	35 - 65	± 5
N° 10	2.000	20 - 45	25 - 50	± 5
N° 40	0.425	15 - 30	15 - 30	± 2
N° 200	0.075	5 - 15	5 - 15	± 2

Table 3. Specific conditions of base materials (DNIT, 2013, 2022).

Parameters	Base requirements	Reference
Simple compression strength	2.1 and 2.5 MPa	DNIT 167/2013 - ES
Diametrical compression tensile strength	0.25 and 0.35 MPa	DNIT 167/2013 - ES
Maximum expansion	0.50%	DNIT 141/2022-ES
ISC / CBR $N \leq 5 \times 10^6$	> 60%	DNIT 141/2022-ES
ISC / CBR $N > 5 \times 10^6$	$\geq 80\%$	DNIT 141/2022-ES
Abrasion for Los Angeles	< 55%	DNIT 141/2022-ES or DNIT 167/2013 - ES
Liquidity Limit	$\leq 25\%$	DNIT 141/2022-ES
Plasticity Limit	$\leq 6\%$	DNIT 141/2022-ES
Sand equivalent	> 30%	DNIT 141/2022-ES
Shape index	≥ 0.5	DNIT 167/2013 - ES
Lamellarity index	< 20%	DNIT 167/2013 - ES
Durability	< 12%	DNIT 167/2013 - ES

N: Equivalent Number of Standard Single Axle Operations of 80 kN Dual Wheels.

eliminated. 27 articles presented the title, abstract and text aligned with the research theme, which were selected for RSL to present the state of the art. One book chapter was included to assist in the evaluation of the results of the article portfolio.

3. State of the art

The properties of the base layer material significantly influence the stiffness of the layer. From the configuration in Figure 4, where a load (P) is applied to the pavement surface, generating a deflection (d), consequently causing strain due to tensile stresses (Et) in the lower region of the asphalt layer, leading to the formation of cracks. The load (P) is then transferred to the granular layer, and subsequently, the compressive deformations (Ec) are transferred to the top of the subgrade layer, causing shear in the subgrade. Therefore, the type of base layer used in flexible or semi-rigid pavement can affect its overall performance (Francois et al., 2019; Bernucci et al., 2008).

This section will present the mechanical properties that influence the performance of the base layer and the overall pavement structure.

3.1 Resilient modulus

The pavement layers are subjected to cyclic loads of varying intensities and different frequencies. The resilient modulus (MR) measures the recoverable deformation under repeated loading. This parameter is obtained through the dynamic triaxial test, which is used to calculate stresses and strains (Bernucci et al., 2008; Nokkaew, 2018).

The resilient modulus is the ratio of cyclic stress to recoverable deformation, indicating the material's ability to withstand load (Khan et al., 2023; Mousa et al., 2021).

The results of various studies have been compiled in Table 4 to illustrate the mechanical behavior of pavement structures containing different RAP mixtures applied in the base layers of pavements. The resilient modulus test is used

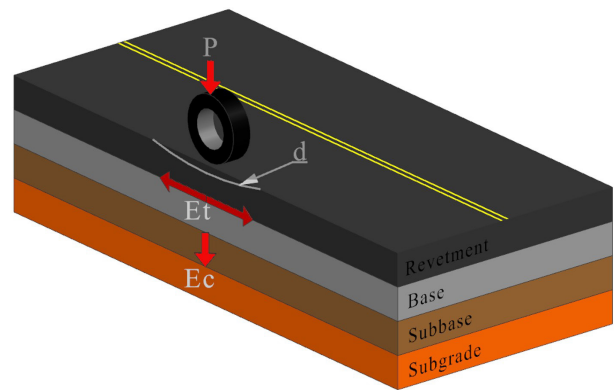


Figure 4. Stresses, Strains, and Displacement in Asphalt Pavement. Adapted from Franco (2007), Francois et al. (2019) and Bernucci et al. (2008). (P: load; Et: tensile stresses; d: deflection; Ec: compressive deformations).

during the detailed design phase and typically yields values ranging from 150 to 300 MPa for granular materials such as graded crushed stone (GCS), and from 5,000 to 10,000 MPa for chemically stabilized materials (DNER, 2006).

From the cyclic triaxial tests Francois et al. (2019) obtained the resilience modulus for bases stabilized with Portland cement, asphalt, calcium chloride, and geogrid. The chemically stabilized base with Portland cement exhibited the highest modulus at 818 MPa, followed by the asphalt sections at 294 MPa, calcium chloride at 262 MPa, and mechanically stabilized base with geogrid at 190 MPa. The Portland cement-stabilized base demonstrated the highest resilience modulus, confirming its suitability. In contrast, the geogrid showed the lowest resilience modulus, indicating that this material is not the most appropriate for resisting the cyclic stresses of flexible pavement.

The results for the base with 100% RAP material and the material with 50% RAP and 50% natural aggregate showed similar resilience modulus values to that of 100% natural aggregate, around 300 MPa, especially at increased confining pressures. This conclusion supports the inclusion of

Table 4. Approximate results of the Resilience Module of the bases with RAP.

Reference	Mixtures	MR (MPa)
Francois et al. (2019)	100% RAP	334
Francois et al. (2019)	100% RAP + Bitumen (3.62 l/m ²)	294
Francois et al. (2019)	100% RAP + CaCl ₂ (3.40 lm ²)	262
Francois et al. (2019)	100% RAP + Cem. (17.4 kg/m ²)	818
Francois et al. (2019)	100% RAP + Geogrid	190
Mousa et al. (2021)	0% RAP + 100% NA	314
Mousa et al. (2021)	20% RAP + 80% NA	378
Mousa et al. (2021)	40% RAP + 60% NA	450
Mousa et al. (2021)	60% RAP+ 40% NA	500
Mousa et al. (2021)	80% RAP + 20% NA	630
Mousa et al. (2021)	100% RAP+ 0% NA	727
Nokkaew (2018)	100% RAP	480.3
Nokkaew (2018)	100% RMP	359.4
Nokkaew (2018)	100% NA	180.4

Legend: Natural aggregate (NA), Asphalt layer mixed with the underlying base layer (RMP) and Cement (Cem.).

RAP as a base material in the design of new pavements and rehabilitation projects, suggesting a sustainable perspective. (Plati & Cliatt, 2019).

Mousa et al. (2021), reports that the resilience modulus increases with the amount of RAP in the mixtures, surpassing the resilience modulus of natural aggregate. The addition of RAP to the mixture enhances the cohesion between particles due to the asphalt binder coating the RAP/aggregate, making the mixture stiffer. The mixtures with 0%, 20%, 40%, 60%, 80%, and 100% RAP achieved resilience modulus values of 314 MPa, 378 MPa, 450 MPa, 500 MPa, 630 MPa, and 727 MPa, respectively. When evaluating the samples using computed tomography, it is observed that natural aggregates have a higher air void content compared to samples containing RAP, suggesting that lower air voids may lead to higher resilience modulus values.

According to investigations conducted by Mousa et al. (2021) natural aggregates have resilience modulus values ranging from 161 to 361 MPa, while mixtures containing 20% to 100% RAP show resilience modulus values between 178 MPa and 461 MPa. This variation depends on the type of aggregate (geological origin, that is, inherent properties), the type of RAP, and the gradation of the materials.

RAP obtained from the milling of surface asphalt and RPM generated by the complete removal of the asphalt layer mixed with the underlying base layer was studied by Nokkaew (2018), according to the gradation distribution, both materials were classified as A-1-a; however, they exhibited different resilience modulus values. The RAP showed an MR of 480.3 MPa, while the RPM had a lower MR of 359.4 MPa. In comparison to crushed limestone, which is traditionally used for base construction, it can be stated that the recycled materials studied demonstrate a higher MR and can be used in base and sub-base layers, given that the MR of crushed limestone is 180.4 MPa.

Additionally, Nokkaew (2018) reports that pavement structures with RAP, after a 20-year simulation, exhibited

lower longitudinal cracking at 949 cm/km, while RPM and crushed limestone showed 2,182 cm/km and 3,000 cm/km, respectively, against a normative value of 34,090 cm/km. Fatigue cracking (alligator cracking) was also lower for RAP aggregate at 0.5%, compared to 0.8% for RPM and 1.8% for crushed limestone, with a normative value of 25%. The plastic deformation of RAP was lower than that of crushed limestone, although RAP and RPM tend to develop higher plastic deformation under repeated loading. The International Roughness Index (IRI) for RAP, RPM, and crushed limestone was 220.8, 226.4, and 248.8, respectively, with a normative value of 244.3. Thus, these studied materials do not compromise pavement service life, and since RAP is derived from asphalt coating, it possesses high quality.

Chemically stabilized bases with cementitious materials exhibit higher resilience modulus values (Francois et al., 2019).

The resilience modulus increases with the amount of RAP in the mixture, surpassing the resilience modulus of natural aggregate (NA). Adding RAP enhances particle cohesion due to the asphalt binder coating the RAP/aggregate, resulting in a stiffer mixture (Euch Khay et al., 2015; Mousa et al., 2021; Nokkaew, 2018).

3.2 California Bearing Ratio

The California Bearing Ratio (CBR) is a penetrometer test developed to assess the subgrade's potential for failure (plastic deformations) in roads. It is expressed as a percentage and defined as the ratio between the pressure needed to achieve a specific penetration of a piston into the test specimen and the pressure required to achieve the same penetration in a reference standard material (standardized crushed stone). The strength measured in the CBR test is an indirect response that combines both the cohesion and the friction angle of the material (Bernucci et al., 2008).

According to the Federal Highway Administration (FHWA), RAP has a CBR value of around 20% to 25%, which can reach values above 150% when mixed with approximately 60% natural aggregate (FHWA, 2016).

The results of several studies were grouped in Table 5 to show the mechanical behavior of pavement structures containing

different RAP mixtures applied to the pavement base layers. The limits required by the DNIT (2022) are $ISC / CBR N \leq 5 \times 10^6$ with at least 60%, and $ISC / CBR N > 5 \times 10^6$ being greater than or equal to 80%.

The Limerock Bearing Ratio (LBR) test is a variant of the California Bearing Ratio (CBR) test. Generally,

Table 5. Approximate results of the California Support Index (CBR) of bases with RAP.

Reference	Mixtures	CBR (%)
Bleakley & Cosentino (2013)	100% RAP	30
Bleakley & Cosentino (2013)	0% RAP + 100% NA	162
Bleakley & Cosentino (2013)	25% RAP + 75% NA	99
Bleakley & Cosentino (2013)	50% RAP + 50% NA	55
Bleakley & Cosentino (2013)	50% RAP + 50% NA + 1% emulsion CSS-1HF	127
Bleakley & Cosentino (2013)	50% RAP + 50% NA + 2% emulsion CSS-1HF	105
Bleakley & Cosentino (2013)	50% RAP + 50% NA + 3% emulsion CSS-1HF	107
Bleakley & Cosentino (2013)	50% RAP + 50% NA + 1% emulsion SS-1H	106
Bleakley & Cosentino (2013)	50% RAP + 50% NA + 2% emulsion SS-1H	103
Bleakley & Cosentino (2013)	50% RAP + 50% NA + 3% emulsion SS-1H	100
Bleakley & Cosentino (2013)	50% RAP + 50% NA + 1% Cem.	175
Bleakley & Cosentino (2013)	50% RAP + 50% NA + 2% Cem.	288
Bleakley & Cosentino (2013)	50% RAP + 50% NA + 3% Cem.	396
Ayan et al. (2014)	100% Recycled Concrete	36
Ayan et al. (2014)	100% NA	32
Ayan et al. (2014)	50% Recycled Concrete + 50% RAP	29
Mousa et al. (2021)	0% RAP + 100% NA	85
Mousa et al. (2021)	20% RAP + 80% NA	65
Mousa et al. (2021)	40% RAP + 60% NA	45
Mousa et al. (2021)	60% RAP + 40% NA	30
Mousa et al. (2021)	80% RAP + 20% NA	25
Mousa et al. (2021)	100% RAP + 0% NA	15
Seferoğlu et al. (2018)	0% RAP + 100% NA	180
Seferoğlu et al. (2018)	10% RAP + 90% NA	149
Seferoğlu et al. (2018)	20% RAP + 80% NA	140
Seferoğlu et al. (2018)	30% RAP + 70% NA	90
Seferoğlu et al. (2018)	30% RAP + 70% NA + 1% Cem.	205
Seferoğlu et al. (2018)	30% RAP + 70% NA + 2% Cem.	320
Seferoğlu et al. (2018)	30% RAP + 70% NA + 3% Cem.	380
Seferoğlu et al. (2018)	40% RAP + 60% NA	80
Seferoğlu et al. (2018)	40% RAP + 60% NA + 1% Cem.	140
Seferoğlu et al. (2018)	40% RAP + 60% NA + 2% Cem.	150
Seferoğlu et al. (2018)	40% RAP + 60% NA + 3% Cem.	280
Seferoğlu et al. (2018)	50% RAP + 50% NA	50
Seferoğlu et al. (2018)	50% RAP + 50% NA + 1% Cem.	85
Seferoğlu et al. (2018)	50% RAP + 50% NA + 2% Cem.	180
Seferoğlu et al. (2018)	50% RAP + 50% NA + 3% Cem.	185
Seferoğlu et al. (2018)	60% RAP + 40% NA	45
Seferoğlu et al. (2018)	60% RAP + 40% NA + 1% Cem.	80
Seferoğlu et al. (2018)	60% RAP + 40% NA + 2% Cem.	130
Seferoğlu et al. (2018)	60% RAP + 40% NA + 3% Cem.	145
Seferoğlu et al. (2018)	100% RAP + 0% NA	31
Seferoğlu et al. (2018)	100% RAP + 0% NA + 1% Cem.	65
Seferoğlu et al. (2018)	100% RAP + 0% NA + 2% Cem.	80
Seferoğlu et al. (2018)	100% RAP + 0% NA + 3% Cem.	140

specifications require a minimum LBR of 100% (equivalent to a CBR of 80%) for base layer materials. To facilitate analysis, we will convert the LBR results obtained by Bleakley & Cosentino (2013).

The test with 100% natural aggregate (NA) without chemical stabilization achieved an average CBR of 130%. The mixture of 25% RAP with 75% natural aggregate without chemical stabilization achieved an average CBR of 79%, while the mixture of 50% RAP with 50% natural aggregate without chemical stabilization achieved an average CBR of 42%. The mixtures of 50% RAP with 50% natural aggregate stabilized with 1%, 2%, and 3% CSS-IHF emulsion achieved CBR values of 102%, 84%, and 86%, respectively. The mixtures of 50% RAP with 50% natural aggregate stabilized with 1%, 2%, and 3% SS-IH emulsion achieved CBR values of 85%, 82%, and 80%, respectively. The mixtures of 50% RAP with 50% natural aggregate stabilized with 1%, 2%, and 3% Portland cement achieved CBR values of 140%, 230%, and 317%, respectively. Only mixtures with 50% or more natural aggregate have the potential to achieve a CBR above 80%, with the most promising results obtained using Portland cement for chemical stabilization (Bleakley & Cosentino, 2013).

A study conducted by Ayan et al. (2014) examined the performance of 100% recycled materials for untreated sub-base layers. Natural aggregate (NA) samples were compared with samples of recycled concrete aggregate (RCA) and samples with a 50% RAP and 50% RCA blend. The CBR values obtained were 32%, 47%, and 29%, respectively. It was observed that during compaction RCA particles broke down into smaller particles more readily compared to NA and RAP. A significant difference between RCA and RAP properties is that the bitumen-coated aggregates in RAP create a more flexible material, which is less prone to breakage during compaction compared to RCA. Samples with a 50% RAP and 50% RCA blend achieved a CBR value of 29%, which falls below the minimum requirement (CBR < 30%) for unbound sub-base applications. However, a treatment applied to this blend could potentially improve the material's performance for such applications.

In line with other literature, Mousa et al. (2021), report that the California Bearing Ratio (CBR) significantly decreases with the increasing amount of RAP in mixtures for base and sub-base layers. This reduction may be attributed to decreased friction between the aggregates due to the bitumen coating on the aggregate grains. The mixtures containing 0%, 20%, 40%, 60%, 80%, and 100% RAP yielded CBR values of 85%, 65%, 45%, 30%, 25%, and 15%, respectively. Based on these CBR results, the authors conclude that up to 60% RAP can be utilized in sub-base layers, while only 20% RAP is suitable for base layers.

Seferoğlu et al. (2018) also observed that as the concentration of RAP increased, the CBR value decreased. Tests were conducted with natural aggregates and RAP. The mixtures that presented a CBR lower than 120% were

stabilized with 1%, 2%, and 3% cement. The mixtures with 100% natural aggregate, 10% RAP, 20% RAP, and 30% RAP had CBR values of 180%, 149%, 140%, and 90%, respectively. Table 5 illustrates the other results obtained in the research. The mixture with 100% RAP had a CBR of 31%, which is insufficient for use as a base layer. However, with the addition of 1%, 2%, and 3% cement, the CBR values increased to 65%, 80%, and 140%, respectively. Thus, it is possible to create a base layer with 100% RAP, provided it receives appropriate treatment to meet normative requirements.

The SP-CS (gravelly sands and clay) and GW-GC (gravelly soil with clays) soils were mixed with 0%, 20%, 40%, and 60% RAP. Without any treatment, it was observed that the mixtures decreased the CBR value with the increase of RAP. The CBR values obtained for SP-CS/RAP were 30%, 25%, 20%, and 18%, while the values for GW-GC/RAP were 83%, 63%, 51%, and 46%. It can be concluded that 60% RAP reduces the CBR by half, and from 40% RAP, the rate of CBR reduction approaches a constant value (Ghanizadeh et al., 2018).

The CBR significantly decreases with the increase in the amount of RAP in the mixtures for base and sub-base layers. This reduction may be due to the decreased friction between the aggregates caused by the asphalt binder coating the aggregate grains (Mousa et al., 2021; Seferoğlu et al., 2018; Ghanizadeh et al., 2018).

3.3 Unconfined compressive strength

Unconfined compressive strength (UCS) is a test that involves applying axial loading to a cylindrical specimen of cohesive soil using a simple press (Bernucci et al., 2008).

The results of several studies were grouped in Table 6 to show the mechanical behavior of pavement structures containing different RAP mixtures applied to the pavement base layers. The limits required by the DNIT (2013) for unconfined compressive strength at 7 days range between 2.1 MPa and 2.5 MPa.

Euch Khay et al. (2015) added a content of 6% Portland cement with 100%, 75%, 50%, 25%, and 0% RAP, achieving unconfined compressive strength at 7 days of 5.85 MPa, 6.75 MPa, 8.30 MPa, 10.55 MPa, and 12.50 MPa, respectively, and at 28 days of 8.35 MPa, 9.95 MPa, 11.30 MPa, 14.70 MPa, and 18.05 MPa. As the RAP content increased, the compressive strength decreased. The indirect tensile strength was around 11 to 12% of the compressive strength, while the flexural strength was about 13% to 19% of the compressive strength. In pavement layer structures, a minimum curing period of 7 days is recommended.

When analyzing the compressive stresses in bases stabilized with Portland cement, bitumen, calcium chloride, and geogrid, it is observed that the bituminous section and the one containing geogrid exhibited similar compressive deformation at the top of the subgrade layer, subjected to similar impacts regarding susceptibility to wheel track

Table 6. Approximate results of unconfined compressive strength of bases with RAP.

Reference	Mixtures	UCS (MPa)
Euch Khay et al. (2015)	100% RAP + 6% Cem. + 0% NA	5.85
Euch Khay et al. (2015)	75% RAP + 6% Cem. + 25% NA	6.75
Euch Khay et al. (2015)	50% RAP + 6% Cem. + 50% NA	8.30
Euch Khay et al. (2015)	25% RAP + 6% Cem. + 75% NA	10.55
Euch Khay et al. (2015)	0% RAP + 6% Cem. + 100% NA	12.50
Fedrigio et al. (2017)	20% RAP + 80% Recycled gravel + 2% Cem. (modified)	2.50
Fedrigio et al. (2017)	50% RAP + 50% Recycled gravel + 2% Cem. (modified)	2.80
Fedrigio et al. (2017)	20% RAP + 80% Recycled gravel + 4% Cem. (modified)	4.80
Fedrigio et al. (2017)	50% RAP + 50% Recycled gravel + 4% Cem. (modified)	4.50
Fedrigio et al. (2017)	20% RAP + 80% Recycled gravel + 4% Cem. (intermediary)	3.10
Fedrigio et al. (2017)	50% RAP + 50% Recycled gravel + 4% Cem. (intermediary)	3.90
Fedrigio et al. (2017)	20% RAP + 80% Recycled gravel + 6% Cem. (intermediary)	5.50
Fedrigio et al. (2017)	50% RAP + 50% Recycled gravel + 6% Cem. (intermediary)	5.10
Pratikso et al. (2017)	100% RAP + 1.5% Cem.	1.18
Pratikso et al. (2017)	100% RAP + 3% Cem.	1.58
Pratikso et al. (2017)	100% RAP + 4.5% Cem.	2.01
Pratikso et al. (2017)	100% RAP + 6% Cem.	2.57
Pratikso et al. (2017)	100% RAP + 7.5% Cem.	3.44
Ungureanu et al. (2020)	100% RAP + 3.5% Cem.	1.44
Ghanizadeh et al. (2018)	0% RAP + 100% soil (SP-SC) + 3% Cem.	2.76
Ghanizadeh et al. (2018)	20% RAP + 80% soil (SP-SC) + 3% Cem.	2.55
Ghanizadeh et al. (2018)	40% RAP + 60% soil (SP-SC) + 3% Cem.	2.26
Ghanizadeh et al. (2018)	60% RAP + 40% soil (SP-SC) + 4% Cem.	2.43
Ghanizadeh et al. (2018)	0% RAP + 100% soil (GW-GC) + 3% Cem.	3.61
Ghanizadeh et al. (2018)	20% RAP + 80% soil (GW-GC) + 3% Cem.	3.43
Ghanizadeh et al. (2018)	40% RAP + 60% soil (GW-GC) + 3% Cem.	2.23
Ghanizadeh et al. (2018)	60% RAP + 40% soil (GW-GC) + 5% Cem.	2.26
Kasu et al. (2020)	100% NA + 2.5% Cem.	2.60
Kasu et al. (2020)	100% NA + 5% Cem.	5.00
Kasu et al. (2020)	100% NA + 7.5% Cem.	7.50
Kasu et al. (2020)	100% NA + 10% Cem.	11.62
Kasu et al. (2020)	80% NA + 20% RAP + 2.5% Cem.	2.50
Kasu et al. (2020)	80% NA + 20% RAP + 5% Cem.	4.80
Kasu et al. (2020)	80% NA + 20% RAP + 7.5% Cem.	7.20
Kasu et al. (2020)	80% NA + 20% RAP + 10% Cem.	10.60
Kasu et al. (2020)	60% NA + 40% RAP + 2.5% Cem.	2.00
Kasu et al. (2020)	60% NA + 40% RAP + 5% Cem.	4.50
Kasu et al. (2020)	60% NA + 40% RAP + 7.5% Cem.	7.00
Kasu et al. (2020)	60% NA + 40% RAP + 10% Cem.	9.20
Kasu et al. (2020)	40% NA + 60% RAP + 2.5% Cem.	1.23
Kasu et al. (2020)	40% NA + 60% RAP + 5% Cem.	3.20
Kasu et al. (2020)	40% NA + 60% RAP + 7.5% Cem.	6.00
Kasu et al. (2020)	40% NA + 60% RAP + 10% Cem.	8.50
Adresi et al. (2019)	0% RAP+ 65% NA+ 30% Sand + 5% Filler + 3% Cem. (Saturated 25 °C 7 days)	3.50
Adresi et al. (2019)	0% RAP+ 65% NA+ 30% Sand + 5% Filler + 3% Cem. (Dry 50 °C 7 days)	5.00
Adresi et al. (2019)	40% RAP+ 20% NA+ 40% Sand + 0% Filler + 3% Cem. (Saturated 25 °C 7 days)	4.50
Adresi et al. (2019)	40% RAP+ 20% NA+ 40% Sand + 0% Filler + 3% Cem. (Dry 50 °C 7 days)	3.50
Adresi et al. (2019)	60% RAP+ 0% NA+ 40% Sand + 0% Filler + 3% Cem. (Saturated 25 °C 7 days)	4.00
Adresi et al. (2019)	60% RAP+ 0% NA+ 40% Sand + 0% Filler + 3% Cem. (Dry 50 °C 7 days)	3.00
Adresi et al. (2019)	80% RAP+ 0% NA+ 0% Sand + 20% Filler + 3% Cem. (Saturated 25 °C 7 days)	1.50
Adresi et al. (2019)	80% RAP+ 0% NA+ 0% Sand + 20% Filler + 3% Cem. (Dry 50 °C 7 days)	1.30
Adresi et al. (2019)	0% RAP+ 65% NA+ 30% Sand + 5% Filler + 5% Cem. (Saturated 25 °C 7 days)	7.50
Adresi et al. (2019)	0% RAP+ 65% NA+ 30% Sand + 5% Filler + 5% Cem. (Dry 50 °C 7 days)	8.40

Table 6. Continued...

Reference	Mixtures	UCS (MPa)
Adresi et al. (2019)	40% RAP+ 20% NA+ 40% Sand + 0% Filler + 5% Cem. (Saturated 25 °C 7 days)	5.30
Adresi et al. (2019)	40% RAP+ 20% NA+ 40% Sand + 0% Filler + 5% Cem. (Dry 50 °C 7 days)	4.00
Adresi et al. (2019)	60% RAP+ 0% NA+ 40% Sand + 0% Filler + 5% Cem. (Saturated 25 °C 7 days)	4.40
Adresi et al. (2019)	60% RAP+ 0% NA+ 40% Sand + 0% Filler + 5% Cem. (Dry 50 °C 7 days)	3.00
Adresi et al. (2019)	80% RAP+ 0% NA+ 0% Sand + 20% Filler + 5% Cem. (Saturated 25 °C 7 days)	3.60
Adresi et al. (2019)	80% RAP+ 0% NA+ 0% Sand + 20% Filler + 5% Cem. (Dry 50 °C 7 days)	2.00
Adresi et al. (2019)	0% RAP+ 65% NA+ 30% Sand + 5% Filler + 7% Cem. (Saturated 25 °C 7 days)	9.50
Adresi et al. (2019)	0% RAP+ 65% NA+ 30% Sand + 5% Filler + 7% Cem. (Dry 50 °C 7 days)	10.00
Adresi et al. (2019)	40% RAP+ 20% NA+ 40% Sand + 0% Filler + 7% Cem. (Saturated 25 °C 7 days)	5.90
Adresi et al. (2019)	40% RAP+ 20% NA+ 40% Sand + 0% Filler + 7% Cem. (Dry 50 °C 7 days)	5.00
Adresi et al. (2019)	60% RAP+ 0% NA+ 40% Sand + 0% Filler + 7% Cem. (Saturated 25 °C 7 days)	4.90
Adresi et al. (2019)	60% RAP+ 0% NA+ 40% Sand + 0% Filler + 7% Cem. (Dry 50 °C 7 days)	3.60
Adresi et al. (2019)	80% RAP+ 0% NA+ 0% Sand + 20% Filler + 7% Cem. (Saturated 25 °C 7 days)	3.60
Adresi et al. (2019)	80% RAP+ 0% NA+ 0% Sand + 20% Filler + 7% Cem. (Dry 50 °C 7 days)	2.00
Goldoni et al. (2023)	20% RAP+ 38% NA+ 42% Gravel + 2% Cem.	1.60
Goldoni et al. (2023)	20% RAP+ 38% NA+ 42% Gravel + 4% Cem.	3.30
Goldoni et al. (2023)	20% RAP+ 38% NA+ 42% Gravel + 6% Cem.	8.30
Goldoni et al. (2023)	45% RAP+ 33% NA+ 22% Gravel + 2% Cem.	1.80
Goldoni et al. (2023)	45% RAP+ 33% NA+ 22% Gravel + 4% Cem.	4.00
Goldoni et al. (2023)	45% RAP+ 33% NA+ 22% Gravel + 6% Cem.	7.50
Goldoni et al. (2023)	70% RAP + 30% NA + 2% Cem.	1.80
Goldoni et al. (2023)	70% RAP + 30% NA + 4% Cem.	4.40
Goldoni et al. (2023)	70% RAP + 30% NA + 6% Cem.	5.60

deformation in flexible pavements. In contrast, the compressive deformations for the sections with CaCl_2 and Portland cement were lower than those obtained for the untreated 100% RAP control section, indicating that these sections had a positive impact on the entire section's ability to resist wheel track rutting (Francois et al., 2019).

The statistical analysis conducted by Noulmanee et al. (2021) indicated that the type of aggregate and the water/cement ratio influence the compressive strength. The lower the water/cement ratio, the higher the compressive strength of the layer. Regarding the type of aggregate, three types were studied: (1) lateritic soil, (2) RAP and crushed stone, and (3) RAP with crushed stone and lateritic soil. The RAP with crushed stone tended to produce the highest compressive strength value among the three materials, while the lateritic soil produced the lowest compressive strength value.

Fedrego et al. (2017) emphasizes that strength is not the only property to consider when designing mixtures for base layers. Some mixtures, although they exhibit acceptable strength values, may fail due to cracking from shrinkage or erosion. When using RAP with crushed stone and varying cement contents of 2%, 4%, and 6% with different compaction energies, it was concluded that increasing the cement content raises the unconfined compressive strength but also increases the brittleness of the mixtures, making them more susceptible to cracking. However, it is important to highlight that a greater compaction effort can offset the use of low cement contents, thereby reducing costs and the risks of shrinkage cracking.

Like other authors, Pratikso et al. (2017) highlight the influence of cement content in mixtures and curing time. A variation of 1.5% to 7.5% cement, with increments of 1.5%, was tested with 100% RAP, resulting in strengths of 1.18 MPa, 1.58 MPa, 2.01 MPa, 2.57 MPa, and 3.44 MPa at 7 days. The cement levels that meet the UCS requirements for base layers are between 6% and 7.5%.

One of the important factors in gaining strength is curing. Mixtures with 100% RAP stabilized with 3.5% hydraulic binder using Portland cement clinker achieved unconfined compressive strengths of 1.44 MPa, 2.22 MPa, and 3.22 MPa at 7, 28, and 60 days, respectively (Ungureanu et al., 2020). Therefore, UCS increases with longer curing times, (Ungureanu et al., 2020; Ghanizadeh et al., 2018) other researchers have drawn similar conclusions.

A percentage of 0%, 3%, 4%, 5%, and 6% cement is added to two mixtures: SP-CS soil (gravelly sands and clay) with RAP and GW-GC soil (gravelly soil with clay) with RAP, maintaining a fixed soil/RAP ratio. The UCS of the treated material increases with the increase in cement content; increasing the cement by 1% while keeping RAP constant results in an average strength increase from 376 kPa to 410 kPa for the SP-CS and GW-GC soils, respectively. Conversely, when the cement content is fixed and the percentage of RAP in the mixtures is increased, the UCS decreases. The reason for this reduction in strength is that the asphalt coating around the RAP material decreases the friction between the aggregates (Ghanizadeh et al., 2018).

The Portland Cement Association suggests that the UCS should be greater than 2100 kPa at 7 days; lower values reduce the pavement's load-bearing capacity, and thus less than 3% cement leads to low strength. Comparing the UCS results for the two soils, it can be concluded that the base treated with 0% and 20% RAP containing GW-GC soil is approximately 793 kPa higher than samples containing SP-SC soil, while the UCS value for the base treated with 60% RAP containing SP-SC soil is about 405 kPa higher than samples containing GW-GC soil. It is concluded that at higher levels of RAP, the effect of cement on increasing UCS is greater for the base treated with SP-SC compared to the base treated with GW-GC soil (Ghanizadeh et al., 2018).

Various studies report that UCS decreases with an increase in RAP content and increases with the amount of cement. This behavior was also reported by Kasu et al. (2020). The bituminous film on the RAP aggregate consists of a hydrophobic material that is chemically incompatible with the hydrophilic cement paste, which results in lower strength in the transition zone at the interface between the aggregate and the cement paste. This conclusion is consistent with other literature. In the experiments, the percentages of cement varied from 2.5% to 10% with increments of 2.5%. Five sample types were produced: 100% natural aggregate with and without cement treatment, 80% natural aggregate plus 20% RAP, 60% natural aggregate plus 40% RAP, and 40% natural aggregate plus 60% RAP. The results obtained ranged from 1.23 MPa to 11.62 MPa. After a statistical study, the author revealed that the optimal mixture concerning strength and durability contains 30% RAP and 70% natural aggregate for all cement dosages. It was suggested that for an economical design, mixtures should have a UCS greater than 1.75 MPa to be applied in sub-base layers, greater than 4.5 MPa for base layers, and greater than 7 MPa for rigid pavement bases.

In order to increase the recycling rate of RAP, Adresi et al. (2019) studied its mechanical properties. Due to the inadequate grading of RAP, percentages of natural aggregate, sand, and filler were added to the RAP aggregate to achieve a well-graded material. The obtained mixtures were stabilized with 3%, 5%, and

7% cement. The results show that the increase in cement content significantly increases the strength of the mixture. The UCS of samples with 5% cement was approximately three times higher than that of samples with 3% cement. The results presented for the mixtures with 5% cement and variations of 0%, 40%, 60%, and 80% RAP were 7.5 MPa, 5.3 MPa, 4.4 MPa, and 3.6 MPa, respectively, with failure occurring after 7 days of curing under saturated conditions at 25 °C. Under dry conditions at 50 °C, the results were 8.4 MPa, 4 MPa, 3 MPa, and 2 MPa. The UCS values were higher for the mixtures without RAP tested at a temperature of 50 °C compared to those tested at 25 °C.

Goldoni et al. (2023) investigates the feasibility of using high RAP (Reclaimed Asphalt Pavement) contents in base layers stabilized with cementitious materials. In mixtures containing 70% RAP and 30% stone dust with 2%, 4%, and 6% cement, the UCS (Unconfined Compressive Strength) values were approximately 1.8 MPa, 4.4 MPa, and 5.6 MPa, respectively. The author highlights those mixtures with 70% RAP and 30% stone dust have lower packing density, resulting in lower strength compared to mixtures composed of 20% RAP, 38% stone dust, supplemented with gravel, and those with 45% RAP, 33% stone dust, also supplemented with gravel. The UCS values for these gravel-included mixtures, with additions of 2%, 4%, and 6% cement, are approximately 1.6 MPa, 3.3 MPa, 8.3 MPa, and 1.8 MPa, 4 MPa, 7.5 MPa, respectively. This led the author to conclude that the RAP content does not have a significant influence on the mechanical behavior of the mixtures. However, the results indicate that increasing cement content enhances mechanical strength, particularly for mixtures with lower RAP content.

3.4 Reflections from literature

Table 7 presents the strength values of different RAP mixtures from various countries, which meet the requirements of the Brazilian standard DNIT (2013). Strength values below the standard are subject to pathologies that may appear prematurely in the pavement, compromising its durability, while excessively high strength values may lead to increased costs during implementation.

Table 7. Unconfined compressive strength of bases with RAP.

Reference	Mistures	UCS (MPa)
Fedriago et al. (2017)	20% RAP + 80% Recycled gravel +2% Cem. (modified)	2.50
Fedriago et al. (2017)	50% RAP + 50% Recycled gravel +2% Cem. (modified)	2.80
Pratikso et al. (2017)	100% RAP + 4.5% Cem.	2.01
Ghanizadeh et al. (2018)	0% RAP + 100% soil (SP-SC) + 3% Cem.	2.76
Ghanizadeh et al. (2018)	20% RAP + 80% soil (SP-SC) + 3% Cem.	2.55
Ghanizadeh et al. (2018)	40% RAP + 60% soil (SP-SC) + 3% Cem.	2.26
Ghanizadeh et al. (2018)	60% RAP + 40% soil (SP-SC) + 4% Cem.	2.43
Ghanizadeh et al. (2018)	40% RAP + 60% soil (GW-GC) + 3% Cem.	2.23
Ghanizadeh et al. (2018)	60% RAP + 40% soil (GW-GC) + 5% Cem.	2.26
Kasu et al. (2020)	80% NA + 20% RAP+ 2.5% Cem.	2.50

The mix designs will depend on the characteristics of the materials; however, Tables 4, 5 and 6 present previous studies that can contribute to the theoretical foundation of future research and provide technical support for the development of highway pavement projects. The optimal cement content range for RAP-containing mixtures is between 2.5% and 5%.

Few studies address the long-term behavior of RAP applied in base layers; generally, research focuses on strength parameters and does not present durability results.

From an environmental perspective, few studies have been conducted on the application of recycled materials in base and sub-base layers of asphalt pavements, as well as studies on life cycle analysis (LCA).

Cost analyses of base layers with cement-treated RAP are rarely studied.

4. Conclusion

The study reviews the effects of recycled materials, as well as RAP (Reclaimed Asphalt Pavement) as a material for the base and sub-base layers of flexible pavements. Based on the results, the following conclusions were drawn:

- a) The addition of RAP and chemical stabilization improves the load-bearing capacity of the granular layers, resulting in a significant increase in the stiffness of the base layer;
- b) The resilience modulus increases with the amount of RAP in the mixtures;
- c) The CBR (California Bearing Ratio) significantly decreases with the increase in the amount of RAP in the mixtures; however, reinforced or treated RAP exhibits high-strength properties, making it suitable for supporting traffic loads and, consequently, its applicability in the base and sub-base layers;
- d) As the RAP content increases, the compressive strength decreases. The reason for the reduction in strength is that the asphalt surrounding the RAP material decreases the friction between the aggregates;
- e) Although the topic has been explored in the academic field, there are gaps that need to be researched, such as durability, life cycle assessment (LCA), and economic viability.

Finally, the SLR (literature review) shows that the results of incorporating RAP as an aggregate for the base and sub-base layers of flexible pavements are promising.

Acknowledgements

Part of the study was financed by FAPEMIG through scholarships.

Declaration of interest

The authors have no conflicts of interest to declare.

Authors' contributions

Emiliane Karolina Gonçalves dos Santos Braga Santana: conceptualization, investigation, methodology, validation, visualization, writing-original draft, writing – review & editing. Júnia Nunes de Paula: project administration, supervision, writing – review & editing.

Data availability

The datasets generated and analyzed during the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was translated with the assistance of generative artificial intelligence (GenAI) by ChatGPT, with the aim of translating the text from Portuguese to US English. The entire process of using this tool was supervised, reviewed and, when necessary, edited by the authors. The authors assume full responsibility for the content of the publication that involved the assistance of GenAI.

List of symbols and abbreviations

d	deflection
BGS	Simple graded gravel
CaCl ₂	Calcium chloride
CBR	California Bearing Ratio
CO ₂	Carbon dioxide
CSS-1H	Cationic asphalt emulsion
DNER	Departamento Nacional de Estradas de Rodagem
DNIT	Departamento Nacional de Infraestrutura de Transportes
GW-GC	Rocky soil with clays
LBR	Limerock Bearing Ratio
LCA	life cycle analysis
MR	Resilience module
N	number of repetitions of a standard axis
NA	Natural aggregate
P	load
RAP	Reclaimed Asphalt Pavement
RPM	Asphalt layer mixed with the underlying base layer
SP-CS	Soil with Stony Sand and Clay
SS-1H	Anionic asphalt emulsion
SRL	Literature review
UCS	Unconfined compressive strength
Ec	compressive deformations
Et	tensile stresses

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Case de Sucesso: Estabilização e Descaracterização da Barragem B2A

O domínio técnico da Fonntes Geotécnica sobre reologia e comportamento de materiais se concretiza em projetos robustos e reconhecidos no setor. Um dos exemplos mais emblemáticos é sua atuação no Complexo Fernandinho, onde, desde 2017, a empresa conduz os estudos e intervenções voltadas à estabilização e descaracterização da Barragem B2A.

A estrutura apresentava fatores de segurança abaixo do exigido pela legislação. Para reverter esse quadro, a Fonntes desenvolveu uma solução integrada baseada no rebaixamento do nível freático, com escavações controladas a montante e instalação de poços tubulares profundos para bombeamento contínuo.

A Fonntes foi responsável por toda a cadeia de estudos: análises tensão-deformação, modelagem hidráulica do Canal de Cintura, adequação dos taludes e definição de parâmetros críticos, sempre com base em extensas investigações geotécnicas e referências técnicas atualizadas.

O projeto também incluiu o Acompanhamento Técnico de Obras (ATO), com monitoramento contínuo por piezômetros, marcos superficiais, topografia e geofones, garantindo controle rígido das deformações e segurança operacional.

Como resultado, a estrutura obteve, no segundo semestre de 2024, a Declaração de Condição de Estabilidade (DCE) e iniciou o processo de descaracterização em seis etapas, todas baseadas nos critérios estabelecidos durante a fase de estabilização.

A atuação da Fonntes na Barragem B2A reafirma sua capacidade de transformar conhecimento técnico em soluções práticas e seguras, destacando-se como referência em engenharia geotécnica aplicada à mineração moderna.



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Geotechnical surveys

Sondagem para Pesquisa Mineral

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Geotechnical Testing Laboratory
(soil, rock and aggregate mechanics)

Obra Geotécnica

Geotechnical Work

Controle Tecnológico, Monitoramento e Instrumentação Geotécnica

Technological Control, Monitoring and
Geotechnical Instrumentation

Prospecção geofísica

Geophysical prospecting

Consultoria e Projeto Geotécnico

Geotechnical Consulting and Design

Inovação e Desenvolvimento Plataforma GeoPlus

Innovation and Development
GeoPlus Platform



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Guide for Authors

Soils and Rocks is an international scientific journal published by the Brazilian Association for Soil Mechanics and Geotechnical Engineering (ABMS) and by the Portuguese Geotechnical Society (SPG). The aim of the journal is to publish original papers on all branches of Geotechnical Engineering. Each manuscript is subjected to either a single-blind or double-blind peer-review process, as the authors choose. The journal's policy of screening for plagiarism includes the use of a plagiarism checker on all submitted manuscripts.

Soils and Rocks embraces the international Open Science program and is striving to meet all the recommendations. Authors are invited to deposit their submissions in reliable preprint servers. Authors who wish to pre-deposit their articles as preprints should contact the editors prior to submission for more information about the process and to sign the Consent Form. Authors are encouraged to make available all content (data, program codes, and other materials) underlying the manuscript text prior to or at the time of publication. Exceptions are allowed in cases of legal and ethical issues.

Soils and Rocks provides a manuscript template available at:

<https://drive.google.com/file/d/17BGwQaD79WSPBVm6BAYATv1rAqNodCZW/view?usp=sharing>

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1. Category of papers

Submissions are classified into one of the following categories:

Article – an extensive and conclusive dissertation about a geotechnical topic, presenting original findings.

Technical Note – presents a study of smaller scope or results of ongoing studies, comprising partial results and/or particular aspects of the investigation.

Case Study – report innovative ways to solve problems associated with design and construction of geotechnical projects. It also presents studies of the performance of existing structures.

Review Article – a summary of the State-of-the-Art or State-of-the-Practice on a particular subject or issue and represents an overview of recent developments.

Discussion – specific discussions about published papers.

Authors are responsible for selecting the correct category when submitting their manuscript. However, the manuscript category may be altered based on the recommendation of the Editorial Board. Authors are also requested to state the category of paper in their cover letter.

When submitting a manuscript for review, the authors should indicate the category of the manuscript, and is also understood that they:

- assume full responsibility for the contents and accuracy of the information in the paper;
- assure that the paper has not been previously published, and is not being submitted to any other journal for publication.

2. Paper length

Full-length manuscripts (Article, Case Study) should be between 4,000 and 8,000 words. Review articles should have up to 10,000 words. Technical Notes have a word count limit of 3,500 words. Discussions have a word count limit of 1,000 words. These word count limits exclude the title page, notation list (e.g., symbols, abbreviations), acknowledgments and references, but include any appendices. Each single column and double column figure or table is considered as equivalent to 150 and 300 words, respectively.

3. Scientific style

The manuscripts should be written in UK or US English, in the third person and all spelling should be checked in accordance with a major English Dictionary. The manuscript should be able to be readily understood by a Civil Engineer and avoid colloquialisms. Unless essential to the comprehension of the manuscript, direct reference to the names of persons, organizations, products or services is not allowed. Flattery or derogatory remarks about any person or organization should not be included.

The author(s) of Discussion Papers should refer to himself (herself/themselves) as the reader(s) and to the author(s) of the paper as the author(s).

The International System (SI) units must be used. The symbols are recommended to be in accordance with Lexicon in 14 Languages, ISSMFE (2013) and the ISRM List of Symbols. Use italics for single letters that denote mathematical constants, variables, and unknown quantities, either in tables or in the text.

4. Submission requirements and contents

A submission implies that the following conditions are met:

- the authors assume full responsibility for the contents and accuracy of the information presented in the paper;
- the manuscript contents have not been published previously, except as a lecture or academic thesis;
- the manuscript is not under consideration for publication elsewhere;
- the manuscript is approved by all authors;
- the manuscript is approved by the necessary authorities, when applicable, such as ethics committees and institutions that may hold intellectual property on contents presented in the manuscript;
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- the authors are aware that the manuscript will be subjected to plagiarism check.

The author(s) must upload two digital files of the manuscript to the Soils and Rocks submission system. The size limit for each submission file is 20 MB. The manuscript should be submitted in docx format (Word 2007 or higher) or doc format (for older Word versions). An additional PDF format file of the manuscript is also required upon submission. Currently, the journal is not accepting manuscripts prepared using LaTeX.

The following documents are required as minimum for submission:

- cover letter;
- manuscript with figures and tables embedded in the text (doc or docx format);
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4.1 Cover letter

The cover letter should include: manuscript title, submission type, authorship information, statement of key findings and work novelty, and related previous publications if applicable. When authors opt for the double-blind peer review process, information on Acknowledgements and Authors' Contributions also should be included.

4.2 Title page

The title page is the first page of the manuscript and must include:

- A concise and informative title of the paper. Avoid abbreviations, acronyms or formulae. Discussion Papers should contain the title of the paper under discussion. Only the first letter of the first word should be capitalized.
- Full name(s) of the author(s). The first name(s) should not be abbreviated. The authors are allowed to abbreviate middle name(s).
- The corresponding author should be identified by a pound sign # beside his/her and in a footnote.

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- Affiliation address and e-mail must appear below each author's name.
- The 16-digit ORCID of the author(s) – mandatory
- Link to the authors' CV Lattes – mandatory for authors affiliated with Brazilian institutions only
- Main text word count (excluding abstract and references) and the number of figures and tables

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Figures, tables or text passages previously published elsewhere may be reproduced under permission from the copyright owner(s) for both the print and online format. The authors are required to provide evidence that such permission has been granted at the moment of paper submission.

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Authors are required to disclose conflicting interests that could inappropriately bias their work. For that end, a section entitled "Declaration of interest" should be included following any acknowledgments and prior to the "Authors' contributions" section. In case of the absence of conflicting interests, the authors should still include a declaration of interest.

4.5 Authors' contributions

Authors are required to include an author statement outlining their individual contributions to the paper according to the CRediT taxonomy (as per <https://credit.niso.org>). The minimum requirements of contribution to the work for recognition of authorship are: a) Participate actively in the discussion of results; b) Review and approval of the final version of the manuscript. A section entitled "Authors' contributions" should be included after the declaration of interest section, and should be formatted with author's name and CRediT role(s), according to the example:

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4.6 Data availability

A data availability statement must be included in any original research article. Authors should include information on where

data supporting the findings reported in the article can be found, if applicable. Persistent hyperlink to make available archived datasets evaluated or produced during the study are to be included. “Data” is the minimal dataset necessary to understand, replicate and develop based on the results presented in the article. Cases in which sharing research data is not possible, data availability is still to be stated in the manuscript including any requirements for access.

4.7 Use of generative artificial intelligence for textual, visual, and data enhancement

The use of generative artificial intelligence (GenAI) has proven to be a valuable tool for translation and for improving the quality of texts, images, graphic elements, as well as for the treatment and compilation of data and results in scientific publications. However, Soils and Rocks considers that at no time can GenAI tools be considered as the author or co-author of the work since authorship and co-authorship are exclusively attributed to human action. Such tool should be used as aid and should be always supervised by humans.

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5. Plagiarism checking

Soils and Rocks follows the ethics of the Committee on Publication Ethics (COPE), of the Code of Ethics American Educational Research Association, the Code of Good Scientific Practice - FAPESP and the Council of Science Editors (CSE) concerning publication. Any material published in is in agreement with the SciELO Guidelines on Best Practices for Strengthening Ethics in Scientific Publication.

Submitted papers are expected to contain at least 50 % new content and the remaining 50 % should not be verbatim to previously published work.

All manuscripts are screened for similarities. Currently, the Editorial Board uses the plagiarism checker Plagius (www.plagius.com) to compare submitted papers to already published works. Manuscripts will be rejected if more than 20 % of content matches previously published work, including self-plagiarism. The decision to reject will be under the Editors’ discretion if the percentage is between 10 % and 20 %.

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from the Journal if the complaint is confirmed. Authors may also be subjected to the penalties.

Corrections and retractions may be corrected through an Erratum in case that errors and shortcomings do not imply in “Misconduct in publication”, which will be recognized as fabrication, falsification, and plagiarism (as per SciELO Guidelines on Best Practices for Strengthening Ethics in Scientific Publication).

IMPORTANT OBSERVATION: Mendeley software plug-in (suggested in this guide) for MS-Word can be used to include the references in the manuscript. This plug-in uses a field code that sometimes includes automatically both title and abstract of the reference. Unfortunately, the similarity software adopted by the Journal (Plagius) recognizes the title and abstract as an actual written text by the field code of the reference and consequently increases considerably the percentage of similarity. Please do make sure to remove the abstract (if existing) inside Mendeley section where the adopted reference is included. This issue has mistakenly caused biased results in the past. The Editorial Board of the journal is now aware of this tendentious feature.

6. Formatting instructions

The text must be presented in a single column, using ISO A4 page size, left, right, top, and bottom margins of 25 mm, Times New Roman 12 font, and line spacing of 1.5. All lines and pages should be numbered.

The text should avoid unnecessary italic and bold words and letters, as well as too many acronyms. Authors should avoid capitalizing words and whenever possible to use tables with distinct font size and style of the regular text.

Figures, tables and equations should be numbered in the sequence that they are mentioned in the text.

Abstract

Please provide an abstract between 150 and 250 words in length. Abbreviations or acronyms should be avoided. The abstract should state briefly the purpose of the work, the main results and major conclusions or key findings.

Keywords

A minimum of three and a maximum of six keywords must be included after the abstract. The keywords must represent the content of the paper. Keywords offer an opportunity to include synonyms for terms that are frequently referred to in the literature using more than one term. Adequate keywords maximize the visibility of your published paper.

Examples:

Poor keywords – piles; dams; numerical modeling; laboratory testing

Better keywords – friction piles; concrete-faced rockfill dams; material point method; bender element test

List of symbols

A list of symbols and definitions used in the text must be included before the References section. Any mathematical constant,

variable or unknown quantity should appear in italics. List lower case characters first, then upper case. Latin alphabet characters must appear before Greek alphabet characters.

Appendix

Use the Appendix when necessary to provide supporting or supplementary information that is part of the study (e.g. tables, graphs, or datasets). The Appendix should be placed after the References. For more than one appendix, organize the titles with letters like Appendix A, Appendix B, and Appendix C.

6.1 Citations

References to other published sources must be made in the text by the last name(s) of the author(s), followed by the year of publication. Examples:

- Narrative citation: [...] while Silva & Pereira (1987) observed that resistance depended on soil density
- Parenthetical citation: It was observed that resistance depended on soil density (Silva & Pereira, 1987).

In the case of three or more authors, the reduced format must be used, e.g.: Silva et al. (1982) or (Silva et al., 1982). Do not italicize “et al.”

Two or more citations belonging to the same author(s) and published in the same year are to be distinguished with small letters, e.g.: (Silva, 1975a, b, c.).

Standards must be cited in the text by the initials of the entity and the year of publication, e.g.: ABNT (1996), ASTM (2003).

6.2 References

A customized style for the Mendeley software is available and may be downloaded from:

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Full references must be listed alphabetically at the end of the text by the first author's last name. Several references belonging to the same author must be cited chronologically. Provide DOI (Digital Object Identifier) whenever available.

Some formatting examples are presented here:

Journal Article

Bishop, A.W., & Blight, G.E. (1963). Some aspects of effective stress in saturated and partly saturated soils. *Géotechnique*, 13(2), 177-197. <https://doi.org/10.1680/geot.1963.13.3.177>.

Castellanza, R., & Nova, R. (2004). Oedometric tests on artificially weathered carbonatic soft rocks. *Journal of Geotechnical and Geoenvironmental Engineering*, 130(7), 728-739. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2004\)130:7\(728\)](https://doi.org/10.1061/(ASCE)1090-0241(2004)130:7(728)).

Fletcher, G. (1965). Standard penetration test: its uses and abuses. *Journal of the Soil Mechanics Foundation Division*, 91, 67-75.

Indraratna, B., Kumara, C., Zhu S-P., Sloan, S. (2015). Mathematical modeling and experimental verification of fluid flow through deformable rough rock joints. *International Journal*

of Geomechanics, 15(4): 04014065-1-04014065-11. [https://doi.org/10.1061/\(ASCE\)GM.1943-5622.0000413](https://doi.org/10.1061/(ASCE)GM.1943-5622.0000413).

Garnier, J., Gaudin, C., Springman, S.M., Culligan, P.J., Goodings, D., König, D., ... & Thorel, L. (2007). Catalogue of scaling laws and similitude questions in geotechnical centrifuge modelling. *International Journal of Physical Modelling in Geotechnics*, 7(3), 01-23. <https://doi.org/10.1680/ijpmg.2007.070301>.

Bicalho, K.V., Gramelich, J.C., & Santos, C.L.C. (2014). Comparação entre os valores de limite de liquidez obtidos pelo método de Casagrande e cone para solos argilosos brasileiros. *Comunicações Geológicas*, 101(3), 1097-1099 (in Portuguese).

Book

Lambe, T.W., & Whitman, R.V. (1979). *Soil Mechanics, SI version*. John Wiley & Sons.

Das, B.M. (2012). *Fundamentos de Engenharia Geotécnica*. Cengage Learning (in Portuguese).

Head, K.H. (2006). *Manual of Soil Laboratory Testing - Volume I: Soil Classification and Compaction Tests*. Whittles Publishing.

Bhering, S.B., Santos, H.G., Manzatto, C.V., Bognola, I., Fasolo, P.J., Carvalho, A.P., ... & Curcio, G.R. (2007). *Mapa de solos do estado do Paraná*. Embrapa (in Portuguese).

Book Section

Yerro, A., & Rohe, A. (2019). Fundamentals of the Material Point Method. In *The Material Point Method for Geotechnical Engineering* (pp. 23-55). CRC Press. <https://doi.org/10.1201/9780429028090>.

Sharma, H.D., Dukes, M.T., & Olsen, D.M. (1990). Field measurements of dynamic moduli and Poisson's ratios of refuse and underlying soils at a landfill site. In *Geotechnics of Waste Fills - Theory and Practice* (pp. 57-70). ASTM International. <https://doi.org/10.1520/STP1070-EB>.

Cavalcante, A.L.B., Borges, L.P.F., & Camapum de Carvalho, J. (2015). Tomografias computadorizadas e análises numéricas aplicadas à caracterização da estrutura porosa de solos não saturados. In *Solos Não Saturados no Contexto Geotécnico* (pp. 531-553). ABMS (in Portuguese).

Proceedings

Jamiolkowski, M.; Ladd, C.C.; Germaine, J.T., & Lancellotta, R. (1985). New developments in field and laboratory testing of soils. *Proc. 11th International Conference on Soil Mechanics and Foundation Engineering*, San Francisco, August 1985. Vol. 1, Balkema, 57-153.

Massey, J.B., Irfan, T.Y. & Cipullo, A. (1989). The characterization of granitic saprolitic soils. *Proc. 12th International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro. Vol. 6, Publications Committee of XII ICSMFE, 533-542.

Indraratna, B., Oliveira D.A.F., & Jayanathan, M. (2008b). Revised shear strength model for infilled rock joints considering overconsolidation effect. *Proc. 1st Southern Hemisphere International Rock Mechanics Symposium*, Perth. ACG, 16-19.

Barreto, T.M., Repsold, L.L., & Casagrande, M.D.T. (2018). Melhoramento de solos arenosos com polímeros. *Proc. 19º Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica*, Salvador. Vol. 2, ABMS, CBMR, ISRM & SPG, 1-11 (in Portuguese).

Thesis

Lee, K.L. (1965). *Triaxial compressive strength of saturated sands under seismic loading conditions* [Doctoral dissertation]. University of California at Berkeley.

Chow, F.C. (1997). *Investigations into the behaviour of displacement pile for offshore foundations* [Doctoral thesis, Imperial College London]. Imperial College London's repository. <https://spiral.imperial.ac.uk/handle/10044/1/7894>.

Araki, M.S. (1997). *Aspectos relativos às propriedades dos solos porosos colapsíveis do Distrito Federal* [Master's dissertation]. University of Brasília (in Portuguese).

Sotomayor, J.M.G. (2018). *Evaluation of drained and non-drained mechanical behavior of iron and gold mine tailings reinforced with polypropylene fibers* [Doctoral thesis, Pontifical Catholic University of Rio de Janeiro]. Pontifical Catholic University of Rio de Janeiro's repository (in Portuguese). <https://doi.org/10.17771/PUCRio.acad.36102>.*

* official title in English should be used when available in the document.

Report

ASTM D7928-17. (2017). Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis. *ASTM International, West Conshohocken, PA*. <https://doi.org/10.1520/D7928-17>.

ABNT NBR 10005. (2004). Procedure for obtention leaching extract of solid wastes. *ABNT - Associação Brasileira de Normas Técnicas*, Rio de Janeiro, RJ (in Portuguese).

DNIT. (2010). Pavimentação - Base de solo-cimento - Especificação de serviço DNIT 143. *DNIT -Departamento Nacional de Infraestrutura de Transportes*, Rio de Janeiro, RJ (in Portuguese).

USACE. (1970). Engineering and Design: Stability of Earth and Rock-Fill Dams, Engineering Manual 1110-2-1902. *Corps of Engineers*, Washington, D.C.

Web Page

Soils and Rocks. (2020). *Guide for Authors*. Soils and Rocks. Retrieved in September 16, 2020, from <http://www.soilsandrocks.com/>.

6.3 Artworks and illustrations

Each figure should be submitted as a high-resolution image, according to the following mandatory requirements:

- Figures created as a TIFF or JPEG file format with minimum resolution of 500 dpi.

- Size the figures according to their final intended size. Single-column figures should have a width of up to 82 mm. Double-column figures should have a maximum width of 170 mm.
- Use Times New Roman for figure lettering. Use lettering sized 8-10 pt. for the final figure size.
- Lines should have 0.5 pt. minimum width in drawings.
- Titles or captions should not be included inside the figure itself.

Figures must be embedded in the text near the position where they are first cited. Cite figures in the manuscript in consecutive numerical order. Denote figure parts by lowercase letters (a, b, c, etc.). Please include a reference citation at the end of the figure caption for previously published material. Authorization from the copyright holder must be provided upon submission for any reproduced material.

Figure captions must be placed below the figure and start with the term "Figure" followed by the figure number and a period. Example:

Figure 1. Shear strength envelope.

Do not abbreviate "Figure" when making cross-references to figures.

All figures are published in color for the electronic version of the journal; however, the print version uses grayscale. Please format figures so that they are adequate even when printed in grayscale.

Accessibility: Please make sure that all figures have descriptive captions (text-to-speech software or a text-to-Braille hardware could be used by blind users). Prefer using patterns (e.g., different symbols for dispersion plot) rather than (or in addition to) colors for conveying information (then the visual elements can be distinguished by colorblind users). Any figure lettering should have a contrast ratio of at least 4.5:1

Improving the color accessibility for the printed version and for colorblind readers: Authors are encouraged to use color figures because they will be published in their original form in the online version. However, authors must consider the need to make their color figures accessible for reviewers and readers that are colorblind. As a general rule of thumb, authors should avoid using red and green simultaneously. Red should be replaced by magenta, vermillion, or orange. Green should be replaced by an off-green color, such as blue-green. Authors should prioritize the use of black, gray, and varying tones of blue and yellow.

These rules of thumb serve as general orientations, but authors must consider that there are multiple types of color blindness, affecting the perception of different colors. Ideally, authors should make use of the following resources: 1) for more information on how to prepare color figures, visit <https://jfly.uni-koeln.de/>; 2) a freeware software available at <http://www.vischeck.com/> is offered by Vischeck, to show how your figures would be perceived by the colorblind.

6.4 Tables

Tables should be presented as a MS Word table with data inserted consistently in separate cells. Place tables in the text near the

position where they are first cited. Tables should be numbered consecutively using Arabic numerals and have a caption consisting of the table number and a brief title. Tables should always be cited in the text. Any previously published material should be identified by giving the original source as a reference at the end of the table caption. Additional comments can be placed as footnotes, indicated by superscript lower-case letters.

When applicable, the units should come right below the corresponding column heading. Horizontal lines should be used at the top and bottom of the table and to separate the headings row. Vertical lines should not be used.

Table captions must be placed above the table and start with the term “Table” followed by the table number and a period. Example:

Table 1. Soil properties.

Do not abbreviate “Table” when making cross-references to tables. Example:

Table 1. Soil properties

Parameter	Symbol	Value
Specific gravity of the sand particles	G_s	2.64
Maximum dry density (Mg/m ³)	$\rho_{d(max)}$	1.554
Minimum dry density (Mg/m ³)	$\rho_{d(min)}$	1.186
Average grain-size (mm)	d_{50}	0.17
Coefficient of uniformity	C_u	1.97

6.5 Mathematical equations

Equations must be submitted as editable text, created using MathType or the built-in equation editor in MS Word. All variables must be presented in italics.

Equations must appear isolated in a single line of the text. Numbers identifying equations must be flushed with the right margin. International System (SI) units must be used. The definitions of the symbols used in the equations must appear in the List of Symbols.

Do not abbreviate “Equation” when making cross-references to an equation.

7. Reviewing procedures

All manuscripts are evaluated following a **single-blind** or a **double-blind** peer-review process. In a single-blind evaluation

the reviewers are kept anonymous. In a double-blind evaluation, both the reviewers and authors are not identified to each other.

Authors **must opt** for either a single or double-blind evaluation process during the submission of their manuscripts. The Associated Editor appointed to your manuscript chooses appropriate reviewers based on the manuscript’s subject and may or may not appoint reviewers suggested by the authors.

Manuscripts submitted to the Journal enters a pre-review assessment by the Editor-in-Chief who decides whether it can be sent for further review. Three criteria are considered in this first evaluation: i) adequacy to the Journal’s scope; ii) compliance with the instructions in the Guidelines for Authors; iii) paper is an original manuscript. Following, an associate editor is assigned to the submission and the paper is sent at least to two independent reviewers, which evaluates the scientific content. Reviewers may recommend to the Associate Editor either: a) accept as it is; b) accept with minor revision by authors; c) accept with major revision by authors; d) reject encouraging further resubmission after extensive changes; e) reject without any possibility of further resubmission.

Typically, two to three evaluations are required before the Associated Editor makes a decision recommendation to the Editor-in-Chief, which may be either: a) accept for publication; b) requires format revision; c) requires content revision due to similarity check; d) requires minor revision; e) requires major revision; f) reject. Currently, invited reviewers are given a deadline of 40 days to provide their assessments.

The deadline for authors to submit revised manuscripts is 20 days for minor changes, whereas for major changes it is 30 days.

8. Instructions for authors opting for a double-blind evaluation procedure

Authors opting for a double-blind evaluation of the manuscript must: 1) Clearly indicate their option for a double-blind evaluation in the cover letter; 2) Remove from the submitted manuscript files any indication of authorship;

- First page: remove all authorship details, such as institution, email, and ORCID;
- Acknowledgements and Authors’ Contributions: please omit these sections since they often can reveal details regarding the manuscript’s authorship. The information from these sections should be **included in the cover letter** and, at a later stage, in the manuscript after it is accepted for publication.

Soils and Rocks
An International Journal of Geotechnical and Geoenvironmental Engineering

Volume 48, N. 2, April-June 2025

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